BAXTER CREEK MULTI-FAMILY HOUSING
11965 SAN PABLO AVENUE, EL CERRITO
FOUNDATION INVESTIGATION & RECOMMENDATIONS

LAWRENCE B. KARP CONSULTING ENGINEER
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LAWRENCE B. KARP  CONSULTING ENGINEER
June 24, 2017

11965 San Pablo Avenue LLC
Charles Oewel
1606 Juanita Lane, Suite A
Tiburon, CA 94920

Subject: Baxter Creek Apartments
Proposed Multi-Family Housing Project
11965 San Pablo Avenue, El Cerrito
Foundation Investigation & Recommendations

Dear Mr. Oewel:

Pursuant to the scope of work authorized, this report describes the results of a site specific subsurface exploration program, review of published literature and reports by private consultants, research of my files and other data applicable to the site, numerous site inspections, and discussions with you and your architect, Martin Wilson of DeCredico Architects.

The purpose of this report is to provide you and your design professionals with geotechnical recommendations for design of the building foundations and sitework, and to provide your contractor with information concerning construction of a new multi-family housing structure with underground parking on the site.

To complete the design process, final plans should be reviewed by the undersigned before a building permit is applied for; and during construction all foundation and related sitework should be observed by a licensed civil engineer especially qualified in soil mechanics and foundation engineering. Observations should be followed by a final report provided to the City of El Cerrito attesting to satisfactory completion of the project substantially pursuant to the recommendations herein, subject to design modifications due to alternatives or alterations to the project before construction and site and foundation conditions are encountered during construction.

Project

The site is a diamond shaped parcel, resulting from adjustments, having a total area of about 23,900 square feet (0.503 acre) located on the west side of San Pablo Avenue between Bissell Avenue and BART in El Cerrito. The area generally slopes very gently toward San Francisco Bay southwest of MacDonald Avenue. The project consists of demolishing a former Taco Bell, excavation (about 9,500 bench CY) with shoring for underground parking, building a foundation system over the total footprint of all the building components consisting of a stiffened mat, and constructing a new 8 story multi-family apartment house (DeCredico 2017). Design, shoring, waterproofing, and construction of a subterranean garage in submerged ground are special foundation aspects for the project. Sitework will include ramps for the underground parking, minor grading, drainage, pavements, and landscaping.
Geology

The area and present site of the Taco Bell building, access driveways and parking areas for the restaurant now covered by Portland cement concrete, is underlain by surficial soils consisting of undifferentiated older alluvium [Qoa], unconsolidated, deposited during the Pleistocene Epoch, Quaternary Period, less than 1.5 million years old which are described in a monograph (Bishop et al 1973) and later mapped with more accuracy by Nilsen (1975) and Dibblee (1980). There are no areas of mass instability at or near the building site.

Soils

Alluvium is comprised of unconsolidated sand, silt, clay, and minor gravels in various proportions and densities. The formation generally consists of interfingered layers of clayey gravel, sandy silty clay, and various clay-silt-sand mixtures of a typical alluvial fan deposit brought down from the Berkeley Hills, probably formed in a dry climate with infrequent torrential rains. Within the deposit there are stream cut channels which are now creeks or were eventually filled.

The clay constituents contain montmorillonite and where not intermixed with predominately granular soils these soils are plastic which swell and become weaker in a saturated state as compared to the material being unsaturated and with drier moisture contents. The clays in the alluvium are known to be moderately expansive. Surficial clays are known to be highly expansive.

Expansive soil, clays where volumetric change (shrink-swell potential and low strength with variations in moisture content) is a common cause of foundation distress (Meehan & Karp 1994). Foundations for housing structures and other structures, such as retaining walls founded in expansive soil, require special design consideration. This consideration generally does not apply to the foundations of larger structures such as the proposed building because of the relatively deep foundation system that is recommended herein. The surficial clays can be managed if sitework is properly designed and constructed. Damage to surface improvements from expansive soils may be mitigated by engineering foundations and flatwork with the installation and continued maintenance of landscaping and proper drainage.

Seismicity

The site is located in the earthquake active San Francisco Bay Area which is seismically dominated by the presence of the San Andreas Fault System. In the theory of plate tectonics, the San Andreas is the boundary between the northward moving Pacific Plate (west of the fault) and North American Plate (east of the fault) which is manifested by the San Andreas Fault System. The faults in the system produce dextral horizontal shear movements resulting from the relative motion of the Pacific and North American plates.

The northwestward movement of the Pacific Plate relative to the North America persistently causes right-lateral slip across the major faults and deformation between the faults. In the Bay Area, this movement is distributed across a complex system of strike-slip, right lateral parallel and subparallel faults. The Hayward fault is about 1.0 mile to the northeast and the San Andreas fault is about 19 miles to the west. The Hayward fault ruptured on 6/10/1836 and 10/21/1868 (estimated M = 7.0 and 6.8, respectively), and last severely shook (M = 4.5) the area on 12/17/54 (Toppozada 1986).
The San Andreas fault ruptured on 4/18/06 (estimated $M = 8.0$) and last shook the area severely on 10/17/89 (Loma Prieta, $M = 7.1$). A relatively recent earthquake that epicentered near the site occurred on 3/22/57 (Daly City, $M = 5.3$), and another earthquake epicentered along the San Andreas on 10/1/69 (Santa Rosa, $M = 5.7$). On 8/24/14 an earthquake of high magnitude occurred between the Hayward-Rogers Creek fault and the Concord-Green Valley fault (South Napa, $M = 6.0$) causing extensive damage to buildings in downtown Napa.

The site is situated at approximate USGS Elevation $+40$ and outside the boundaries of the Alquist-Priolo Special Study Zone (SSZ; CDM&G 1982). The lot is not crossed by an active fault so the hazard from direct fault offset is remote. Bedrock occurs at relatively shallow depths at the site (31 to 34 feet from the existing asphaltic concrete surface) so future earthquakes will characteristically produce moderate frequency ground shaking but seismic motions will be amplified by the very soft ground above bedrock (depth 9 to 31 feet). Maximum moment magnitudes (scaled size of earthquakes in terms of energy released) are San Andreas $M_w = 7.9$, Hayward $M_w = 6.9$.

The USGS has forecasted a 67% probability that one or more earthquakes of $M = 7.0$ (0.20 to 0.45g) or greater will occur on the San Andreas or Hayward faults by the year 2020 (Peterson 1996). The average earthquake recurrence interval for the East Bay is roughly 220 years, give or take 100 years or so. As for ground rupturing, there has been a quiescent period of seismic activity after the great 1906 earthquake on the San Andreas fault and there has been no rupturing along the Hayward fault for more than 100 years.

The latest research, including very recent trenching by the U.S. Geological Survey at the Mira Vista Country Club in the Berkeley Hills east of the site, indicates that the Northern Segment of the Hayward Fault is overdue for a characteristic major earthquake (Schwartz & Lettis 1998). Based on history and theory, the site will be subjected to strong ground shaking from earthquakes generated by the San Andreas and Hayward faults as well as by any other active faults in the Bay Area.

2016 CBC Appendix J §103.2 requires a soils report for all walls retaining cuts or fills and 2016 CBC §1803.6 requires that certain information appear in a geotechnical investigation report submitted to the building official for permit although not all of that information may be directly applicable to a given project. Where lateral loads and resistances are given herein, recommendations are for use with 2016 CBC §1803.5.11 where seismic forces may or may not be added to earth loads for design; for this project they are included in the recommended lateral pressures for shoring.

The project is located in Contra Costa County, 1800 feet from an area in Alameda County, also in the Richmond Quadrangle, where the presence of potentially liquefiable soils has been identified (CGS 2003). Seismic liquefaction (cyclic mobility), a phenomenon that occurs when loose granular (not sticky) soils that are saturated undergo a rapid loss in shear strength as a consequence of ground shaking, may occur because liquefaction is possible in significant deposits of loose and saturated granular soils (sands, gravels and non-cohesive silts). Soils at the site are predominately sands, gravels and clays intermixed or layered with cohesive (sticky) soils. Stiff clayey and dense granular soils do not liquefy so they are not susceptible to liquefaction at least to the extent that a structure at the subject site would be affected.

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1 The moment magnitude scale is used to measure earthquake magnitude $M_w$ taking into account the size of the fault rupture, the stiffness of rock, and the amount of the movement of the fault using values that can be estimated from the size of several types of seismic waves; while the older Richter scale is a numerical scale used to measure the magnitude $M$ of an earthquake using values based on the size of the earthquake's largest seismic waves.
Earthquake Hazards

The site is 2.42 miles from the Alameda County portion of the seismic hazard zone map of the Richmond Quadrangle for "areas where historical occurrence of liquefaction, or local geological, geotechnical and ground-water conditions indicate a potential for permanent ground displacement such that mitigation as defined in Public Resources Code 2693[c] would be required."

Seismic hazards are classified as primary and secondary. Primary effect is ground rupture (surface faulting). Secondary seismic hazards include ground shaking (potential building damage mitigated by designing to the 2016 CBC as discussed in the next paragraph), liquefaction and densification (appropriate concerns for the site), lateral spreading, landslides, lurching, tsunamis, and seiches. Consideration of primary effects indicates there are no active faults crossing the site nor is it within the Alquist-Priolo earthquake fault hazard zone so there is no likelihood of ground rupture. Consideration of secondary effects indicates an earthquake of moderate to high magnitude generated on any fault in the Bay Area will cause considerable ground shaking at the site. Parameters for ground motion are given on pages 5 though 7 herein.

Seismic design provisions of the 2016 CBC prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead plus live loads. The prescribed lateral forces are smaller than the equivalent forces that would be associated with a major earthquake. Structures must be able to (1) resist minor earthquakes without damage; (2) resist moderate earthquakes without structural damage but with some nonstructural damage; and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to building code requirements does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well constructed structure will not collapse or cause loss of life in a major earthquake.

LIQUEFACTION: Liquefaction is a phenomenon where saturated cohesionless soils (and soils with low cohesion), are subject to a temporary, but effectively total, loss of shear strength because of pore pressure build-up under the reversing cyclic shear stresses induced by earthquakes. Soils most susceptible to liquefaction are generally loose, clean, saturated, uniformly graded, fine-grained sand and silt of low plasticity with a low percentage of fine-grained clay particles. Significant deposits of clean submerged potentially liquefiable soils are not generally present, which is the controlling screening parameter for building sites (Castro & Poulos 1977, CGS 2008).

DENSIFICATION: Densification of loose sand above the groundwater table during earthquake shaking could cause settlement of the ground surface. In addition, densification of liquefiable soils, below the groundwater level could cause detrimental settlement at the ground surface. Loose sand layers were not encountered in the geotechnical drilling, and the granular soils encountered at the exploratory exploration were generally dense to very dense. Although the building will settle in service, the potential for earthquake-induced densification is low.

SPREADING: Lateral spreading is a failure within a nearly horizontal soil layer that could occur due to liquefaction. Spreading causes the overlying soil mass to move toward a free face or down a gentle slope. There are no significant potentially liquefiable strata and the lot is level so lateral spreading will not occur.

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LOCAL FLATLAND CHARACTERISTICS: As mapped by the California Geological Survey (CGS 2003), the proposed project site is located within the extension of a potential liquefaction hazard zone (and/or by extension a soft ground amplification hazard zone) which potential occurrence may be predicted by an investigation of soil conditions not only at the specific site of a proposed project but also appropriately in the immediate area of the project. Analyses of the results of liquefaction/amplification evaluations by different investigators (Karp 2015) gives the broadest view of the potentials (general opinion is that the upper 40 feet of soil may be subject to liquefaction). Such use of adjacent data is permitted by ASCE 7-10 §11.8.2 and those provisions have been adopted into 2016 CBC §1803.2.

Accordingly, effects of ground shaking may be mitigated by designing structures to resist lateral forces in accordance with the 2016 CBC. Ground shaking and differential movements due to soil/structure interaction may be further mitigated by engineering all construction and by providing a fully integrated foundation and substructure system. Engineering for the building superstructure should be designed to exceed the minimum lateral force seismic resisting requirements set forth in Chapter 16 of the 2016 CBC as adopted into the El Cerrito Municipal Code. Structurally interconnecting all foundation components will cause the system to move as a unit, if it moves at all. This concept has been employed in the design and construction recommendations herein.

Ground Motion Parameters

The National Earthquake Hazards Reduction Program ("2009 NEHRP") document "Recommended Provisions for Seismic Regulations for New Buildings and Other Structures" (FEMA 450-1) feeds into the ASCE (American Society of Civil Engineers) 7-10 “Minimum Design Loads for Buildings & Other Structures” (ASCE 2013) development process, and ASCE 7 in turn serves as the primary referenced standard in the 2015 International Building Code (2015 IBC). The 2016 California Building Code (2016 CBC) is the City of El Cerrito’s building code, which is the State’s iteration and adoption of the 2015 IBC.

Ground motion parameters, for this report of a subsurface investigation for the project site, were determined for the site using USGS ASCE 7 (2010 with 2013 Errata) based calculation tools derived from published ground motion maps. Seismic ground motion values for use in characterizing and classifying the site for structural design of the new residence are as follows:

**General:**

Site Location (USGS):

| Latitude 37.93086°N |
| Longitude -122.32272°W |

Risk Category (2016 CBC Table 1604.5): II

Seismic Importance Factor $I_c$ (ASCE 7 Table 1.5-2): 1.00

**Mapped Acceleration Parameters (2016 CBC §1613.3.1):**

Determination of Maximum Considered Earthquake (MCE) spectral response accelerations, mapped at short (0.2 second) period $S_s$ and at a full second (1.0 second) period $S_1$, for the site:

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2 "Buildings and other structures that represent a substantial hazard to human life in the event of failure."

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Site Soil Classification (2010 ASCE 7-10 w/March 2013 Errata): D

Short period (0.20 second) mapped spectral acceleration $S_v$: 2.312g
Site Coefficient $F_v$ (2016 CBC Table 1613.3.3(1); function/Site Class B & $S_v$): 1.0
Adjusted MCE 0.20 second period spectral response acceleration $S_{Mx-B} = F_vS_v$: 2.312g

One second period mapped spectral acceleration $S_v$: 0.960g
Site Coefficient $F_v$ (2016 CBC Table 1613.3.3(2); function/Site Class D & $S_v$): 1.5
Adjusted MCE one second period spectral response acceleration $S_{M1-B} = F_vS_v$: 1.440g

Design Spectral Response Acceleration Parameters (2016 CBC §1613.3.3):

Determination of Seismic Design Category (SDC) is based on occupancy or use and level of expected soil/rock-modified seismic ground motion at the site.

Site Classification definitions are dependent on geotechnical data (2016 CBC §1613.2.1; ASCE 7 §§20.3.2, 20.3.3(3) e.g. shear wave velocity (at this site: $v_s = 600$ to 1,200 ft/sec)$^3$.

Defined Site Classification (2016 CBC §1613.3.2 & ASCE 7 Table 20.3-1): D

Site Coefficient $F_v$ (2016 CBC Table 1613.3.3(1); function/Site Class D & $S_v$): 1.0
Adjusted MCE 0.20 second period spectral response acceleration $S_{Mx-D} = F_vS_v$: 2.312g
5% damped short period design spectral acceleration $S_{DS} = 0.67S_{Mx-D} = 0.67(2.312)$: 1.549g

Site Coefficient $F_v$ (2016 CBC Table 1613.3.3(2); function/Site Class D & $S_v$): 1.5
Adjusted MCE one second period spectral response acceleration $S_{M1-D} = F_vS_v$: 1.440g
5% damped one sec. period design spectral acceleration $S_{DI} = 0.67S_{M1-D} = 0.67(1.440)$: 0.965g

Seismic Design Categories (SDC); Risk Category II, $S_v \geq 0.75$ (2016 CBC §1613.3.5, ASCE 7 §11.6):

Short period response acceleration $SDC_{DS}$ (2016 CBC Table 1613.3.5(1); $0.50g \leq S_{DS}$): D
One second period response accel. $SDC_{DI}$ (2016 CBC Table 1613.3.5(2); $0.20g \leq S_{DI}$): D

Mapped MCE Geometric Mean Peak Ground Acceleration, PGA (ASCE 7 §11.8.3; SDC = D):

PGA (USGS output for Location and Class): 0.889g
Site Coefficient $F_{PGA}$ (Site Class B, ASCE Table 11.8-1, PGA $\geq 0.50$): 1.000
Peak Ground Acceleration adjusted for site class effects $PGA_m = F_{PGA}PGA$: 0.889g

$^3$ ASCE 7-10 §20.1 provides “where site-specific data are not known to a depth of 100 feet, appropriate soil properties are permitted to be estimated by the licensed design professional preparing the soils report based on known geologic conditions.” Definition of Site Class: ASCE 710 (page 132) Table §20.3-1 “Average properties in the top 100 feet.”

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Reporting of the above ground motion parameters, recently required per ASCE 7-10 (2013) and calculated for the subject building, should be used for analysis. Lateral force resisting systems must meet seismic detailing requirements and limitations set forth in ASCE 7 (2016 CBC §1604.10).

Ground motion parameters listed above are conditional for design providing all excavations and foundations are verified during construction by a qualified civil/structural engineer and geotechnical observations and special inspections are provided by a civil engineer especially qualified in the field of soil mechanics and foundation engineering. Geologists do not qualify (B&P Code §7839).

Mitigation of Seismic Hazards

California’s Special Publication 117A (CGS 2008) mandates countermeasures to liquefaction and landslide hazards in mapped hazard areas. As the Contra Costa County portion of the Richmond Quad has no mapped seismic hazards, it could be surmised that no mitigation countermeasures are necessary; however a proper assumption would be that ground conditions are the same as mapped for nearby Albany and Berkeley.

Due to the foundation system recommended, there is minimal risk of liquefaction, lateral spreading, or lurching that will damage a structure built on the subject site if the foundation recommendations given in this report are followed. There is a low to moderate risk of tsunami as the building will be founded at about average USGS Elev. +40 (there is no land survey map at the time of this report).

Field Exploration

Two state-of-the-art Cone Penetration Tests (CPT) were advanced at the site. There were previous CPT soundings but no evidence remains other than four logs (Engeo 1997a).

Due to prolonged rainfall in early 2017 and the unavailability of drillers acceptable to CCEH, a decision was made to retain Gregg to undertake a CPT program which was performed on 5/18/17. Two CPTs probes were advanced with holes backfilled with grout upon completion as mandated by CCEH. The process was intermittently observed by CCEH inspector and environmental specialist (Salvatore Ruiz). Materials registered at depths as the CPT probes advanced were:

CPT preparation: Hand augered. Fill 0-5 feet, olive brown silty clay (Munsel 2.5Y 4/4).

CPT-1 probe encountered: 5 to 8 feet clay with sand and gravel, then 8 to 16 feet clay, then 16 to 25 feet organics with clay lenses, then 25 to 30 feet clay, and then 30 to 34 feet to sandstone bedrock (CPT Log, Figure B, page 30).

CPT-2 probe encountered: 5 to 8 feet clay with sand and gravel, then 8 to 16 feet clay, then 16 to 25 feet organics with clay lenses, then 25 to 30 feet clay, and then 30 to 34 feet to sandstone bedrock (CPT Log, Figure C, page 31).

The CPT-1 probe was advanced to depth of 58.2 feet, and the CPT-2 probe was advanced to 56.7 feet. Locations of the CPTs are shown on the Site Plan, Figure A (page 29). Details applicable to CPT-1 are shown on Figure B (page 30) and details applicable to CPT-2 shown on Figure C (page 31). Information for CPT explorations is shown in the Appendix.

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Foundation Recommendations

Ground Supported Stiffened Raft Slab

The garage will occupy an area excavated to an approximate nominal depth of 12 feet below grade which will result in removal of soil above subgrade weighing an approximate average of 1.300 psf. The structural plans for the building have not been developed and there are no preliminary calculations available, but a fair estimate is that the building will weigh about 1,500 psf. This means that the building’s weight will be essentially compensated by the excavated ground such that the ultimate net bearing pressure of the building will be, after rebound and settlement, very low.

Ground below subgrade is predominately sands which are submerged and confined, there are negligible soft soils below subgrade that could be consolidated from the small differential in bearing pressure between the present overburden and the new building. Added to the relatively light loads and strong characteristics of the underlying soils and the very small net loads that will be exerted by the building is the fact that because the buildings are really two separate buildings with one very large footprint in the ground, the design of the foundation system will have to be stiff enough to distribute the building loads across the excavated site without any concentrated loads.

Ultimate settlement of the building will be negligible and will not be discernable if the concrete ramps and slabs are reinforced and scored with crack organizers in accordance with standard practice. The CPTs cover two relatively small areas of the building’s total footprint (about 21,300 square feet) so there may be areas discovered where relatively soft soils may be encountered during excavation, perhaps even during shoring operations before excavation. The project’s geotechnical engineer may make determinations for modifications to the foundation design during or after excavation, if necessary.

The mat slab between stiffening beams should be placed over at least 8 inches of imported crushed rock aggregate base (e.g. Class 2) for cushion/working surface, subgrade preparation, capillary break, and drainage (at least during construction). Raft beams may be poured neat without being underlain by aggregate. The rough raft must be designed to provide permanent support for the cantilevered reinforced concrete garage walls.

Waterproofing (2016 CBC §1805), warranted, of walls and slabs to provide a “bathtub spec” is the sole responsibility of the contractor, not any architect or engineer. Architect should ensure that the construction contract for the project provides for adequate long term performance of the waterproofing. After the basement walls are completed, a 6 inch topping (finish) slab should be integrated with the rough mat slab and stiffening beams below. There must be a dampproofing/waterproofing system between the rough slab and the aggregate using an approved membrane system (e.g. Stego®, Griffolyn®) installed per manufacturer’s specifications. The contractor, not the architect or any engineer, has sole responsibility for the sufficiency of the required dampproofing/waterproofing to ensure all interior spaces will stay permanently dry for a period without maintenance decided by the architect and owner.

The mat slab will become a raft when connected to the stiffening grade beams below the mat by integration of all concrete components. Reinforcing penetrations must be sealed for waterproofing. The raft should be:

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A. placed over about 8 inches of Class 2 aggregate base or other similar crushed rock aggregate base approved, before import, by the project's geotechnical engineer;

B. placed on a membrane to provide a moisture barrier; membrane should be covered with 1 to 1½ inches of slightly moist sand, spread evenly, to assist in preventing puncture damage during construction and curing after concreting. This recommendation is provided because it is considered by some to be standard practice, however it assumes extreme care in placing the sand and maintaining its placement during concreting (sharp rocks, reinforcing bar supports, and concrete screed supports that penetrate the membrane must not be used);

C. proportioned and reinforced per minimum geotechnical requirements regardless of code minimums: Slab shall be 6 inches thick. Concrete mix must contain a minimum of 5½ sacks of Portland cement per cubic yard, water/cement ratio no greater than 0.50 by weight.

D. reinforced with a minimum of #5 bars at 8 inches on center, both ways, top and bottom with the bars positively positioned and held, concrete must be mechanically vibrated;

E. articulated on the surface into panels (using wet struck, not saw cut, “crack organizer” joints) with individual areas configured (preferred score locations may be shown on the plans) without reentrant corners that are no larger than about 64 square feet and additionally shall not have any dimension exceeding about 8 feet.

Where slabs are on grade, such as ramps, prior to placing of aggregate base and underdrainage, ground surfaces (which must be approved before placing aggregate) shall be compacted and proof-rolled to provide a smooth, firm surface for support, and then thoroughly soaked. Reinforcing shown on the structural drawings (subject to geotechnical minimums specified above to control bending and cracking), using positively supported deformed bars (no mesh), must be placed both ways top and bottom in the center of the mat slab. Actual amount and spacing of mat reinforcement should be in accordance with the service loadings and spans anticipated subject to the minimum amount of steel specified above regardless of Code minimum requirements for “shrinkage and temperature” steel.

The system should be constructed as a grid, all grade beams should be designed according to structural requirements. In addition, any footings located adjacent to other footing excavations or utility trenches must also be constructed with the closest edge of their bearing surfaces below an imaginary 1.5 horizontal to 1 vertical (1.5h:1v) plane projected upward from the closest edge of the bottom of the adjacent excavation (into undisturbed competent soil) or other footing. Foundations must contain minimum flexural reinforcing \( \rho_{min} = 200b_wd_f, \) whether or not an interpretation of the building code section (ACI 318 §10.5.1) deems the requirement applicable to foundations. Where necessary, foundations shall be stepped in accordance with provisions of the 2016 CBC.

**Maximum Bearing Pressures**

Foundation system must be designed for an allowable bearing pressure of 4,000 psf plus one-third for seismic. Although no bearing foundations other than the raft are anticipated, all such foundation elements should be proportioned so that allowable vertical loads subject to these allowable values being increased by one-third for environmental (wind/seismic) loads are not exceeded, as follows:

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Allowable Bearing Pressures

<table>
<thead>
<tr>
<th>Footing Depth (feet)</th>
<th>Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>-Below lowest point of original adjacent grade-</td>
<td>-Undisturbed Soil-</td>
</tr>
<tr>
<td></td>
<td>-Dead + Live-</td>
</tr>
<tr>
<td>1.5</td>
<td>1,500</td>
</tr>
<tr>
<td>2.0</td>
<td>2,000</td>
</tr>
<tr>
<td>2.5</td>
<td>2,500</td>
</tr>
</tbody>
</table>

Groundwater

Observations and a measurement into a grated collection box for drainage around the existing Taco Bell building in February 2017 found the GWT was at about 9 feet. In the hand augered boring in March 2017 the groundwater table was not found at 5 feet. In May 2017 the GWT was at about 7 feet from the surface of the concrete parking lot pavement.

Lateral Loads

Basement Retaining Walls

CANTILEVER ("unrestrained") retaining walls (and basement walls) must be designed to resist lateral earth pressures plus additional lateral pressures that may be caused by surcharge (construction and/or service, pages 15-16) loads applied at the ground surface behind the walls. Unrestrained walls retaining level or sloped surfaces should be designed for the active state to resist the following:

<table>
<thead>
<tr>
<th>Surface Slope Retained Soil (h:v)</th>
<th>Surface Slope Angle (degrees)</th>
<th>Equivalent Fluid Pressure (psf/ft-depth)</th>
</tr>
</thead>
<tbody>
<tr>
<td>level</td>
<td>0</td>
<td>35</td>
</tr>
<tr>
<td>5:1</td>
<td>11.3</td>
<td>40</td>
</tr>
<tr>
<td>4:1</td>
<td>14</td>
<td>45</td>
</tr>
<tr>
<td>3:1</td>
<td>18.4</td>
<td>50</td>
</tr>
<tr>
<td>2:1</td>
<td>26.6</td>
<td>55</td>
</tr>
<tr>
<td>1.5:1</td>
<td>33.7</td>
<td>70</td>
</tr>
<tr>
<td>1:1</td>
<td>45</td>
<td>85</td>
</tr>
</tbody>
</table>

Although not anticipated, design for walls with other backslope inclinations may use values derived by straight line interpolation of nearest values. Fill slopes steeper than 2h:1v are not allowed (2016 CBC Appendix J §107.6) and unsupported cut slopes (without retaining walls) steeper than 2h:1v are also not allowed (2016 CBC Appendix J §106.1). Equivalent Fluid Pressures given are unfactored; a minimum factor of safety of 1.5 must be provided for sliding and overturning (2016 CBC §1807.3.3).
The lateral pressure design values given above are predicated on the use of free draining backfill materials installed with sufficient drainage being provided behind the walls to prevent the build-up of hydrostatic pressures from surface and subsurface water infiltration; a drainage wedge against an excavated face inclined 4h:7v (approximately 30° from vertical) should be placed behind all walls if the recommended equivalent fluid pressures are relied upon. Site retaining walls free of the building should be constructed with hand dug pit (pier) foundations using a shape factor of the ratio of the diagonal dimension to the smallest side of the pit applied to “h” (2016 CBC §1807.3.2.1).

REstrained retaining walls and restrained foundation or basement walls having level and inclined backslopes may be designed for the appropriate equivalent fluid pressures given above providing the pressure diagram is converted (for the same height) from triangular distribution of equivalent fluid pressure (EFP) to rectangular distribution after reducing the abscissa of the diagram by 35% (multiply by 0.65). This mathematical conversion has the appropriate effects of redistributing the stresses, increasing the overall design load, and raising the height of the resultant.

For design of restrained retaining components, the abscissa of an appropriate lateral pressure diagram for restrained loading (rectangular distribution) is developed for this site using values for expected backfill of $\gamma_w = 122$pcf and $\phi_w = 30^\circ$. Therefore, as the active pressure coefficient $K_a = \tan^2(45^\circ-\phi/2) = 0.33$, the abscissa of the diagram for earth pressure is $0.625K_a\gamma_wH = 25H$ psf/ft of horizontal length. Consistent with 2016 CBC §1803.5.11, the value used for design should be 30H which includes 5H due to theoretical earthquake motions although retaining walls move with their backfill. This approach is equivalent to providing a factor of safety of 1.5 to 20H pursuant to 2016 CBC §1807.2.3.

Frictional resistance for restraint to sliding may only be taken at a full value of $\mu = 0.25$ for bearing soils that are predominately sandy; for silts and clays, no more than one-half of the imposed load may be used to calculate resistance due to friction (2016 CBC §1806.2). Friction should not be used to calculate resistance to seismic forces.

Near-surface overburden soils at the site have a high expansion potential and tendency to "creep". In addition to other loads, continuous foundation elements parallel to the modest slope contours and founded in natural soils should be designed to resist a uniform lateral pressure of 250 psf/ft depth acting against the net projected area of concrete. However, as excavation for the underground garage will remove the potentially expansive soil, therefore creep loads may be neglected. All new retaining walls, cantilever or restrained, should be of reinforced concrete founded on reinforced concrete footings or pits designed and constructed in general compliance with the above specifications or, where not noted for retaining walls, as noted herein for building foundations.

As with all other foundation construction, possible ramp footings or pit (hand-dug pier) depths and their locations may be adjusted by the project’s geotechnical engineer during construction to account for actual ground conditions that are encountered during excavation or where adverse drainage conditions exist that cannot be permanently corrected. Also, as conditions may be found to vary somewhat across the construction, it is possible that not only the geotechnical recommendations herein may be modified during excavation and construction, but the structural design based on any of the recommendations may be modified as well. This very effective coordination between the intended design and actual construction is sometimes referred to as the “observational” approach (Peck 1969).
All wall foundations adjacent to living areas or in areas where moisture penetration through the walls is not desirable must be protected with membrane waterproofing and dampproofing in compliance with 2016 CBC and as specified herein. As the performance of dampproofing and waterproofing, and other weathertightness and weather requirements for this project are responsibilities of the contractor, the project’s architect and/or any engineer are not responsible for their design or construction.

All retaining wall (and subgrade basement wall) construction must conform to the requirements of the 2016 CBC and current standards, and specifications herein. Testing for concrete compressive strength will not be necessary if specified \( f'_c < 2,500 \) psi, except for hand dug piers which could be considered similar to drilled piers where construction requires special inspection.

All concrete for retaining walls must be placed against undisturbed soil, other approved concrete, or suitable formwork. Plans must be reviewed by the undersigned prior to submittal for permit. All site and foundation construction must be intermittently observed by the project’s geotechnical engineer, and a final report upon completion shall be provided to the City of El Cerrito.

**Surcharge**

Any superimposed loading, service or construction, including retained earth in excess of the recommended tabulated lateral earth pressures shall be considered surcharge and provided for in the design of any retaining structure or foundation. The recommended (page 11) foundation load of 30H includes 5H surcharge load however extra superimposed loading, during construction or in service, may be considered as surcharge and provided for in the design of any future site retaining structure or foundation. Uniformly distributed loads may be considered as equivalent added depth of retained earth. The effects of surcharge loading due to continuous or external isolated loads\(^5\) may be determined by the following formulae or by an equivalent method:

\[
R = \frac{0.3 \cdot P \cdot h^2}{x^2 + h^2}
\]

Location of lateral resultant:

\[
d = x \left[ \left( \frac{x^2}{h^2} + 1 \right) \left( \tan^{-1} \frac{h}{x} \right) - \left( \frac{h}{x} \right) \right], \text{ where:}
\]

\(R = \) Resultant lateral force measured in pounds per foot of wall width.\(^6\)

\(P = \) Resultant surcharge load of continuous or isolated footings measured in pounds per foot of length parallel to the wall.

\(x = \) distance of resultant load from back face of wall, measured in feet.

\(^5\) Loads applied within a horizontal distance equal to the height of a retaining wall stem height, measured from the back face of the wall, shall be considered surcharge.

\(^6\) For isolated footings having a width parallel to the wall less than 3 feet, "R" may be reduced to one-sixth the calculated value. The resultant lateral force “R” shall be assumed to be uniform for the length of footing parallel to the wall, and to diminish uniformly to zero at the distance “x” beyond the ends of the footing. Vertical pressure due to surcharge applied to the top of the wall footing may be considered to spread uniformly within the limits of the stem and planes making an angle of 45 degrees with the vertical.
h = depth below point of application of surcharge loading to top of wall footing in feet.

d = depth of lateral resultant below point of application of surcharge loading in feet.

\[
\frac{h}{x} = \tan^{-1}(x)
\]

Lateral Resistance

Resistance to lateral soil loads for bearing type foundations (footings, mat, raft), integrated and/or with hand-dug foundation ("underpinning") pits (hand dug piers), and shoring, may be provided as follows:

a. the restraint due to passive resistance provided by lateral bearing of the hand-dug piers on soil per 2016 CBC §1807.3.2.1 (non-constrained) or 2016 CBC§1807.3.2.2 (constrained), or

b. transfer of moment into the mat slab, and/or tiebacks or anchors, as follows:

Pits (individual hand-dug piers that are eventually interconnected with each other through other foundation elements, perhaps along the ramp to the garage, if confined in competent soil; passive resistance (soil that may act against the outside or downhill face of the foundation elements) may be used to counter lateral loads.  

Tiebacks or anchors for this project (possible but not probable) along the garage walls of that are grouted in drilled holes (or helical anchors) may be used to resist lateral loads. For grouted anchors, a bond stress value of 1,000 pounds per square foot\(^8\) of contact area between the grout and the projected face of the drilled hole (1.57 kips per linear foot of 6 inch diameter grouted tieback length) may be estimated for each tieback, however no tieback shall be less than 35 feet deep.

Unbonded (not grouted) length of tieback rod shall be the length of the tieback between the "phi line" (\(\phi_{eq} = 30^\circ\)) and the rear of the reinforced concrete pit.  

---

\(^7\) Allowable bearing pressure (to resist vertical loads) in reasonably expected competent sedimentary materials is 4,000 psf plus 20% for each additional foot of depth to a maximum value of 3 times (6,000 psf, unless a lower or higher value for observed conditions is determined by the project’s geotechnical engineer-of-record during excavation of hand-dug pits; deep pits are not anticipated).

\(^8\) Allowable bond stress is 5 psi maximum.

\(^9\) An influence line projected upward at 60° from the bottom of the wall to and past the anchor.
A lateral resistance equal to an equivalent fluid pressure (allowable stress) of 200 psf\(^6\) per foot of depth with an increase of this value for each foot of depth from the soil interface to a maximum value of 15 times (3,000 psf, unless otherwise determined by the project's geotechnical engineer), plus one-third increase for environmental (wind or seismic) loading pursuant to 2016 CBC §1806.1 and a shape factor for the diameter ("b") of the pier consistent with 2016 CBC §1807.3.2.1 (the ratio of the diagonal across an individual rectangular pier divided by the rear side of the pier) subject to the following notes and values:

A. Unless a point of fixity under overburden is determined, the top one foot should be neglected in the calculations for lateral resistance to foundation pier loads and there is adequate distance to adjacent pits and between them;

B. the in-situ soil providing the passive resistance is competent (as verified by the project’s geotechnical engineer-of-record) and undisturbed (concrete is cast neat) or within skip wood lagging (pressure treated for ground contact);

C. friction between the foundation bearing surfaces and the supporting subgrade materials; a coefficient of sliding friction \(\mu = 0.33\) may be used for calculations. This recommendation is providing the bearing surface is predominately sandy soil (for silts and clays, friction shall be limited to one-half the normal force imposed by the foundation or mat on the soil); and

D. passive resistance by soil that may act against the sides of formed footings; a resistance equal to an equivalent fluid pressure of 200 psf/ft-depth as noted above and footnoted (unless a higher value for certain specified conditions is determined by the project’s geotechnical engineer during construction) providing [a] the upper 6 inches are neglected from the effective depth used in the calculations, [b] there is adequate distance to adjacent footings or nearby excavations, and [c] the in-situ soil providing the passive resistance is undisturbed (concrete is poured neat).

If the closest distance between adjacent footings is less than 12 feet in the direction towards the “moving” footing, only a proportion of the calculated passive resistance should be allowed, as follows:

<table>
<thead>
<tr>
<th>Nearest Distance Between Footings (feet)</th>
<th>Proportion of Passive Resistance (percent)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>50</td>
</tr>
<tr>
<td>12 &amp; &gt;12</td>
<td>100</td>
</tr>
</tbody>
</table>

\(^{10}\) Presumptive (allowable stress) lateral bearing for reasonably dense sedimentary materials (adapted from 2016 CBC Table §1806.2).
The above allowable pressures may be increased by one-third for all environmental loading (wind or seismic) and other increases in allowable stresses allowed by the 2016 CBC. These allowable bearing pressures are net values; therefore, the weight of the footings can be neglected for design purposes. Building foundation (basement) walls below grade should be designed as restrained retaining walls for the laterally loaded conditions described in the following section.

Expansive soils exist at the site, however differential foundation movements will be minimized because the surficial soils that swell (in the winter) and shrink (in the summer), will be resisted by a deep rigid foundation system. With a rigid raft foundation system, control of drainage is desirable for several reasons but stability of the site and building will not be dependent alone upon drainage.

Building foundation walls (basement walls) below grade should be designed as restrained retaining walls for the laterally loaded conditions described in a following section. All construction must conform to the minimum requirements of the 2016 California Building Code.

All excavations, fills, forms, and placement of reinforcement for all structural concrete must be approved by the project’s geotechnical engineer prior to concreting. All concrete must be placed against undisturbed soil, other concrete that has been prepared and approved, or suitable formwork, and consolidated (mechanically vibrated). All site and foundation construction must be intermittently observed by the project’s geotechnical engineer and a final report provided to the City of El Cerrito upon completion of the project.

E. Passive resistance (soil that may act against the downhill face of foundation elements) may be used in calculations to counter lateral loads including active soil pressures.

F. A lateral bearing resistance (equivalent fluid pressure) of 150 psf/ft depth, per 2016 CBC Table §1806.2 (to a maximum value of 1,500 psf unless a higher value appropriate for observed conditions anticipated for embedment depth into rock) is determined by the project’s geotechnical engineer during excavation may be used for the purpose of determining constrained or non-constrained embedment depth (to a maximum of 12 feet for drilled piers which are not anticipated) in accordance with 2016 CBC §1807.3.2, providing:

1. Passive resistance begins at bottom of grade beams for calculating resistance to lateral loads on pits;
2. there is adequate distance between pits (“piers”), pits are placed inboard of building corners, pits are not aligned perpendicular to a slope, pit depths are staggered (pit bottoms are not at equal depths); and
3. the in-situ soil providing the passive resistance is undisturbed (concrete is poured neat) and excavations are approved prior to placing reinforcing, and the concrete is mechanically vibrated to fill the excavation, with the pits and other excavations considered “forms” (ACI 318 §5.10.8).

Shoring

Recommendations for design of temporary braced shoring of excavations (ways and means by a Class A General Engineering Contractor) to facilitate placement of membrane waterproofing, drainage, and construction of permanent foundation or retaining walls are as follows:

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Lateral Pressures for Temporary Shoring

\[ H = \text{height in feet} \]

<table>
<thead>
<tr>
<th>Load</th>
<th>Lateral Pressure(^{12})</th>
<th>Condition</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil</td>
<td>20H lb/ft(^2) per ft ht</td>
<td>Restrained</td>
<td>Rectangular(^{13})</td>
</tr>
<tr>
<td>Surcharge</td>
<td>12H lb/ft(^2) per ft ht</td>
<td>Restrained</td>
<td>Rectangular</td>
</tr>
<tr>
<td>Surcharge</td>
<td>8H lb/ft(^2) per ft ht</td>
<td>Unrestrained</td>
<td>Rectangular</td>
</tr>
</tbody>
</table>

**Shotcrete**

The state of the art for building reinforced concrete walls, particularly on the property line, is the use of shotcrete (pneumatically placed concrete). Shotcrete allows concrete to be placed against vertical surfaces (e.g. shored excavations) without forms, and skilled finishers produce a permanent exposed surface that is superior to that of poured-in-place concrete.


Strength shall be per structural design. 2016 CBC §1913.4.3 requires lap splices in reinforcing bars by the noncontact lap splice method with at least 2 inches clearance between bars. Any rebound or accumulated loose aggregate shall be removed from the surfaces to be covered prior to placing the initial or any succeeding layers of shotcrete. Rebound shall not be reused as aggregate. Unfinished placement of shotcrete shall not be allowed to stand for more than 30 minutes unless all edges are sloped to a thin edge. Before placing additional material adjacent to previously applied work, sloping and square edges shall be cleaned and wetted. In-place shotcrete which exhibits sags or sloughs, segregation, honeycombing, sand pockets or other obvious defects shall be removed and replaced. Shotcrete above sags and sloughs shall be removed and replaced while still plastic.

**Earthwork**

A moderate amount of site excavation of natural soils may be necessary to achieve the grades shown on the design drawings, and this may be accomplished by using small sized mechanized equipment. Imported backfill materials (other than drain rock, aggregate base, concrete, and landscaping soil) could be required for replacement of unsuitable materials.

Site excavation shall be carefully coordinated to prevent loss of ground, loss of lateral and subjacent support, and caving of banks. Grades, lateral support, temporary slopes, control of groundwater and other drainage, protection of adjacent property, and all safety precautions, are all sole responsibilities of the Contractor.

\(^{11}\) No consideration of soil strengthening by any means. Lateral pressures are cumulative.

\(^{12}\) Per linear foot of shoring width.

\(^{13}\) Rankine: 30 lb/ft\(^2\)/ft ht EFP (Equivalent Fluid Pressure), 30H active (unrestrained, triangular distribution), 30H(2/3) = 20H (restrained, rectangular distribution).

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The contractor is also solely responsible for over-excavation beyond that required for construction shown on the plans. Replacement of over-excavated or other disturbed soil must include placement of non-expansive materials compacted under the supervision of the project’s geotechnical engineer. It is necessary that the project’s geotechnical engineer be afforded the opportunity to observe the materials in the borrow area prior to importation to verify suitability. Imported soil must be of a low to non-expansive nature that meets the following criteria:

<table>
<thead>
<tr>
<th>Imported Fill Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plasticity Index</td>
</tr>
<tr>
<td>4 - 12 %</td>
</tr>
<tr>
<td>Liquid Limit</td>
</tr>
<tr>
<td>less than 30%</td>
</tr>
<tr>
<td>Passing #200 Sieve</td>
</tr>
<tr>
<td>less than 30%</td>
</tr>
</tbody>
</table>

**Structural Fill**

No significant structural fill is anticipated for this project, however if fills are required for any purpose they must be placed on benches excavated into competent soil, and compacted in thin lifts. No fills more than 2 feet thick should be placed without on-site engineering which will include subdrainage. In the event fill is required for any purpose, for instance replacement of overexcavated soil or grade changes to comply with the final site plan, outline specifications are as follows:

1. No fill slope shall be created over, nor shall any foundations constructed on, existing uncontrolled fills. The project’s geotechnical engineer must observe and approve subgrade conditions prior to installation of any paving or flatwork.

2. Maximum fill excavations should not exceed inclinations parallel to an imaginary plane sloped at a maximum steepness of 2 horizontal to 1 vertical (2h:1v, about 27°) in accordance with 2016 CBC Appendix J §107.6.

3. Temporary slopes should not exceed a vertical face height of five feet without shoring and OSHA permit; slope inclinations may be flattened or steepened (excepting fill slopes) if in the opinion of the project’s geotechnical engineer the quality of materials encountered warrants.

4. All fill material must be approved by the project’s geotechnical engineer. Fill material shall be a soil or soil-rock mixture which is free from organic matter or other deleterious substances.

5. Fill material shall not contain rocks or rock fragments over 6 inches in greatest dimension and not more than 15 percent shall be over 2½ inches. Some rocks may be incorporated into the lower portions of larger fills if the rocks are widely spaced and if the spacing method is approved by the project’s geotechnical engineer.

6. On-site material having an organic content of less than 3 percent by volume is suitable for use as fill in all areas except where non-expansive fill is required below slabs-on-grade. Imported fill shall be non-expansive with a Plasticity Index of 12% or less as noted in the above table.

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7. All fills must be keyed into existing undisturbed natural soils to the satisfaction of the engineer. Subdrainage in addition to foundation or retaining wall backdrainage specified herein may be required and shall be installed as directed by engineer.

8. All fill shall be considered structural. Fill less than 3 feet thick shall be compacted by mechanical means to produce a minimum degree of compaction of 90 percent as determined by ASTM Test Designation D 1557-12. Fill greater than 3 feet in thickness shall be compacted to at least 95 percent relative compaction.

9. Field density tests may be performed in accordance with either ASTM Test D 1556-07 (Sand-Cone Method) or ASTM Test Designation D 6938-10 (Nuclear Method).

10. Location and number of field density tests shall be determined by the engineer. Test results and a record of the engineer's observation verifying compliance with these specifications shall be the basis upon which satisfactory completion of work is judged.

Utility Trench Backfill

Utility trench backfilling in all areas should be carefully controlled; poor compaction may cause excessive settlements and divert subdrainage that may result in soil instability.

11. Minimum compaction of trench backfill is 85% of maximum dry density to within 36 inches of finished subgrade. Between 12 inches and 36 inches below grade, trench backfill shall be compacted to a minimum of 90% of maximum dry density. The upper 12 inches of trench backfill should be compacted to 95% of maximum dry density.

12. The maximum dry density and optimum moisture content of any backfill soils may be required by the project's geotechnical engineer to be determined by ASTM designations (§9 above) and these laboratory results may be compared with field density measurements.

Pavement & Flatwork

A driveway and ramp will access the underground garage. The driveway pavement should be reinforced Portland cement concrete as asphaltic concrete (AC, or "blacktop") has a very poor performance record for sites in clay soil as compared to reinforced concrete, even where only low or moderately expansive soils are present. Reinforced Portland cement concrete pavements in expansive soil are more cost effective as they will last indefinitely.

Pavement

The driveway should be reinforced concrete slab-on-grade subject to the following:

1. Concrete shall be 5½ inches thick minimum, reinforced with #5 deformed bars spaced at 12 inch on center both ways placed in the middle of the section. Add one #5 bar along all edges. Transverse bars should be placed first with the longitudinal bars being the upper layer of reinforcing steel.

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The reinforcing mat should be supported by small concrete blocks ("dobees") cast with wires, to the satisfaction of the project's geotechnical engineer. Minimum lap at splices, intersections, and corners to be 24 inches for all bars regardless of size or bond calculations. [Welded wire mesh is not acceptable unless it is installed as additional steel over the minimum #5 deformed reinforcing specified.]

II. Concrete pavement shall be placed over at least 6 inches of compacted aggregate base rock ("AB") set on competent conditioned and rolled soil as verified by the project's geotechnical engineer, and surfaces shall be inclined away from buildings and slopes. Exterior horizontal concrete surfaces to be non-slip and shall be broken up with "crack organizers" into panels not exceeding a 5 feet dimension in any direction. Aggregate should conform to Caltrans [§26-1.02A] Class 2 (3/4" max.), or the following:

<table>
<thead>
<tr>
<th>Aggregate Grading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sieve Size</td>
</tr>
<tr>
<td>1 inch</td>
</tr>
<tr>
<td>3/4 inch</td>
</tr>
<tr>
<td>No. 4</td>
</tr>
<tr>
<td>No. 30</td>
</tr>
<tr>
<td>No. 200</td>
</tr>
</tbody>
</table>

Aggregate base (crushed rock) should be as produced by Kaiser Sand & Gravel, Pine Hollow Road, Clayton CA 94517; phone [plant] (925) 672-4955 & fax [office] (925) 426-4198, or Syar Industries, Lake Herman Plant, Lake Herman Road, Vallejo, CA 94590; phone [plant] (707) 643-3261 & fax [office] (707) 257-2630.

III. Aggregate base must be placed on flat, prepared, and approved subgrade soils. For areas of native subgrade are unsuitable, those areas should be over-excavated and soil replaced with at least two 8 inch thick (before compaction) lifts of non-expansive controlled fill, moisture conditioned to slightly above optimum water content, and compacted as structural fill specified herein or as directed by the project's geotechnical engineer.

Flatwork

Placement of flatwork directly on any potentially expansive soils must be avoided.

IV. Flatwork (including pavement) should be divided into sections by constructing with wet-cut joints that will provide "crack organizers" for relief due to shrinkage and temperature stresses. When fresh, the concrete surface must be protected from exposure to summer sun, wind, and all concrete must be carefully cured.

V. Soil surface must be moisture conditioned and proof rolled prior to installing the aggregate base ("AB", as noted for pavements but base thickness may be 4 inches). The base for asphaltic concrete pavements should be compacted at a moisture content sufficient to obtain 95 percent relative compaction in all lifts.

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Care should be taken to obtain full compaction throughout the width and depth of the layer within the paved area, including a 3 foot wide perimeter strip outside the paving. Approval must be obtained from the project’s geotechnical engineer prior to pouring of concrete. Exterior flatwork, soil, and landscaped areas must slope to drain away from the building.

VI. Concrete flatwork to be 4½ inches thick minimum, reinforced with #4 deformed bars spaced at 12 inch centers both ways placed in the center. Horizontal concrete surfaces to be non-slip. All reinforcing shall be positively supported by steel chairs or small concrete blocks cast with wires, to the satisfaction of the Engineer. All flatwork should be scored with “crack organizers” to form panels with no dimension larger than 4 feet.

Drainage

Stabilization of the site improvements may be greatly improved by improving existing drainage and installing a proper drainage system for the new building. The architectural site plan (JoeDeCredicoStudio 2017) should be supplemented by a civil engineering site plan. All surface water from the roof and ground surfaces that surround the building should be properly collected and discharged. Subsurface water must be intercepted, collected, and discharged to avoid foundation movements associated with ground deformations due to softening and impairment of lateral support. Even though the foundation system will provide rigidity for the building, permanent seasonal stabilization of the site will be greatly improved by installing and maintaining a proper perimeter backdrainage and surface drainage. Design and construction of the project shall include collection of all roof and other surface drainage, and discharge of the collected stormwater into the local existing storm drainage system.

All piping must be rigid ABS or PVC (SDR-26 sewer pipe or Schedule 40 piping), buried. Corrugated tubing is absolutely prohibited for any purpose on this project.

Recommendations for drainage are as follows:

SURFACE WATER. The roof must be guttered. All roof gutters and scuppers shall drain into downspouts that are collected by tight lines, sloped towards exit and buried in well-tamped soil. Roof and other surface water may be collected in the same pipe, but subsurface water must be carried by separate piping (which may be placed in the same trench) to a point of discharge to approved stormwater facilities such as through the curb and into the street gutter along San Pablo Avenue.

All tight lines (non-perforated piping) to daylight in the natural watercourse or watershed channel and shall exit upon non-erodible surfaces. All drainage construction must be tested for operation efficiency.

All storm water from roof and ground surfaces that surround the building must be properly collected and discharged. The ground surface around the building should be graded or otherwise established to provide a positive drainage gradient away from the buildings of at least 2½ percent for a distance of at least 8 feet. Surface water so directed should be collected in catch basins specified by the project’s geotechnical engineer or surface ditches and carried in a system of non-perforated pipes to a point of suitable discharge.

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SUBSURFACE WATER. All parallel-to-contour (perpendicular to slope), and other perimeter foundations, grade beam walls, foundation walls, and retaining walls must be continuously backdrained by subdrainage facilities consisting of perforated RIGID pipe, sloped to discharge and buried in filter rock. Solid walled piping ("tight") lines must transport groundwater collected by perforated piping (and all roof and other surface drainage collected in catch basins and transported by tight lines) from the subsurface collection facilities to approved stormwater collection facilities.

Continuous subdrainage facilities away from any backdrained perimeter foundations are not anticipated, but if determined to be necessary during construction by the project's geotechnical engineer they should be installed in a gravel (drain rock) filled trench, lined with geotextile (filter fabric) and containing perforated rigid piping, and the piping should extend along locations recommended by the project's geotechnical engineer. About 4 inches of clean drain rock should be placed in the cleaned and lined trench, and then perforated rigid pipe laid over the rock (the pipe placed with perforations facing down so the water rises into the pipe).

Lengths of pipe from the trench to the discharge point should form a "tight" line (solid walled rigid pipe, no perforations). Rounded gravel (drain rock) should be used to fill the trench to about one foot from the ground surface, with a soil plug over the folded geotextile to complete the assembly.

The perforated pipe in the bottom of the trench will act as a subdrain and will stabilize nearby groundwater levels at the "invert" (lowest point inside the pipe); the pipe (placed with perforations down) should slope to drain about one-quarter inch per foot (2½ inches rise in any 10 foot run).

Before fully filling the trench, the upper part of the trench may be used to carry a separate rigid "tight" line (solid walled pipe) which may be used to transport roof and other surface water away from the building. Tight lines may join the solid walled section of the subdrainage piping between the building area and the point of discharge, which must be into existing (long term) stormwater collection facilities.

Cuts for retaining walls, made at an inclination of about 60°, should be lined with geotextile and a perforated drain pipe (perforations placed down) installed in the base of the cut over at least 6 inches of drain rock. All drain pipe, perforated and "tight" line, should slope to drain, and should be fitted with cleanout risers at the high-point of each line. The remainder of excavations behind retaining walls should be filled with gravel (drain rock) to within 12 inches of the surface, then after wrapping the geotextile over the drain rock, a plug of the native clayey soil should be placed and tamped to bring the grade up with the high point necessary for adequate surface drainage.

Outline specifications for the filter fabric, pipe, and drain rock materials are as follows:

1. Geotextile (filter fabric) should be "Trevira Spunbond" Type 011/140, 4 ounce per square yard (140 grams per square meter), 100% non-woven (needlepunched) continuous filament polyester engineering fabric, resistant to ultraviolet light exposure and biological and naturally encountered chemicals.

   Geotextile should be as manufactured by Hoechst Celanese Corporation, Technical Fibers Group, P. O. Box 5887, Spartanburg SC 29304; phone (800) 845-7597. Sales representative is ATCO, 4025 Nelson Avenue, Concord CA 94520; phone (925) 686-4430.

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2. All drain pipe must be RIGID bell (expanded or pre-welded collar) and spigot smooth wall PVC (polyvinyl chloride) gravity sewer pipe, PVC Schedule 40 pipe, or ABS pipe. Corrugated tubing must not be used on this project. Recommended RIGID pipe is 4 inches in diameter PVC SDR-26 gravity sewer (meeting ASTM D3034 and ASTM F679) for subsurface drains and collection lines.

3. Cleanouts should be through drop and junction boxes rather than risers with threaded plugs. Where used to collect subsurface seepage, the pipe should be PVC Schedule 40 perforated with one-half inch round holes spaced at 5 inch centers, staggered, in rows 120° apart. When installed, the holes in the piping must face down.

Pipe should be as manufactured by J-M Eagle, 1051 Sperry Road, Stockton CA 95206. Local distributor is Pace Supply, 425 Market Street, Oakland CA 94607; phone (510) 318-6900 or (877) 689-7223 & fax (510) 318-6794; or Contech Construction Products Inc., Roseville CA 95678 [sales engineer is John Lewis, 1864 Genoa Court, Livermore, CA 94550; cell phone (925) 292-0666].

4. Drain rock should be 3/4 inch, well-graded, and clean. River-run gravels, NOT crushed rock, should be used in subdrainage assemblies. Open graded drain rock must be separated from the soil with suitable geotextile. Alternatively, Caltrans [§68-1.025] Class 2 Permeable Material (Sand Equivalent ≥ 75) or 3/8 inch pea gravel may be used without geotextile separation between the gravel and the soil; HOWEVER the pipe itself must be wrapped with geotextile fabric to prevent gravel from being lost through the perforations.

Filter (drain) rock may be obtained as follows: 3/4” drain rock (should be river run gravel, not crushed rock). Pea Gravel, and Class 2 Permeable are available from Hansen Aggregates Mid-Pacific, 4501 Tidewater Avenue, Oakland CA 94601; phone [office](510) 261-8532 & fax (510) 534-7418, [plant] (510) 261-8532 & fax (510) 534-7415 or 3/4” drain rock (must be river run, not crushed). Pea Gravel, and Class 2 Permeable are available from RMC Pacific Materials, 1544 Stanley Boulevard, Pleasanton CA 94566; phone (925) 846-2824 & fax (925) 226-2112.

Catch basins shall be installed where shown on the architectural site plan or as required by the project’s geotechnical or civil engineer in the field. Outline specifications for catch basins and grates (if not supplied with basins) are as follows:

5. Catch basins to be precast concrete as manufactured by Christy Concrete Products, 44100 Christy Avenue, Fremont CA 95438; phone (800) 486-7070 or (510) 657-7070 & fax (800) 486-6804 or as manufactured by Santa Rosa Cast Products Co., 471 West College Avenue, Santa Rosa CA 95401; phone (707) 546-5016 & fax (707) 571-7768.

Grates or grilles for catch basins, unless provided with the basins or otherwise not available from the basin manufacturer, shall be cast iron as manufactured by AB&I, 7825 San Leandro Street, Oakland CA 94621; phone (510) 632-3467.

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Wet Weather Construction

Although it is possible that construction can proceed during or immediately following the wet winter months, a number of geotechnical problems may occur that may increase costs and cause delays.

The water content of on-site soils will increase during the winter and rise significantly above optimum moisture content for compaction of subgrade or backfill materials. If this occurs, the contractor may be unable to achieve the recommended levels of compaction without using special measures. It is likely that the Contractor will have to:

A. Install and maintain tents and/or temporary drainage facilities;
B. wait until the materials dry enough to become workable;
C. dispose of the wet soils and import dry soils;
D. import crushed rock for use as working surfaces; or
E. intermix lime or cement with native materials to absorb water to achieve workability.

If foundation excavations or utility trenches are open during winter rains, then caving of the excavation or trench walls may occur. Also, if the foundation excavations fill with water during construction or if saturated materials are encountered at the anticipated bottom of the excavation, the excavations, and therefore the depth of the foundation section, may need to be extended to greater depths than would be necessary if dry weather construction took place.

From experience, it has been conclusively determined that increased clean-up costs will occur and there will be greater safety hazards if work proceeds during wet weather.

Plan Review

All construction must conform to and be in accordance with 2016 California Building Code and all other applicable City of El Cerrito laws, ordinances, and regulations. OSHA safety rules must be observed, when applicable, throughout the course of construction, and the contractor is solely responsible for safety unless otherwise agreed with the owner. The project’s geotechnical engineer must have the opportunity for a general review of soil and foundation geotechnical aspects of any plans and specifications prepared by others that may affect the subject work at this site (for instance, landscaping plans) in order that the recommendations herein are incorporated in any other designs and that they are properly interpreted and implemented during construction.

It is the responsibility of the owner’s representatives to make available to the geotechnical engineer copies of the plans in advance of actual construction. Prior to construction, the City of El Cerrito may require that the geotechnical engineer provide a letter stating that he or she will be the inspector of record within 10 days of building permit issuance; failure to provide this verification may result in suspension of the building permit. There are numerous other developmental requirements imposed by the City of El Cerrito and County of Contra Costa which may be subject to change or interpretation at any time.

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Safety

The Contractor shall furnish all materials and equipment required for the work, and shall be responsible for the design of, and the ways-and-means of constructing, all temporary ground and building shoring including but not limited to the use of tieback and anchor equipment. Grades, temporary slopes, control of groundwater, protection of existing improvements off the property, drainage during construction, waterproofing and dampproofing of walls and slabs, and all safety precautions, are the sole responsibilities of the Contractor. The Contractor shall be solely responsible for the safety of all persons at the jobsite during the entire construction period.

Construction Verification

It is recommended that construction of this project be coordinated by an experienced design professional (preferably the project architect), who should consult periodically with the project’s structural and geotechnical engineers.

All foundation and site construction should be observed by a geotechnical engineer (a civil engineer especially qualified in the field of soil mechanics and foundation engineering). All foundation, slab, and superstructure construction should be observed by an experienced architect or civil engineer especially qualified in the field of structural engineering for buildings.

Geotechnical engineering services should be provided during the excavation, drainage, and compaction phases of the work. These observations are to verify compliance with the design concepts, specifications and recommendations and to allow changes in the event that subsurface conditions differ from those anticipated prior to the start of construction. The project’s geotechnical engineer should observe the following aspects of construction:

A. Demolition, site stripping and debris removal, and excavations;
B. subgrade preparation (scarification and recompaction operations) for all purposes, borrow areas for imported fill, and restoration of grade where lowered;
C. placement and compaction of engineered fill and utility or other trench fill;
D. pier drilling and foundation excavations, subgrades, and aggregate bases;
E. building of all components of the building’s foundation system;
F. construction of site retaining walls, backdrainage, stairs, abutments, and flatwork; construction of foundation walls and basement wall backdrainage;
G. construction of surface and subdrainage facilities including the installation of roof downspout discharge piping; and
H. finish and landscape grading around building perimeters, and pavements.

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In order to effectively accomplish these observations, it is recommended that a pre-construction meeting be held to develop procedures for communication throughout the project. It is requested that the Owner’s representative or the Contractor contact the geotechnical engineer at least 48 hours prior to the commencement of any of the construction phases listed above.

The City of El Cerrito will likely require that the project’s geotechnical engineer provide certification that foundation related Special Inspections have been performed in accordance with the 2016 California Building Code and other regulations that may be noted on the permit documents.

Limitations

If the configuration or size of the planned improvements change appreciably, supplemental recommendations for design and construction will be provided. Reference to this report should be cited in full on the site, foundation, and landscape drawings for the project. The analysis and recommendations submitted in this report are based on two current cone penetration tests and reports for a previous project at the site, and exploratory borings or test pits for other housing projects in the area, data from published and private geotechnic literature, and evaluation of the performance of nearby existing buildings. Observations include other structures on nearby properties, evaluation of geotechnical data gathered for the surrounding area, and information gained from local experience. Some variation of the nature and extent of the expected soil conditions across the site will probably become evident during construction, and it may be necessary to modify the recommendations given in this report accordingly.

It is the responsibility of the contractors, designers, or other representatives of the owners, to give timely notice to the geotechnical engineer for inspection of excavations, earthwork, foundation and retaining wall construction, pavements, flatwork, and drainage facilities.

Engineers and/or Architects are not responsible for unauthorized changes made to their design documents [Business & Professions Code §§6735(b), 5536.25(a)] and Engineers and/or Architects do not have responsibility for defective construction on projects where either has stamped and signed design documents but was not hired to observe construction [Business & Professions Code §§6735.1, 5536.25(b)]. The services provided herein consist of professional opinions, conclusions, and recommendations that are made in accordance with the scope of work assigned and generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties, either indicated or implied.

Yours truly,

Lawrence B. Karp

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This report, and the ideas or designs incorporated therein, are instruments of professional service. They are the property of Lawrence B. Karp and they are not to be used in whole or part on any other project without the express written authority of Lawrence B. Karp.
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CPT-1
SITE TO BE EXCAVATED
FOR PROPOSED EIGHT STORY
MULTI-FAMILY HOUSING BUILDING

cement not shown

APN 513-340-059
City of Richmond
City of El Cerrito

cement not shown

elevations vary across the site 0 to 4 feet

CPT-2

concrete curbs
at parking areas, and driveways,
ot shown
(to be removed)

GRAPHIC SCALE

0 40
IN FEET

Base: Grant Deed Recorded December 23, 2016 at OR 2016-0279475

SCALE: 1 INCH = 40 FEET

LAURENCE B. KARP
CONSULTING
GEOTECHNICAL ENGINEER
100 TRES MESAS
ORINDA, CALIFORNIA 94563
(925) 254-1222

SITE PLAN

Baxter Creek Apartments
11965 San Pablo Avenue
El Cerrito, CA

DATE PROJECT FIGURE:
June 2017 21712 A
Site: BAXTER CREEK  
Sounding: CPT-02  
Engineer: L.KARP  
Date: 5/18/17 02:13

Max Depth: 56.759 (ft)  
Avg Interval: 0.322 (ft)

SBT: Soil Behavior Type (Robertson 1990)
The Cone Penetration Test (CPT) data collected are presented in graphical and electronic form in the report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings deeper than 30m, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson (Guide to Cone Penetration Testing, 2015). The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling & Testing Inc. does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software. Some interpretation methods require input of the groundwater level to calculate vertical effective stress.

An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on $q$, $f$, and $u$. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.

---

**Figure SBT (After Robertson et al., 1986)** – Note: Colors may vary slightly compared to plots

---

**CPT KEY**

Baxter Creek Apartments
11965 San Pablo Avenue
El Cerrito, CA

<table>
<thead>
<tr>
<th>DATE</th>
<th>PROJECT</th>
</tr>
</thead>
<tbody>
<tr>
<td>June 2017</td>
<td>21712</td>
</tr>
</tbody>
</table>

**FIGURE:** D
Appendix

Information about Cone Penetration Testing
Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*.

The cone takes measurements of tip resistance ($q_t$), sleeve resistance ($f_s$), and penetration pore water pressure ($u_z$). Measurements are taken at either 2.5 or 5 cm intervals during penetration to provide a nearly continuous profile. CPT data reduction and basic interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored electronically for further analysis and reference. All CPT soundings are performed in accordance with revised ASTM standards (D 5778-12).

The 5mm thick porous plastic filter element is located directly behind the cone tip in the $u_z$ location. A new saturated filter element is used on each sounding to measure both penetration pore pressures as well as measurements during a dissipation test (PPDT). Prior to each test, the filter element is fully saturated with oil under vacuum pressure to improve accuracy.

When the sounding is completed, the test hole is backfilled according to client specifications. If grouting is used, the procedure generally consists of pushing a hollow tremie pipe with a “knock out” plug to the termination depth of the CPT hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.
Gregg 15cm² Standard Cone Specifications

<table>
<thead>
<tr>
<th>Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cone base area</td>
</tr>
<tr>
<td>Sleeve surface area</td>
</tr>
<tr>
<td>Cone net area ratio</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Cone load cell</strong></td>
</tr>
<tr>
<td>Full scale range</td>
</tr>
<tr>
<td>Overload capacity</td>
</tr>
<tr>
<td>Full scale tip stress</td>
</tr>
<tr>
<td>Repeatability</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sleeve load cell</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full scale range</td>
</tr>
<tr>
<td>Overload capacity</td>
</tr>
<tr>
<td>Full scale sleeve stress</td>
</tr>
<tr>
<td>Repeatability</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Pore pressure transducer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full scale range</td>
</tr>
<tr>
<td>Overload capacity</td>
</tr>
<tr>
<td>Repeatability</td>
</tr>
</tbody>
</table>

Note: The repeatability during field use will depend somewhat on ground conditions, abrasion, maintenance and zero load stability.
Cone Penetration Test (CPT) Interpretation

Gregg uses a proprietary CPT interpretation and plotting software. The software takes the CPT data and performs basic interpretation in terms of soil behavior type (SBT) and various geotechnical parameters using current published empirical correlations based on the comprehensive review by Lunne, Robertson and Powell (1997). The interpretation is presented in tabular format using MS Excel. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

The following provides a summary of the methods used for the interpretation. Many of the empirical correlations to estimate geotechnical parameters have constants that have a range of values depending on soil type, geologic origin and other factors. The software uses ‘default’ values that have been selected to provide, in general, conservatively low estimates of the various geotechnical parameters.

Input:
1. Units for display (Imperial or metric) (atm. pressure, \( p_a = 0.96 \) tsf or 0.1 MPa)
2. Depth interval to average results (ft or m). Data are collected at either 0.02 or 0.05m and can be averaged every 1, 3 or 5 intervals.
3. Elevation of ground surface (ft or m)
4. Depth to water table, \( z_w \) (ft or m) – input required
5. Net area ratio for cone, \( a \) (default to 0.80)
6. Relative Density constant, \( C_r \) (default to 350)
7. Young’s modulus number for sands, \( \alpha \) (default to 5)
8. Small strain shear modulus number
   a. for sands, \( S_0 \) (default to 180 for SBTn 5, 6, 7)
   b. for clays, \( C_S \) (default to 50 for SBTn 1, 2, 3 & 4)
9. Undrained shear strength cone factor for clays, \( N_{nt} \) (default to 15)
10. Over Consolidation ratio number, \( k_{oc} \) (default to 0.3)
11. Unit weight of water, \( \gamma_w = 62.4 \) lb/ft\(^3\) or 9.81 kN/m\(^3\)

Column
1. Depth, \( z \) (m) – CPT data is collected in meters
2. Depth (ft)
3. Cone resistance, \( q_c \) (tsf or MPa)
4. Sleeve resistance, \( f_s \) (tsf or MPa)
5. Penetration pore pressure, \( u \) (psi or MPa), measured behind the cone (i.e. \( u_2 \))
6. Other – any additional data
7. Total cone resistance, \( q_t \) (tsf or MPa) \( q_t = q_c + u(1-a) \)

Revised 02/05/2015
<table>
<thead>
<tr>
<th>No.</th>
<th>Parameter Description</th>
<th>Formula/Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>Friction Ratio, $R_t$ (%)</td>
<td>$R_t = \left( \frac{f_s}{q_t} \right) \times 100%$</td>
</tr>
<tr>
<td>9</td>
<td>Soil Behavior Type (non-normalized), SBT</td>
<td>see note</td>
</tr>
<tr>
<td>10</td>
<td>Unit weight, $\gamma$ (pcf or kN/m$^3$)</td>
<td>see note</td>
</tr>
<tr>
<td>11</td>
<td>Total overburden stress, $\sigma_v$ (tsf)</td>
<td>$\sigma_v = \sigma_z$</td>
</tr>
<tr>
<td>12</td>
<td>In-situ pore pressure, $u_o$ (tsf)</td>
<td>$u_o = \gamma_w (z - z_w)$</td>
</tr>
<tr>
<td>13</td>
<td>Effective overburden stress, $\sigma'_v$ (tsf)</td>
<td>$\sigma'_v = \sigma_v - u_o$</td>
</tr>
<tr>
<td>14</td>
<td>Normalized cone resistance, $Q_{nt}$</td>
<td>$Q_{nt} = \left( \frac{q_t - \sigma_v}{\sigma'_v} \right)$</td>
</tr>
<tr>
<td>15</td>
<td>Normalized friction ratio, $F_t$ (%)</td>
<td>$F_t = \left( \frac{f_s}{q_t - \sigma_v} \right) \times 100%$</td>
</tr>
<tr>
<td>16</td>
<td>Normalized Pore Pressure ratio, $B_q$</td>
<td>$B_q = \frac{u - u_o}{(q_t - \sigma_v)}$</td>
</tr>
<tr>
<td>17</td>
<td>Soil Behavior Type (normalized), SBT$_n$</td>
<td>see note</td>
</tr>
<tr>
<td>18</td>
<td>SBT$_{n}$ Index, $l_c$</td>
<td>see note</td>
</tr>
<tr>
<td>19</td>
<td>Normalized Cone resistance, $Q_{mn}$ (n varies with $l_c$)</td>
<td>see note</td>
</tr>
<tr>
<td>20</td>
<td>Estimated permeability, $k_{SBT}$ (cm/sec or ft/sec)</td>
<td>see note</td>
</tr>
<tr>
<td>21</td>
<td>Equivalent SPT $N_{60}$, blows/ft</td>
<td>see note</td>
</tr>
<tr>
<td>22</td>
<td>Equivalent SPT $(N_{1})_{60}$, blows/ft</td>
<td>see note</td>
</tr>
<tr>
<td>23</td>
<td>Estimated Relative Density, $D_r$ (%)</td>
<td>see note</td>
</tr>
<tr>
<td>24</td>
<td>Estimated Friction Angle, $\phi'$, (degrees)</td>
<td>see note</td>
</tr>
<tr>
<td>25</td>
<td>Estimated Young's modulus, $E_t$ (tsf)</td>
<td>see note</td>
</tr>
<tr>
<td>26</td>
<td>Estimated small strain Shear modulus, $G_0$ (tsf)</td>
<td>see note</td>
</tr>
<tr>
<td>27</td>
<td>Estimated Undrained shear strength, $s_u$ (tsf)</td>
<td>see note</td>
</tr>
<tr>
<td>28</td>
<td>Estimated Undrained strength ratio $s_u/\sigma'_v$</td>
<td>see note</td>
</tr>
<tr>
<td>29</td>
<td>Estimated Over Consolidation ratio, OCR</td>
<td>see note</td>
</tr>
</tbody>
</table>

**Notes:**

1. Soil Behavior Type (non-normalized), SBT (Lunne et al., 1997 and table below)

2. Unit weight, $\gamma$ either constant at 119 pcf or based on Non-normalized SBT (Lunne et al., 1997 and table below)

3. Soil Behavior Type (Normalized), SBT$_n$ Lunne et al. (1997)

4. SBT$_n$ Index, $l_c$ 

   
   $l_c = ((3.47 - \log Q_{mn})^2 + (\log F_t + 1.22)^2)^{0.5}$

5. Normalized Cone resistance, $Q_{mn}$ (n varies with $l_c$)

   $Q_{mn} = \left( \frac{(q_t - \sigma_v)}{\sigma'_v} \right)$ and recalculate $l_c$, then iterate:

   - When $l_c < 1.64$, $n = 0.5$ (clean sand)
   - When $l_c > 3.30$, $n = 1.0$ (clays)
   - When $1.64 < l_c < 3.30$, $n = (l_c - 1.64)0.3 + 0.5$

   Iterate until the change in $n$, $\Delta n < 0.01$

Revised 02/05/2015
Estimated permeability, \( k_{SBT} \) based on Normalized SBT\(_n\) (Lunne et al., 1997 and table below)

<table>
<thead>
<tr>
<th>Equivalent SPT ( N_{60} ) blows/ft</th>
<th>Lunne et al. (1997)</th>
</tr>
</thead>
<tbody>
<tr>
<td>[ \frac{(q_p/p_a)}{N_{60}} = 8.5 \left( \frac{1 - I_c}{4.6} \right) ]</td>
<td></td>
</tr>
</tbody>
</table>

Equivalent SPT \( (N_1)_{60} \) blows/ft where \( C_n = (p_a/\sigma'_{vo})^{0.5} \)

<table>
<thead>
<tr>
<th>Relative Density, ( D_r ) (%)</th>
<th>( D_r^2 = Q_{lm} / C_{Dr} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( Only SBT_n 5, 6, 7 &amp; 8 )</td>
<td>Show ‘N/A’ in zones 1, 2, 3, 4 &amp; 9</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Friction Angle, ( \phi' ), (degrees)</th>
<th>[ \tan \phi' = \frac{1}{2.68} \left[ \log \left( \frac{q_c}{\sigma'_{vo}} \right) + 0.29 \right] ]</th>
</tr>
</thead>
<tbody>
<tr>
<td>( Only SBT_n 5, 6, 7 &amp; 8 )</td>
<td>Show ‘N/A’ in zones 1, 2, 3, 4 &amp; 9</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Young's modulus, ( E_s )</th>
<th>( E_s = \alpha q_t )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( Only SBT_n 5, 6, 7 &amp; 8 )</td>
<td>Show ‘N/A’ in zones 1, 2, 3, 4 &amp; 9</td>
</tr>
</tbody>
</table>

Small strain shear modulus, \( G_0 \)

a. \[ G_0 = S_0 (q_t - \sigma'_{vo} p_a)^{1/3} \] For \( SBT_n 5, 6, 7 \)

b. \[ G_0 = C_0 q_t \] For \( SBT_n 1, 2, 3 \& 4 \)

<table>
<thead>
<tr>
<th>Undrained shear strength, ( s_u )</th>
<th>( s_u = (q_t - \sigma'_{vo}) / N_k )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( Only SBT_n 1, 2, 3, 4 &amp; 9 )</td>
<td>Show ‘N/A’ in zones 5, 6, 7 &amp; 8</td>
</tr>
</tbody>
</table>

Over Consolidation ratio, OCR

<table>
<thead>
<tr>
<th>OCR = ( k_{OCR} Q_{11} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( Only SBT_n 1, 2, 3, 4 &amp; 9 )</td>
</tr>
</tbody>
</table>

The following updated and simplified SBT descriptions have been used in the software:

<table>
<thead>
<tr>
<th>SBT Zones</th>
<th>SBT(_n) Zones</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 sensitive fine grained</td>
<td>1 sensitive fine grained</td>
</tr>
<tr>
<td>2 organic soil</td>
<td>2 organic soil</td>
</tr>
<tr>
<td>3 clay</td>
<td>3 clay</td>
</tr>
<tr>
<td>4 clay &amp; silty clay</td>
<td>4 clay &amp; silty clay</td>
</tr>
<tr>
<td>5 clay &amp; silty clay</td>
<td></td>
</tr>
<tr>
<td>6 sandy silt &amp; clayey silt</td>
<td></td>
</tr>
</tbody>
</table>

Revised 02/05/2015
<table>
<thead>
<tr>
<th></th>
<th></th>
<th>5</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>silty sand &amp; sandy silt</td>
<td>silty sand &amp; sandy silt</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>sand &amp; silty sand</td>
<td>sand &amp; silty sand</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>sand</td>
<td>6</td>
<td>sand &amp; silty sand</td>
</tr>
<tr>
<td>10</td>
<td>sand</td>
<td>7</td>
<td>sand</td>
</tr>
<tr>
<td>11</td>
<td>very dense/stiff soil*</td>
<td>8</td>
<td>very dense/stiff soil*</td>
</tr>
<tr>
<td>12</td>
<td>very dense/stiff soil*</td>
<td>9</td>
<td>very dense/stiff soil*</td>
</tr>
</tbody>
</table>

*heavily overconsolidated and/or cemented

Track when soils fall with zones of same description and print that description (i.e. if soils fall only within SBT zones 4 & 5, print 'clays & silty clays')
**Estimated Permeability** (see Lunne et al., 1997)

<table>
<thead>
<tr>
<th>SBT&lt;sub&gt;n&lt;/sub&gt;</th>
<th>Permeability (ft/sec)</th>
<th>(m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$3 \times 10^{-8}$</td>
<td>$1 \times 10^{-8}$</td>
</tr>
<tr>
<td>2</td>
<td>$3 \times 10^{-7}$</td>
<td>$1 \times 10^{-7}$</td>
</tr>
<tr>
<td>3</td>
<td>$1 \times 10^{-8}$</td>
<td>$3 \times 10^{-10}$</td>
</tr>
<tr>
<td>4</td>
<td>$3 \times 10^{-8}$</td>
<td>$1 \times 10^{-8}$</td>
</tr>
<tr>
<td>5</td>
<td>$3 \times 10^{-6}$</td>
<td>$1 \times 10^{-6}$</td>
</tr>
<tr>
<td>6</td>
<td>$3 \times 10^{-4}$</td>
<td>$1 \times 10^{-4}$</td>
</tr>
<tr>
<td>7</td>
<td>$1 \times 10^{-2}$</td>
<td>$1 \times 10^{-2}$</td>
</tr>
<tr>
<td>8</td>
<td>$3 \times 10^{-6}$</td>
<td>$1 \times 10^{-6}$</td>
</tr>
<tr>
<td>9</td>
<td>$1 \times 10^{-8}$</td>
<td>$3 \times 10^{-9}$</td>
</tr>
</tbody>
</table>

**Estimated Unit Weight** (see Lunne et al., 1997)

<table>
<thead>
<tr>
<th>SBT</th>
<th>Approximate Unit Weight (lb/ft&lt;sup&gt;3&lt;/sup&gt;)</th>
<th>(kN/m&lt;sup&gt;3&lt;/sup&gt;)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>111.4</td>
<td>17.5</td>
</tr>
<tr>
<td>2</td>
<td>79.6</td>
<td>12.5</td>
</tr>
<tr>
<td>3</td>
<td>111.4</td>
<td>17.5</td>
</tr>
<tr>
<td>4</td>
<td>114.6</td>
<td>18.0</td>
</tr>
<tr>
<td>5</td>
<td>114.6</td>
<td>18.0</td>
</tr>
<tr>
<td>6</td>
<td>114.6</td>
<td>18.0</td>
</tr>
<tr>
<td>7</td>
<td>117.8</td>
<td>18.5</td>
</tr>
<tr>
<td>8</td>
<td>120.9</td>
<td>19.0</td>
</tr>
<tr>
<td>9</td>
<td>124.1</td>
<td>19.5</td>
</tr>
<tr>
<td>10</td>
<td>127.3</td>
<td>20.0</td>
</tr>
<tr>
<td>11</td>
<td>130.5</td>
<td>20.5</td>
</tr>
<tr>
<td>12</td>
<td>120.9</td>
<td>19.0</td>
</tr>
</tbody>
</table>
Pore Pressure Dissipation Tests (PPDT)

Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals can be used to measure equilibrium water pressure (at the time of the CPT). If conditions are hydrostatic, the equilibrium water pressure can be used to determine the approximate depth of the ground water table. A PPDT is conducted when penetration is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure \( u \) with time is measured behind the tip of the cone and recorded.

Pore pressure dissipation data can be interpreted to provide estimates of:
- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation \( c_h \)
- In situ horizontal coefficient of permeability \( k_h \)

In order to correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until it reaches equilibrium, *Figure PPDT*. This time is commonly referred to as \( t_{100} \), the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992 and Lunne et al. 1997.

A summary of the pore pressure dissipation tests are summarized in Table 1.

**Water Table Calculation**

\[
D_{\text{water}} = D_{\text{cone}} - H_{\text{water}}
\]

where \( H_{\text{water}} = U_e \) (depth units)

*Figure PPDT*
Seismic Cone Penetration Testing (SCPT)

Seismic Cone Penetration Testing (SCPT) can be conducted at various intervals during the Cone Penetration Test. Shear wave velocity (Vs) can then be calculated over a specified interval with depth. A small interval for seismic testing, such as 1-1.5m (3-5ft) allows for a detailed look at the shear wave profile with depth. Conversely, a larger interval such as 3-6m (10-20ft) allows for a more average shear wave velocity to be calculated. Gregg’s cones have a horizontally active geophone located 0.2m (0.66ft) behind the tip.

To conduct the seismic shear wave test, the penetration of the cone is stopped and the rods are decoupled from the rig. An automatic hammer is triggered to send a shear wave into the soil. The distance from the source to the cone is calculated knowing the total depth of the cone and the horizontal offset distance between the source and the cone. To calculate an interval velocity, a minimum of two tests must be performed at two different depths. The arrival times between the two wave traces are compared to obtain the difference in time (Δt). The difference in depth is calculated (Δd) and velocity can be determined using the simple equation: \[ v = \frac{\Delta d}{\Delta t} \]

Multiple wave traces can be recorded at the same depth to improve quality of the data.

A complete reference on seismic cone penetration tests is presented by Robertson et al. 1986 and Lunne et al. 1997.

A summary the shear wave velocities, arrival times and wave traces are provided with the report.
Gregg Drilling & Testing, Inc. conducts groundwater sampling using a sampler as shown in Figure GWS. The groundwater sampler has a retrievable stainless steel or disposable PVC screen with steel drop off tip. This allows for samples to be taken at multiple depth intervals within the same sounding location. In areas of slower water recharge, provisions may be made to set temporary PVC well screens during sampling to allow the pushing equipment to advance to the next sample location while the groundwater is allowed to infiltrate.

The groundwater sampler operates by advancing 44.5mm (1⅛ inch) hollow push rods with the filter tip in a closed configuration to the base of the desired sampling interval. Once at the desired sample depth, the push rods are retracted; exposing the encased filter screen and allowing groundwater to infiltrate hydrostatically from the formation into the inlet screen. A small diameter bailer (approximately ⅜ or ⅝ inch) is lowered through the push rods into the screen section for sample collection. The number of downhole trips with the bailer and time necessary to complete the sample collection at each depth interval is a function of sampling protocols, volume requirements, and the yield characteristics and storage capacity of the formation. Upon completion of sample collection, the push rods and sampler, with the exception of the PVC screen and steel drop off tip are retrieved to the ground surface, decontaminated and prepared for the next sampling event.

For a detailed reference on direct push groundwater sampling, refer to Zemo et. al., 1992.
Soil Sampling

Gregg Drilling & Testing, Inc. uses a piston-type push-in sampler to obtain small soil samples without generating any soil cuttings, Figure 55. Two different types of samplers (12 and 18 inch) are used depending on the soil type and density. The soil sampler is initially pushed in a "closed" position to the desired sampling interval using the CPT pushing equipment. Keeping the sampler closed minimizes the potential of cross contamination. The inner tip of the sampler is then retracted leaving a hollow soil sampler with inner 1⅛” diameter sample tubes. The hollow sampler is then pushed in a locked "open" position to collect a soil sample. The filled sampler and push rods are then retrieved to the ground surface. Because the soil enters the sampler at a constant rate, the opportunity for 100% recovery is increased. For environmental analysis, the soil sample tube ends are sealed with Teflon and plastic caps. Often, a longer "split tube" can be used for geotechnical sampling.

For a detailed reference on direct push soil sampling, refer to Robertson et al, 1998.
Ultra-Violet Induced Fluorescence (UVOST)

Gregg Drilling conducts Laser Induced Fluorescence (LIF) Cone Penetration Tests using a UVOST module that is located behind the standard piezocone, Figure UVOST. The laser induced fluorescence cone works on the principle that polycyclic aromatic hydrocarbons (PAH’s), mixed with soil and/or groundwater, fluoresce when irradiated by ultra violet light. Therefore, by measuring the intensity of fluorescence, the lateral and vertical extent of hydrocarbon contamination in the ground can be estimated.

The UVOST module uses principles of fluorescence spectrometry by irradiating the soil with ultra violet light produced by a laser and transmitted to the cone through fiber optic cables. The UV light passes through a small window in the side of the cone into the soil. Any hydrocarbon molecules present in the soil absorb the light energy during radiation and immediately re-emit the light at a longer wavelength. This re-emission is termed fluorescence. The UVOST system also measures the emission decay with time at four different wavelengths (350nm, 400nm, 450nm, and 500nm). This allows the software to determine a product “signature” at each data point. This process provides a method to evaluate the type of contaminant. A sample output from the UVOST system is shown in Figure Output. In general, the typical detection limit for the UVOST system is <100 ppm and it will operate effectively above and below the saturated zone.

With the capability to push up to 200m (600ft) per day, laser induced fluorescence offers a fast and efficient means for delineating PAH contaminant plumes. Color coded logs offer qualitative information in a quick glance and can be produced in the field for real-time decision making. Coupled with the data provided by the CPT, a complete site assessment can be completed with no samples or cuttings, saving laboratory costs as well as site and environmental impact.
Figure Output
Hydrocarbons detected with UVOST
- Gasoline
- Diesel
- Jet (Kerosene)
- Motor Oil
- Cutting fluids
- Hydraulic fluids
- Crude Oil

Hydrocarbons rarely detected using UVOST
- Extremely weathered gasoline
- Coal tar
- Creosote
- Bunker Oil
- Polychlorinated bi-phenols (PCB's)
- Chlorinated solvent DNAPL
- Dissolved phase (aqueous) PAH's

Potential False Positives (fluorescence observed)
- Sea-shells (weak-medium)
- Paper (medium-strong depending on color)
- Peat/meadow mat (weak)
- Calcite/calcareous sands (weak)
- Tree roots (weak-medium)
- Sewer lines (medium-strong)

Potential False Negatives (do not fluoresce)
- Extremely weathered fuels (especially gasoline)
- Aviation gasoline (weak)
- "Dry" PAHs such as aqueous phase, lamp black, purifier chips
- Creosotes (most)
- Coal tars (most) gasoline (weak)
- Most chlorinated solvents
- Benzene, toluene, zylenes (relatively pure)
Main Plot:
Signal (total fluorescence) versus depth where signal is relative to the Reference Emitter (RE). The total area of the waveform is divided by the total area of the Reference Emitter yielding the %RE. This %RE scales with the NAPL fluorescence. The fill color is based on relative contribution of each channel’s area to the total waveform area (see callout waveform). The channel-to-color relationship and corresponding wavelengths are given in the upper right corner of the main plot.

Callouts:
Waveforms from selected depths or depth ranges showing the multi-wavelength waveform for that depth.

The four peaks are due to fluorescence at four wavelengths and referred to as “channels”. Each channel is assigned a color.

Various NAPLs will have a unique waveform “fingerprint” due to the relative amplitude of the four channels and/or broadening of one or more channels.

Basic waveform statistics and any operator notes are given below the callout.

Conductivity Plot:
The Electrical Conductivity (EC) of the soil can be logged simultaneously with the UVOST data. EC often provides insight into the stratigraphy. Note the drop in EC from 10 - 13 ft, indicating a shift from consolidated to unconsolidated stratigraphy. This correlates with the observed NAPL distribution.

Rate Plot:
The rate of probe advancement. ~ 0.8in (2cm) per second is preferred.

A noticeable decrease in the rate of advancement may be indicative of difficult probing conditions (gravel, angular sands, etc.) such as that seen here at ~5 ft.

Notice that this log was terminated arbitrarily, not due to "refusal", which would have been indicated by a sudden rate drop at final depth.

Info Box:
Contains pertinent log info including name and location.

Note A:
Time is along the x axis. No scale is given, but it is a consistent 320ns wide.
The y axis is in mV and directly corresponds to the amount of light striking the photodetector.

Note B:
These two waveforms are clearly different. The first is weathered diesel from the log itself while the second is the Reference Emitter (a blend of NAPLs) always taken before each log for calibration.

Note C:
Callouts can be a single depth (see 3rd callout) or a range (see 4th callout). The range is noted on the depth axis by a bold line. When the callout is a range, the average and standard deviation in %RE is given below the callout.
**Waveform Signal Calculation**

<table>
<thead>
<tr>
<th></th>
<th>Reference Emitter</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH1</td>
<td>4820</td>
<td>4923</td>
</tr>
<tr>
<td>CH2</td>
<td>8108</td>
<td>5743</td>
</tr>
<tr>
<td>CH3</td>
<td>6249</td>
<td>4166</td>
</tr>
<tr>
<td>CH4</td>
<td>2984</td>
<td>1735</td>
</tr>
<tr>
<td>Total</td>
<td>22161</td>
<td>16587</td>
</tr>
<tr>
<td><strong>Area (pVs)</strong></td>
<td><strong>Percent RE</strong></td>
<td></td>
</tr>
<tr>
<td>CH1</td>
<td>22.3</td>
<td>75%</td>
</tr>
<tr>
<td>CH2</td>
<td>25.9</td>
<td></td>
</tr>
<tr>
<td>CH3</td>
<td>18.8</td>
<td></td>
</tr>
<tr>
<td>CH4</td>
<td>7.8</td>
<td></td>
</tr>
</tbody>
</table>

**Data Files**

<table>
<thead>
<tr>
<th>File Type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>*.lif.raw.bin</td>
<td>Raw data file. Header is ASCII format and contains information stored when the file was initially written (e.g. date, total depth, max signal, gps, etc., and any information entered by the operator). All raw waveforms are appended to the bottom of the file in a binary format.</td>
</tr>
<tr>
<td>*.lif.plt</td>
<td>Stores the plot scheme history (e.g. callout depths) for associated Raw file. Transfer along with the Raw file in order to recall previous plots.</td>
</tr>
<tr>
<td>*.lif.jpg</td>
<td>A jpg image of the OST log including the main signal vs. depth plot, callouts, information, etc.</td>
</tr>
<tr>
<td>*.lif.dat.txt</td>
<td>Data export of a single Raw file. ASCII tab delimited format. No string header is provided for the columns (to make importing into other programs easier). Each row is a unique depth reading. The columns are: Depth, Total Signal (%RE), Ch1%, Ch2%, Ch3%, Ch4%, Rate, Conductivity Depth, Conductivity Signal, Hammer Rate. Summing channels 1 to 4 yields the Total Signal.</td>
</tr>
<tr>
<td>*.lif.sum.txt</td>
<td>A summary file for a number of Raw files. ASCII tab delimited format. The file contains a string header. The summary includes one row for each Raw file and contains information for each file including: the file name, gps coordinates, max depth, max signal, and depth at which the max signal occurred.</td>
</tr>
<tr>
<td>*.lif.log.txt</td>
<td>An activity log generated automatically located in the OST application directory in the 'log' subfolder. Each OST unit the computer operates will generate a separate log file per month. A log file contains much of the header information contained within each separate Raw file, including: date, total depth, max signal, etc.</td>
</tr>
</tbody>
</table>

**Common Waveforms** (highly dependent on soil, weathering, etc.)

- Diesel
- Gas
- Kerosene
- Motor Oil