GEOTECHNICAL INVESTIGATION

Open Door Community Health Centers
Tydd Street
Eureka, California

Assessor's Parcel Numbers 002-191-027, -028, and -031

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January 11, 2011
LACO Project No. 7119.02

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Assessor's Parcel Number 002-191-027, -028, and -031

LACO Project Number 7119.02

1.0 INTRODUCTION

LACO Associates (LACO) performed a Geotechnical Investigation in support of the design and construction of the new Open Door Community Health Centers (ODCHC) in the City of Eureka, California. The project site is located in the northeast section Eureka between Myrtle Avenue and U.S. Highway 101, and is accessed from the intersection of West Avenue and Tydd Street. Currently the area encompasses three undeveloped vacant parcels. The general site location is illustrated on Figure 1.

The investigation described in this report was performed in accordance with the scope of Engineering Services outlined in our agreement with ODCHC dated November 1, 2010, and approved on November 15, 2010. The primary purposes of this investigation were to explore and characterize subsurface soil conditions at the site and to develop geotechnical recommendations and design criteria for permitting, foundation support, and earthwork construction for the new building.

The scope of Engineering Services for this investigation consisted of a field exploration program including geotechnical test borings; performing laboratory tests on selected soil samples to evaluate pertinent engineering and index properties of the subsurface soils encountered during our exploration; geotechnical analyses and assessment of potential geologic hazards; developing recommendations for foundation support and earthwork; and preparation of this report to present our findings, conclusions and recommendations to be utilized for permitting, design, and construction of the proposed development. Specifically, this report includes the following:

- Description of site terrain, local and regional geology based on available published maps and literature, and our field exploration.
- Interpreted descriptions of subsurface soil and groundwater conditions based on our field exploration, laboratory testing, and geotechnical analyses.
- Assessment of potential earthquake-related hazards (including surface fault rupture, liquefaction, differential settlement, and site instability) with discussion of possible mitigation measures, as necessary.
- Seismic design parameters in accordance with the applicable portions of the most recent California Building Code (CBC), including site soil classification, seismic design category, and spectral response classifications. No site-specific ground motion assessment was performed as part of this investigation.
- Discussion of appropriate foundation support options and estimates of static and dynamic settlement.
- Recommended foundation and retaining wall design criteria, including:

    - Allowable bearing pressures or capacities for dead, live and wind or seismic loads
    - Minimum foundation embedment
    - Allowable lateral earth pressures including passive pressure and sliding resistance values
- Recommendations for exterior site drainage requirements.
- Recommendations for earthwork construction including site and subgrade preparation; fill material quality, placement, and compaction requirements; criteria for temporary excavation support; and general dewatering.
- Recommendations for pavement section thicknesses.
- Recommendations for observation of footing excavation and foundation installation.

Specifically excluded from our scope of work was an environmental assessment for the presence or absence of any hazardous, toxic, or corrosive materials. Although we have explored subsurface conditions as part of this investigation, we have not conducted any analytical laboratory testing of samples obtained for the presence of hazardous, toxic, or corrosive materials.

1.1 Limitations

This report has been prepared for the exclusive use of our CLIENT (ODCHC), our CLIENT's contractors and sub-consultants, and appropriate public authorities for specific application to development of the site. LACO has endeavored to comply with the generally accepted geotechnical engineering standard of care common to the local area. LACO makes no other warranty, express or implied. A brochure prepared by ASFE (Association of Firms Practicing in the Geosciences) has been included as Attachment 1 of this report. We recommend that all individuals reading this report also read this brochure.

The findings, analyses, and recommendations contained in this report are based on data obtained from subsurface explorations and laboratory tests. The exploration methods used indicate subsurface conditions only at specific locations where samples were obtained, only at the time they were obtained, and only to the depths penetrated. Samples cannot always be relied upon to accurately reflect stratigraphic variations that commonly exist between sampling locations, nor do they necessarily represent conditions at any other time. Results of sample testing obtained during this project will be retained on file in our office. Unless directed otherwise by our CLIENT, collected samples will be discarded after 60 days following the issuance of this report.

The recommendations included in this report are based in part on assumptions about subsurface conditions that may only be confirmed during earthwork. Accordingly, the validity of these recommendations is contingent upon LACO being retained to provide additional professional services during project design and construction. LACO cannot assume responsibility or liability for the adequacy of the report recommendations when they are applied in the field unless LACO is retained to observe and test during project construction. Please contact us to further discuss the

extent of such observations and tests required to check the validity of our recommendations.

This report's findings, conclusions and/or recommendations should not be used if the nature, design, or location of the proposed development is changed. If changes are contemplated, LACO should be consulted to review their impact on the applicability of the findings, conclusions and/or recommendations contained in this report. Also, LACO will not be responsible for any claims, damages, or liability associated with any other party's interpretation of the subsurface data or reuse of this report for other projects, or at other locations, without our express written authorization.

2.0 PROJECT DESCRIPTION

Pertinent project site location information is listed in Table 1 below.

Table 1. Project Location Information

Latitude and Longitude	41.8019° North and -124.1444° West
Assessor Parcel Number(s)	002-191-027, -028, and -031
United States Geologic Survey Quadrangle (USGS)	Eureka 7.5-minute topographic quadrangle

Pre-design concept plans for the proposed development call for constructing a two-story medical building presumably of wood-frame construction and having a concrete slab-on-grade foundation, and totaling approximately 26,000 square-feet in size. The lower floor footprint will be approximately 200 feet in length and between 60 and 80 feet in width. The portion of the building that abuts Tydd Street will include a drive-through covered main entrance. Additional onsite improvements include paved parking areas, landscaping, drainage, and widening of the portion of Tydd Street fronting the new building. The actual building loads are not yet known, but are assumed to be in the light to moderate range for the type of construction anticipated.

3.0 FIELD EXPLORATION AND LABORATORY TESTING

The geotechnical field exploration and laboratory testing programs were performed for the current investigation as described below.

3.1 Field Exploration Program

Four borings were drilled and sampled at the locations shown on Figure 2. Boring locations were limited by the accessibility of the two-wheel drive, truck-mounted drilling rig. The boring locations were located in the field on a surveyed base map. Borings were drilled by Clear Heart Drilling, Inc. (Santa Rosa, California), on December 20 and 21, 2010, using a CME 75 drilling rig fitted with hollow-stem augers and an automatic safety hammer. All borings were advanced to depths of between 25- and 50-feet below existing grade.

Borings were logged in the field by a Professional Geologist from our office in general accordance with ASTM D 2488 (Visual-Manual Procedure). Upon their completion, borings were backfilled with drill cuttings and bentonite grout to existing grade. Soil samples collected from the exploration locations were submitted to the LACO materials testing laboratory for further examination and testing. The final boring logs were prepared based on the field logs in conjunction with the laboratory test results.

3.2 Laboratory Testing

Laboratory testing of selected soil samples was conducted to evaluate their physical characteristics and engineering properties. Samples were tested for in-place density, moisture content, percent fines (silt and clay), and shear strength. A summary of the testing results are presented below and at the corresponding sample locations on the Soil Profile Logs in Appendix A. Laboratory Data Sheets are presented in full in Appendix B.

Table 2. Laboratory Testing Results Summary

Boring	Depth (ft)	Soil Type(s)	Dry Density (pcf)	Moisture Content (%)	Friction Angle (ø)	Cohesion (psf)	Void Ratio	Minus #200 Sieve %
TB-1	4	SP/SM	--	--	--	--	--	13
TB-1	6	SP/SM	92.0	26.1	--	--	0.8	13
TB-1	7	SC	92.9	25.8	33.7	317	--	18
TB-2	4	SM	--	--	--	--	--	26
TB-2	6.5	SP/SM	--	--	--	--	--	13
TB-2	11.5	SP/SM	--	--	--	--	--	10
TB-2	16.5	SM	--	--	--	--	--	18
TB-2	21.5	SM	--	--	--	--	--	20
TB-2	23.5	SM	108.4	21.1	--	--	0.5	16
TB-3	3.5	SC	105.2	21.1	34	490	0.6	26
TB-4	7.5	SP/SC	107.3	19.0	38	609	0.5	13

4.0 SITE AND SUBSURFACE CONDITIONS

The following report sections describe the current project site and surface conditions, the geologic and seismic settings of the site vicinity, and subsurface soil and groundwater conditions encountered at our exploration locations.

4.1 Site Conditions

The portion of the project site to be developed is situated at an elevation of approximately 21 to 32 feet relative to mean sea level. The Tydd Street access is surfaced with asphalt concrete (AC) paving and gravel shoulders. The remainder of the site is undeveloped and consists of a nearly-level, open, grassy field flanked by gentle slopes that descend toward wetland areas. All development will be

located on the level areas of the site at the higher elevations, outside of the wetland and wetland buffer zones. Overhead and buried utilities are present along Tydd Street.

4.2 Geologic Setting

Based on a review of the site, our soil boring logs, and published geologic maps (California Department of Mines and Geology, CDMG, 1980; McLaughlin *et al*., 2000), the project site is underlain by a minimum of 50-feet of uplifted Quaternary aged marine terrace deposits (map symbol Qt, Figures 3A and 3B). Published mapping prepared by CDMG (1980) indicates these terrace deposits consist of Hookton Formation sediments composed of shallow marine and non-marine sands and gravel. The published map compilation by McLaughlin (2000) indicates the site to be underlain by Pleistocene to Holocene age non-marine terrace deposits. On the basis of our test borings and soil textures, we interpret the sediments within the upper 50-feet of the ground surface to be of shallow marine origin. Sediments are composed primarily of silty sand grading downward to poorly graded sand and poorly graded sand with silt. The relative density of the soil profile, based on Standard Penetration Testing (SPT), is consistent with Pleistocene age marine terrace deposits composed of Hookton Formation sediments elsewhere in Humboldt County.

4.3 Seismicity

The project site is not located within a "Fault Rupture Hazard Zone" (California Geological Survey, CGS, 2007; CGS, 2010). However, the site is located within a seismically active region subject to frequent moderate to large earthquakes. The regional tectonic framework is controlled by the Cascadia subduction zone (CSZ), wherein oceanic crust of the Juan de Fuca/Gorda plate is being actively subducted beneath the leading edge of the North American plate. The CSZ in its entirety extends from the southern Humboldt County coast to British Columbia. Plate convergence along the Gorda segment of the CSZ is occurring at a rate of approximately 30 to 40 millimeters per year (mm/yr) (Heaton & Kanamori, 1984). Rupture along the entire CSZ boundary may produce an earthquake with a maximum moment magnitude (M_w) of 9.0 or greater (Satake, 2003).

Upper plate crustal deformation associated with the subduction of the Gorda plate is expressed as a 90-kilometer (km) wide fold and thrust belt that comprises the accretionary complex along the North American plate margin (Carver, 1987). Faults associated with the offshore and onshore portions of the CSZ fold and thrust belt include the Little Salmon and Mad River fault zones.

The Little Salmon and Fickle Hill faults are the closest recognized active fault traces to the project site, and are located a distance of 6.5 miles and 5.5 miles, respectively, from the site (CDMG, 1983; CDMG, 1991; CGS, 2010). Both faults are northwest-striking, northeast-dipping, low-angle thrust faults. The upper-bound earthquakes considered likely to occur on the Little Salmon and Fickle Hill faults have an estimated maximum moment magnitude M_w of 7.1 and 6.9, respectively (International Conference of Building Officials, 1998).

Based on the record of historical earthquakes (approximately 150 years), faults within the plate boundary zone and internally deforming Gorda Plate have produced numerous small-magnitude and several moderate to large (i.e., magnitude greater than 6) earthquakes affecting the local area. Several active regional seismic sources in addition to those mentioned above are proximal to the project site and have the potential to produce strong ground motions. These seismic sources include:

- The northern segment of the San Andreas transform fault that represents the boundary between the stable North American plate and the northwest-migrating Pacific plate.
- The Mendocino fault, an offshore, high-angle, east-west-trending, right-lateral strike-slip fault that forms the boundary between the Gorda and Pacific plates.
- Faults within the internally-deforming Gorda plate consisting of high-angle, northeast-trending, left-lateral, strike-slip faults.

4.4 Subsurface Conditions

Native soils encountered at each of the four test locations consist predominantly of shallow marine sediments to the depths explored. Soil texture graded vertically, from shallow to deep, from lean clay (CL) and silt (ML), to silty sand (SM), to poorly graded sand (SP) and poorly graded sand with silt and/or clay (SP/SM/SC). The lean clay and silt were encountered in the shallow subsoils (less than about 5-feet in depth), and are interpreted to represent *in situ* soil development within an eolian cap. With the exception of these shallow subsoils, soils were typically non-plastic to low plasticity. Relative densities were typically medium dense to dense. In the two deep borings, stiff bluish gray elastic silt containing abundant shell fragments was encountered between approximately 46 feet and 49 feet below the ground surface (bgs). The elastic silt appears to be interbedded and in abrupt contact with the marine sand.

A summary of the generalized soil types underlying the project site is presented below. Detailed descriptions of the soils encountered in each boring for the current investigation is provided Appendix A.

Table 3. Soil Profile Summary

Depth (ft)	Primary Soil Type(s)	Consistency[1]
0	ML	soft/--
2.5	CL	medium stiff/--
5	SM	--/loose to medium dense
10	SP	--/medium dense
15	SM	--/medium dense
20	SC	--/medium dense
25	SM, SC	--/dense
35	SP	--/medium dense to dense
40	SM	--/medium dense
45	MH	stiff/--
50	SC, SM	--/medium dense to dense

Note: (1) consistency of cohesive materials (relative density of cohesionless materials) based on average SPT blow counts (N, uncorrected for depth).

4.5 Existing Fills

No fill soils were encountered at our test boring locations. However, a previous field exploration performed with a backhoe by Whitchurch Engineering, Inc. (2006; Appendix C), identified several areas of the site to be underlain by undocumented fill soils (Figure 2). Test pit excavations at that time exposed the presence of up to 10 feet of fill in the eastern-half of the eastern-most parcel. Fill soils are present in those areas along the crest, and down slope of the descending grade break, near the southeasterly building corner of the proposed structure. The topographic high and over-steepened slope below the descending grade break in the western portion of the site, also suggest the presence of a spoils pile and side cast fill material, respectively. On the basis of existing topography, it should be anticipated that fill material up to several feet thick will be present beneath the southeasterly building corner and parking area east of the new structure. The portion of the new structure located on relatively level ground appears to be sited on relatively undisturbed native soils.

4.6 Groundwater Conditions

At the time of our field exploration during the beginning of the winter wet-season, free groundwater was observed in our test borings at a depth of approximately 15-feet bgs. As reported during the Whitchurch Engineering, Inc. investigation conducted in April and May 2005, free groundwater was encountered within their test pit excavations at a depth of approximately of 10-feet bgs at the higher elevations of the site, and as shallow as 3-feet bgs near the toe of the descending slope. A seasonal variation of at least several feet in groundwater elevation is expected to occur at this site. Therefore, it should be anticipated that saturated soil conditions will generally be present during the wet season

below about 10-feet, but possibly as close as 3-feet (as measured from the ground surface), depending on the portion of the site being excavated.

4.7 Geologic/Geotechnical-related Hazards

The primary geologic/geotechnical-related hazards associated with this site include seismic ground shaking, liquefaction, and settlement. The assessments for these and other potential geologic/geotechnical-related hazards are presented below.

4.7.1 Seismic Ground Shaking

As noted in Section 4.3 above, the project site is situated within a seismically active area proximal to multiple seismic sources capable of generating moderate to strong ground motions. Given the proximity of multiple active seismic sources (the Little Salmon fault to the south, Fickle Hill fault to the north, and Cascadia subduction zone offshore), as well as other active faults within and offshore of northern California, there is high probability that the project site will experience strong ground shaking during the economic life span of the proposed development.

The spectral response accelerations prescribed by the 2007 CBC as related to seismic analysis and design for the proposed structures are presented in Section 5.4 of this report.

4.7.2 Surface Fault Rupture

The Fickle Hill fault is the nearest active fault in proximity to the project site, located approximately 5.5 miles to the northeast (CDMG, 1983). The project site, however, is not located within an Alquist-Priolo Earthquake Fault Rupture Zone. Based on the distance between the project site and the nearest active fault trace, the potential for surface fault rupture to occur within the boundaries of the subject parcel is low.

4.7.3 Liquefaction

Liquefaction is a soil behavior phenomenon in which soil strength is rapidly decreased due to high excess pore-water pressure generated by strong earthquake ground motions. Geologically young (i.e., less than 11,000 years old), and relatively unconsolidated granular soils and/or artificial fills located below the groundwater surface are susceptible to liquefaction (Youd and Perkins, 1978). Relatively clean, loose, uniformly graded sand and non-plastic silts are typically most susceptible to liquefaction. In addition to the necessary adverse soil and groundwater conditions, the ground acceleration must be high, and the duration of the shaking must be sufficient, for liquefaction to occur. As discussed previously in this report, the soil profile and laboratory data gathered during this investigation indicate that the soils underlying the proposed developments are predominantly medium dense to dense, coarse grained soils of late Pleistocene age.

Adverse effects associated with liquefaction include localized ground settlement resulting from soil densification, ground cracking, the partial and/or complete loss of structural load bearing and

confinement, amplification of seismic ground motion, and lateral spreading resulting from competent strata being mobilized atop a liquefied soil layer toward an unsupported slope face (such as the descending slope along the southerly edge of the project site).

SPT blow counts of the *in-situ* subsoils from four locations were analyzed for liquefaction and coseismic settlement potential using the program LiqIT (by GeoLogismiki version 4.7.6.1). The Liquefaction Analysis Report sheets are included in Appendix C. The likelihood for liquefaction to occur is indicated where the Factor of Safety (F.S.) is less than 1. According to LiqIT analysis, no liquefiable layers are present below the groundwater table to the maximum depths explored (50-feet).

On the basis of the soil conditions encountered at our boring locations, and the results of our liquefaction analysis, we conclude there is a low potential for liquefaction to occur at the site in response to strong earthquake ground motions. This low likelihood that liquefaction will occur is also consistent with the subsoils inferred late Pleistocene age.

4.7.4 Site Instability/Landsliding

Lateral spreading, which is the lateral displacement of surficial soils, is usually associated with liquefaction of the underlying soils. The potential liquefaction hazard at the site is considered to be low based on our quantitative assessment. Due to the age and density of the underlying soils, the potential for liquefaction-induced lateral spreading to occur is also considered low.

The project site is located on a nearly level surface. The closest slopes to the site are the gradually descending slopes into the wetland area southerly of the building site. According to the Humboldt County Community Development Service, the low gradient slopes on which the site is located are considered “Relatively Stable” (HCCDS, 2004).

On the basis of the current morphology exhibited by the slopes in the vicinity of the project site, the absence of past or incipient instability, and our qualitative evaluation, the hazard posed by landsliding to the new development is considered low.

4.7.5 Tsunami

The most recent tsunami hazard maps published by the State of California (CGS, 2009) indicate that the site is outside a predicated tsunami run-up zone. On the basis of this mapping, the risk of tsunami inundation at the site is considered low.

4.7.6 Soil Swelling or Shrinkage Potential

Expansion potential represents a significant structural hazard to buildings founded on some plastic clay soils where site conditions cause a seasonal fluctuation in soil moisture. Due to the presence of non-plastic granular soils at this site composed primarily of fine to medium sand, or low plasticity

silt and clays, the risk of expansive soil movement (shrink or swell) at this site is considered low to negligible.

4.7.8 Settlement

Static settlement is the result of compressive consolidation of soil beneath an applied load. The consolidation generally results from a reduction in voids within the soil. In dry soils, the settlement of the soil occurs relatively rapidly. However, in saturated soils the voids are filled with water that must be drained to accommodate the settlement. In fine-grained soils, the rate at which water moves through the soil is slow compared to granular soils. As a result, settlement of the saturated fine grain soils occurs more slowly. Given the typical coarse-grained nature and permeability of the soils at this site, total settlement under applied loads is anticipated to occur relatively rapidly (several months).

Although specific building loads for the new structure were not available at the time of this report, we assume a light to moderately-loaded structure typical of two-story, wood-frame construction. As such, ultimate settlement is not likely to exceed approximately1.0- inch for total loads in the range of 1 to 2 kips per lineal foot for walls and 50 kips for columns; post-construction differential settlement is not likely to exceed 0.25 inches between adjacent footings or along a continuous footing.

On the basis of our quantitative liquefaction analysis, dynamic settlement resulting from a liquefaction event (if any) is not likely to exceed approximately 0.25 inches.

5.0 DISCUSSION AND GEOTECHNICAL RECOMMENDATIONS

On the basis of our investigation, it is our opinion that the project is feasible as currently proposed. However, the herein reported geotechnical-related conditions, risks, and hazards must be considered, and our recommendations adhered to, during both the design and construction phases of the project. To reduce the potential for adverse settlement, the new structure should be supported on a conventionally-reinforced concrete slab-on-ground foundation system which, in turn, gains support on compacted engineered fill consisting of partial-excavation of any loosely consolidated fill materials and/or soft, native topsoil.

The key geotechnical issues for design and construction of the proposed structure include the following:

- The presence of variable thickness fill soils near the location of the southeasterly building corner, which are susceptible to settlement under new or increased loading conditions. These fill soils are typically composed of a mixture of silt, sand, and gravel with construction debris containing concrete rubble, rebar, and wood.
- The location of the project site within a seismically active region which is subject to strong earthquake ground motions resulting from a multitude of onshore and offshore seismic sources. The new structure that falls under code-required design

requirements will, at a minimum, need to resist moderate levels of seismic ground motion without experiencing structural damage, and to resist very strong seismic ground motion having intensity equal to, or greater than, the Maximum Considered Earthquake (MCE) without collapse but with some structural damage.

- Groundwater control within the low-lying parking areas, east of the proposed structure, during and after construction due to the presence of shallow, perched groundwater and/or saturated soil conditions.

Our preliminary recommendations presented in the following sections include initial provisions to mitigate these conditions to acceptable levels. Mitigation of all potentially adverse effects resulting from relatively infrequent upper bound seismic events may be impractical due to cost considerations. As such, the CLIENT may be willing to accept some degree of risk and co-incident damage to the structure depending on their budgetary constraints.

5.1 Foundations

Foundation plans are not yet known. The following recommendations are suitable for foundation support and design, subject to the conditions presented:

- Design bearing pressures should be no more than 2,000 pounds per square foot (psf) for dead and live loads, and no more than 2,500 psf for total load combinations including wind or seismic forces. Resistance to lateral forces may be computed using friction or passive pressure against foundation elements. A friction factor of 0.35 is considered appropriate between the undersurface of concrete foundations and the supporting soils. A passive pressure consisting of both an equivalent fluid component weighing 250 pounds per cubic foot (pcf) is recommended against the sides of foundations. In computations, if friction and passive pressures are combined, the lesser value should be reduced by 50 percent. Also, passive pressure should be neglected in the upper 12 inches of the ground surface, unless footings are confined by pavements or slabs.
- Footing concrete should be placed neat against undisturbed soil or rock, if possible. The materials exposed in footing excavations should not be allowed to dry before placing concrete. If shrinkage cracks appear in the footing excavation materials, these materials should be thoroughly moistened to close all cracks prior to concrete placement.
- The foundation system should be designed and constructed in accordance with the minimum standards of the current edition of the CBC, and the recommendations contained herein.
- The foundation excavations should be observed by LACO prior to placement of foundation forms or reinforcing steel.

5.2 Floor Slabs

The concrete floor slab-on-grade should have a minimum thickness of 4-inches and be designed to resist cracking from bending, tension, or shearing forces as required by the CBC. The floor slab should also be designed to accommodate the anticipated floor storage loads from medical equipment

and, as such, may be thickened and/or structurally integrated with both continuous and isolated foundations, as needed.

The floor slab should be underlain by at least 4 inches of underslab rock consisting of clean, ¾-inch material to act as a capillary moisture break. To reduce the possibility of moisture migration through the floor slab, a minimum 15-mil plastic membrane (vapor retarder) should be placed on top of the underslab rock. Joints between the sheets and utility piping openings should be lapped and taped. To help protect the membrane from puncture during steel and concrete placement, and to possibly aid in concrete finishing, the membrane should be covered with at least 2-inches of clean sand.

5.3 Retaining Walls

New retaining walls (if any) can be backfilled with the onsite soils that were found to have a low expansion potential. Over-compaction of wall backfill should also be avoided because increased compaction effort can result in lateral pressures significantly greater than those recommended below. Light-weight compaction equipment should be used to reduce the potential for overstressing the wall.

An active soil pressure may be used for design of retaining walls, if the wall is able to move at least one-tenth of a percent of the planned wall height and the corresponding backfill settlement is not a concern. The static active lateral soil pressure will be a triangular pressure distribution calculated using an equivalent fluid weight of 35 pcf. In addition, one-third of any live load behind the walls should be applied as an added design surcharge. For drained backfill slope angles of between 4:1 (feet horizontal to feet vertical) and 2:1, if any, the active equivalent fluid weight should be increased to 55 pcf.

Section 1802.2.7 of the 2007 CBC requires a determination of lateral pressures due to earthquake motions on structure retaining walls in Seismic Design Categories D, E and F. We assume that retaining structures, where planned, will be in Seismic Design Category D. Seismic lateral forces were estimated using Mononobe-Okabe analysis and a pseudo-static horizontal seismic force. For this analysis, the Peak Ground acceleration (PGA) was estimated as 0.40g (S_{DS}/2.5). The psuedo-static acceleration used in this analysis was 0.20g (PGA/2). Based on this analysis, the total seismic lateral force will be equal to approximately 45 percent of the static lateral force. However, in contrast to the static force (which is assumed to act at heights of H/3 and H/2 above the base of the wall, where H equals the wall height), the resultant of the seismic increase should be assumed to act at a height of 0.6H above the base of the wall.

Retaining walls may be supported on spread footings that are design in accordance with the foundation bearing and lateral resistance recommendations presented above. Backfill behind retaining walls should generally consist of onsite granular fill material that is free of rock sizes greater than about 4 inches in largest dimension. Backfill should be compacted to between 90

percent and 92 percent relative compaction. To reduce backfill pressures, we recommend that the backfill placed within five feet of the wall (measured horizontally) be compacted with lightweight, hand-operated compaction equipment. Over-compaction of this backfill can greatly increase wall pressures and/or deflections.

Seasonal groundwater may collect near the base of the wall. To provide drainage, immediately behind the retaining wall, and for a minimum thickness of 1-foot (horizontally), the wall should be backfilled with drain rock conforming to the requirements of the State of California Department of Transportation (Caltrans) Standard Specifications (68-1.025) for Class 1 Permeable Material if the drain rock will be separated from native soils with a drainage geotextile Caltrans Standard Specifications for "Filter Fabric," 88-1.03, or Class 2 Permeable Material if a drainage geotextile is not used. Alternatively, a prefabricated drainage board, such as Miradrain (or equivalent) may be used in lieu of the drain rock. The drain rock (or drainboard) should enclose a 4-inch-diameter (minimum) perforated drainpipe at the base of the structure which discharges into a tight drain pipe outletting into the site storm drain system or, alternatively, by weep holes in the wall where soil erosion at the wall base is not a concern. The backfill should be capped with approximately 1 foot of topsoil, clay, or pavement so that surface infiltration does not overload the drainage system.

5.4 Seismic Design

We recommend the proposed building be designed and constructed to withstand seismic shaking as required by the CBC. Based on the site conditions as encountered at test boring locations, we have classified the site as Site Class D consisting of a "stiff soil profile" (Section 1613.5.2, 2007 CBC). On this basis, the design spectral response accelerations Ss, S_1, Fa, Fv, S_{MS}, S_{M1}, S_{DS} and S_{D1} were determined using the United States Geological Survey (USGS) seismic calculator software, "Seismic Hazard Curves, Response Parameter, Design Parameters: Seismic Hazard Curves and Uniform Hazard Response Spectra", version 5.0.9a dated November 4, 2010, utilizing the American Society of Civil Engineers (ASCE) Standard 7-05, Minimum Design Loads for Buildings and Other Structures analysis option. Calculated values are presented below.

Table 4. Summary of Seismic Design Factors

Site Class	F_a	Fv	Ss	S_1	S_{MS}	S_{M1}	S_{DS}	S_{D1}
D	1.0	1.5	1.480	0.704	1.480	1.056	0.987	0.704

The criteria are defined as follows:

Fa – Short period coefficient to modify 0.2-second period of mapped spectral response accelerations for Site Class other than Site Class B.

Fv – Long period coefficient to modify 1.0-second period of mapped spectral response accelerations for Site Class other than Site Class B.

Ss – Mapped spectral response acceleration, 5 percent damped, at 0.2-second period for Site Class B (in %g).

S_1 – Mapped spectral response acceleration, 5 percent damped, at 1.0-second period for Site Class B (in %g).

S_{MS} – Maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-second for Site Class effects (in %g).

S_{M1} – Maximum considered earthquake spectral response acceleration, 5 percent damped, at 1.0-second period for Site Class effects (in %g).

S_{DS} – Design spectral response acceleration, 5 percent damped, at 0.2-second period (in %g).

S_{D1} – Design spectral response acceleration, 5 percent damped, at 1.0-second period (in %g).

5.5 Earthwork

The following sections provide earthwork recommendations to suitably prepare the site for construction of the new structure. Recommendations for site and subgrade preparation; excavation criteria and groundwater control; fill and backfill quality and compaction; and surface drainage control are presented.

5.5.1 Site Preparation

The proposed building and related development will be located within an undeveloped area, portions of which were previously used as a spoils site. As such, any old construction rubble, fill material, sod, topsoil, and/or any other debris material encountered at, or below, the existing ground surface should be removed from the new construction area. All earthwork including, but not limited to, site clearing, grubbing, and stripping should be conducted during dry weather conditions, if feasible. If wet-weather site preparation is to be conducted, care should be taken to prevent excessive rutting of and/or mixing of the loose/soft surficial soils with new imported fill, underslab rock and/or aggregate base materials.

All active or inactive utility lines within the construction area should be relocated, abandoned, or fully protected during new construction and later operation of the planned development. Pipelines to be abandoned in place should generally be filled with sand-cement slurry. If existing utilities are removed, the resulting excavations should be backfilled with compacted fill. In areas where existing foundations are removed, the resulting excavations should be backfilled with compacted fill.

5.5.2 Building Area Preparation

The area to support the concrete slab-on-grade foundation system should be prepared as follows:

1. Remove any existing old fill to its full depth and native topsoil to a depth of at least 12-inches below the planned slab subgrade elevation.
2. Scarify, moisture condition, and re-compact the upper 12-inches of exposed soils to a minimum of two percent above optimum moisture content and to 90 percent of the same soils maximum dry density (ASTM D1557 method).
3. Following compaction of the exposed soils, place and suitably compact at least 12-inches of native soil or imported granular fill to achieve the new planned slab subgrade elevation. Excessively soft or yielding soils identified during excavation and subsequent fill compaction should either be removed and replaced with additional compacted engineered fill or otherwise suitably mitigated through supplemental recommendations provided by the project Geotechnical Engineer, as appropriate.

5.5.3 Excavation Conditions

Based on the subsurface conditions encountered at the test locations, excavations needed for earthwork construction will be made in soils that consist predominantly of granular material. These soils will most likely become weaker and be prone to yielding if exposed to prolonged periods of rain; the saturated granular soils may also be subject to sloughing and/or caving.

All temporary excavations and construction slopes should be designed, planned, constructed, and maintained by the contractor and should conform to all applicable local, state, and federal regulations including the current Occupational Safety and Health Administration (OSHA) Excavation and Trench Safety Standards. To help minimize the risk of ground movement and/or settlement, construction equipment, building materials, excavated soil, vehicular traffic, and other similar loads should not be allowed near the top of any unshored (unbraced) excavation. Where the stability of adjoining buildings, walls, pavements, or other similar improvements may be endangered by excavation operations, and to protect personnel working in the excavation, support systems such as shoring, bracing, or underpinning may be required to provide structure and trench wall stability.

Excavation operations are dependent on construction methods and schedules and, as such, the contractor shall be solely responsible for the design, installation, maintenance, and performance of all shoring, bracing, underpinning, and other similar excavation-related systems. Under no circumstances should anything written herein be inferred to mean that LACO assumes any

responsibility for temporary excavations or the safety thereof. Nor does LACO assume any responsibility for the design, installation, maintenance, and performance of any shoring, bracing, underpinning, or other similar excavation-related systems.

5.5.4 Fill Quality and Compaction Standard

Fill materials used to support foundations, floor slabs, sidewalks, and/or pavements should be composed of soil material having a low expansion potential, and be free of organic content, debris, and/or other deleterious matter. Engineered (compacted) fills should be placed on a prepared grade as specified above. The fill material should not generally contain rocks larger than 3-inches in greatest dimension, or more than 15 percent larger than 2-inches. Additionally, the material should typically meet with the following specifications:

Plasticity Index:	less than15%
Liquid Limit:	less than 40%
Percent passing No. 200 sieve:	50 maximum, 5 minimum

As noted in the table below, engineered fill should be compacted to a minimum of 90 percent of the same soils maximum dry density (ASTM D1557 method). A qualified Field Technician should be present to observe fill placement and perform field density tests (ASTM D6938 method) at random locations throughout each lift to check that the specified compaction is being achieved by the contractor. The structure or reinforced fill should be placed (in loose lifts less than 8-inches-thick) on a prepared grade as specified above.

Table 5. Engineered Fill Placement Specifications

Location	Compaction Recommendation (ASTM D1557)	Moisture Content (Percent Optimum)
Beneath Floor Slab	90%	**-1 to +3 percent**
Supporting Footings	90%	**-1 to +3 percent**
Within 5-feet beyond the perimeter of the building pad	90%	**-1 to +3 percent**
Roadway fill placed within 2-feet of the base of the Pavement	95%	**-1 to +3 percent**
Utility trenches within building and pavement areas	95%	**-1 to +3 percent**
Utility trenches beneath landscape areas	90%	**-1 to +3 percent**

5.5.5 Site Drainage

Final site grading should provide for surface drainage away from structures and foundations; minimum compaction of disturbed soils may be required to reduce the amount of percolation of

water into the underlying soils. Ponding of surface water should not be allowed adjacent to the structure. Grades should be sloped away from the structure at a minimum gradient of five percent in landscaped areas, and two percent in paved areas for a horizontal distance of at least 10-feet. Rainwater collected at the building roof levels should generally be transported via gutters, downspouts, and tightlines that discharge onto pavement areas or directly into the site storm water system.

5.5.6 Pavement Design

The proposed project includes the construction of new parking areas, access roadways, and the extension of Tydd Street. Where planned, pavement structural sections should be designed to withstand the anticipated traffic loads over the tested (or conservatively assumed) supporting subgrade strength for the design life of the development. A flexible pavement system may be used for this site consisting of AC placed over compacted Caltrans Class 2 Aggregate Base (AB), which in turn rests on a properly prepared subgrade. Previous Resistance (R-) Value tests conducted on soil samples for other projects (but similar to the shallow soils encountered at the proposed development) lead us to conclude that the anticipated subgrade soils have an R-Value of between 25 and 40 (at 300 exudation pressure). Our asphalt concrete pavement thickness recommendations presented herein are based on the assumption that the pavement subgrade soils will consist of the onsite, untreated native soils. For these soils, we assumed an R-Value of 25 (minimum). We selected a Traffic Index (T.I.) range of 4 to 8. The Caltrans Flexible Pavement Design Method was used to provide the recommended pavement sections presented below. These pavement section thicknesses and corresponding T.I.s should be checked by the project Civil Engineer for their applicability prior to final design and use.

ASPHALT CONCRETE PAVEMENT SECTION ALTERNATIVES R-VALUE = 25		
	Pavement Section (inches)*	
Traffic Index	AC	AB
4	2.0	6.0
4.5	2.5	6.0
5	2.5	7.0
5.5	3.0	8.0
6	3.0	9.5
6.5	3.5	10.0
* AC = Type B Asphalt Concrete AB = Class 2 Aggregate Base (Minimum R-Value = 78)		

5.5.7 Pavement Subgrade Preparation

On the basis of our field observations at our field exploration locations, and those provided within the Whitchurch Engineering Report, we estimate that at up to 1.5-feet of native

topsoil/undocumented fill material will be required to be removed to reach a suitable grade for placement and compaction of new fill, and/or compaction of planned pavement subgrade.

Compaction standards for pavement sections should conform to Caltrans Test Methods Cal 216 and 231 with a minimum relative compaction of 95 percent within the top 6-inches of subgrade. Pavement subgrade should be visually inspected to check its suitability, i.e., proof-rolled with heavy-duty earthmoving equipment to check that a firm and unyielding condition is observed and approved by the Geotechnical Engineer prior to covering with completed construction. Pumping areas or soft spots may have to be over-excavated and replaced with properly moisture-conditioned fill material that is placed, compacted, and tested as recommended in this report.

For convenience, compaction testing may be performed using ASTM methods in lieu of Caltrans methods provided the specified relative compaction noted in the preceding paragraph is adhered to.

6.0 CONSULTATION, OBSERVATION, AND TESTING

To check for conformance with the specific recommendations contained within this report, and that assumptions made in the preparation of this report are valid, LACO should be retained for the following:

- Monitor site grading and inspect exposed grades prior to placement of engineered fills.
- Inspect foundation excavations prior to placement of any forms or reinforcing steel.
- Monitor the placement of engineered fill.
- Test all engineered fill to check that the required relative compaction is achieved.

7.0 REFERENCES

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LACO ASSOCIATES CONSULTING ENGINEERS 21 W 4TH ST. EUREKA, CA 95501 (707)443-5054	PROJECT	GEOTECHNICAL INVESTIGATION	BY	JB	FIGURE
	CLIENT	OPEN DOOR COMMUNITY HEALTH CENTER	DATE	1/10/11	1
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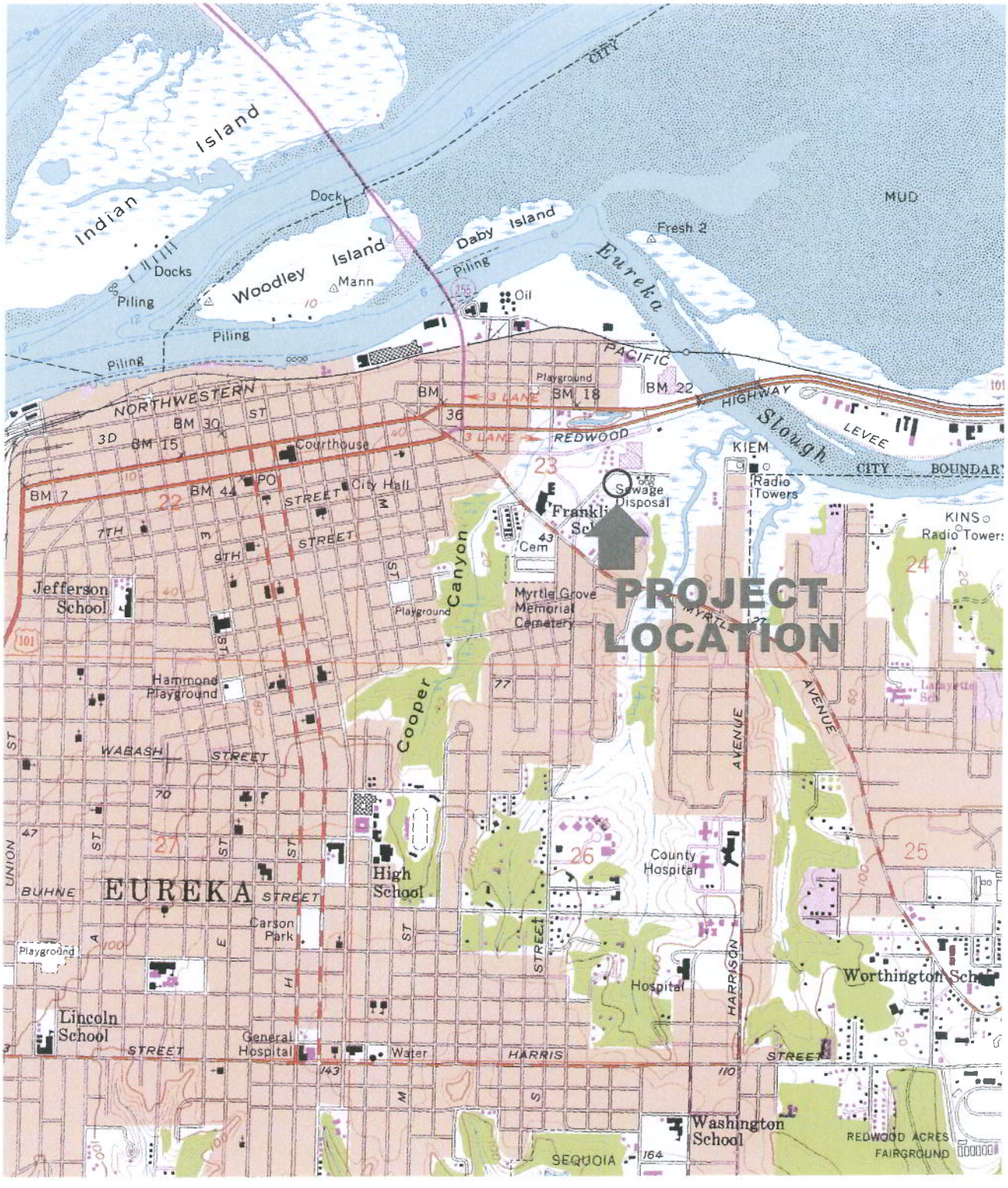
		LOCATION MAP	SCALE	AS SHOWN	7119.02



LOCATION MAP
1"=2,000'

0 1000' 2000'

SCALE: 1"=2000'

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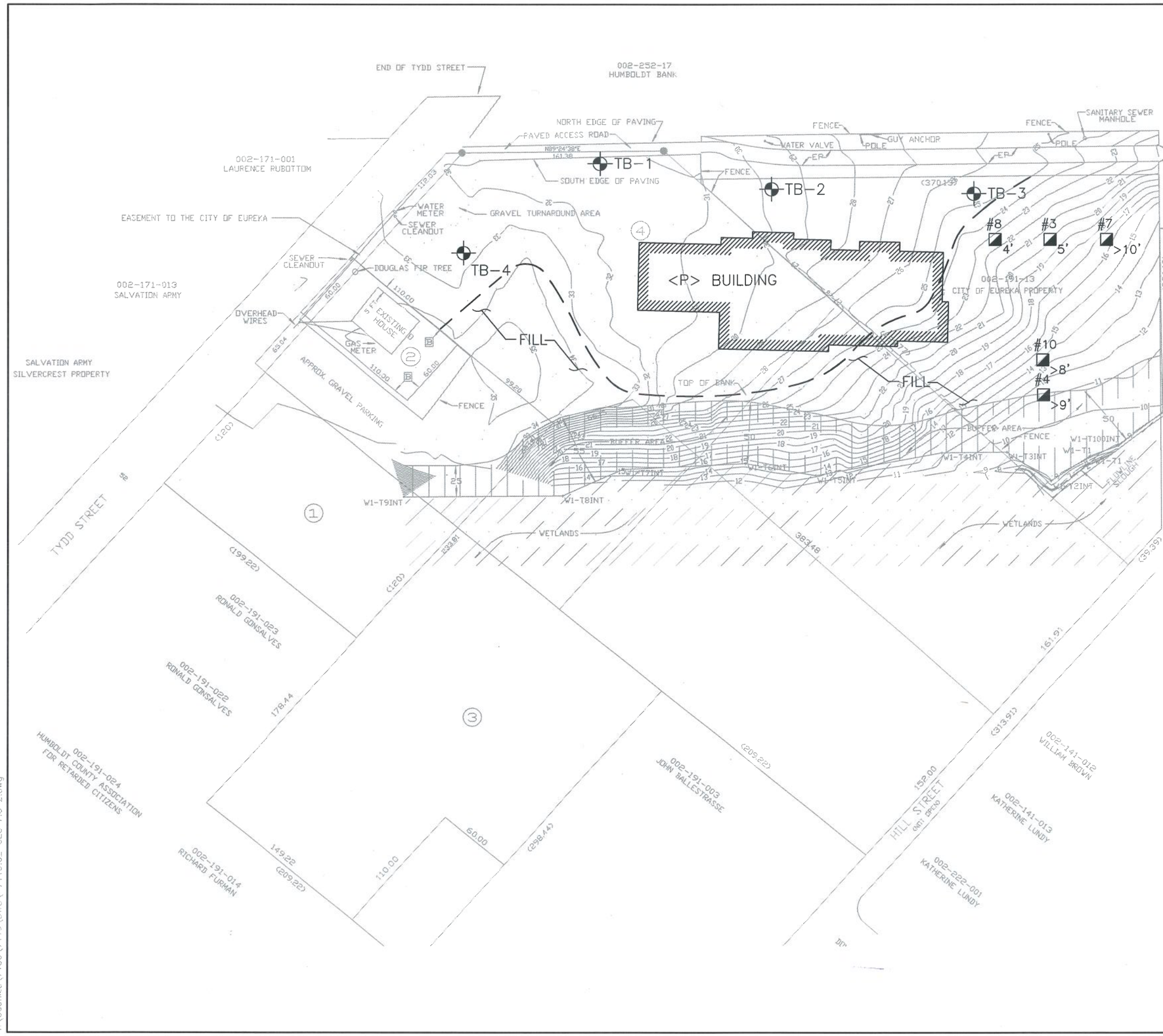

LEGEND

- #3 5' — TEST PIT LOCATION WITH FILL THICKNESS (WHITCHURCH, 2006)
- TEST BORING LOCATION (LACO, 2011)
- <P> BUILDING (LOCATION APPROXIMATE)
- APPROXIMATE LIMIT OF FILL SOILS
- 30 — <E> CONTOUR IN FEET (ABOVE MEAN SEA LEVEL APPROX.)
- <P> PROPOSED
- TB TEST BORING

NOTE:

BASE MAPPING PROVIDED BY KELLY-O'HERN ASSOCIATES, REVISION DATE, JANUARY 2005.

LACO ASSOCIATES
CONSULTING ENGINEERS
21 W 4TH ST. EUREKA, CA 95501 (707)443-5054

NO.	REVISION	BY	CHK	DATE

GEOTECHNICAL INVESTIGATION
SITE PLAN W/BORING LOCATIONS

OPEN DOOR COMMUNITY HEALTH CLINIC
TYDD STREET, EUREKA, CA.

SCALE	AS SHOWN
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FIGURE	2

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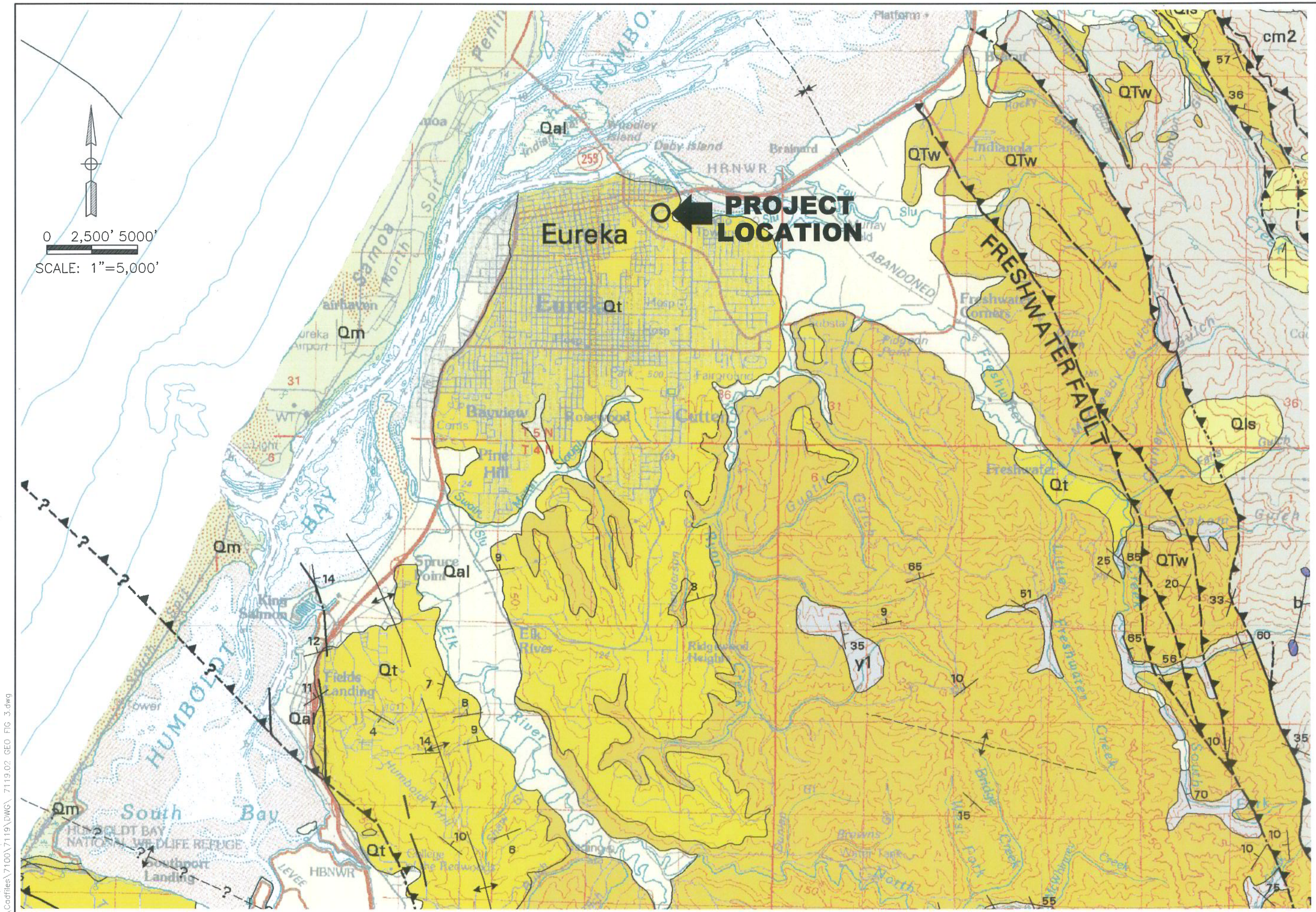
I:\Codfiles\7100\7119\DWG\ 7119.02 GEO FIG 2.dwg



LACO ASSOCIATES
CONSULTING ENGINEERS
21 W 4TH ST. EUREKA, CA 95501 (707)443-5054

NO.	REVISION	BY	CHK	DATE

GEOTECHNICAL INVESTIGATION
GEOLOGIC MAP

OPEN DOOR COMMUNITY HEALTH CLINIC
TYDD STREET, EUREKA, CA.

SCALE	AS SHOWN
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FIGURE	3A

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T:\Cadfiles\7100\7119\DWG\ 7119.02 GEO FIG 3.dwg

LACO ASSOCIATES CONSULTING ENGINEERS 21 W 4TH ST. EUREKA, CA 95501 (707)443-5054	PROJECT GEOTECHNICAL INVESTIGATION	BY JB	FIGURE 3B
	CLIENT OPEN DOOR COMMUNITY HEALTH CENTER	DATE 1/10/11	
	LOCATION TYDD STREET, EUREKA, CA.	CHECK GAV	JOB NO. 7119.02
	GEOLOGIC MAP LEGEND	SCALE N.T.S.	

DESCRIPTION OF MAP UNITS

QUATERNARY AND TERTIARY OVERLAP DEPOSITS

- **Qal** — Alluvial deposits (Holocene and late Pleistocene?)
- **Qm** — Undeformed marine shoreline and aolian deposits (Holocene and late Pleistocene)
- **Qt** — Undifferentiated nonmarine terrace deposits (Holocene and Pleistocene)
- **Qls** — Landslide deposits (Holocene and Pleistocene)
- **QTog** — Older alluvium (Pleistocene and [or] Pliocene)
- **QTw** — Marine and nonmarine overlap deposits (late Pleistocene to middle Miocene)
- **Ti** — Volcanic rocks of Fickle Hill (Oligocene)

COAST RANGES PROVINCE

FRANCISCAN COMPLEX

-- Coastal Belt --

Coastal terrane (Pliocene to Late Cretaceous)

Sedimentary, igneous, and metamorphic rocks of the Coastal terrane (Pliocene to Late Cretaceous):

- **co1** — Melange
- **co2** — Melange
- **co3** — Broken sandstone and argillite
- **co4** — Intact sandstone and argillite
- **cob** — Basaltic Rocks (Late Cretaceous)
- **cols** — Limestone (Late Cretaceous)
- **m** — Undivided blueschist (Jurassic?)

King Range terrane (Miocene to Late Cretaceous)

- **Krp** — Igneous and sedimentary rocks of Point Delgada (Late Cretaceous)
- **m** — Undivided blueschist blocks (Jurassic?)

Sandstone and argillite of King Peak (middle Miocene to Paleocene[?]):

- **krk1** — Melange and (or) folded argillite
- **krk2** — Highly folded broken formation
- **krk3** — Highly folded, largely unbroken rocks
- **krl** — Limestone
- **krc** — Chert
- **krb** — Basalt

False Cape terrane (Miocene? to Oligocene?)

- **fc** — Sedimentary rocks of the False Cape terrane (Miocene? to Oligocene?)

Yager terrane (Eocene to Paleocene?)

Sedimentary rocks of the Yager terrane (Eocene to Paleocene?):

- **y1** — Sheared and highly folded mudstone
- **y2** — Highly folded broken mudstone, sandstone, and conglomeratic sandstone
- **y3** — Highly folded, little-broken sandstone, conglomerate, and mudstone
- **Ycgl** — Conglomerate

-- Central belt --

Melange of the Central belt (early Tertiary to Late Cretaceous):

Unnamed Metasandstone and meta-argillite (Late Cretaceous to Late Jurassic):

- **cm1** — Melange
- **cm2** — Melange
- **cb1** — Broken formation
- **cb2** — Broken formation
- **cwr** — White Rock metasandstone of Jayko and others (1989) (Paleogene and [or] Late Cretaceous)
- **chr** — Harman Ridge graywacke of Jayko and others (1989) (Cretaceous?)
- **cfs** — Fort Seward metasandstone (age unknown)
- **cls** — Limestone (Late to Early Cretaceous)
- **cc** — Chert (Late Cretaceous to Early Jurassic)
- **bs** — Basaltic rocks (Cretaceous and Jurassic)
- **m** — Undivided blueschist blocks (Jurassic?)
- **gs** — Greenstone
- **c** — Metachert
- **yb** — Metasandstone of Yolla Bolly terrane, undivided
- **b** — Melange block, lithology unknown

-- Eastern Belt --

Pickett Peak terrane (Early Cretaceous or older)

Metasedimentary and metavolcanic rocks of the Pickett Peak terrane (Early Cretaceous or older):

- **ppsm** — South Fork Mountain Schist
- **mb** — Chinquapin Metabasalt Member (Irwin and others, 1974)
- **ppv** — Valentine Springs Formation
- **mv** — Metabasalt and minor metachert

Yolla Bolly terrane (Early Cretaceous to Middle Jurassic?)

Metasedimentary and metaigneous rocks of the Yolla Bolly terrane (Early Cretaceous to Middle Jurassic?):

- **ybt** — Taliaferro Metamorphic Complex of Suppe and Armstrong (1972) (Early Cretaceous to Middle Jurassic?)
- **ybc** — Chicago Rock melange of Blake and Jayko (1983) (Early Cretaceous to Middle Jurassic)
- **gs** — Greenstone
- **c** — Metachert
- **ybh** — Metagraywacke of Hammerhorn Ridge (Late Jurassic to Middle Jurassic)
- **c** — Metachert
- **gs** — Greenstone
- **sp** — Serpentinite
- **ybd** — Devils Hole Ridge broken formation of Blake and Jayko (1983) (Early Cretaceous to Middle Jurassic)
- **c** — Radiolarian chert
- **ybi** — Little Indian Valley argillite of McLaughlin and Ohlin (1984) (Early Cretaceous to Late Jurassic)

Yolla Bolly terrane

- **yb** — Rocks of the Yolla Bolly terrane, undivided

GREAT VALLEY SEQUENCE AND COAST RANGE OPHIOLITE

Elder Creek(?) terrane

- **ecms** — Mudstone (Early Cretaceous)

Coast Range ophiolite (Middle and Late Jurassic):

- **ecg** — Layered gabbro
- **ecsp** — Serpentinite melange

Del Puerto(?) terrane

Rocks of the Del Puerto(?) terrane:

- **dpms** — Mudstone (Late Jurassic)

Coast Range ophiolite (Middle and Late Jurassic):

- **dpt** — Tuffaceous chert (Late Jurassic)
- **dpb** — Basaltic flows and keratophyric tuff (Jurassic?)
- **dpd** — Diabase (Jurassic?)
- **dpsp** — Serpentinite melange (Jurassic?)
- **sp** — Undivided Serpentinized peridotite (Jurassic?)

KLAMATH MOUNTAINS PROVINCE

Undivided Great Valley Sequence:

- **Ks** — Sedimentary rocks (Lower Cretaceous)

GREAT VALLEY SEQUENCE OVERLAP ASSEMBLAGE

Hayfork terrane

Eastern Hayfork subterrane:

- **eh** — Melange and broken formation (early? Middle Jurassic)
- **ehls** — Limestone
- **ehsp** — Serpentinite

Western Hayfork subterrane:

- **whu** — Hayfork Bally Meta-andesite of Irwin (1985), undivided (Middle Jurassic)
- **whwg** — Wildwood (Chanchefulla Peak of Wright and Fahan, 1988) pluton (Middle Jurassic)
- **whwp** — Clinopyroxenite
- **whjl** — Diorite and gabbro plutons (Middle? Jurassic)

Rattlesnake Creek terrane

- **rcm** — Melange (Jurassic and older)
- **rcls** — Limestone
- **rcc** — Radiolarian chert
- **rcls** — Volcanic Rocks (Jurassic or Triassic)
- **rcic** — Intrusive complex (Early Jurassic or Late Triassic)
- **rcp** — Plutonic rocks (Early Jurassic or Late Triassic)
- **rcum** — Ultramafic rocks (age uncertain)
- **rcpd** — Blocky peridotite

Western Klamath terrane

Smith River subterrane:

- **srs** — Galice? formation (Late Jurassic)
- **srv** — Pyroclastic andesite
- **srgb** — Glen Creek gabbro-ultramafic complex of Irwin and others (1974)
- **srpd** — Serpentinized peridotite

MAP SYMBOLS

- Contact
- Fault
- Thrust fault
- Trace of the San Andreas fault associated with 1906 earthquake rupture

Strike and dip of bedding:

- Inclined
- Vertical
- Horizontal
- Overturned
- Approximate
- Joint
- Strike and dip of cleavage

Shear foliation:

- Inclined
- Vertical

Folds:

- Synclinal or synformal axis
- Anticlinal or antiformal axis
- Overturned syncline
- Landslide

Melange Blocks:

- Serpentinite
- Chert
- Blueschist
- Greenstone
- Fossil locality and number

GEOLOGY OF THE CAPE MENDOCINO, EUREKA, GARBERVILLE, AND SOUTHWESTERN PART OF THE HAYFORK 30 X 60 MINUTE QUADRANGLES AND ADJACENT OFFSHORE AREA, NORTHERN CALIFORNIA (McLaughlin et al., 2000)

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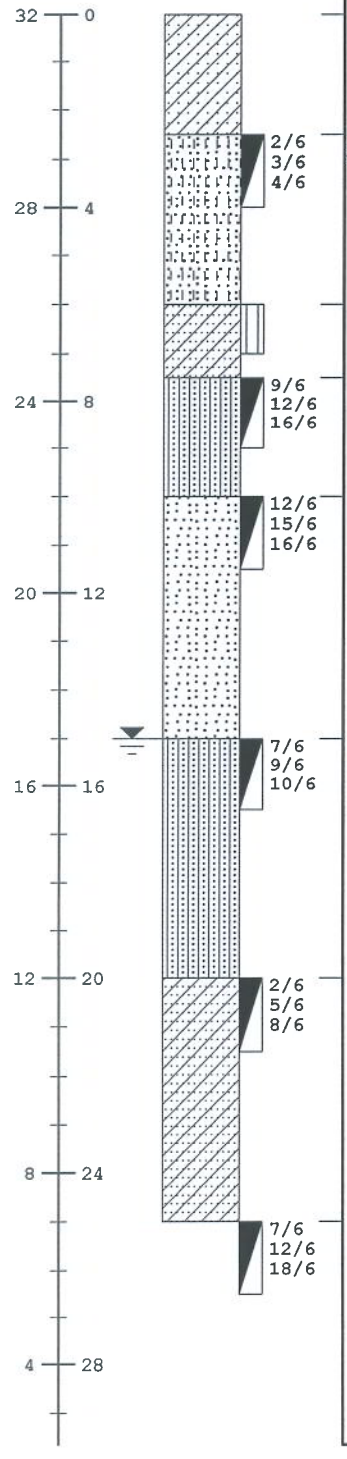
APPENDIX A

Soil Profile Logs

GEOTECHNICAL BORING LOG

Boring No. TB-1

PROJECT: GEOTECHNICAL INVESTIGATION
PROJECT NO.: 7119.02
BORING LOCATION: NW CORNER OF SITE
DATE: 12-20-10
DRILLING METHOD: 6-5/8-INCH OD RHSA
ELEVATION: 32 FEET MSL (APPROX)
DRILLER: CLEAR HEART DRILLING
LOGGED BY: GAV
DEPTH TO WATER: **INITIAL** ▽ : 15 FEET **COMPLETION** ▼ : 15 FEET
SITE GEOLOGY: QUATERNARY HOOKTON FORMATION



ELEVATION/ DEPTH	SOIL SYMBOLS, SAMPLERS AND TEST DATA	USCS	Description	Water Content %	Dry Density pcf	STANDARD PENETRATION TEST DEPTH	STANDARD PENETRATION TEST N
		CL	SANDY LEAN CLAY, 10YR 4/3 Brown, wet, soft, medium plasticity, low toughness, weak cementation.				
	2/6 3/6 4/6	SP-SM	POORLY GRADED SAND w/SILT, 10YR 4/4 Dark Yellowish Brown, wet, loose, non-plastic, low dry strength, weak cementation, sand fine to medium, few fine roots. <#200=13.37%			2.5-4	7
		SC	CLAYEY SAND, moist, medium plasticity, medium toughness, weak cementation. <#200=13.32%	26.1	92.0		
	9/6 12/6 16/6	SM	SILTY SAND, 7.5YR 4/6 Strong Brown, moist, medium dense, non-plastic, medium dry strength, medium cementation, strongly oxidized.			7.5-9	28
	12/6 15/6 16/6	SP	<#200=17.63%			10-11.5	31
			POORLY GRADED SAND, 2.5Y 4/4 Olive Brown, dry to moist, dense, non-plastic, medium dry strength, weak to medium cementation, sand fine to medium.				
	7/6 9/6 10/6	SM	SILTY SAND, 2.5Y 5/4 Light Olive Brown, wet, medium dense, non-plastic, medium dry strength, medium cementation, sand fine.			15-16.5	19
	2/6 5/6 8/6	SC	CLAYEY SAND, 10YR 6/8 Brownish Yellow, wet, medium dense, medium plasticity, medium to high dry strength, strong cementation, sand medium, slightly oxidized.			20.0-21.5	13
	7/6 12/6 18/6	SP	POORLY GRADED SAND, 2.5y 5/4 Light Olive Brown, wet, dense, non-plastic, medium to high dry strength, medium to strong cementation, moderately oxidized.			25.0-26.5	30

STANDARD PENETRATION TEST CURVE

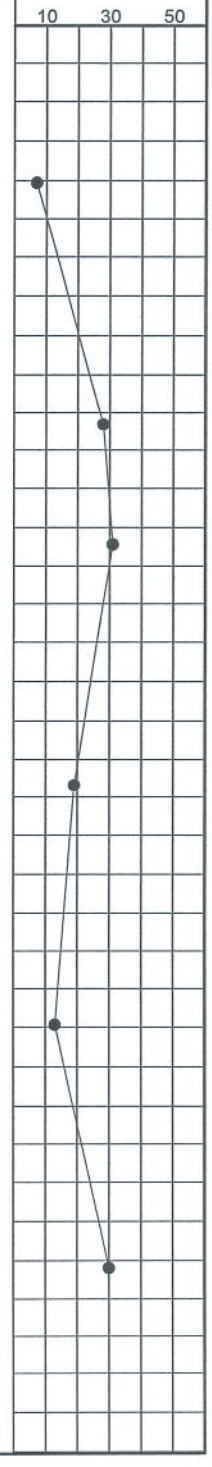
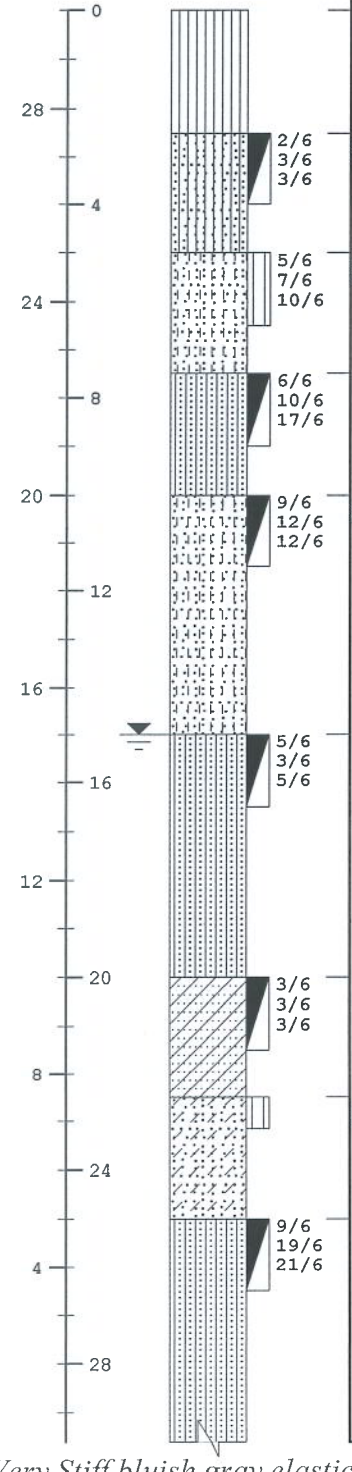


Figure 1

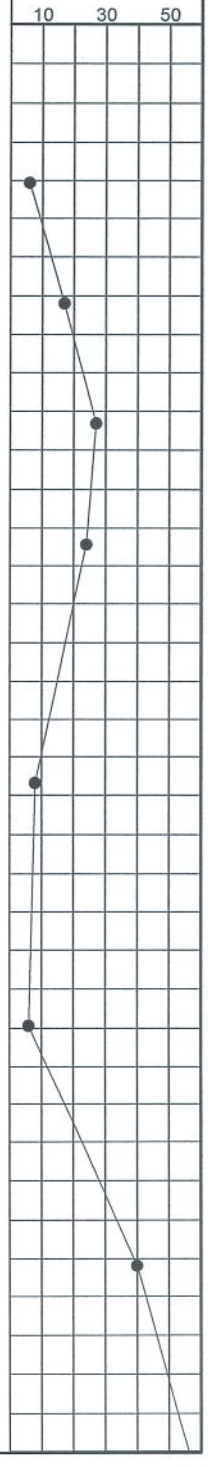
GEOTECHNICAL BORING LOG

Boring No. TB-2

PROJECT: GEOTECHNICAL INVESTIGATION
PROJECT NO.: 7119.02
BORING LOCATION: NORTH CENTER EDGE OF SITE
DATE: 12-20-10
DRILLING METHOD: 6-5/8-INCH OD RHSA
ELEVATION: 30 FEET MSL (APPROX)
DRILLER: CLEAR HEART DRILLING
LOGGED BY: GAV
DEPTH TO WATER: **INITIAL** ▽ : 15 FEET **COMPLETION** ▼ : 15 FEET
SITE GEOLOGY: QUATERNARY HOOKTON FORMATION



ELEVATION/ DEPTH	SOIL SYMBOLS, SAMPLERS AND TEST DATA	USCS	Description	Water Content %	Dry Density pcf	STANDARD PENETRATION TEST DEPTH	N
		ML	SILT, 10YR 3/3 Dark Brown, wet, soft, low plasticity, low toughness, non-cemented.				
	2/6 3/6 3/6	SM-ML	SILT grading to SILTY SAND, 10YR 5/6 Yellowish Brown, moist, loose, low plasticity, medium dry strength, weak cementation, sand fine.			2.5-4	6
	5/6 7/6 10/6	SP-SM	#200=25.78% POORLY GRADED SAND w/SILT, 10YR 4/6 Dark Yellowish Brown,, moist, medium dense, non-plastic, medium dry strength, weak cementation, sand fine to medium.			5.0-6.5	17
	6/6 10/6 17/6	SM	<#200=12.51% SILTY SAND, 2.5Y 4/4 Olive Brown, moist, medium dense, low plasticity, medium dry strength, medium cementation, sand medium, trace amount of clay.			7.5-9	27
	9/6 12/6 12/6	SP-SM	POORLY GRADED SAND w/SILT, 2.5Y 5/4 Light Olive Brown, dry, medium dense, non-plastic, high dry strength, medium cementation, manganese-oxide nodules. <#200=9.95%			10-11.5	24
	5/6 3/6 5/6	SM	SILTY SAND, 2.5Y 5/4 Light Olive Brown, moist to wet, loose, low plasticity, medium dry strength, medium cementation, weakly oxidized, sand fine, trace amount of clay. <#200=18.08%			15-16.5	8
	3/6 3/6 3/6	SC	CLAYEY SAND, 5Y 7/2 Light Gray, wet, loose, medium plasticity, high dry strength, medium cementation, sand fine to medium. <#200=19.96%			20.0-21.5	6
		SP-SC	Dense POORLY GRADED SAND to stiff CLAYEY SAND.	21.1	108.4		
	9/6 19/6 21/6	SM	SILTY SAND, 2.5Y 4/4 Olive Brown, wet, dense, non-plastic, medium dry strength, weak cementation, sand fine.			25.0-26.5	40



Very Stiff bluish gray elastic silt observerd on lead auger after pulling out.

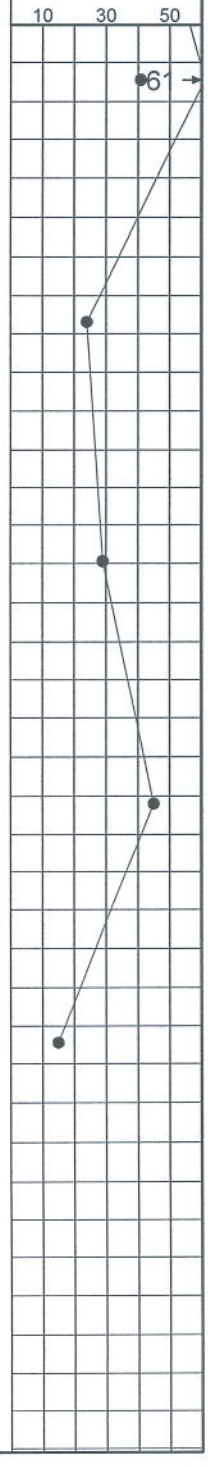
Figure 2

GEOTECHNICAL BORING LOG

Boring No. TB-2

PROJECT: GEOTECHNICAL INVESTIGATION
PROJECT NO.: 7119.02
BORING LOCATION: NORTH CENTER EDGE OF SITE
DATE: 12-20-10
DRILLING METHOD: 6-5/8-INCH OD RHSA
ELEVATION: 30 FEET MSL (APPROX)
DRILLER: CLEAR HEART DRILLING
LOGGED BY GAV
DEPTH TO WATER: INITIAL ▽ : 15 FEET **COMPLETION** ▼ : 15 FEET
SITE GEOLOGY: QUATERNARY HOOKTON FORMATION

ELEVATION/ DEPTH	SOIL SYMBOLS, SAMPLERS AND TEST DATA	USCS	Description	Water Content %	Dry Density pcf	STANDARD PENETRATION TEST DEPTH	N
0 / 32	13/6 24/6 37/6	SP-SM	POORLY GRADED SAND w/SILT, 2.5Y 5/4 Light Olive Brown, wet, very dense, non-plastic, medium dry strength, weak cementation, sand fine to medium.			30.0-31.5	61
-4 / 36	11/6 12/6 12/6	SP	POORLY GRADED SAND, 2.5Y 4/3 Olive Brown, wet, medium dense, non-plastic, medium dry strength, weak cementation, weakly oxidized, sand medium.			35.0-36.5	24
-8 / 40	5/6 11/6 18/6	SM	SILTY SAND, 2.5Y 4/4 Olive Brown, wet, medium dense, non-plastic, high dry strength, medium cementation, strongly oxidized, sand fine to medium.			40.0-41.5	29
-12 / 44	17/6 18/6 27/6	SP-SM	POORLY GRADED SAND w/SILT, 2.5Y 4/4 Olive Brown, wet, dense, non-plastic, medium dry strength, weak cementation with strongly cemented layer in cutting shoe, weakly oxidized, sand medium.			45.0-46.5	45
-16 / 48	5/6 4/6 11/6	SC	CLAYEY SAND, 5Y 6/3 Pale Olive, wet, medium dense, medium plasticity, high dry strength, medium cementation, moderately oxidized, sand fine to medium.			50.0-51.5	15
-20 / 52							
-24 / 56							
-28							



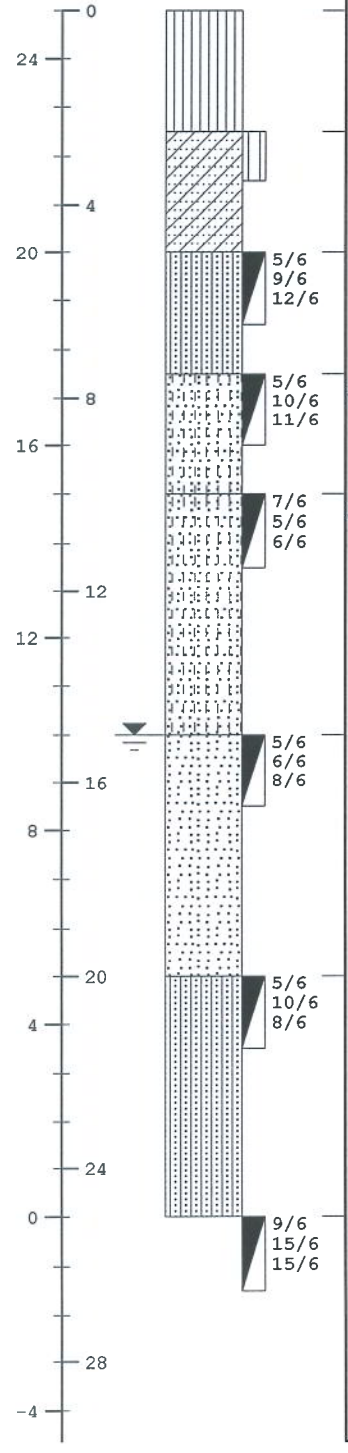
Very Stiff bluish gray elastic silt observerd on lead auger after pulling out.

Figure 2

GEOTECHNICAL BORING LOG

Boring No. TB-3

PROJECT: GEOTECHNICAL INVESTIGATION
PROJECT NO.: 7119.02
BORING LOCATION: NE CORNER OF SITE
DATE: 12-20-10
DRILLING METHOD: 6-5/8-INCH OD RHSA
ELEVATION: 25 FEET MSL (APPROX)
DRILLER: CLEAR HEART DRILLING
LOGGED BY: GAV
DEPTH TO WATER: **INITIAL** ▽ : 15 FEET **COMPLETION** ▼ : 15 FEET
SITE GEOLOGY: QUATERNARY HOOKTON FORMATION



ELEVATION/ DEPTH	SOIL SYMBOLS, SAMPLERS AND TEST DATA	USCS	Description	Water Content %	Dry Density pcf	STANDARD PENETRATION TEST DEPTH	STANDARD PENETRATION TEST N
		ML	SILT, 10YR 3/3 Dark Brown, wet, soft, low plasticity, low toughness, non-cemented.				
		SC	CLAYEY SAND, 10YR 4/4 Dark Yellowish Brown, wet, loose, medium plasticity, low dry strength, weak cementation, sand fine to medium.	21.1	105.2		
	5/6 9/6 12/6	SM	<#200=25.69% phi=34 degrees cohesion=490 psf			5.0-6.5	21
	5/6 10/6 11/6	SP-SM	SILTY SAND, 7.5YR 4/6 Strong Brown, moist, medium dense, non-plastic, medium dry strength, medium cementation.			7.5-9	21
	7/6 5/6 6/6	SP-SM	POORLY GRADED SAND w/SILT, 7.5YR 4/6 Strong Brown, moist, medium dense, low plasticity, medium dry strength, medium cementation.			10-11.5	11
			SAME, color change to 2.5Y 4/4 Olive Brown, moist, medium dense, non-plastic.				
	5/6 6/6 8/6	SP	POORLY GRADED SAND, 2.5Y 5/4 Light Olive Brown, wet, medium dense, non-plastic, medium dry strength, medium cementation, sand fine to medium.			15-16.5	14
	5/6 10/6 8/6	SM	SILTY SAND, 2.5Y 5/4 Light Olive Brown, wet, medium dense, non-plastic, medium dry strength, medium cementation, sand fine.			20.0-21.5	18
	9/6 15/6 15/6	SM	SAME, wet, dense, non-plastic.			25.0-26.5	30

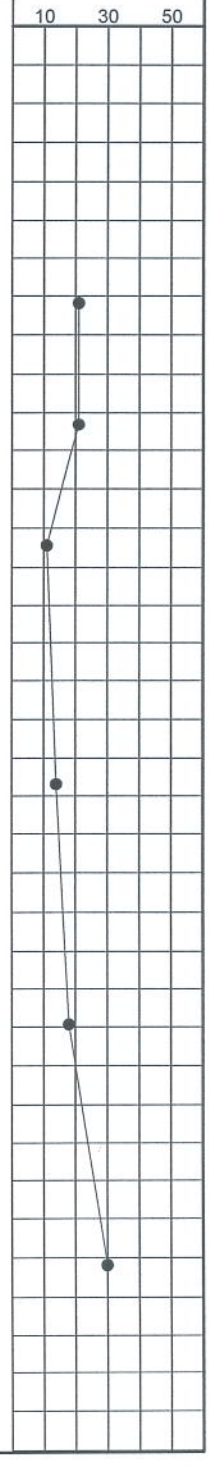
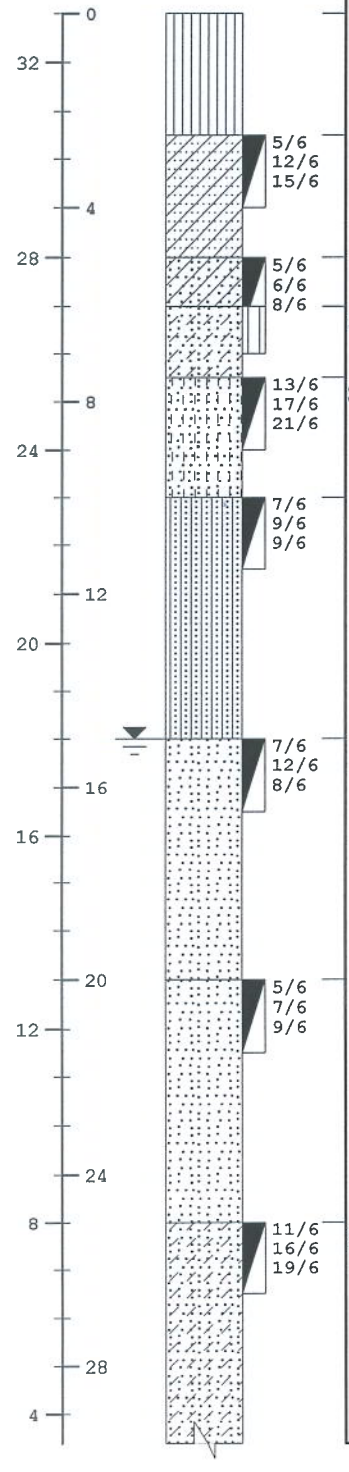


Figure 3

GEOTECHNICAL BORING LOG

Boring No. TB-4

PROJECT: GEOTECHNICAL INVESTIGATION
PROJECT NO.: 7119.02
BORING LOCATION: WEST EDGE OF SITE
DATE: 12-20-10
DRILLING METHOD: 6-5/8-INCH OD RHSA
ELEVATION: 33 FEET MSL (APPROX)
DRILLER: CLEAR HEART DRILLING
LOGGED BY: GAV
DEPTH TO WATER: **INITIAL** ▽ : 15 FEET **COMPLETION** ▼ : 15 FEET
SITE GEOLOGY: QUATERNARY HOOKTON FORMATION



ELEVATION/ DEPTH	SOIL SYMBOLS, SAMPLERS AND TEST DATA	USCS	Description	Water Content %	Dry Density pcf	STANDARD PENETRATION TEST DEPTH	N
		ML	SILT, 10YR 3/3 Dark Brown, wet, soft, low plasticity, low toughness, non-cemented.				
	5/6 12/6 15/6	SC	CLAYEY SAND, 10YR 5/6 Yellowish Brown, moist, medium dense, low plasticity, medium dry strength, weak cementation, sand fine.			2.5-4	27
	5/6 6/6 8/6	CL	SANDY LEAN CLAY, 10YR 4/6 Dark Yellowish Brown, moist, stiff, medium plasticity, medium dry strength, weak cementation, sand fine.			5.0-6.5	14
		SP-SC		19	107.3		
	13/6 17/6 21/6	SP-SM	POORLY GRADED SAND w/CLAY, moist, medium plasticity, medium toughness, weak cementation.			7.5-9	38
	7/6 9/6 9/6	SM	<#200=13.32% phi=38 degrees cohesion=609 psf			10-11.5	18
			POORLY GRADED SAND w/SILT, 2.5Y 5/4 Light Olive Brown, dry, dense, non-plastic, high dry strength, medium cementation.				
	7/6 12/6 8/6	SP	SILTY SAND, 2.5Y 5/4 Light Olive Brown, dry, medium dense, non-plastic, high dry strength, medium cementation.			15-16.5	20
			POORLY GRADED SAND, 2.5Y 5/4 Light Olive Brown, wet, medium dense, non-plastic, medium dry strength, medium cementation, weakly oxidized, sand fine to medium.				
	5/6 7/6 9/6	SP	SAME, wet, medium dense, trace amount of clay.			20.0-21.5	16
	11/6 16/6 19/6	SP-SC	POORLY GRADED SAND w/CLAY, 2.5Y 4/4 Olive Brown, wet, dense, low plasticity, medium dry strength, weak cementation, sand fine to medium.			25.0-26.5	35

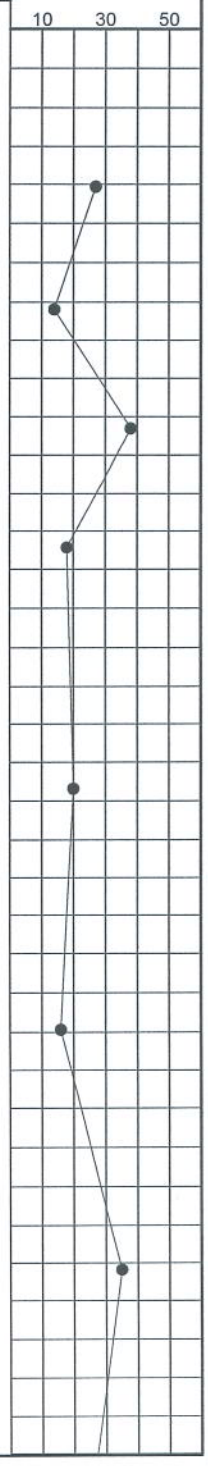
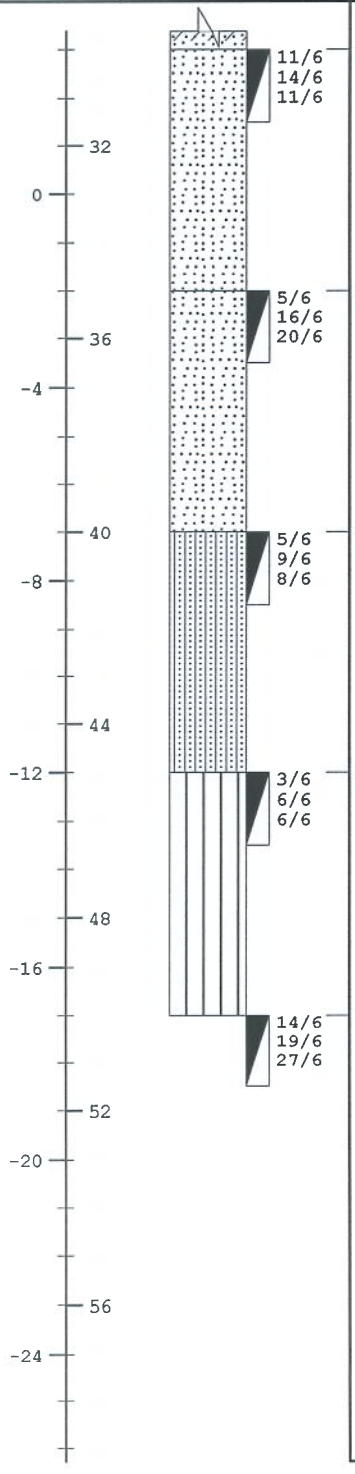


Figure 4

GEOTECHNICAL BORING LOG

Boring No. TB-4

PROJECT: GEOTECHNICAL INVESTIGATION
PROJECT NO.: 7119.02
BORING LOCATION: WEST EDGE OF SITE
DATE: 12-20-10
DRILLING METHOD: 6-5/8-INCH OD RHSA
ELEVATION: 33 FEET MSL (APPROX)
DRILLER: CLEAR HEART DRILLING
LOGGED BY: GAV
DEPTH TO WATER: **INITIAL** ▽ : 15 FEET **COMPLETION** ▼ : 15 FEET
SITE GEOLOGY: QUATERNARY HOOKTON FORMATION



ELEVATION/ DEPTH	SOIL SYMBOLS, SAMPLERS AND TEST DATA	USCS	Description	Water Content %	Dry Density pcf	STANDARD PENETRATION TEST DEPTH	N
	11/6 14/6 11/6	SP	POORLY GRADED SAND, 2.5Y 5/4 Light Olive Brown, wet, medium dense, non-plastic, medium dry strength, weak cementation, sand fine to medium.			30.0-31.5	25
	5/6 16/6 20/6	SP	POORLY GRADED SAND, 2.5Y 4/3 Olive Brown, wet, dense, non-plastic, medium dry strength, weak cementation, sand medium.			35.0-36.5	36
	5/6 9/6 8/6	SM	SILTY SAND, 2.5Y 4/4 Olive Brown, wet, medium dense, non-plastic, high dry strength, medium cementation, strongly oxidized, sand fine to medium.			40.0-41.5	17
	3/6 6/6 6/6	MH	ELASTIC SILT, Bluish Gray, wet, stiff, high plasticity, abundant shell fragments.			45.0-46.5	12
	14/6 19/6 27/6	SM	SILTY SAND, 2.5Y 4/4 Olive Brown, wet, dense, low plasticity, medium dry strength, weak cementation, sand fine to medium.			50.0-51.5	46

STANDARD PENETRATION TEST CURVE

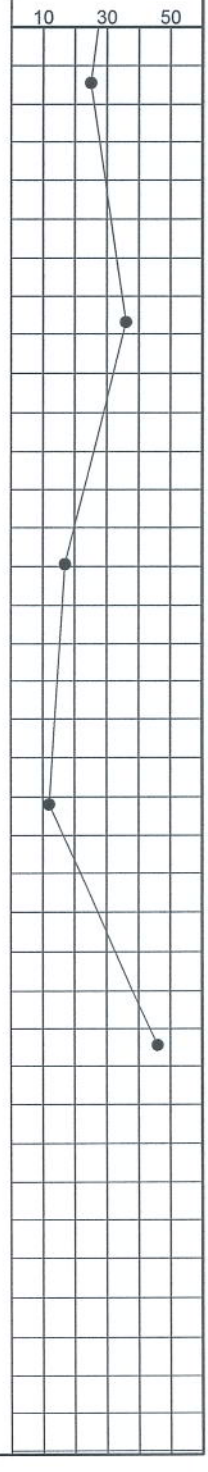


Figure 4

APPENDIX B

Laboratory Test Data

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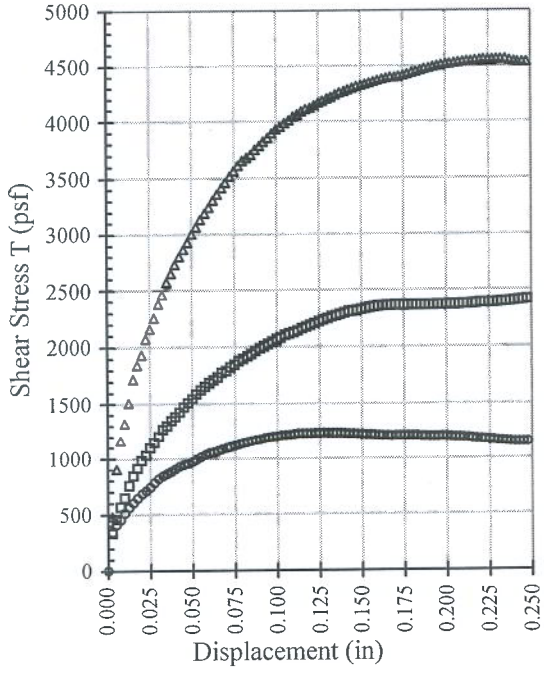
DIRECT SHEAR
ASTM D-3080

PROJECT	ODCHC		JOB NO.	7119.02	SHEET
CLIENT	ODCHC		SAMPLE ID	10-179EK	1 of 1
LOCATION	TB-1 @ 7'	TEST BY	DLR	DATE	12/29/10
SOIL TYPE	CLAYEY SAND	CHECKED BY		CHECK DATE	

Shear Stress -vs- Displacement



Sample Dimensions

Diameter (inch)	2.5
Height (inch)	1.0

	Point 1	Point 2	Point 3	Average
Water Content (%)	25.6	25.7	26.2	25.8
Dry Density (pcf)	91.4	94.2	93.0	92.9
Void Ratio*	0.8	0.8	0.8	0.8
Peak Shear Stress (psf)	1229.9	2464.5	4566.7	

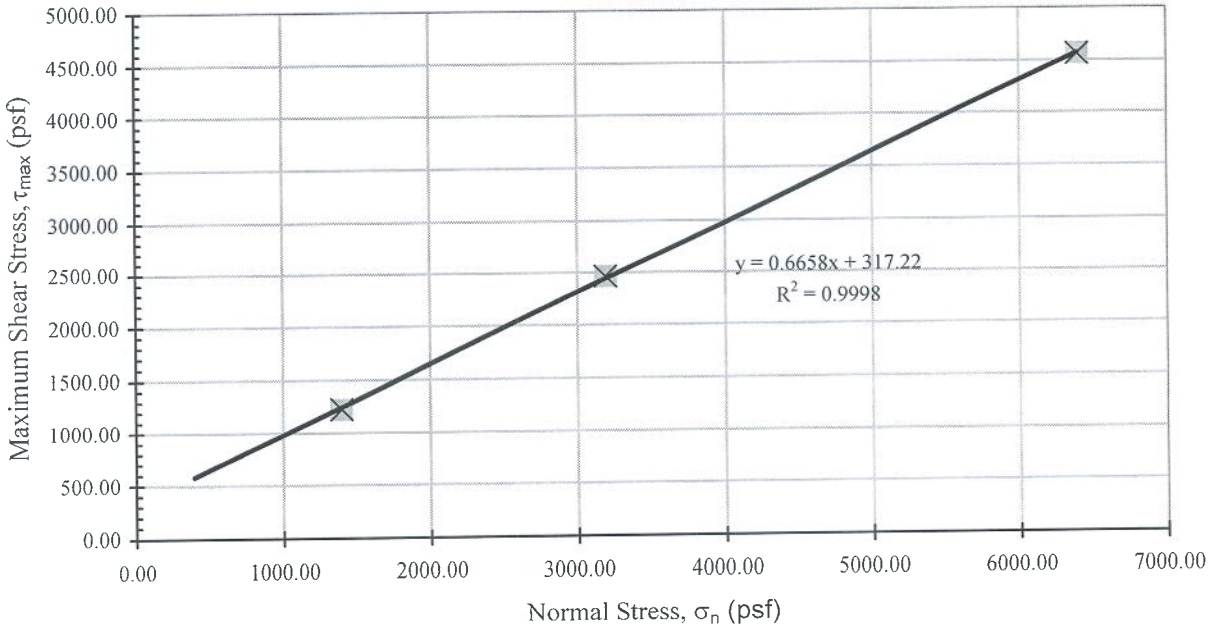
*Void ratio calculation assumes a specific gravity of 2.65

c - intercept (cohesion)	317
phi (internal friction angle)	33.7°

Notes: UNDISTURBED SAMPLE FROM 3' SHELBY

0

Maximum Shear Stress -vs- Normal Stress



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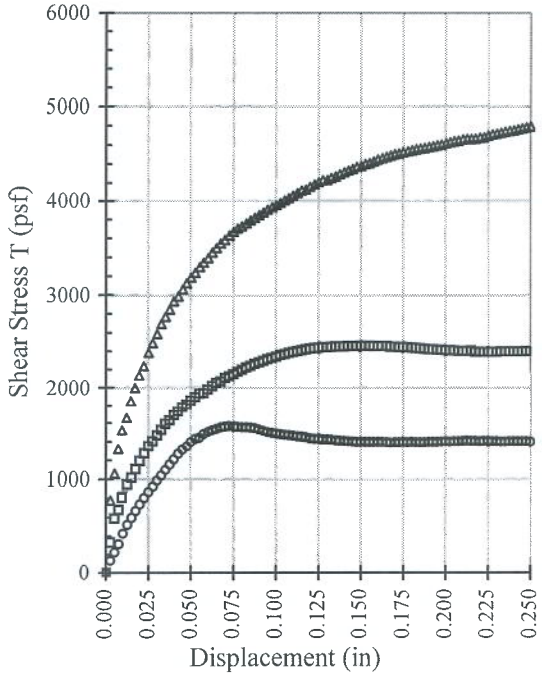
DIRECT SHEAR
ASTM D-3080

PROJECT	ODCHC		JOB NO.	7119.02	SHEET
CLIENT	ODCHC		SAMPLE ID	10-179EK	1 of 1
LOCATION	TB-3 @ 3.5'	TEST BY	DLR	DATE	1/5/11
SOIL TYPE	CLAYEY SAND	CHECKED BY		CHECK DATE	

Shear Stress -vs- Displacement



Sample Dimensions

Diameter (inch)	2.5
Height (inch)	1.0

	Point 1	Point 2	Point 3	Average
Water Content (%)	18.6	19.2	18.6	18.8
Dry Density (pcf)	101.2	102.8	102.8	102.3
Void Ratio*	0.6	0.6	0.6	0.6
Peak Shear Stress (psf)	1564.5	2455.3	4864.6	

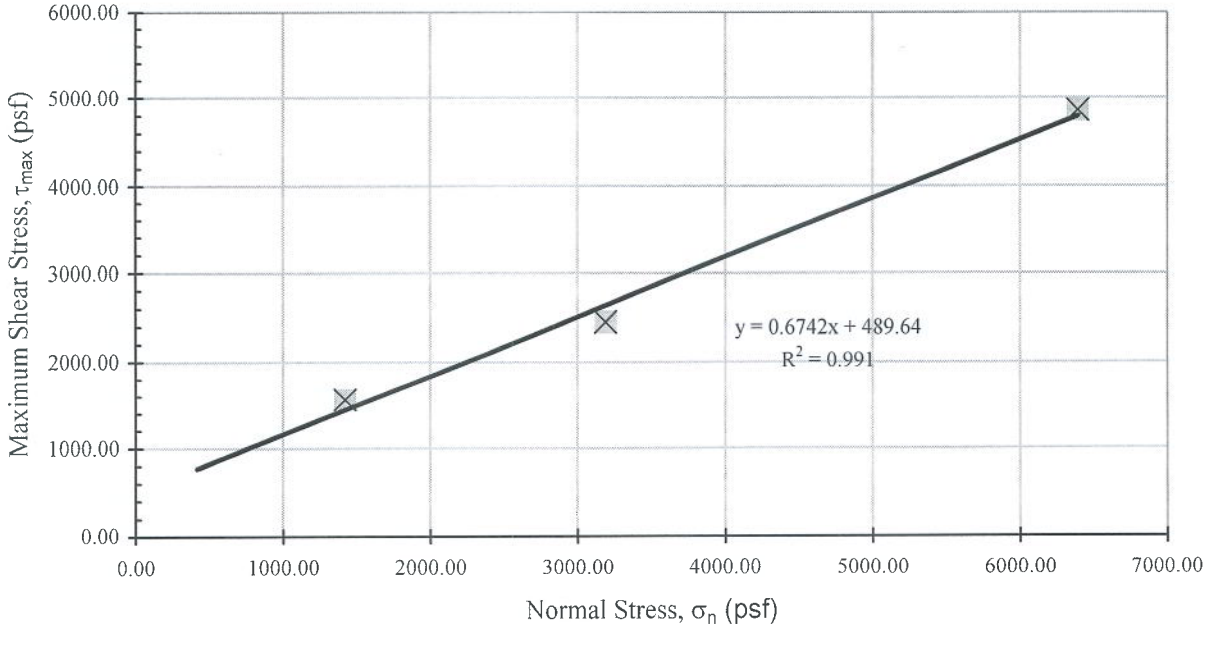
*Void ratio calculation assumes a specific gravity of 2.65

c - intercept (cohesion)	490
phi (internal friction angle)	34°

Notes: UNDISTURBED SAMPLE FROM 3' SHELBY

0

Maximum Shear Stress -vs- Normal Stress



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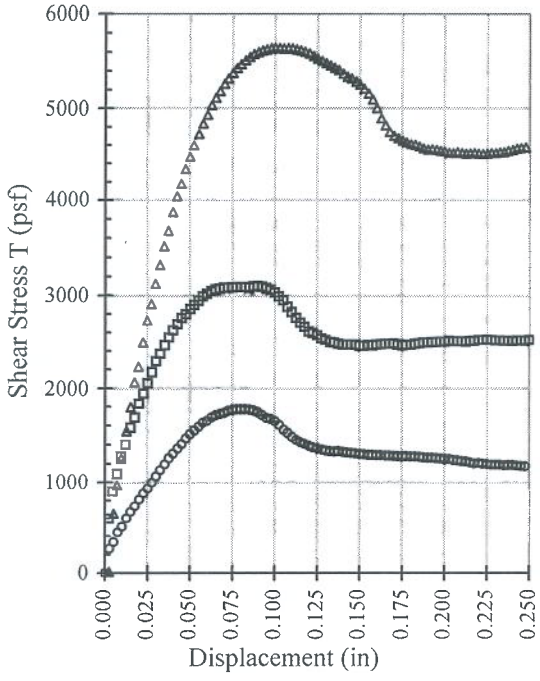
DIRECT SHEAR
ASTM D-3080

PROJECT	ODCHC		JOB NO.	7119.02	SHEET
CLIENT	ODCHC		SAMPLE ID	10-179EK	1 of 1
LOCATION	TB-4 @ 7.5'	TEST BY	DLR	DATE	12/29/10
SOIL TYPE	POORLY GRADED SAND w/CLAY	CHECKED BY		CHECK DATE	

Shear Stress -vs- Displacement



Sample Dimensions

Diameter (inch)	2.5
Height (inch)	1.0

	Point 1	Point 2	Point 3	Average
Water Content (%)	18.8	21.5	20.6	20.3
Dry Density (pcf)	106.2	103.4	106.4	105.3
Void Ratio*	0.6	0.6	0.6	0.6
Peak Shear Stress (psf)	1771.4	3096.0	5643.2	

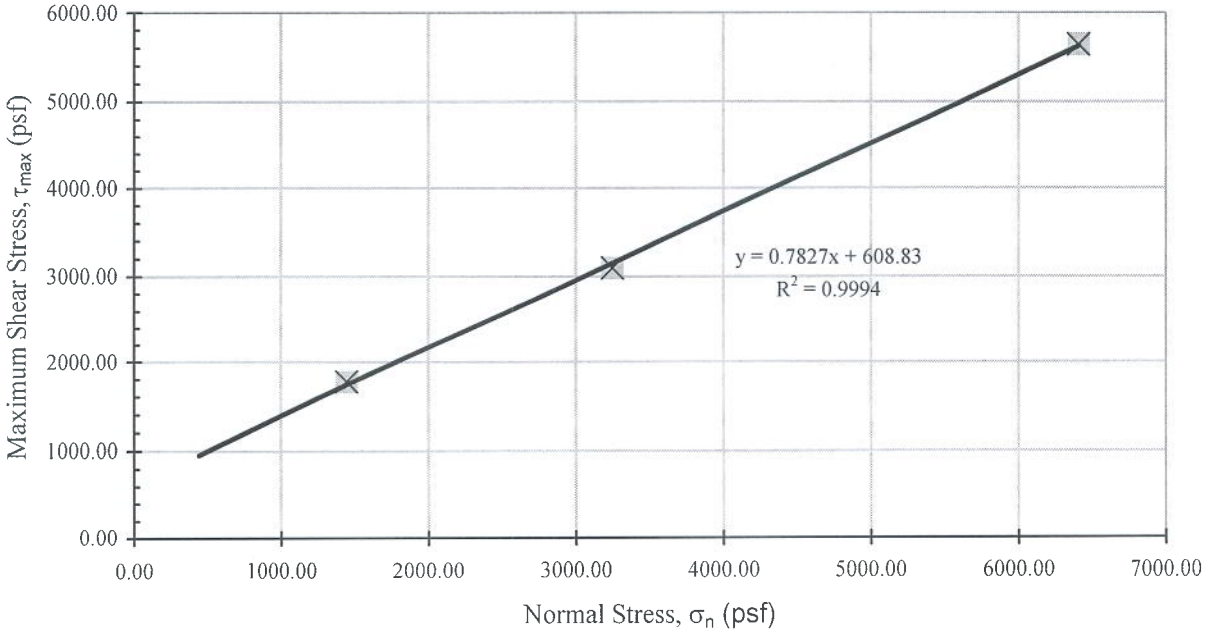
*Void ratio calculation assumes a specific gravity of 2.65

c - intercept (cohesion)	609
phi (internal friction angle)	38°

Notes: UNDISTURBED SAMPLE FROM 3' SHELBY

0

Maximum Shear Stress -vs- Normal Stress



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MOISTURE / DENSITY
ASTM D-2216 / 2937

PROJECT	ODCHC		JOB NO.	7119.02/01/2500	SHEET
CLIENT	ODCHC		LAB ID	10-179EK	1 of 1
LOCATION	TYDD ST	TEST BY	DLR	DATE	12/28/10
SOIL TYPE	VARIOUS	CHECKED BY		CHECK DATE	

Sample Location	TB-1	TB-2	TB-3	TB-4				
Sample Depth (ft bgs)	6.0'-7.0'	22.5'-23.5	2.5'-3.5'	6.5'-7.5'				
Soil Type (USCS)	SC	SP/SC	SC	SP/SC				
Moisture Content (%)	26.1	21.1	21.1	19.0				
Wet Density (pcf)	116.0	130.9	127.5	127.7				
Dry Density (pcf)	92.0	108.4	105.2	107.3				
Void Ratio*	0.8	0.5	0.6	0.5				
% Saturation	86.4	104.2	97.6	92.7				
Compressive Strength** (tsf)	0.9	0.6	1.0	1.1				

*Void ratio calculation assumes a specific gravity of 2.65

**Unconfined compressive strength as measured with a pocket penetrometer. Average of three readings.

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FINER THAN #200 SIEVE
ASTM D-1140

PROJECT	ODCHC		JOB NO.	7119.02	SHEET
CLIENT	ODCHC		SAMPLE ID	10-179EK	1 of 1
LOCATION	TYDD STREET	TEST BY	DLR	DATE	12/29/10
SOIL TYPE	VARIOUS	CHECKED BY		CHECK DATE	

TB-1@2.5'-4.0'

(B)	Net sample (Dry)	235.6	gms
(C)	Dry sample after washing	204.1	gms
	Total Material finer than 200 sieve	31.5	gms
(A)	% Material finer than 200 sieve	**13.37%**	

A=[(B-C)/B]X100

TB-1@5.0'-6.0'

(B)	Net sample (Dry)	327.4	gms
(C)	Dry sample after washing	283.8	gms
	Total Material finer than 200 sieve	43.6	gms
(A)	% Material finer than 200 sieve	**13.32%**	

A=[(B-C)/B]X100

TB-1@6.0'-7.0'

(B)	Net sample (Dry)	220.1	gms
(C)	Dry sample after washing	181.3	gms
	Total Material finer than 200 sieve	38.8	gms
(A)	% Material finer than 200 sieve	**17.63%**	

A=[(B-C)/B]X100

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FINER THAN #200 SIEVE
ASTM D-1140

PROJECT	ODCHC		JOB NO.	7119.02	SHEET
CLIENT	ODCHC		SAMPLE ID	10-179EK	1 of 1
LOCATION	TYDD STREET	TEST BY	DLR	DATE	12/29/10
SOIL TYPE	VARIOUS	CHECKED BY		CHECK DATE	

TB-2@2.5'-4.0'

(B)	Net sample (Dry)	238.6	gms
(C)	Dry sample after washing	177.1	gms
	Total Material finer than 200 sieve	61.5	gms
(A)	% Material finer than 200 sieve	**25.78%**	

A=[(B-C)/B]X100

TB-2@5.0'-6.5'

(B)	Net sample (Dry)	196.7	gms
(C)	Dry sample after washing	172.1	gms
	Total Material finer than 200 sieve	24.6	gms
(A)	% Material finer than 200 sieve	**12.51%**	

A=[(B-C)/B]X100

TB-2@10.0'-11.5'

(B)	Net sample (Dry)	192.9	gms
(C)	Dry sample after washing	173.7	gms
	Total Material finer than 200 sieve	19.2	gms
(A)	% Material finer than 200 sieve	**9.95%**	

A=[(B-C)/B]X100

TB-2@15.0'-16.5'

(B)	Net sample (Dry)	243.4	gms
(C)	Dry sample after washing	199.4	gms
	Total Material finer than 200 sieve	44.0	gms
(A)	% Material finer than 200 sieve	**18.08%**	

A=[(B-C)/B]X100

TB-2@20.0'-21.5'

(B)	Net sample (Dry)	371.7	gms
(C)	Dry sample after washing	297.5	gms
	Total Material finer than 200 sieve	74.2	gms
(A)	% Material finer than 200 sieve	**19.96%**	

A=[(B-C)/B]X100

TB-2@22.0'-23.5'

(B)	Net sample (Dry)	302.0	gms
(C)	Dry sample after washing	252.2	gms
	Total Material finer than 200 sieve	49.8	gms
(A)	% Material finer than 200 sieve	**16.49%**	

A=[(B-C)/B]X100

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FINER THAN #200 SIEVE
ASTM D-1140

PROJECT	ODCHC		JOB NO.	7119.02	SHEET
CLIENT	ODCHC		SAMPLE ID	10-179EK	1 of 1
LOCATION	TYDD STREET	TEST BY	DLR	DATE	12/29/10
SOIL TYPE	VARIOUS	CHECKED BY		CHECK DATE	

TB-3@2.5'-3.5'

(B)	Net sample (Dry)	265.5	gms
(C)	Dry sample after washing	197.3	gms
	Total Material finer than 200 sieve	68.2	gms
(A)	% Material finer than 200 sieve	**25.69%**	

A=[(B-C)/B]X100

TB-4@6.5'-7.5'

(B)	Net sample (Dry)	232.7	gms
(C)	Dry sample after washing	201.7	gms
	Total Material finer than 200 sieve	31.0	gms
(A)	% Material finer than 200 sieve	**13.32%**	

A=[(B-C)/B]X100

APPENDIX C

Whitchurch Engineering, Inc.

Soil Profile Logs

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SHEET 1 OF 6

EXPLORATION TEST LOG

PROJECT NAME: TYDD ST.	JOB NO: P1E0541	SAMPLE DATE: 4-7-5

HOLE #: TH-1	HOLE TYPE: B.H	LOGGED BY: TOR	HOLE ELEV: —	DATUM: —

LAB DATA									SOIL DESCRIPTION
MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFIRMED COMPRESSION (PSF)	LIQUID LIMIT	PLASTICITY INDEX	RELATIVE COMPACTION	SAMPLE	SOIL TYPE	DEPTH (FEET)	SOIL, COLOR, MOISTURE, CONSISTENCY, REMARKS; WATER LEVEL(S) AND DATE(S) (UNIFIED SOIL CLASSIFICATION SYSTEM)
		1.5							CLAYEY SILTY SAND, REDDISH BROWN, MOIST MEDIUM DENSITY. FEW ROOTS, COHESIVE
		1.5						1	
								2	CLAYEY SILTY SAND; SAND CONTENT INCREASES W/ DEPTH, STRATIFIED PALE YELLOW BROWN, MOIST. MED. DENSITY. COHESIVE
								3	SILTY SAND, REDDISH ORANGE, MOIST, MED. DENSITY. COHESIVE
								4	SILTY SAND, PALE YELLOW, MOIST, MEDIUM DENSITY. COHESIVE
								5	
								6	
								7	
								8	BOTTOM OF HOLE AT 8' SOIL GENERALLY BECOMES SANDIER WITH DEPTH NO GROUNDWATER OBSERVED
								9	
								10	
								11	
								12	
								13	
								14	
								15	
								16	
								17	
								18	
								19	
								20	
								21	

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SHEET 2 OF 6

EXPLORATION TEST LOG

PROJECT NAME: TYDD ST	JOB NO: P1E0501	SAMPLE DATE: 4-7-5

HOLE #: TH-2	HOLE TYPE: B.H.	LOGGED BY: TOR	HOLE ELEV: —	DATUM: —

LAB DATA | SOIL DESCRIPTION

MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFIRMED COMPRESSION (PSF)	LIQUID LIMIT	PLASTICITY INDEX	RELATIVE COMPACTION	SAMPLE	SOIL TYPE	DEPTH (FEET)	SOIL, COLOR, MOISTURE, CONSISTENCY, REMARKS, WATER LEVEL(S) AND DATE(S) (UNIFIED SOIL CLASSIFICATION SYSTEM)
									SANDY LOAM, DARK, MOIST, LOOSE
		1.0						1	SLIGHTLY SILTY CLAYEY SAND, ORANGISH BROWN, MOIST, MED. DENSITY, COHESIVE
		1.0							
								2	SLIGHTLY SILTY SAND, PALE YELLOWISH BROWN, MOIST, MED. DENSITY, COHESIVE. GROUND WATER SEEPING IN @ 40". SAND CONTENT INCREASES W/ DEPTH
▽								3	
								4	
								5	
								6	CLAYEY SAND, BLUISH BROWNISH WHITE STRATIFIED/MOTTLED/SPLOTCHY, MOIST, MEDIUM DENSITY. HOLE CAVES @ 7'
								7	
								8	SAND CONTENT INCREASES W/ DEPTH
								9	
								10	
								11	
								12	
								13	
								14	
								15	
								16	
								17	
								18	
								19	
								20	
								21	

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SHEET 3 OF 6

EXPLORATION TEST LOG

PROJECT NAME: TYDD ST.	JOB NO: PIE0501	SAMPLE DATE: 4-7-5

HOLE #: TH-3	HOLE TYPE: B.H.	LOGGED BY: TOR	HOLE ELEV: —	DATUM: —

LAB DATA: MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFIRMED COMPRESSION (PSF)	LIQUID LIMIT	PLASTICITY INDEX	RELATIVE COMPACTION	SAMPLE	SOIL TYPE	DEPTH (FEET)	SOIL DESCRIPTION: SOIL, COLOR, MOISTURE, CONSISTENCY, REMARKS; WATER LEVEL(S) AND DATE(S) (UNIFIED SOIL CLASSIFICATION SYSTEM)
									SILTY CLAY, BLUISH GRAY, MOIST, LOOSE FILL SOIL
								1	CLAYEY SILTY SAND, BROWN, MOIST, LOOSE, FILL
		1.75-2.5							SANDY GRAVEL, GRAY, MOIST FILL SOIL
								2	CLAYEY SILTY SAND, REDDISH BROWN, MOIST, DENSE
								3	SLIGHTLY SILTY SAND, ORANGISH BROWN, MOIST, DENSE. FEW PEBBLES TO 1/4" Ø. SOIL IS COHESIVE. SAND CONTENT INCREASES W/ DEPTH
								4	
								5	
								6	@ 64" EXPOSED THE TOP OF A ±7" Ø BLACK IRON PIPE. PIPE BEARS APPROX S. 32° E.
								7	
								8	
								9	
								10	
								11	
								12	
								13	
								14	
								15	
								16	
								17	
								18	
								19	
								20	
								21	

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SHEET 4 OF 6

EXPLORATION TEST LOG

PROJECT NAME: TYDD ST	JOB NO: PIE0501	SAMPLE DATE: 4-7-5

HOLE #: TH-4	HOLE TYPE: B.H.	LOGGED BY: TOR	HOLE ELEV: —	DATUM: —

LAB DATA | SOIL DESCRIPTION

MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFIRMED COMPRESSION (PSF)	LIQUID LIMIT	PLASTICITY INDEX	RELATIVE COMPACTION	SAMPLE	SOIL TYPE	DEPTH (FEET)	SOIL, COLOR, MOISTURE, CONSISTENCY, REMARKS; WATER LEVEL(S) AND DATE(S) (UNIFIED SOIL CLASSIFICATION SYSTEM)
									SILTY CLAYEY SAND, POTTING SOIL, CONC/ ASPHALT CHUNKS, TRASH FILL SOIL
								1	
								2	
								3	
▽		3.0						4	CLAYEY SILTY SAND, MOTTLED ORANGISH BROWN, MOIST
								5	SILTY CLAYEY SAND, BLACKISH BLUE, MOIST, DENSE
								6	SILTY CLAY, BLUISH BLACK, MOIST, DENSE
								7	SANDY CLAY, BLUE, MOIST, DENSE. @ 7', EXPOSED A ±12"ϕ REDWOOD LOG AND
								8	
								9	MANY BARE BRANCHES.
								10	BOTTOM OF HOLE @ 9'
								11	
								12	
								13	
								14	
								15	
								16	
								17	
								18	
								19	
								20	
								21	

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SHEET 5 OF 6

EXPLORATION TEST LOG

PROJECT NAME: TYDD ST	JOB NO: P1E0501	SAMPLE DATE: 4-7-5

HOLE #: TH-5	HOLE TYPE: B.H.	LOGGED BY: TOR	HOLE ELEV: —	DATUM: —

LAB DATA									SOIL DESCRIPTION
MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFIRMED COMPRESSION (PSF)	LIQUID LIMIT	PLASTICITY INDEX	RELATIVE COMPACTION	SAMPLE	SOIL TYPE	DEPTH (FEET)	SOIL, COLOR, MOISTURE, CONSISTENCY, REMARKS; WATER LEVEL(S) AND DATE(S) (UNIFIED SOIL CLASSIFICATION SYSTEM)
		2.0							SANDY SILTY TOPSOIL, YELLOWISH BROWN, MOIST, MEDIUM DENSITY
		2.25						1	SANDY SILTY CLAY, REDDISH BROWN, MOIST, MEDIUM DENSITY
								2	SANDY SILTY CLAY, BROWNISH YELLOW, MOIST @ 39"-50": BAND OF FEW SANDSTONE RUCK'S
								3	
								4	
								5	SLIGHTLY SILTY SAND, LIGHT BROWNISH YELLOW, MOIST, DENSE.
								6	
								7	
								8	AT 93" BECOMES SANDSTONE LIKE
								9	BOTTOM OF HOLE @ 104" NO GROUNDWATER
								10	
								11	
								12	
								13	
								14	
								15	
								16	
								17	
								18	
								19	
								20	
								21	

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SHEET 6 OF 6

EXPLORATION TEST LOG

PROJECT NAME: TYDO ST	JOB NO: P1E 0501	SAMPLE DATE: 4-7-5

HOLE #: TH-6	HOLE TYPE: B.H.	LOGGED BY: TOR	HOLE ELEV: —	DATUM: —

LAB DATA | SOIL DESCRIPTION

MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSION (PSF)	LIQUID LIMIT	PLASTICITY INDEX	RELATIVE COMPACTION	SAMPLE	SOIL TYPE	DEPTH (FEET)	SOIL, COLOR, MOISTURE, CONSISTENCY, REMARKS; WATER LEVEL(S) AND DATE(S) (UNIFIED SOIL CLASSIFICATION SYSTEM)
		1-2 / 1-2						1	SILTY SAND / SANDY SILT, BROWN, MOIST MEDIUM DENSITY
								2	SILTY SAND, YELLOW, MOIST, MEDIUM DENSITY. @ 43"-48" - BAND OF CLAYEY SILTY FINE SAND, REDDISH BROWN - MOIST, MEDIUM DENSITY
								3	
								4	
								5	SAND, SLIGHTLY SILTY, MOIST. SLIGHTLY COHESIVE
								6	
▽								7	BECOMES SANDSTONE LIKE AT BOTTOM
								8	BOTTOM OF HOLE @ 8'
								9	
								10	
								11	
								12	
								13	
								14	
								15	
								16	
								17	
								18	
								19	
								20	
								21	

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SHEET___ OF___

EXPLORATION TEST LOG

PROJECT NAME: PIERSON	JOB NO: PIE 0501	SAMPLE DATE: 5-2-05

HOLE #: 7	HOLE TYPE: BH	LOGGED BY: TOR	HOLE ELEV: ___	DATUM: ___

LAB DATA | SOIL DESCRIPTION

MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSION (TONS/SF)	LIQUID LIMIT	PLASTICITY INDEX	RELATIVE COMPACTION	SAMPLE	SOIL TYPE	DEPTH (FEET)	SOIL, COLOR, MOISTURE, CONSISTENCY, REMARKS, WATER LEVEL(S) AND DATE(S) (UNIFIED SOIL CLASSIFICATION SYSTEM)
									0-6" BROWN SILTY SAND ±20% GRAVELS, VARIOUS CONCRETE/ ASPHALT CHUNKS 2" ϕ, MOIST
								1	
		1.5-2.0						2	6"-9'-8" BROWNISH BLUE, SLIGHTLY SILTY SAND, MOIST
								3	BLUE SANDY GRAVEL TOWARD NORTH WEST END, REBAR + CONC. CHUNKS DIFFICULT TO PENETRATE
								4	UCS 1.5-2.0 TSF @24"
								5	
								6	
								7	
								8	
								9	
▽								10	9'-8"-10' BLUE/BROWN SAND, SILTY WET, GROUNDWATER OBSERVED
									END EXCAVATION @ 10'
								11	

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EXPLORATION TEST LOG

SHEET___ OF___

PROJECT NAME: PIERSON	JOB NO: PIE0501	SAMPLE DATE: 5-2-05

HOLE #: 8	HOLE TYPE: BH	LOGGED BY: TOR	HOLE ELEV: —	DATUM: —

LAB DATA | SOIL DESCRIPTION

MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSION (PSF)	LIQUID LIMIT	PLASTICITY INDEX	RELATIVE COMPACTION	SAMPLE	SOIL TYPE	DEPTH (FEET)	SOIL, COLOR, MOISTURE, CONSISTENCY, REMARKS, WATER LEVEL(S) AND DATE(S) (UNIFIED SOIL CLASSIFICATION SYSTEM)
								1	0-26" BROWN SANDY SILT, FILL MOIST, DENSE
								2	
								3	26-44" BLUISH SANDY GRAVEL MOIST, DENSE, FOUND REBAR (NO GRAVEL LAYER IN SOUTH ½ OF 20' TRENCH)
								4	44"-10½' SLIGHTY CLAYEY SILTY SAND, BROWN/TAN, MOIST, DENSE CLAY CONTENT INCREASES WITH DEPTH
								5	
								6	
								7	
								8	
								9	
								10	GROUNDWATER OBSERVED @ 10.5'
								11	END EXCAVATION @ 10.5'

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SHEET___ OF___

EXPLORATION TEST LOG

PROJECT NAME: PIERSON	JOB NO: PIE0501	SAMPLE DATE: 5-2-05

HOLE #: 9 (WEST)	HOLE TYPE: BH	LOGGED BY: TOR	HOLE ELEV: —	DATUM: —

LAB DATA | SOIL DESCRIPTION

MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSION TONS/SF	LIQUID LIMIT	PLASTICITY INDEX	RELATIVE COMPACTION	SAMPLE	SOIL TYPE	DEPTH (FEET)	SOIL, COLOR, MOISTURE, CONSISTENCY, REMARKS, WATER LEVEL(S) AND DATE(S) (UNIFIED SOIL CLASSIFICATION SYSTEM)
									0-18" BROWN CLAYEY SILTY SAND MOIST, DENSE
								1	
		3.25-4.0							UCS 3.25-4 TSF @ 12" + 18"
									18-82" TAN SILTY SAND, MOIST, DENSE
								2	
								3	
								4	
								5	
								6	
								7	82" - 8' REDDISH BROWN SAND, SIM. TO BEACH SAND
								8	END EXCAVATION @ 8'
								9	
								10	
								11	

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SHEET___ OF___

EXPLORATION TEST LOG

PROJECT NAME: PIERSON	JOB NO: PIE 0501	SAMPLE DATE: 5-2-05

HOLE #: 9 (EAST)	HOLE TYPE: BH	LOGGED BY: TOR	HOLE ELEV: —	DATUM: —

LAB DATA								SOIL DESCRIPTION	
MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSION (PSF)	LIQUID LIMIT	PLASTICITY INDEX	RELATIVE COMPACTION	SAMPLE	SOIL TYPE	DEPTH (FEET)	SOIL, COLOR, MOISTURE, CONSISTENCY, REMARKS, WATER LEVEL(S) AND DATE(S) (UNIFIED SOIL CLASSIFICATION SYSTEM)
								—	0-16" RIVER RUN BACKFILL
								1	
								—	16"- 8' TAN/BROWN SILTY SAND
								2	MOIST, DENSE
								—	2" PIPE, REBAR FOUND @ 6'
								3	
								—	
								4	
								—	
								5	
								—	
								6	
								—	
								7	
								—	
								8	END EXCAVATION @ 8'
								—	
								9	
								—	
								10	
								—	
								11	
								—	

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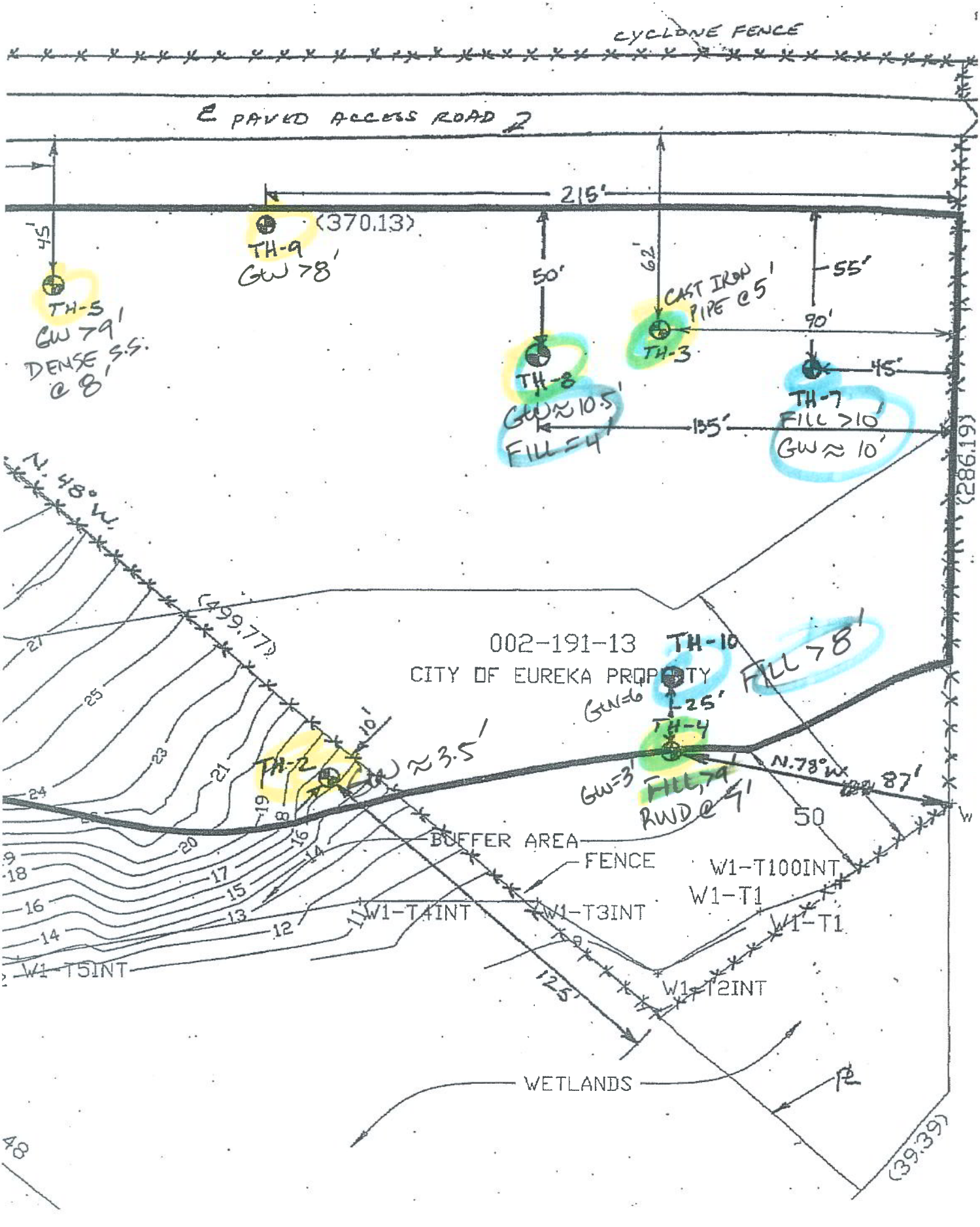
EXPLORATION TEST LOG

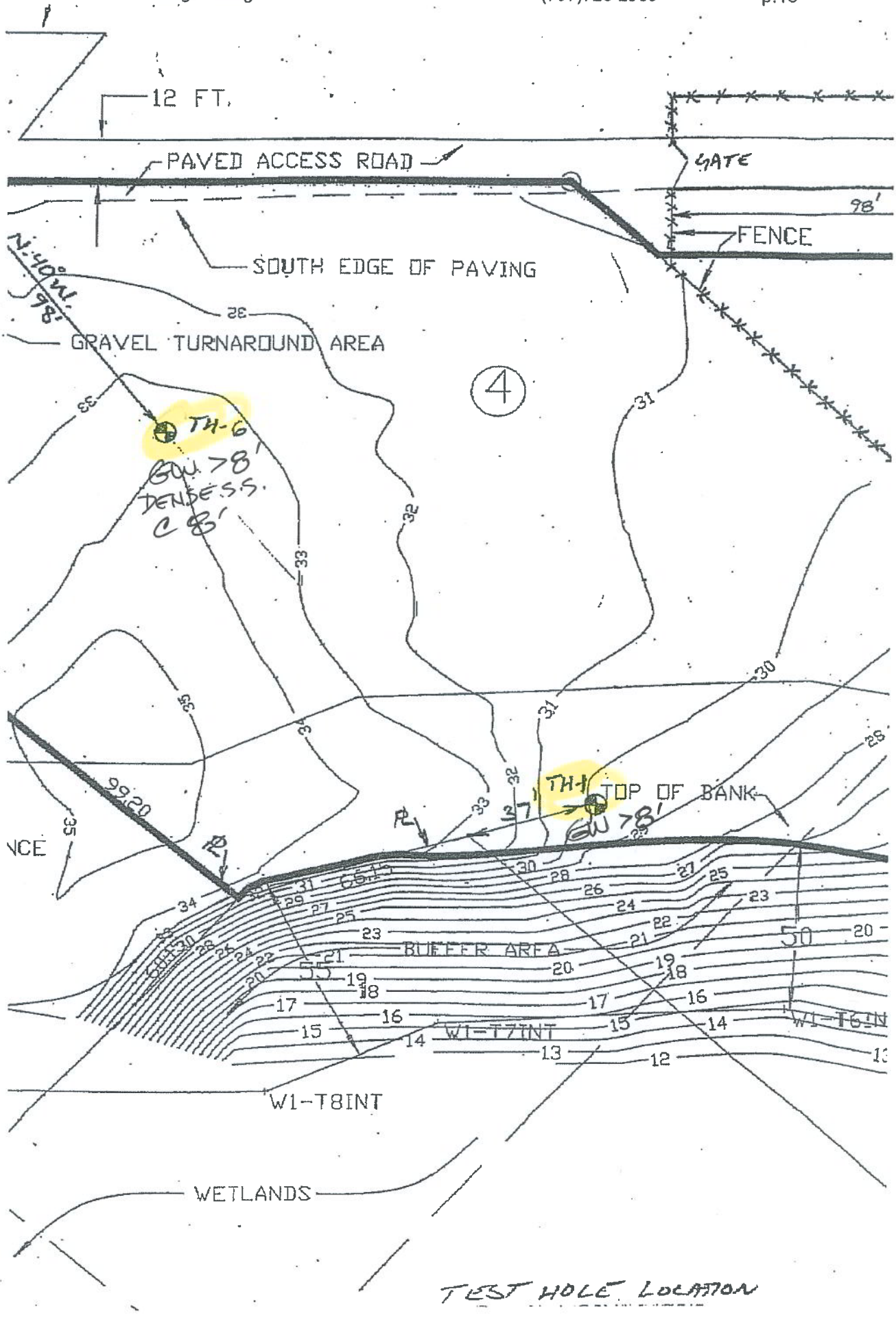
SHEET___ OF___

PROJECT NAME: PIERSON	JOB NO: PIE0501	SAMPLE DATE: 5-2-05

HOLE #: 10	HOLE TYPE: BH	LOGGED BY: TOR	HOLE ELEV: –	DATUM: –

LAB DATA									SOIL DESCRIPTION
MOISTURE CONTENT (%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSION (PSF)	LIQUID LIMIT	PLASTICITY INDEX	RELATIVE COMPACTION	SAMPLE	SOIL TYPE	DEPTH (FEET)	SOIL, COLOR, MOISTURE, CONSISTENCY, REMARKS, WATER LEVEL(S) AND DATE(S) (UNIFIED SOIL CLASSIFICATION SYSTEM)
								–	0-6' DISTURBED FILL CONSISTING
								1	OF CONCRETE, ASPHALT, TRASH
								–	METAL PIPE, + WOOD CHUNKS.
								2	DARK SANDY CLAYEY SILT,
								–	MOIST, SOFT
								3	CUT BOARDS 3" THICK
								–	BLACK 1" ϕ GAS PIPE @ 4'
								4	(UNBROKEN)
								–	
								5	
								–	
								6	GROUNDWATER @ 6'
								–	6'-6'-3" SAND, WHITISH BLUE, WET
								7	END EXCAVATION @ 6'-3"
								–	
								8	
								–	
								9	
								–	
								10	
								–	
								11	
								–	


CYCLONE FENCE
℄ PAVED ACCESS ROAD ℄
215'
(370.13)
TH-9
GW >8'
45'
TH-5
GW >9'
DENSE S.S.
@ 8'
50'
62'
CAST IRON
PIPE @ 5'
55'
90'
TH-3
TH-8
GW≈10.5'
FILL=4'
45'
TH-7
FILL >10'
GW≈10'
135'
(286.19)
N. 48° W.
(459.77)
002-191-13
CITY OF EUREKA PROPERTY
TH-10
GW=6
FILL >8'
25'
TH-4
TH-2
GW≈3.5'
GW=3'
FILL >9'
RWD @ 7'
N.78° W.
87'
50
BUFFER AREA
FENCE
W1-T100INT
W1-T1
W1-T4INT
W1-T3INT
W1-T1
W1-T5INT
W1-T2INT
125'
WETLANDS
(39.39)


12 FT.
PAVED ACCESS ROAD
GATE
98'
FENCE
SOUTH EDGE OF PAVING
N.40°W.
98'
GRAVEL TURNAROUND AREA
4
TH-6
GW >8'
DENSE S.S.
C 8'
TH-1
TOP OF BANK
GW >8'
99.20
BUFFER AREA
55
50
W1-T7INT
W1-T6INT
W1-T8INT
WETLANDS
TEST HOLE LOCATION

ATTACHMENT 1

ASFE Brochure

Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

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