GEOTECHNICAL INVESTIGATION AND BLUFF STABILITY STUDY
5228 CHELSEA STREET
LA JOLLA, CALIFORNIA

Prepared for
Mr. David M. Lessnick
Las Vegas, Nevada

TerraCosta
Consulting Group

Prepared by
TERRACOSTA CONSULTING GROUP, INC.
San Diego, California

Project No. 2918
July 19, 2016

TerraCosta
Consulting Group

Geotechnical Engineering

Coastal Engineering

Maritime Engineering

Project No. 2918
July 19, 2016

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BLUFF STABILITY STUDY
5228 CHELSEA STREET
LA JOLLA, CALIFORNIA

Dear Mr. Lessnick:

In accordance with your request and our Proposal No. 15138 dated February 16, 2016, TerraCosta Consulting Group, Inc. (TerraCosta) has performed a geotechnical investigation and bluff stability study for the proposed construction of a single-family residence to be located at 5228 Chelsea Street in La Jolla, California.

The accompanying report presents the results of our field investigative work, laboratory testing, and engineering analyses of the subsurface conditions at the site, and presents our conclusions and recommendations pertaining to the geotechnical aspects of site development.

We appreciate the opportunity to work with you on this project, and trust this information meets your present needs. If you have any questions or require further information, please give us a call.

Very truly yours,
TERRACOSTA CONSULTING GROUP, INC.

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WFC/GAS/jg
Attachments

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GEOTECHNICAL INVESTIGATION AND BLUFF STABILITY STUDY **5228 CHELSEA STREET** LA JOLLA, CALIFORNIA

1 INTRODUCTION AND PROJECT DESCRIPTION

The subject property is located at 5228 Chelsea Street, near the intersection with Crystal Drive in the La Jolla area of San Diego, California (see Vicinity Map, Figure 1, attached). As we understand, the proposed project consists of demolition of an existing house and site improvements, and construction of a new two-story house with a basement and side-yard swimming pool. The site is located atop a southwesterly facing coastal bluff, which descends approximately 70 feet from the top-of-bluff, down to the Pacific shoreline. The Site Plan (Figure 2, attached) and Generalized Geologic Cross Section (Figure 3, attached) show the general topographic and geologic conditions at the site. Figure 2 also indicates the approximate footprint of the planned residential structure.

1.1 Background

The subject site and surrounding area was part of the U.S. military's Bird Rock Coastal Defense and Anti-Aircraft Training Center during World War II and up until the 1950s, when residential properties began to encroach on the facility. Evidence of the old coastal defense facilities can still be seen in the bluffs today, and our review of documents indicates that the area was subdivided and redeveloped into residential lots in the mid to late 1950s.

2 PURPOSE AND SCOPE OF WORK

The purpose of our study is to provide geotechnical information to assist you and your consultants in project design, and to address City of San Diego and California Coastal Commission concerns regarding the proposed project.

For input in performing our studies and preparing this report, we have reviewed geologic literature, maps, historic aerial stereographic and oblique photographs, and other relevant reports and documents in our files. References are provided at the end of this report.

In particular, our investigation is designed to address the following geotechnical issues:

- The geologic setting of the site;
- Potential geologic hazards;
- Geotechnical characteristics of the on-site soils;
- Groundwater;
- Proposed site grading;
- Foundation design, including allowable soil bearing and earth pressure values;
- On-site and off-site surface water drainage;
- Construction-period stability of cut slopes;
- Gross stability of the coastal bluff, including the location of the 1.5 factor of safety line; and
- Predicted bluff retreat over the next 75 years.

3 FIELD INVESTIGATION AND LABORATORY TESTING

A limited geologic reconnaissance was performed on the subject site and immediately adjacent areas. Our subsurface investigation included the excavation of three 6-inch-diameter hollow-stem auger borings to depths ranging from 6.5 feet to 9 feet. The auger borings were advanced using a limited-access tripod-mounted drill rig. The locations of the auger borings are shown on the Site Plan, Figure 2. A Key to Excavation Logs is presented in Appendix A as included on Figure A-1. Final logs of the test borings are presented as Figures A-2 and A-3. Mapping of the bluff face provided additional data to aid in characterizing the geologic site conditions.

Selected representative samples were tested in the laboratory to classify and evaluate the engineering properties of the on-site soils. Laboratory tests were performed to establish moisture/density relationships, grain size analyses, and strength characteristics. The results of our laboratory testing are presented in Appendix B.

4 GEOLOGY AND SITE CONDITIONS

4.1 Geologic Setting

The coastal plain of San Diego County is characterized by thick sequences of interbedded Eocene and Cretaceous marine siltstones, claystones, sandstones, and conglomerates upon which younger Quaternary-age deposits rest. Coastal bluff retreat, a geomorphic process that has operated for millions of years and continues today along most of San Diego's coastline, has formed steep coastal bluffs ranging up to as high as 300 feet in elevation in San Diego County.

Locally, the project site is situated at the westerly bluff-terminated edge of an approximately 1/2-mile-wide gently westerly-sloping coastal terrace, one of a sequence of well-defined wave-cut abrasion terraces created primarily by higher eustatic sea stands during Pleistocene-age interglacial episodes and, to a lesser degree, by tectonic uplift.

4.2 Site Conditions

The subject 100-foot-wide by 83- to 93-foot-deep property is bounded on the east by Chelsea Street, on the north and south by adjoining residential lots, and on the west by the Pacific shoreline. Based on our review of 1953 aerial photographs, this surface was altered during the grading and construction of the Sun Gold Point residential development. Our review of public records indicates that the house was built in 1951.

From the top-of-bluff, the upper coastal bluff (underlain by local fill soils and Quaternary terrace deposits) descends seaward at an average angle of approximately 2 degrees down to approximate elevation of 70 feet (MSLD) at the top of the near-vertical cliff-forming Mount Soledad Formation. As indicated on Figure 3, the lower cliffed part of the coastal bluff is underlain by the relatively erosion-resistant Mount Soledad Formation.

4.3 Subsurface Conditions

Four soil and geologic units (the lower-bluff Mount Soledad Formation, the upper-bluff late Pleistocene terrace deposits, transient beach deposits, and surficial fill soils) are present in the general site area. These soil units are described below from oldest to youngest.

Mount Soledad Formation: The lower cliff-forming Mount Soledad Formation is a predominantly massive medium-grained sandstone (TMss). The Mount Soledad Formation is mapped as dipping 10 degrees to the east in the area of the project site.

Terrace Deposits: The moderately consolidated, poorly indurated, light reddish-brown, silty to clayey fine sands, underlain by cobble conglomerate, are characteristic of late Pleistocene-age coastal terrace deposits. These old paralic deposits are exposed in the upper bluff above approximate elevation 62 feet. Soils within this generally medium dense to dense geologic unit include nearshore marine and beach deposits, locally interfingered with colluvial soils shed from the hillsides.

Beach Deposits: Unconsolidated transient beach deposits consisting of sands, gravels, and cobble are found at the base of the bluff. The deposits are estimated to be on the order of 4 to 6 feet near the base of the bluff.

Fill Soils: Locally derived fill soils cap the upper 1 to 5 feet of the lot. These soils were likely placed during finish grading of the lots in the late 1940s and early 1950s.

5 GEOLOGIC HAZARDS

5.1 Faulting and Seismicity

The site is located at 32.807° North latitude and 117.263° West longitude, in a moderately-active seismic region of Southern California that is subject to significant hazards from moderate to large earthquakes. Ground shaking from ten major active fault zones could affect the site in the event of an earthquake. The nearest of these, the northerly offshore extension of the Rose Canyon fault zone, trends north-northwest and has been mapped approximately 2.2 miles east-northeast of the site. No known active faults have been mapped on or near the property. A small older inactive fault is located approximately 100 feet north of the site.

5.2 Landslides

Landslides have not been mapped as being present, both on or immediately adjacent to the site. However, bluff failures are known to occasionally occur in the area. These failures have generally occurred because of heavy localized seepage of groundwater, by uncontrolled runoff over the face of the bluff, or by marine erosion acting at the base of the coastal bluff.

6 GROUNDWATER

Groundwater was not encountered in our test borings. However, it should be noted that perched groundwater seepage has been observed along the contact between the terrace deposits and the underlying formational soils at other locations in the general project site area. Groundwater seepage was specifically noted north of the fault located approximately 100 feet north of the subject site.

7 COASTAL ENVIRONMENT

The site is located within the northern portion of the Mission Bay Littoral Cell and is characterized by a rocky sea cliff-bounded shoreline with a few small sandy pocket beaches (U.S. Army Corps of Engineers [USACE], 1988). The Mission Bay littoral cell is an area of sand movement along the coast bounded by Point La Jolla to the north and Point Loma to the south, a distance of approximately 13.5 miles. Under natural conditions, a littoral cell is supplied with sediment by rivers and streams that empty into the ocean within its limits. The sandy material brought to the coast by fluvial action is then incorporated into the beach sands and transported south (in most areas) along the coast by wave action. This longshore transport of sand is ultimately intercepted by a submarine canyon or other sink, where it is diverted offshore and lost to the nearshore environment. The Mission Bay Littoral Cell is primarily supplied with sediment by the San Diego River (USACE, 1988). Because there is no significant source of sand north of the site, the local beaches are comprised primarily of gravel, cobble, and boulder conglomerate.

7.1 Wave Climate

Waves provide nearly all of the energy input that drives shoreline processes along the California coast. As illustrated in Figure 4 (below), incoming waves along the southern California coast fall into three main categories: Longer period northern and southern hemisphere swell, and locally short-period generated seas. North hemisphere swell from the North Pacific Ocean dominate the winter wave conditions off California, while southern hemisphere swell is more important in the summer. Short-period seas are produced by storms sweeping through the area. The offshore islands, shallow banks, submarine canyons and generally complex bathymetry of southern California greatly complicate the wave climate at the coast (Figure 5, below).

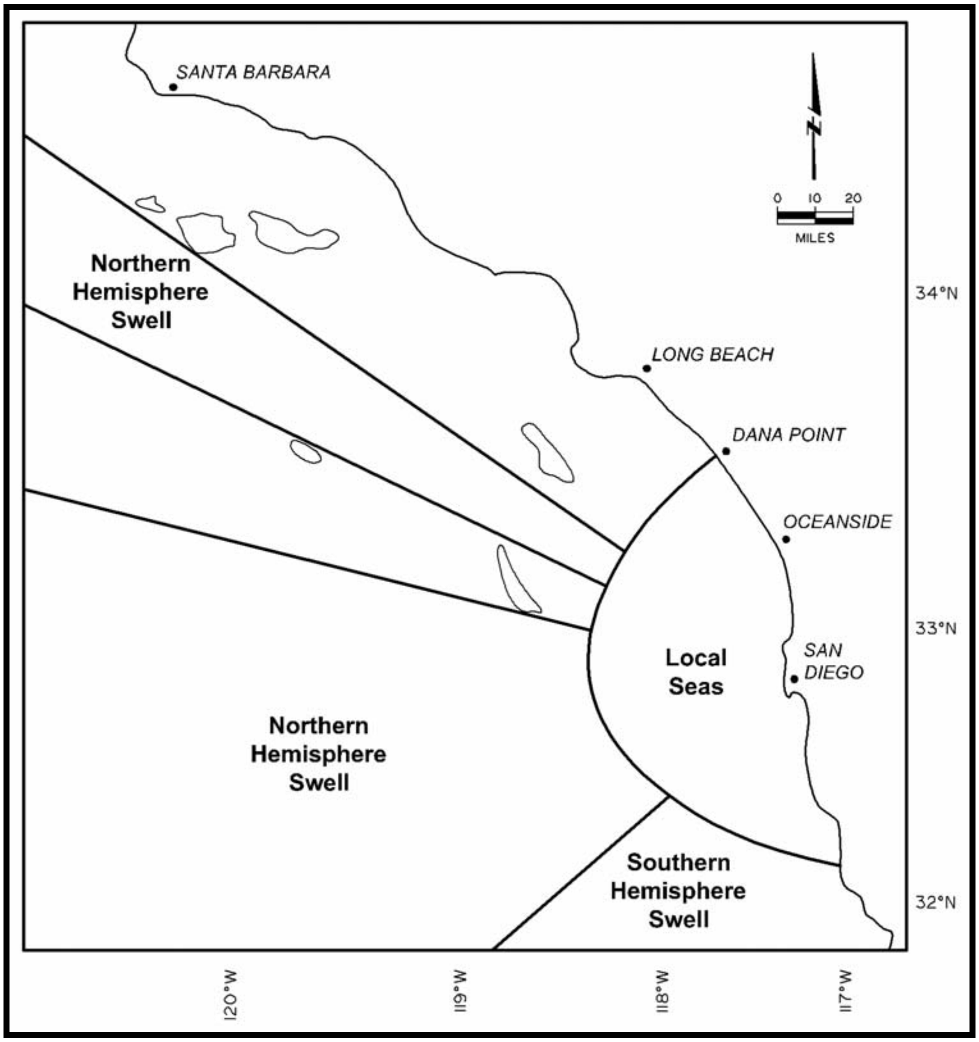

Figure 4. Map Showing Generalized Wave Exposure for Southern California.

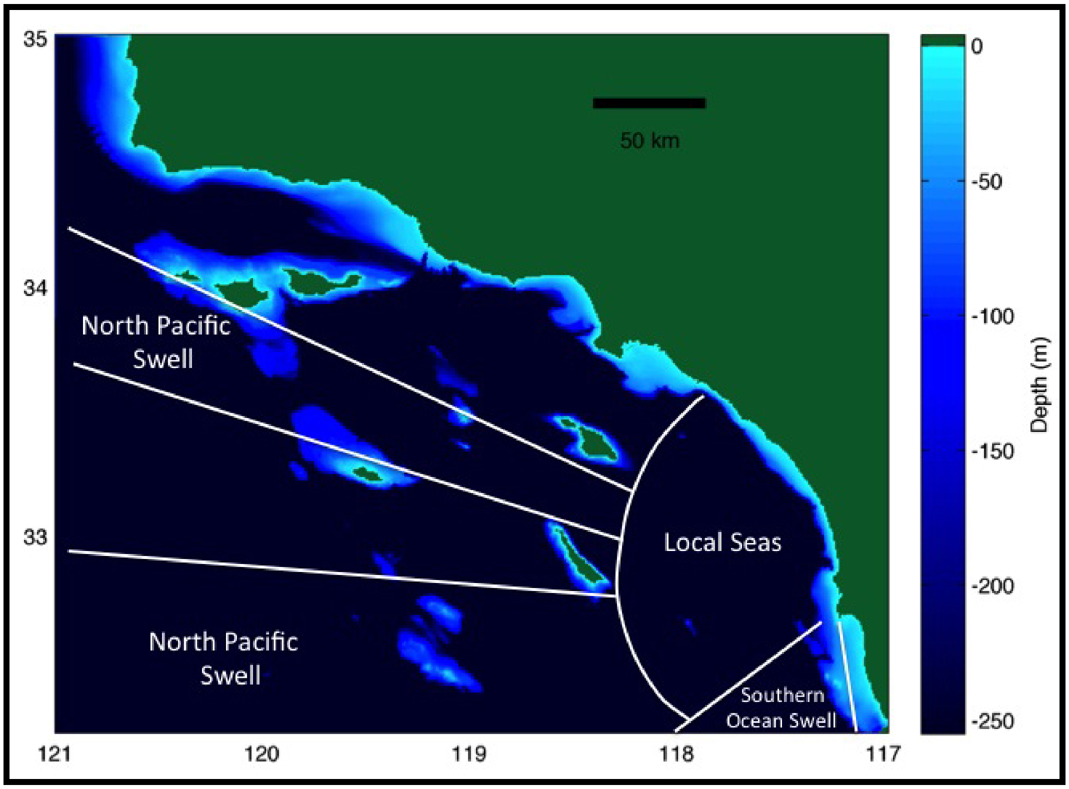

Figure 5. Map Showing Generalized Bathymetry in the Southern California Bight and Wave Exposure Windows at Oceanside.

Coastal orientation, and the islands and banks greatly influence the swell propagating toward shore by partially sheltering southern California, including La Jolla, especially from directions north of west. Figure 5 (above) shows the approximate directions from which incoming swell is blocked by the islands. The coastline fronting the subject site faces south and is therefore also exposed to southern hemisphere swell. Because of the complicated effects of bathymetry and island shadowing, the wave height at the shoreline is sensitive to relatively small changes in the incoming direction of the deep ocean waves.

While waves along the San Diego County shoreline generally range in height from 2 to 5 feet, deep water waves off the coast have been recorded with deep water significant wave heights approaching 10 meters (33 feet).

7.2 Short-Term Sea Level Change

The effect of waves on the coast is highly dependent on the sea level during the wave episode. Large waves at low sea level cause limited erosion, since they break well offshore. When episodes of large waves combine with short-term high sea level from tides and other factors, rapid retreat may occur along vulnerable coastlines.

7.2.1 *Tides*

Tides are caused by the gravitational pull of astronomical bodies; primarily the moon, sun, and planets. Tides along the San Diego coast have a semi-diurnal inequality. On an annual average basis, the lowest tide is about 1.6 feet (MLLW datum) and the highest tide is about 7.1 feet, MLLW datum.

7.2.2 *El Niño*

Large-scale, Pacific Ocean-wide warming periods occur episodically and are related to the El Niño phenomenon. These meteorological anomalies are characterized by low atmospheric pressures and persistent onshore winds. During these events, average sea levels in southern California can rise up to 0.5 foot above normal. Tidal data indicates that six episodes (1914, 1930 through 1931, 1941, 1957 through 1959, 1982 through 1983, and 1997 through 1998 - mild El Niño-type conditions were also reported in 1988 and 1992) have occurred since 1905. Further analysis suggests that these events have an average return period of 14 years, with 0.2-foot tidal departures lasting for two to three years.

The added probability of experiencing more severe winter storms during El Niño periods increases the likelihood of coincident storm waves and higher storm surge. The record water level of 8.35 feet, MLLW, observed in San Diego Bay in January 1983, includes an estimated 0.8 foot of surge and seasonal level rise (Flick and Cayan, 1984), which set the stage for the wave-induced flooding and erosion that marked that winter season.

7.3 Sea Level Rise

Past and possible future changes in mean sea level (MSL) are of interest in design and planning for all coastal cities, as well as for any engineering activities on the coast. Global mean sea level rose at least 300 feet, and perhaps as much as 400 feet, during the past 18,000 years or so (CLIMAP, 1976). Sea level, both globally and along California, rose approximately 0.7 foot over the past century, as shown in Figure 6 (below). Furthermore, evidence suggests that the rate of global mean sea level rise has accelerated since the mid-1800s, or even earlier (Church and White, 2006; Jevrejeva, et al., 2008), and that it has now reached a rate of about 1 foot per century over the past decade or so (Nerem, et al., 2006).

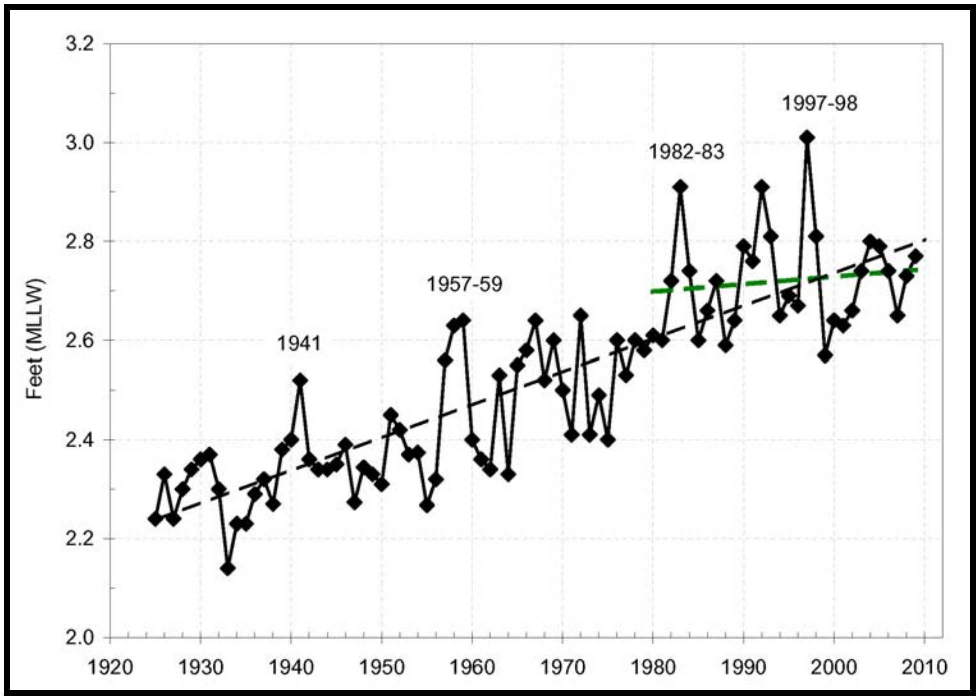

Figure 6. Annual Average Sea Level History at La Jolla, 1925-2007. Broken Line Shows Linear Trend of 0.7 Feet/Century Rise.

Figure 6 is a plot of the annual mean sea levels measured at the La Jolla tide gauge starting in 1925. The linear trend indicates the approximate 0.7 foot per century sea level rise. Also noticeable are the enhanced sea levels during the El Niño episodes of 1941, 1957-59, 1982-83, and 1997-98 (respectively labeled).

A notable feature of the sea level history at La Jolla is the leveling-off of sea level rise since about 1980 (Figure 6, above). The green broken line shows a much reduced trend of about 0.15 foot per century between 1980 and 2009, or about 4.5 times smaller than the overall trend of 0.67 foot per century. A similar reduction in the rate of sea level rise has been noted at San Francisco, which has a similar overall appearance as the La Jolla record, but is a much longer record extending back to 1856.

Figure 7 (below) shows the global distribution of the rate of sea level change for the period of 1993-2012 (University of Colorado, 2012). Note that warm colors (yellow-orange-red) show areas of sea level rise (positive rates), while cool colors (green- blue) indicate falling sea level (negative rates) over the record. Inspection of the North Pacific reveals that sea levels in the western Pacific, especially in the lower latitudes, have risen at a rate of 3-9 mm/year (equivalent to 30-90 cm per century, or about 1-3 feet per century). Conversely, sea levels in the eastern Pacific, extending from Central America north to Washington State, have fallen at a rate of 0-3 mm per year (0-30 cm per century, or 0-1 foot per century). This may explain the coastal tide gauge observations (La Jolla sea level history; Figure 6, above) described above.

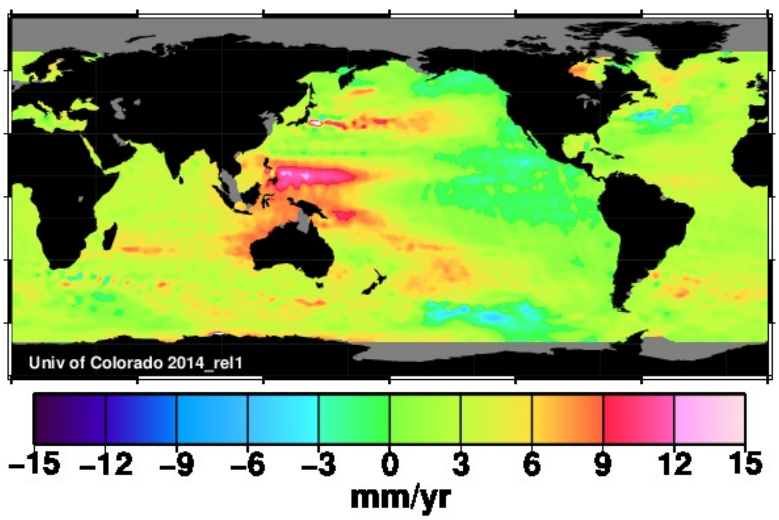

Figure 7. Global Sea Level Change Rates 1993-2012 as derived from satellite altimetry measurements, following University of Colorado (2012).

While the cause of these regional differences undoubtedly lies in the large-scale circulation of the Pacific Ocean and the overlying atmosphere, no detailed explanation is known. However, these observations could be a cause for some concern. If the conditions driving sea level up in the western Pacific and down in the eastern Pacific were to relax or even reverse, sea level along the coast of California could begin to increase at a much higher rate than what has been observed over the past several decades. Future global sea level rise scenarios could further increase the rate of sea level rise.

7.4 Water Levels

Past water elevations are based on the tide gauge data from La Jolla, which has been collected at Scripps Institution of Oceanography (SIO) Pier since 1924. These data are applicable to the San Diego region open-ocean coastline. The tidal and geodetic reference relationships at La Jolla are illustrated in Figure 8 (below).

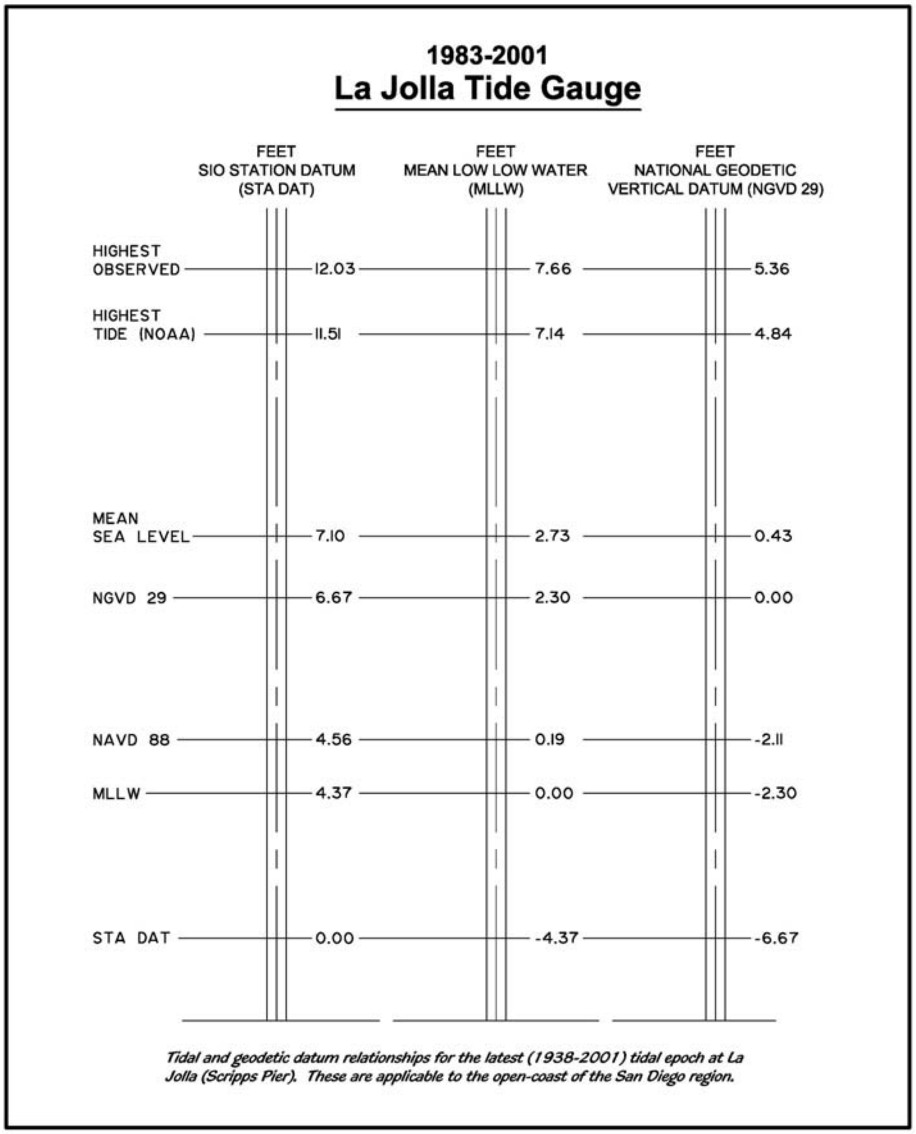

Figure 8. Sea Level Datums.

Tide gauges measure total water level outside the breaker zone, which includes contributions from the tide, as well as storm surges and other factors that raise sea level over the short and long term, including the effects of El Niño. All non-tide sea level influences measured by the tide gauges are termed “non-tide residuals, or “NTR.” Importantly, tide gauges do not include the effects of waves or wave-driven runup. At the shoreline and on beaches, wave-driven runup is a crucial component of the design water elevation and must be determined by means other than tide gauge data. Alternatively, as the back beach becomes flooded during high tide and low beach sand level events, the standard runup formulations may not apply, and other factors, including local shallow-water depth-limited waves, must be considered.

When considering the effects of future sea level rise, the National Academy of Sciences (NAS, 2012) presents a possible global, west-coast, and state-wide future Mean Sea Level Rise (MSLR) for California, Oregon, and Washington (Figure 9, dots) and its range (Figure 9, bars). These are based on the IPCC (2007) mid-range Green House Gas emissions scenarios for the ocean steric (warming) expansion component added to the results of new research projecting the likely contributions of future ice-melt. The resulting projected *global* MSLR relative to 2000 ranged from 0.08-0.23 m (0.26-0.75 ft) by 2030; 0.18-0.48 m (0.59-1.6 ft) by 2050; and 0.50-1.4 m (1.6-4.6 ft) by 2100 (Figure 9, red bars). The global estimates were adjusted for vertical crustal movement (uplift north of Cape Mendocino and down-drop in the south) resulting in the orange bars, also shown in Figure 9 (below). The State of California (2013) used these results of NAS (2012) shown as the updated MSLR guidance in Table 1 (below).

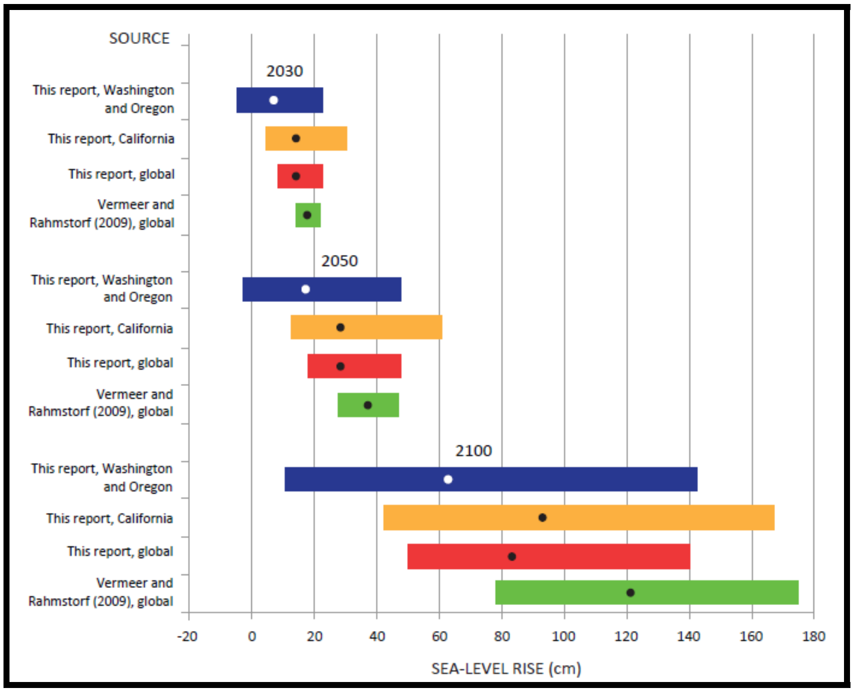

Figure 9. NAS (2012) summary of global, Washington, Oregon, and California (south of Cape Mendocino) MSLR projections for 2030, 2050, and 2100 relative to 2000.

Table 1. Updated MSLR Guidance from State of California (2013)

Time Period	North of Cape Mendocino[3]	South of Cape Mendocino
2000 - 2030	-4 to 23 cm (-0.13 to 0.75 ft)	4 to 30 cm (0.13 to 0.98 ft)
2000 – 2050	-3 to 48 cm (-0.1 to 1.57 ft)	12 to 61 cm (0.39 to 2.0 ft)
2000 – 2100	10 to 143 cm (0.3 to 4.69 ft)	42 to 167 cm (1.38 to 5.48 ft)

8 BLUFF EROSION

This section of coastline is characterized by steep coastal bluffs comprised of relatively erosion-resistant Eocene- and Cretaceous-age strata (Mount Soledad or Cabrillo Formations) at the base of the bluff and a less resistant upper bluff (terrace deposits), with a narrow cobble beach at the base of the bluff. The bluff in the project area is located in a medium to high energy wave environment subject to direct wave impact.

8.1 Lower Bluff Erosion

Review of historical photographs dating back to the 1970s does not reveal a great deal of long-term lower bluff erosion in the general area of this site. Based on our review of historical photographs, we estimate that on the order of 5 to 10 feet of erosion has occurred in the last 45+ years. Younger Eocene-age formations to the north, Solana Beach for example, exhibit erosion rates on the order of 0.4 foot per year. The slightly older Mount Soledad Formation would be expected to have a similar or lower erosion rate, likely on the order of 2 to 3 inches per year.

8.2 Empirical and Analytical Techniques of Erosion Rate Assessment

The scientific community has been actively engaged in developing numerical models to assess rates of shoreline erosion. Numerical models attempt to address both the landward retreat of the sea cliff and the development of the shore platform. In this simplest expression, predictive cliff-erosion models take the following form (Sunamura, 1977):

$$dx/dt \propto \ln\left(\frac{f_w}{f_r}\right)$$

where dx/dt is the horizontal rate of erosion, f_w is the wave force, and f_r is the rock resistance. Similar equations have been developed to describe platform development.

Of particular interest in numerical modeling is the fact that a minimum or critical wave height capable of causing erosion exists, below which, for a given rock lithology, no erosion would occur. Additionally, the rate of erosion increases in logarithmic proportion to increase in wave force, which is substantially less than a linear increase in wave energy. Importantly, however, these numerical models describe the mechanical erosion of intact rock of assumed uniform lithology, and do not account for the accelerated erosion caused by the hydrodynamic component of wave forces that occurs in fractured rock.

When using the preceding equation, and when comparing the site conditions with San Diego's North County Tertiary cliff-forming sediments, the wave force (f_w) is likely similar for the subject site and North County San Diego. Importantly, however, the erosion resistance of the rock (f_r) is likely stronger for the earlier Eocene sediments than for the younger Eocene sediments. This suggests both a more severe storm wave to initiate erosion of the sea cliff, and a corresponding reduction in marine erosion for a given design wave event from the early Eocene sediments than from the North County Tertiary sediments. Thus, one would again conclude that, in the absence of more data, the annualized average erosion rate for the site would be on the order of 2 to 3 inches (0.17 to 0.25 foot) per year, given the more well-defined erosion rate of the later Eocene sediments of 4.8 inches (0.4 foot) per year.

9 SLOPE STABILITY

9.1 Soil Conditions

In order to assess the stability of the upper bluff, slope stability analyses were performed using the following soil strengths.

Fill:
φ = 30 degrees
c = 100 psf
γ_t = 120 pcf

Terrace Deposits:
φ = 33 degrees
c = 300 psf
γt = 120 pcf

Mount Soledad Formation (conglomerate), TMsc:
φ = 33 degrees
c = 1000 psf
γt = 120 pcf

Mount Soledad Formation (sandstone), TMss:
φ = 35 degrees
c = 3500 psf
γt = 120 pcf

9.2 Slope Stability Analyses

The stability of the coastal bluff was evaluated using the computer software GSTABL7. GSTABL7 is a graphical program that uses limit equilibrium theory to compute the factor of safety for earth and rock slopes. The Modified Bishop Method was selected for analyses of the subject slope (see Figures 2 and 3).

Slope stability analyses indicate that the existing static factor of safety with regard to slope stability is greater than 1.5, with a computed factor of safety of 2.33 for Section 1 (Figure 3). Under pseudo-static conditions corresponding to a horizontal seismic coefficient of 0.15 g, the slope has a computed factor of safety greater than 1.1, with a computed factor of safety of 1.96 for Section 1. As such, from both a static and pseudo-static perspective, the slope is considered stable. Summary results of the stability analyses are included in Appendix C.

10 BLUFF-TOP SETBACK REQUIREMENTS

The City of San Diego uses three criteria for evaluating bluff-top setbacks behind which structures may be located. Depending upon the stability of the bluff, the City requires a minimum bluff-top setback of either 40 feet for unstable bluffs or 25 feet for bluffs that have been demonstrated as being stable. Given that the slope in question is stable, the City requires a minimum 25-foot bluff-top setback. In addition, the City requires consideration be given to the minimum setback that would be required to accommodate 75 years of annualized bluff retreat, which in this area we estimate to be 18.75 feet. For this site, we estimate the controlling bluff-top setback to be from the minimum 25-foot setback line.

11 GEOTECHNICAL CONCLUSIONS AND RECOMMENDATIONS

11.1 General

Our investigation did not reveal the presence of any unmitigated adverse geologic conditions on the site, such as faults, adverse bedding, or a high groundwater table, that might preclude development of the currently-proposed new construction.

11.2 **Proposed Site Grading**

We anticipate that the site preparation and earthwork operations for the project will include:

- Demolition of existing structures;
- Clearing and grubbing;
- Removal and recompaction of soils for the support of structural elements, such as the new house, walkways and area flatwork (patios, etc.), pavements, retaining walls; and
- Excavation for foundations and the basement.

We recommend that all grading and site preparation be performed under the observation of the geotechnical engineer, and in accordance with the attached specifications for engineered fill (Appendix D). In addition, we recommend that vegetation, trash, rubble, and other deleterious material be removed from the site prior to grading. All loose and porous topsoil, residual soils, slopewash, and uncontrolled fill soils not removed by the grading operations should be excavated and removed prior to placing additional fill or structural elements. We recommend that the geotechnical engineer confirm the actual depths and extent of removal of unsuitable materials in the field at the time of grading. Based on the results of our exploratory borings and laboratory testing, the deposits of unsuitable materials requiring overexcavation generally range from 1 to 6 feet in depth. As we understand, the proposed building foundations and swimming pool will be below the depths of any unsuitable overburden soils founded on competent terrace formational soils.

Any excavations resulting from utility removals should be properly backfilled, and the backfill compacted in accordance with the specifications provided in Appendix D. Utility trenches under foundations or pavements should be backfilled with material that provides similar stiffness as adjacent areas. In these cases, cement-sand slurries may be warranted, depending on adjacent soil properties.

We recommend that all fill soils be compacted to a minimum density of 90 percent of the maximum dry density, as determined by ASTM Test Method D 1557. Moisture content should be maintained between the optimum moisture content and 3 percent above optimum. We recommend that the geotechnical engineer review the foundation and grading plans to verify that the intent of the recommendations presented herein has been properly interpreted and incorporated into the contract documents. We further recommend that the geotechnical engineer observe the site grading, foundation excavations, construction of retaining walls, and subgrade preparation under concrete slabs and paved areas.

If construction proceeds through the rainy winter months, we recommend that adequate surface drainage be provided to drain water away from any open excavations.

11.3 **Building Foundations**

11.3.1 *Bearing Capacity*

For foundations having a minimum width of 12 inches, and founded a minimum of 18 inches below finished final grade, we recommend an allowable net bearing pressure of 2,000 psf. In addition, we recommend that adjacent footings not be founded above an imaginary plane extending upward at an angle of 45 degrees from the bottom outside edge of an adjacent lower footing. Additionally, we recommend that all footings be adequately reinforced as recommended by a structural engineer experienced with the design of shallow foundation systems. Footing excavations should be cleared of any loose material prior to concrete placement. Lastly, we recommend that the geotechnical engineer inspect all footing excavations.

11.3.2 *Settlement*

We estimate that footings loaded to an allowable bearing pressure of 2,000 psf will settle approximately 1/2 inch or less, with differential settlements on the order of 1/4 inch or less.

11.3.3 *Lateral Resistance*

To provide resistance for design lateral loads of footings and shear keys poured neat against vertical excavations, we recommend using an equivalent fluid pressure of 300 or 450 pcf for properly compacted granular fill or competent formational materials, respectively. These values assume a horizontal surface for the soil mass extending at least 10 feet from the face of the footing or three times the height of the surface generating the passive pressure, whichever is greater. The upper 12 inches of soil in areas not protected by floor slabs or pavements should not be included in design for passive resistance to lateral loads.

If friction is to be used to resist lateral loads, we recommend a coefficient of friction of 0.45 between soil and concrete for either compacted fill or formational soil. If it is desired to combine friction and passive resistance in design, we recommend reducing the friction coefficient by 25 percent.

11.3.4 *Concrete Slabs-on-Grade*

We recommend that concrete slabs-on-grade be designed in accordance with the CBC and the American Concrete Institute's (ACI) Committee Report No. 360. In addition, we recommend that the construction of concrete slabs-on-grade conform to the guidelines and specifications presented in ACI Committee Report No. 302.

11.3.5 *Pipes and Trenches*

Open or backfilled trenches, which are generally aligned in parallel with a footing shall not be below a plane having a downward slope of 1 unit vertical to 2 units horizontal (50% slope) from a line 9 inches above the bottom edge of the footing and not closer than 18 inches form the face of such footing.

Where pipes cross under footings, the footings shall be specially designed. Pipe sleeves shall be provided where pipe crosses through footings or footing walls, and sleeve clearances shall provide for possible footing settlement, but not less than 1 inch.

11.3.6 *Water- and Damp-Proofing Foundation Systems*

As a minimum, we recommend that the basement walls, along with all concrete slabs and foundation systems for the proposed structures be waterproofed and/or damp-proofed in accordance with Chapter 18, Section 1805, of the 2013 CBC.

11.4 **Retaining Walls**

For cantilevered retaining walls that are free to rotate through a horizontal movement of at least 0.002H at the top of the wall (where H is the height of the wall in feet), we recommend the following.

We recommend providing all retaining walls with a backfill drainage system adequate to prevent buildup of hydrostatic pressures.

For cantilevered retaining walls with level backfill, and which retain granular soils that comply with the material requirements of Section 300-3.5 (Structure Backfill) of the Standard Specifications for Public Works Construction (SSPWC), and that extend a minimum distance equal to 80 percent of the height of the wall, we recommend a design lateral earth pressure equivalent to a fluid pressure of 35 pcf. The on-site soils are sandy in nature and, in general, should comply with the requirements of Section 300-3.5 of the SSPWC.

For cantilevered retaining walls with a 2:1 (horizontal to vertical) backfill slope, which retain granular soils that comply with Section 300-3.5, and extend a minimum distance equal to 80 percent of the height of the wall, we recommend a design lateral earth pressure equivalent to a fluid pressure of 55 pcf.

Cantilevered retaining walls subject to vehicular loads (including the garage floor slab) should be designed to resist an equivalent fluid pressure for the active case described above, plus a surcharge load equal to an additional 2 feet of height of equivalent backfill.

We recommend that walls restrained from movement at the top, such as basement walls, be designed for the active case equivalent fluid pressure given above plus an additional uniform load of 8H psf for granular backfill materials in the backfill prism (that zone of soil extending upward and outward on a 0.8 to 1 plane from the bottom outside edge of the retaining wall footing).

Partially restrained retaining walls can be designed for a load reduction if they can be assumed to deflect. The additional uniform pressure that is added to the active condition equivalent fluid pressure should vary linearly from 8H psf uniform pressure to 0 as the calculated deflection at the top of the wall varies from 0 to 0.002H.

For strip footings supporting the proposed retaining walls, we recommend an allowable bearing pressure of 3,000 psf for footings founded a minimum of 6 inches into competent formational soils, and 2,000 psf for footings founded in compacted fill soils. In addition, all footings should be founded a minimum of 18 inches below adjacent ground surface. This recommendation also assumes that the footings will be founded on and within properly compacted fill soils, or on and within competent formational soils.

Resistance for design lateral loads of retaining wall footings should be in conformance with Section 11.3.3.

11.5 Construction Cuts and Excavations

The removal and recompaction of existing fill and formational soils will require construction cuts and excavations. We recommend that construction cuts and excavations comply with the CALOSHA and OSHA recommendations and guidelines.

For all excavations and construction cuts adjacent to, or near, existing buildings, we recommend that lateral support for the existing structures be maintained either by shoring said excavations, or that the existing buildings be underpinned. This may mean that exploratory test pits may have to be excavated in order to assess the depth and type of the existing adjacent building foundation.

The sides of all unshored excavations may be sloped no steeper than 1.5:1 (horizontal to vertical), provided that:

1. The excavation is at least 18 inches out from the face of existing footings; and

2. The excavation does not extend below a plane inclined downward at 2:1 (horizontal to vertical) from a line 9 inches above the bottom edge of the existing footing.

12 LIMITATIONS

Geotechnical engineering and the earth sciences are characterized by uncertainty. Professional judgments presented herein are based partly on our evaluation of the technical information gathered, partly on our understanding of the proposed construction, and partly on our general experience. Our engineering work and judgments rendered meet the current professional standards. We do not guarantee the performance of the project in any respect.

We have investigated only a small portion of the pertinent soil, rock, and groundwater conditions of the subject site. The opinions and conclusions made herein were based on the assumption that those rock and soil conditions do not deviate appreciably from those encountered during our field investigation. We recommend that a soil engineer from our office observe construction to assist in identifying soil conditions that may be significantly different from those encountered in our borings. Additional recommendations may be required at that time.

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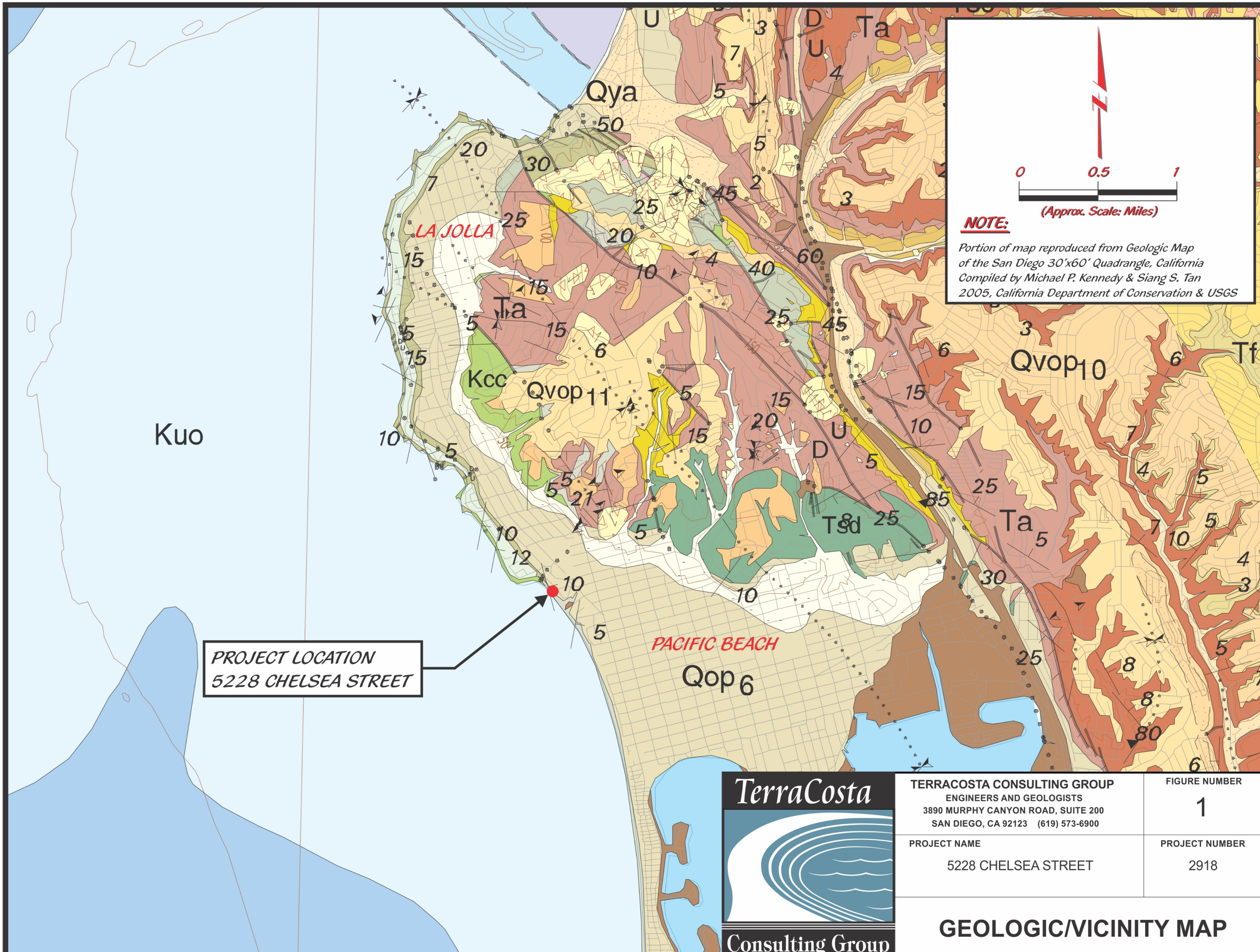

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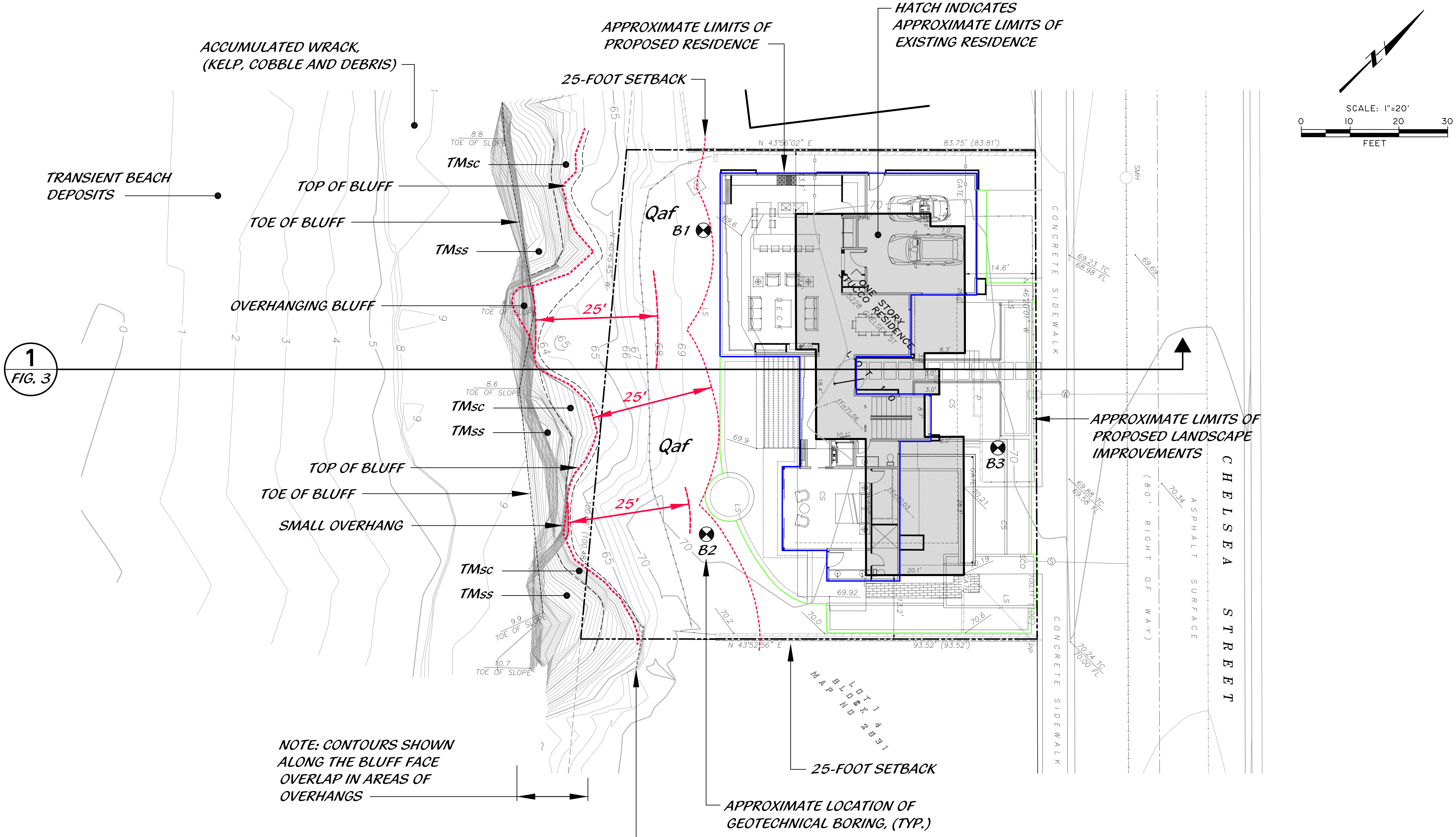
3890 MURPHY CANYON ROAD, SUITE 200
SAN DIEGO, CA 92123 (619) 573-6900

FIGURE NUMBER	PROJECT NAME	PROJECT NUMBER
1	5228 CHELSEA STREET	2918

GEOLOGIC/VICINITY MAP

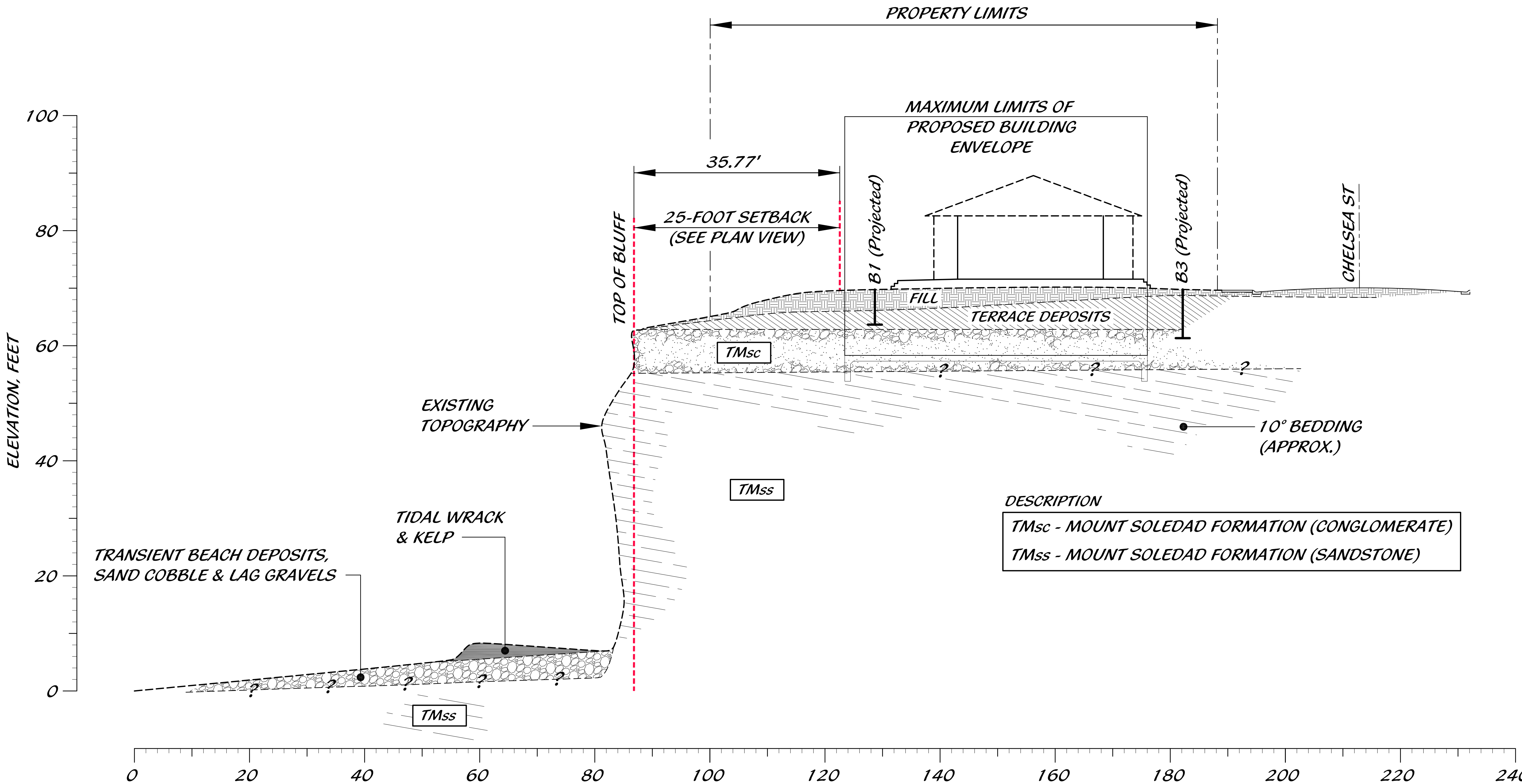


DESCRIPTION

Qaf - FILL
TMsc - MOUNT SOLEDAD FORMATION (CONGLOMERATE)
TMss - MOUNT SOLEDAD FORMATION (SANDSTONE)

SITE PLAN
SCALE: 1"=20'

TerraCosta Consulting Group	TERRACOSTA CONSULTING GROUP ENGINEERS AND GEOLOGISTS 3890 MURPHY CANYON ROAD, SUITE 200 SAN DIEGO, CA 92123 (858) 573-6900	FIGURE NUMBER 2
	PROJECT NAME 5228 CHELSEA STREET	PROJECT NUMBER 2918
	SITE PLAN	



EXISTING & PROPOSED CROSS SECTION

SCALE: 1"=20' (HORIZ. & VERT.)

1

TerraCosta Consulting Group	TERRACOSTA CONSULTING GROUP ENGINEERS AND GEOLOGISTS 3890 MURPHY CANYON ROAD, SUITE 200 SAN DIEGO, CA 92123 (858) 573-6900	FIGURE NUMBER 3
	PROJECT NAME 5228 CHELSEA STREET	PROJECT NUMBER 2918
	CROSS SECTION	

APPENDIX A

LOGS OF EXPLORATORY EXCAVATIONS

TerraCosta
Consulting Group

LOG OF TEST BORING

PROJECT NAME	PROJECT NUMBER	BORING
5228 CHELSEA STREET	2918	LEGEND

SITE LOCATION	START	FINISH	SHEET NO.
La Jolla, California	3/8/2016	3/8/2016	1 of 2

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling	Solid Stem Auger	G. Spaulding	

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Tripod	4	40		n/a

SAMPLING METHOD	NOTES
SPT/Cal	

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
5									
10									
		C	1						
		S	2						
15									

KEY TO EXCAVATION LOGS

WATER TABLE MEASURED AT TIME OF DRILLING

OTHER TESTS

CC	Confined Compression	ppm	parts per million of VOCs*
CL	Chloride Content	R	Resistivity
CS	Consolidation	RV	R-Value
DS	Direct Shear	SA	Sieve Analysis
EI	Expansion Index	SE	Sand Equivalent
GS	Grain Size Analysis	SF	Sulfate
LC	Laboratory Compaction	SG	Specific Gravity
pH	Hydrogen Ion	SW	Swell
PI	Plasticity Index		

PENETRATION RESISTANCE (BLOWS/ft)

Number of blows required to advance the sampler 1 foot.

California Sampler blow counts can be converted to equivalent SPT blow counts by using an end-area conversion factor of 0.67 when using a 140-pound hammer and a 30-inch drop.

SAMPLE TYPE

C ("California Sampler") - An 18-inch-long, 2-1/2-inch I.D., 3-inch O.D., thick-walled sampler. The sampler is lined with eighteen 2-3/8-inch I.D. brass rings. Relatively undisturbed, intact soil samples are retained in the brass rings.

S ("SPT") - a.k.a. Standard Penetration Test, an 18-inch-long, 2-inch O.D., 1-3/8-inch I.D. drive sampler.

KEY TO EXCAVATION LOGS

NOTES ON FIELD INVESTIGATION

Borings were advanced using a limited-access drill rig with a 4-inch solid-stem auger.

Standard Penetration Tests (SPT) and California SamplersSamplers were used to obtain soil samples. The SPT and California Samplers were driven into the soil at the bottom of the borings with a 140-pound hammer falling 30 inches. When the samplers were withdrawn from the boring, the samples were removed, visually classified, sealed in plastic containers, and taken to the laboratory for detailed inspection.

(CONTINUED)

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LOG OF TEST BORING

PROJECT NAME	PROJECT NUMBER	BORING
5228 CHELSEA STREET	2918	**LEGEND**

SITE LOCATION	START	FINISH	SHEET NO.
La Jolla, California	3/8/2016	3/8/2016	2 of 2

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling	Solid Stem Auger	G. Spaulding	

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Tripod	4	40		▼ n/a

SAMPLING METHOD	NOTES
SPT/Cal	

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
25									
30									
35									

KEY TO EXCAVATION LOGS

(CONTINUED)

NOTES ON FIELD INVESTIGATION (Continued)

Free groundwater was not encountered in the borings at the time of drilling.

Classifications are based upon the Unified Soil Classification System and include color, moisture, and consistency. Field descriptions have been modified to reflect results of laboratory inspection where deemed appropriate.

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FIGURE A-1 b

LOG OF TEST BORING

PROJECT NAME	PROJECT NUMBER	BORING
5228 CHELSEA STREET	2918	B-1

SITE LOCATION	START	FINISH	SHEET NO.
La Jolla, California	3/8/2016	3/8/2016	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling	Solid Stem Auger	G. Spaulding	

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Tripod	4	6.5	69	▼ n/a

SAMPLING METHOD	NOTES
SPT/Cal	

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
									FILL **Fine Sandy CLAY (CL)**, mottled gray-brown / light gray, damp
		C	1	19	113.5	12.2			
	65								TERRACE DEPOSITS **Sandy CLAY (CL)**, very stiff to hard, mottled brown / red-brown, damp
5									
		S	2	35		10.7	PI SA		
									Boring terminated at depth of 6.5 feet due to refusal on rock. *No free groundwater encountered at time of excavation.*
	60								
10									
	55								
15									
	50								

TCG_METRIC_LOG(3) 2918.GPJ GDCLOGMT.GDT 6/28/16

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THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE A-2

LOG OF TEST BORING

PROJECT NAME	PROJECT NUMBER	BORING
5228 CHELSEA STREET	2918	B-2

SITE LOCATION	START	FINISH	SHEET NO.
La Jolla, California	3/8/2016	3/8/2016	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling	Solid Stem Auger	G. Spaulding	

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Tripod	4	8.75	70	⊻ n/a

SAMPLING METHOD	NOTES
SPT/Cal	

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
									FILL **Fine Sandy CLAY to Clayey Fine SAND (SC/CL)**, gray-brown, damp
		C	1	15	106.6	11.8	PI SA		
5	65								TERRACE DEPOSITS **Fine Sandy CLAY (CL)**, very stiff, red-brown, damp, with occasional gravel/cobble
		S	2	22					
									- Hard drilling
									MOUNT SOLEDAD FORMATION **Silty to Clayey SAND and GRAVEL CONGLOMERATE (SC-GC)**, very dense, light brown to yellow-brown, damp, with gravel and cobble
		S	3	50/2"					*Boring terminated at depth of 8.75 feet due to refusal on rock.* *No free groundwater encountered at time of excavation.*
10	60								
15	55								

TCG_METRIC_LOG(3) 2918.GPJ GDCLOGMT.GDT 6/28/16

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San Diego, California 92123

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE A-3

LOG OF TEST BORING

PROJECT NAME	PROJECT NUMBER	BORING
5228 CHELSEA STREET	2918	**B-3**

SITE LOCATION	START	FINISH	SHEET NO.
La Jolla, California	3/8/2016	3/8/2016	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling	Solid Stem Auger	G. Spaulding	

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Tripod	4	9	70	⊻ n/a

SAMPLING METHOD	NOTES
SPT/Cal	

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
									FILL **Clayey Fine SAND (SC)**, dark gray-brown, damp to moist
									TERRACE DEPOSITS **Clayey Fine SAND (SC)**, medium dense, red-brown, moist
		S	1	18					
5	65								**Sandy CLAY (CL)**, very stiff, mottled red-brown / red, damp to moist, with occasional gravel
		S	2	21		15.1	PI SA		
									- Hard drilling
									MOUNT SOLEDAD FORMATION **Silty to Clayey SAND & GRAVEL CONGLOMERATE (SC-GC)**, very dense, red-brown, damp
		S	3	84/10"					*Boring terminated at depth of 9.0 feet due to refusal on rock.* *No free groundwater encountered at time of excavation.*
10	60								
15	55								

TCG_METRIC_LOG(3) 2918.GPJ GDCLOGMT.GDT 6/28/16

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THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE A-4

APPENDIX B

LABORATORY TEST RESULTS

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9177 Sky Park Ct. San Diego, CA. 92123

PHYSICAL PROPERTIES OF SOILS

PROJECT:#2918: 5228 Chelsea	LAB NO.: 29693-29693 (page 1 of 1)	PROJECT NO.: 5015-15-0030.12
	SAMPLED BY: G. Spaulding	DATE: 03/08/16
	SUBMITTED BY: G. Spaulding	DATE: 03/08/16
	AUTHORIZED BY: M. Eckert	DATE: 03/08/16
	REVIEWED BY: L. Collins	REPORT DATE: 03/21/16

Sample I.D.	Depth (ft.)	Maximum Density/Optimum Moisture ASTM D1557	Liquid and Plastic Limit ASTM D43189	% -#200 ASTM C 117	Dry Density (pcf)	Moisture Content (%), as received ASTM D 2216
B1-1 (#29690)	2'	*	*	*	113.5	12.2
B1-2 (#29691)	5'	*	33.5/13.6/19.9	64.2	*	10.7
B2-1 (#29692)	2'	*	28.6/15.4/13.2	62.4	106.6	11.8
B3-2 (#29693)	5'	*	36.9/13.6/23.3	68.2	*	15.1

NOTE: *Indicates test not requested

TerraCosta Consulting Inc./ M. Eckert

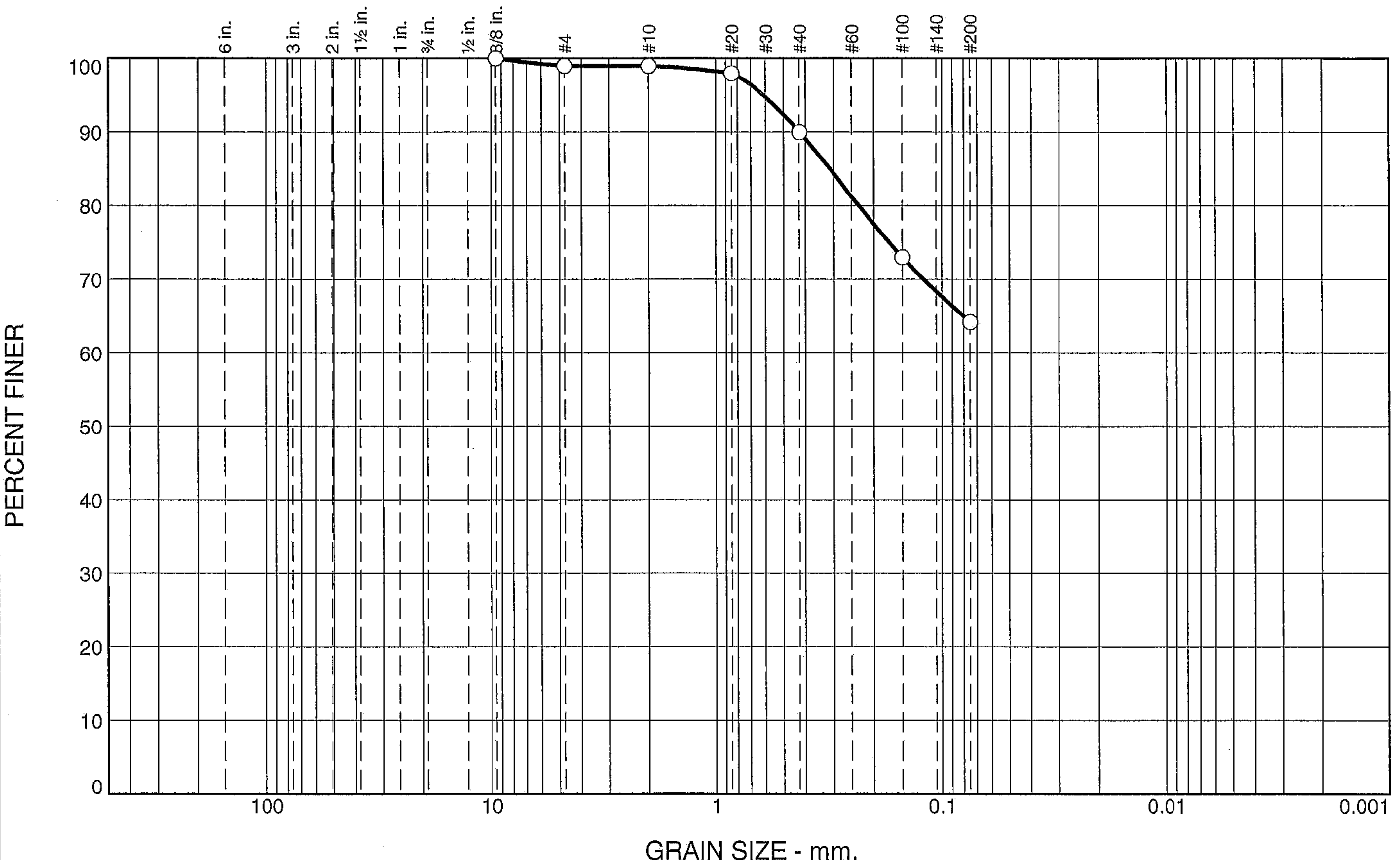
Amec Foster Wheeler

Reviewed By:____________________

David C. Wilson, CE#54734

Senior Principal Engineer

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.0	0.0	9.0	25.8	64.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375"	100.0		
#4	99.0		
#10	99.0		
#20	98.0		
#40	90.0		
#100	73.0		
#200	64.2		

* (no specification provided)

Material Description

Sandy Clay, CL (#29691)

Atterberg Limits

PL= 13.6 LL= 33.5 PI= 19.9

Coefficients

D_{90}= 0.4250 D_{85}= 0.3121 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Sample Number: B1-2 **Depth:** 5.0'

Date: 3/16/16

amec

Client: TerraCosta Consulting Group, Inc.
Project: #5228 Chelsea
Project No: 5015150030.12 **Figure** #29691

Tested By: R. Valles **Checked By:** L. Collins

GRAIN SIZE DISTRIBUTION TEST DATA

3/21/2016

Client: TerraCosta Consulting Group, Inc.
Project: #5228 Chelsea
Project Number: 5015150030.12
Depth: 5.0' **Sample Number:** B1-2
Material Description: Sandy Clay, CL (#29691)
Date: 3/16/16 **PL:** 13.6 **LL:** 33.5 **PI:** 19.9
USCS Classification: CL
Tested by: R. Valles **Checked by:** L. Collins

Sieve Test Data

Sieve Opening Size	Percent Finer
0.375"	100.0
#4	99.0
#10	99.0
#20	98.0
#40	90.0
#100	73.0
#200	64.2

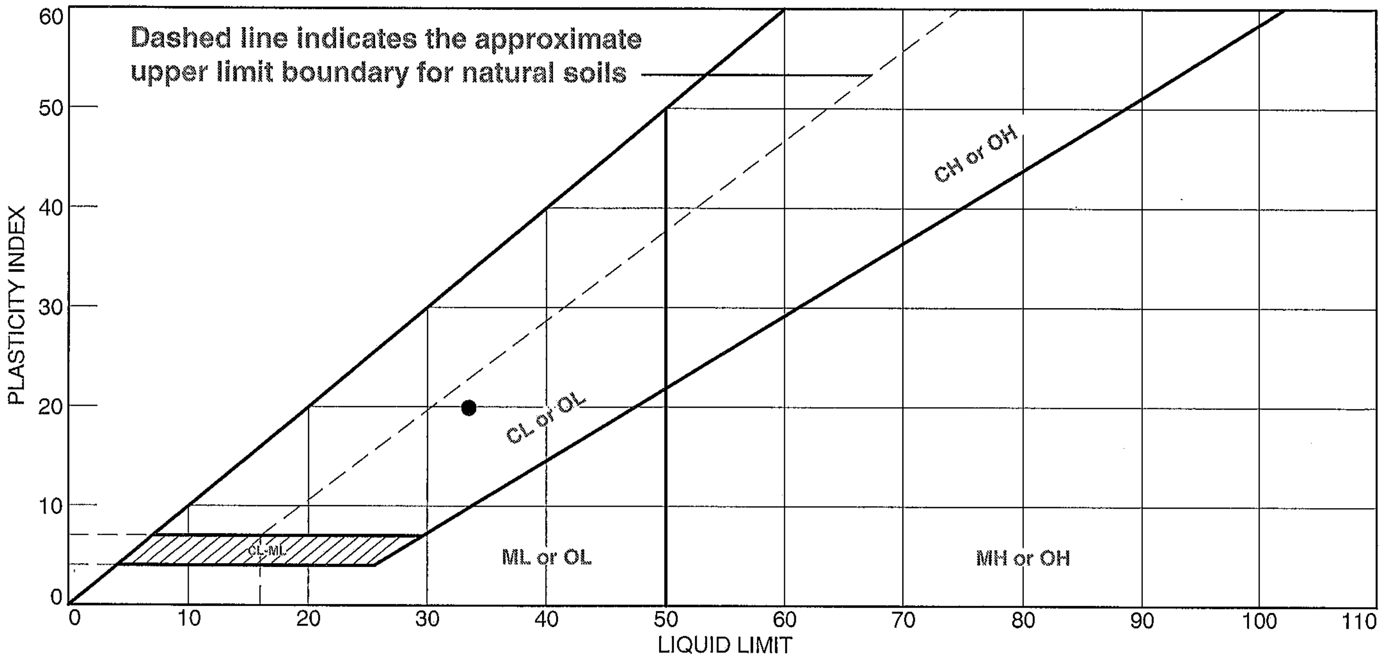
Fractional Components

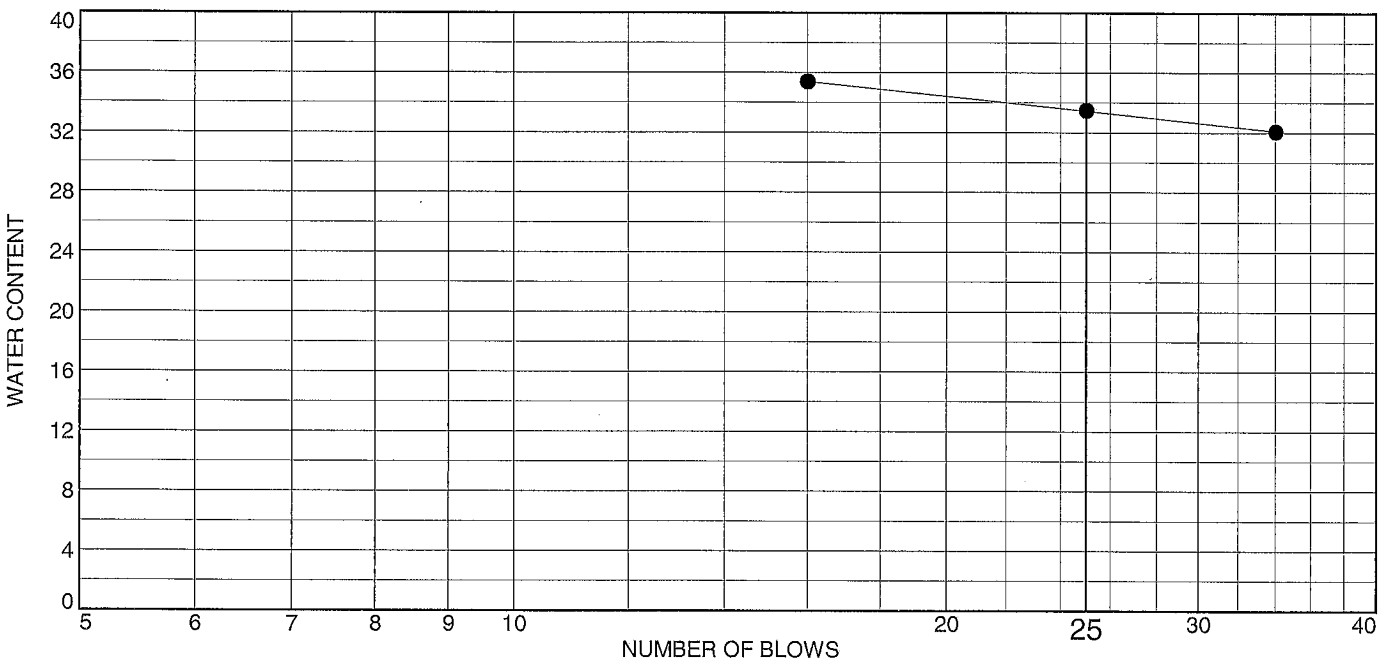
Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	1.0	1.0	0.0	9.0	25.8	34.8			64.2

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
								0.2324	0.3121	0.4250	0.6122

Fineness Modulus
0.51

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy Clay, CL (#29691)	33.5	13.6	19.9	90.0	64.2	CL

Project No. 5015150030.12 **Client:** TerraCosta Consulting Group, Inc.

Project: #5228 Chelsea

Sample Number: B1-2 **Depth:** 5.0'

amec

Remarks:

Figure #29691

Tested By: R. Valles **Checked By:** L. Collins

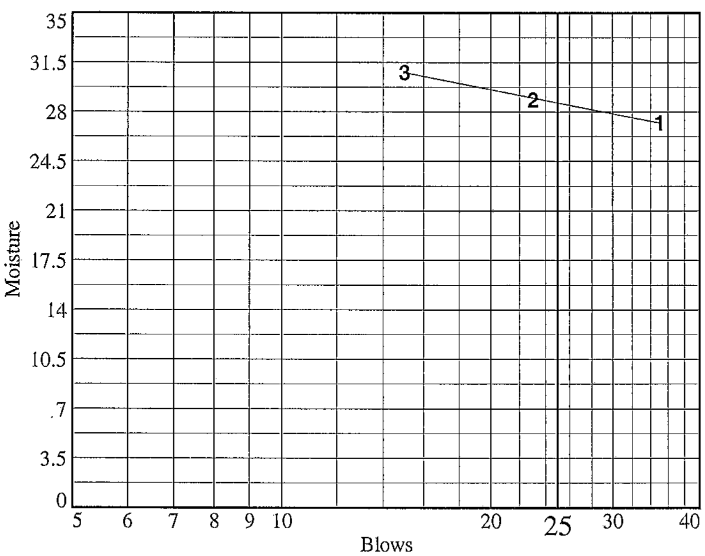
LIQUID AND PLASTIC LIMIT TEST DATA

3/21/2016

Client: TerraCosta Consulting Group, Inc.
Project: #5228 Chelsea
Project Number: 5015150030.12
Depth: 2.0' **Sample Number:** B2-1
Material Description: Sandy Clay, CL (#29692)
%<#40: 82.3 **%<#200:** 62.4 **USCS:** CL **AASHTO:** A-6(6)
Tested by: R. Valles **Checked by:** L. Collins

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	38.98	39.68	39.42			
Dry+Tare	35.16	35.51	35.10			
Tare	21.15	21.09	21.09			
# Blows	35	23	15			
Moisture	27.3	28.9	30.8			

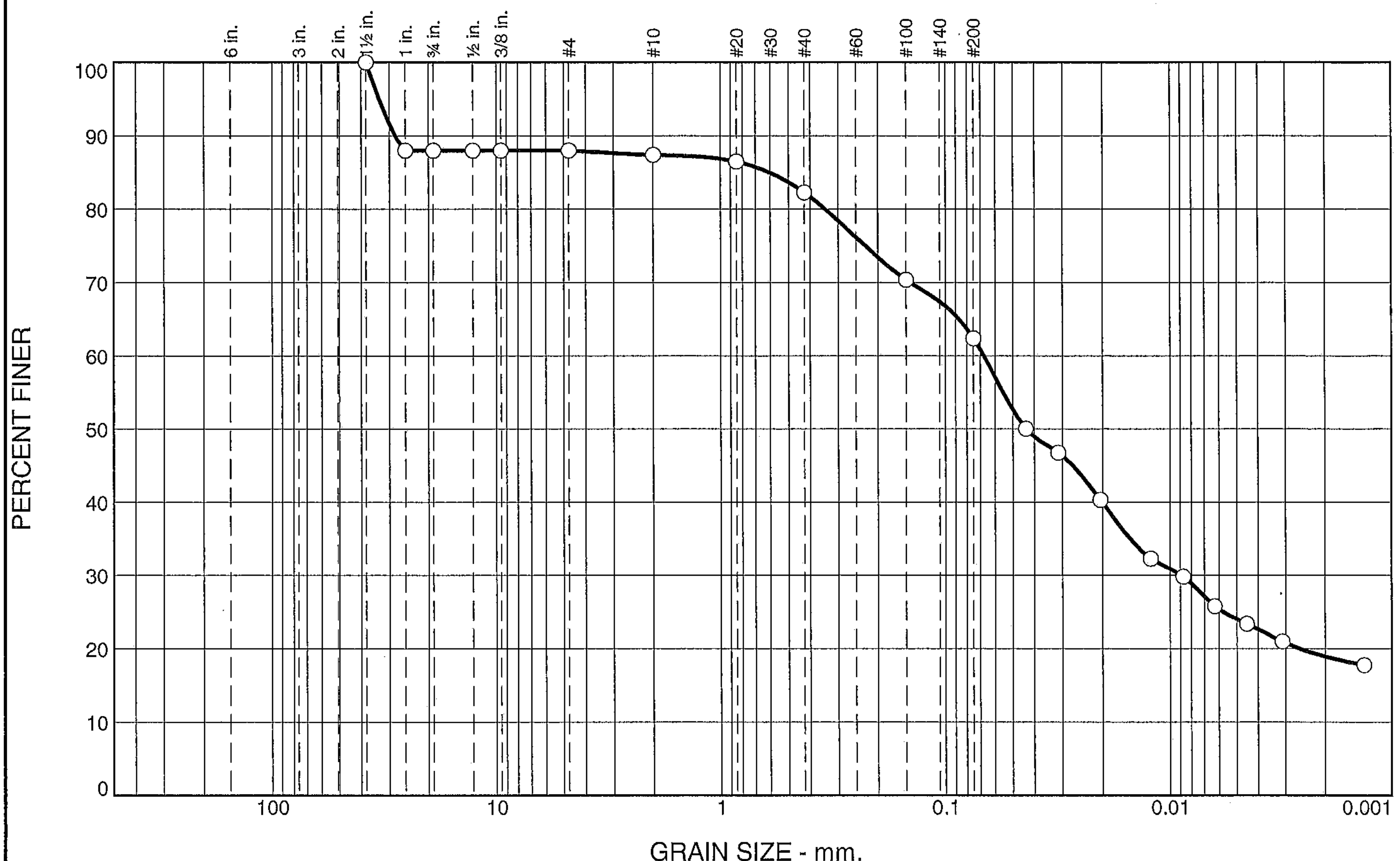


Liquid Limit= 28.6
Plastic Limit= 15.4
Plasticity Index= 13.2

Plastic Limit Data

Run No.	1	2	3	4
Wet+Tare	31.9	31.92		
Dry+Tare	30.48	30.46		
Tare	21.26	21.00		
Moisture	15.4	15.4		

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	12.0	0.0	0.6	5.1	19.9	43.5	18.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.5"	100.0		
1"	88.0		
0.75"	88.0		
0.5"	88.0		
0.375"	88.0		
#4	88.0		
#10	87.4		
#20	86.5		
#40	82.3		
#100	70.4		
#200	62.4		

* (no specification provided)

Material Description

Sandy Clay, CL (#29692)

Atterberg Limits

PL= 15.4 LL= 28.6 PI= 13.2

Coefficients

D_{90}= 28.3574 D_{85}= 0.6038 D_{60}= 0.0675
D_{50}= 0.0437 D_{30}= 0.0089 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(6)

Remarks

Assumed specific gravity of 2.65 used for hydrometer calculations and soil particles smaller than 0.002mm have been classified as clay

Sample Number: B2-1 **Depth:** 2.0'

Date: 3/18/16

amec

Client: TerraCosta Consulting Group, Inc.
Project: #5228 Chelsea

Project No: 5015150030.12 **Figure** #29692

Tested By: R. Valles **Checked By:** L. Collins

GRAIN SIZE DISTRIBUTION TEST DATA

3/21/2016

Client: TerraCosta Consulting Group, Inc.
Project: #5228 Chelsea
Project Number: 5015150030.12
Depth: 2.0' **Sample Number:** B2-1
Material Description: Sandy Clay, CL (#29692)
Date: 3/18/16 **PL:** 15.4 **LL:** 28.6 **PI:** 13.2
USCS Classification: CL **AASHTO Classification:** A-6(6)
Testing Remarks: Assumed specific gravity of 2.65 used for hydrometer calculations and soil particles smaller than 0.002mm have been classified as clay
Tested by: R. Valles **Checked by:** L. Collins

Sieve Test Data

Sieve Opening Size	Percent Finer
1.5"	100.0
1"	88.0
0.75"	88.0
0.5"	88.0
0.375"	88.0
#4	88.0
#10	87.4
#20	86.5
#40	82.3
#100	70.4
#200	62.4

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 87.4
Weight of hydrometer sample =54.66
Hygroscopic moisture correction:
Moist weight and tare = 36.27
Dry weight and tare = 36.17
Tare weight = 25.56
Hygroscopic moisture = 0.9%
Table of composite correction values:

Temp., deg. C:	20.3	21.6	22.5	23.3
Comp. corr.:	-4.0	-4.0	-5.0	-5.0

Meniscus correction only = 0.0
Specific gravity of solids = 2.65
Hydrometer type = 152H
Hydrometer effective depth equation: L = 16.294964 - .164 **x Rm**

Elapsed Time (min.)	Temp. (deg. C.)	Actual Reading	Corrected Reading	K	Rm	Eff. Depth	Diameter (mm.)	Percent Finer
1.00	21.0	35.0	31.0	0.0135	35.0	10.6	0.0438	50.0
2.00	21.0	33.0	29.0	0.0135	33.0	10.9	0.0314	46.8
5.00	21.0	29.0	25.0	0.0135	29.0	11.5	0.0205	40.4
15.00	21.0	24.0	20.0	0.0135	24.0	12.4	0.0122	32.3
30.00	21.0	22.5	18.5	0.0135	22.5	12.6	0.0087	29.9
60.00	20.7	20.0	16.0	0.0135	20.0	13.0	0.0063	25.8
120.00	20.6	18.5	14.5	0.0135	18.5	13.3	0.0045	23.4
250.00	21.5	17.0	13.0	0.0134	17.0	13.5	0.0031	21.0
1440.00	20.1	15.0	11.0	0.0136	15.0	13.8	0.0013	17.8

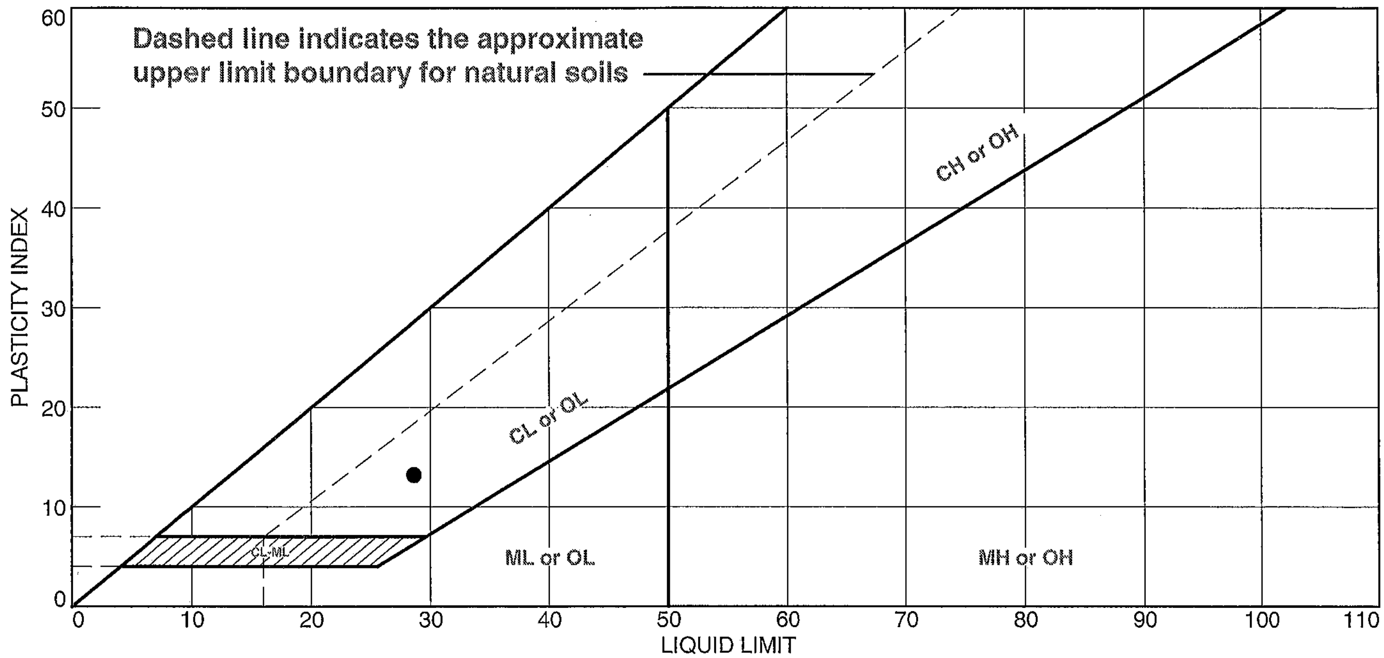
Fractional Components

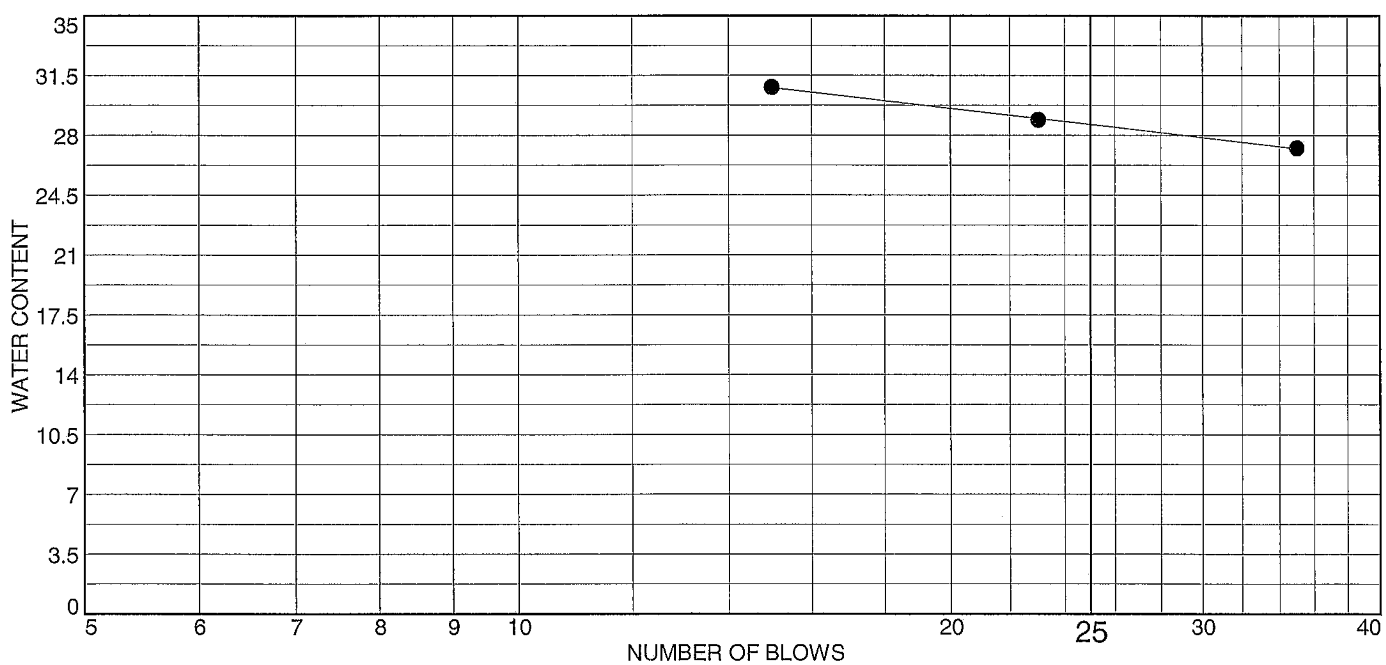
Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	12.0	0.0	12.0	0.6	5.1	19.9	25.6	43.5	18.9	62.4

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
			0.0026	0.0089	0.0201	0.0437	0.0675	0.3425	0.6038	28.3574	33.3687

Fineness Modulus
1.28

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy Clay, CL (#29692)	28.6	15.4	13.2	82.3	62.4	CL

Project No. 5015150030.12 **Client:** TerraCosta Consulting Group, Inc.

Project: #5228 Chelsea

Sample Number: B2-1 **Depth:** 2.0'

amec

Remarks:

Figure #29692

Tested By: R. Valles **Checked By:** L. Collins

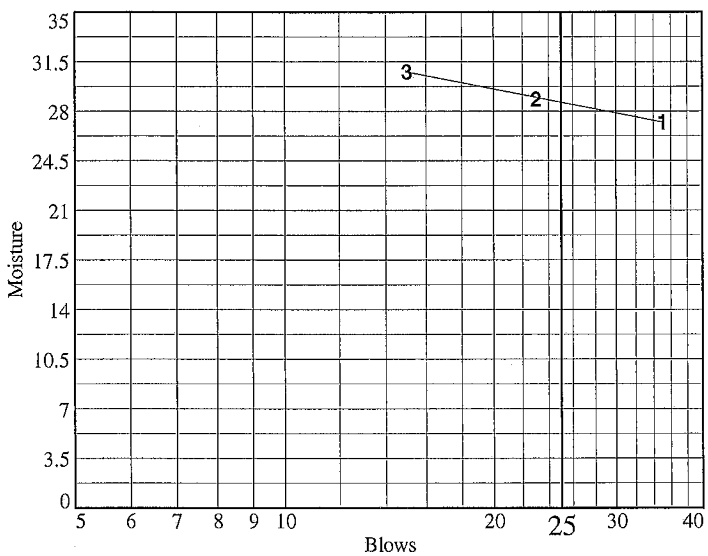
LIQUID AND PLASTIC LIMIT TEST DATA

3/21/2016

Client: TerraCosta Consulting Group, Inc.
Project: #5228 Chelsea
Project Number: 5015150030.12
Depth: 2.0' **Sample Number:** B2-1
Material Description: Sandy Clay, CL (#29692)
%<#40: 82.3 **%<#200:** 62.4 **USCS:** CL **AASHTO:** A-6(6)
Tested by: R. Valles **Checked by:** L. Collins

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	38.98	39.68	39.42			
Dry+Tare	35.16	35.51	35.10			
Tare	21.15	21.09	21.09			
# Blows	35	23	15			
Moisture	27.3	28.9	30.8			

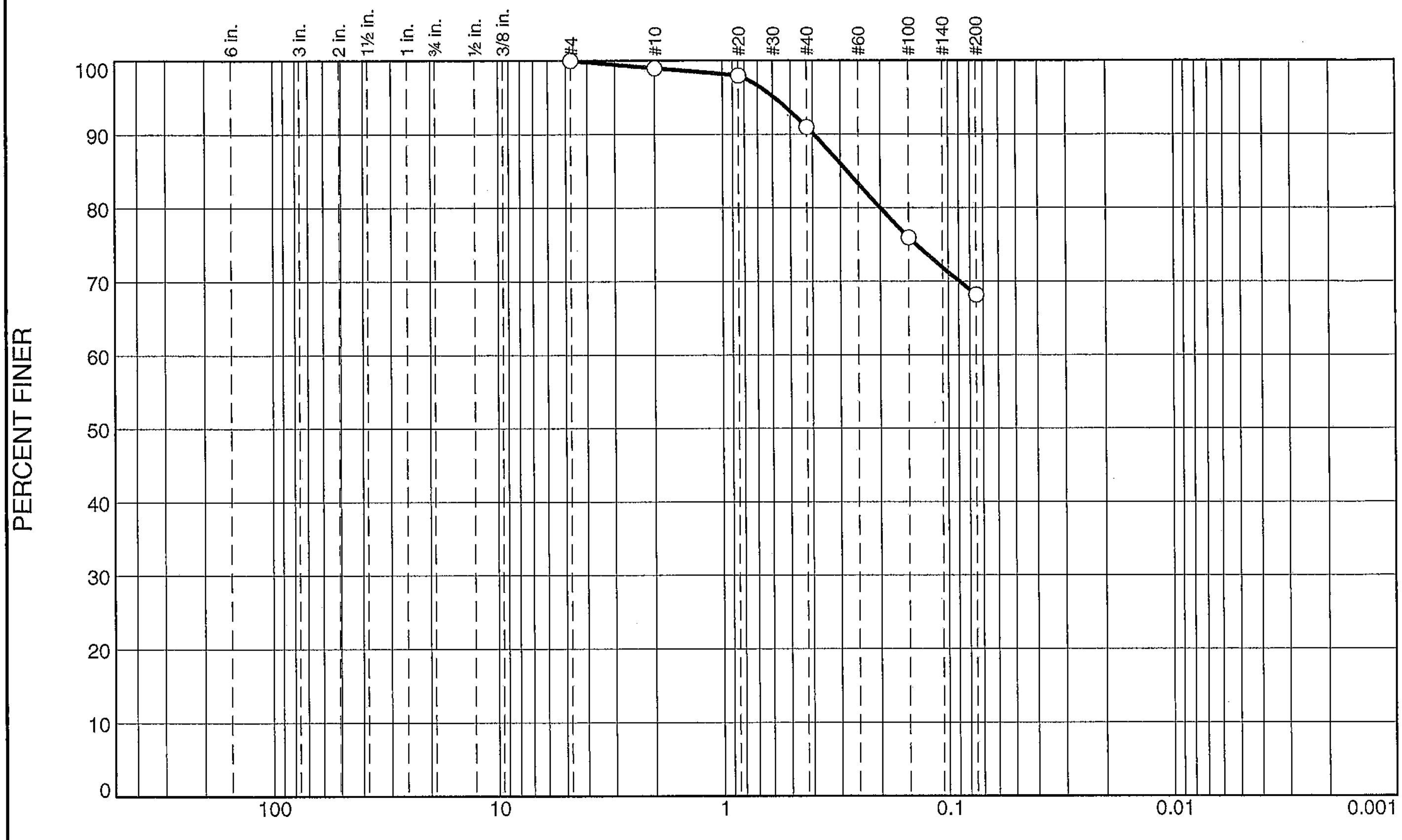


Liquid Limit= 28.6
Plastic Limit= 15.4
Plasticity Index= 13.2

Plastic Limit Data

Run No.	1	2	3	4	
Wet+Tare	31.9	31.92			
Dry+Tare	30.48	30.46			
Tare	21.26	21.00			
Moisture	15.4	15.4			

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	1.0	8.0	22.8	68.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.0		
#20	98.0		
#40	91.0		
#100	76.0		
#200	68.2		

* (no specification provided)

Material Description

Sandy Clay, CL (#29693)

Atterberg Limits

PL= 13.6 LL= 36.9 PI= 23.3

Coefficients

D_{90}= 0.3951 D_{85}= 0.2804 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Sample Number: B3-2 **Depth:** 5.0'

Date: 3/18/16

amec

Client: TerraCosta Consulting Group, Inc.
Project: #5228 Chelsea
Project No: 5015150030.12 **Figure** #29693

Tested By: R. Valles **Checked By:** L. Collins

GRAIN SIZE DISTRIBUTION TEST DATA

3/21/2016

Client: TerraCosta Consulting Group, Inc.
Project: #5228 Chelsea
Project Number: 5015150030.12
Depth: 5.0' **Sample Number:** B3-2
Material Description: Sandy Clay, CL (#29693)
Date: 3/18/16 **PL:** 13.6 **LL:** 36.9 **PI:** 23.3
USCS Classification: CL
Tested by: R. Valles **Checked by:** L. Collins

Sieve Test Data

Sieve Opening Size	Percent Finer
#4	100.0
#10	99.0
#20	98.0
#40	91.0
#100	76.0
#200	68.2

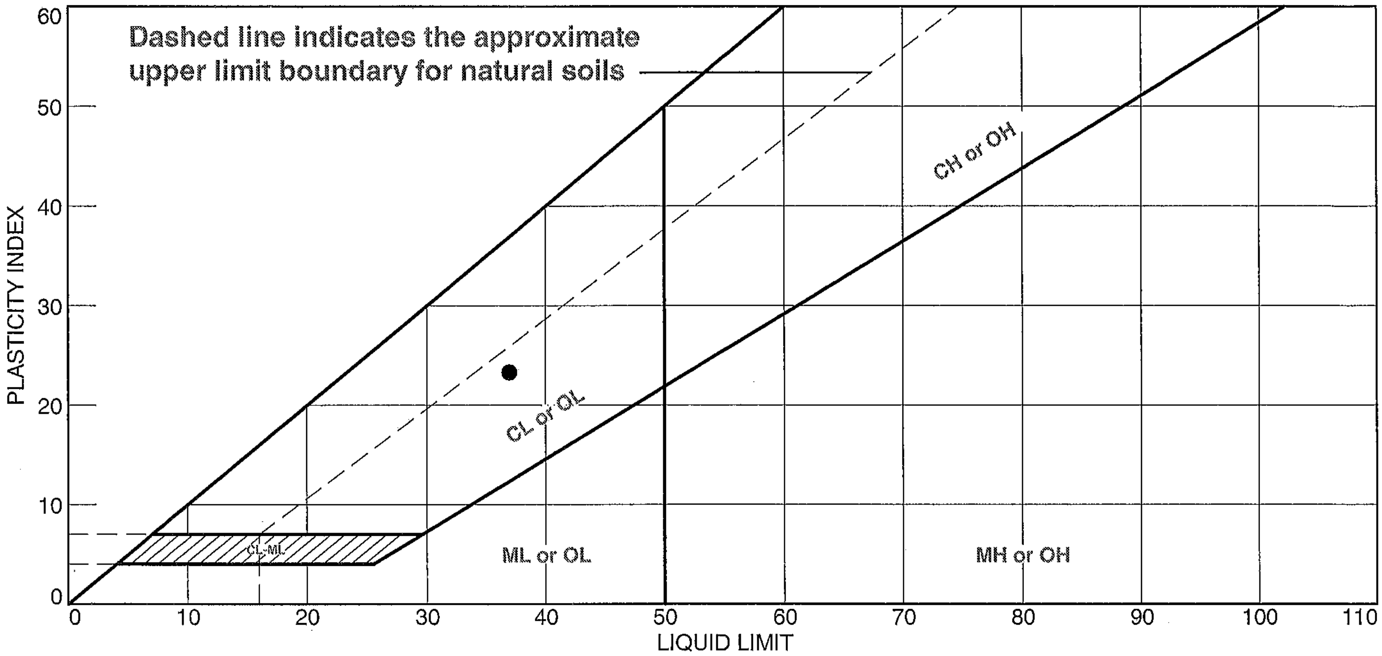
Fractional Components

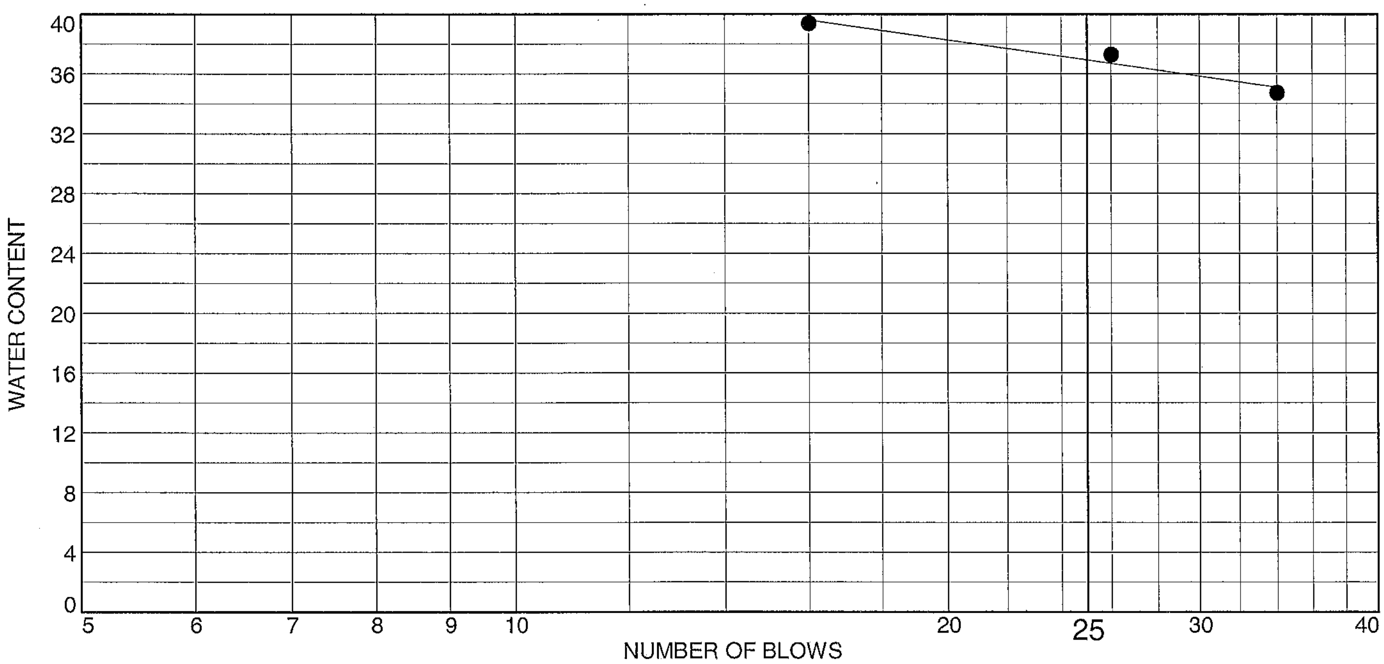
Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	0.0	1.0	8.0	22.8	31.8			68.2

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
								0.2003	0.2804	0.3951	0.5897

Fineness Modulus
0.45

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy Clay, CL (#29693)	36.9	13.6	23.3	91.0	68.2	CL

Project No. 5015150030.12 **Client:** TerraCosta Consulting Group, Inc.

Project: #5228 Chelsea

Sample Number: B3-2 **Depth:** 5.0'

Remarks:

amec

Figure #29693

Tested By: R. Valles **Checked By:** L. Collins

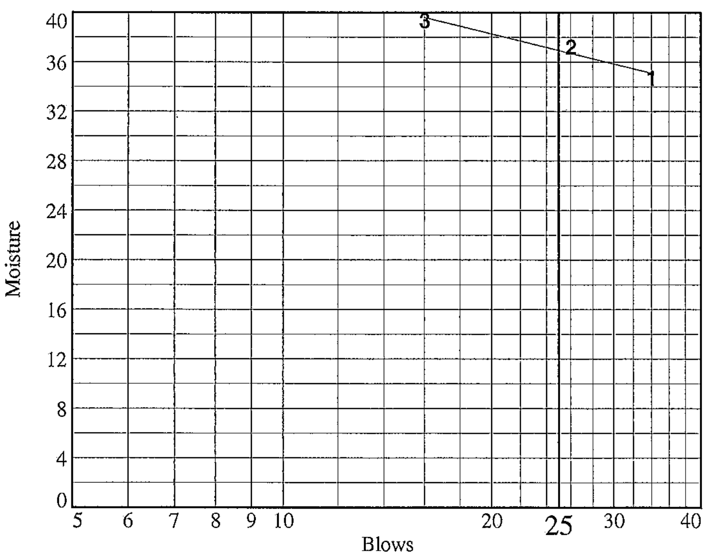
LIQUID AND PLASTIC LIMIT TEST DATA

3/21/2016

Client: TerraCosta Consulting Group, Inc.
Project: #5228 Chelsea
Project Number: 5015150030.12
Depth: 5.0' **Sample Number:** B3-2
Material Description: Sandy Clay, CL (#29693)
%<#40: 91.0 **%<#200:** 68.2 **USCS:** CL
Tested by: R. Valles **Checked by:** L. Collins

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	36.94	38.51	40.83			
Dry+Tare	32.81	33.82	35.20			
Tare	20.92	21.24	20.91			
# Blows	34	26	16			
Moisture	34.7	37.3	39.4			



Liquid Limit= 36.9
Plastic Limit= 13.6
Plasticity Index= 23.3

Plastic Limit Data

Run No.	1	2	3	4	
Wet+Tare	30.73	30.22			
Dry+Tare	29.55	29.13			
Tare	21.13	20.82			
Moisture	14.0	13.1			

APPENDIX C

SUMMARY SLOPE STABILITY ANALYSES

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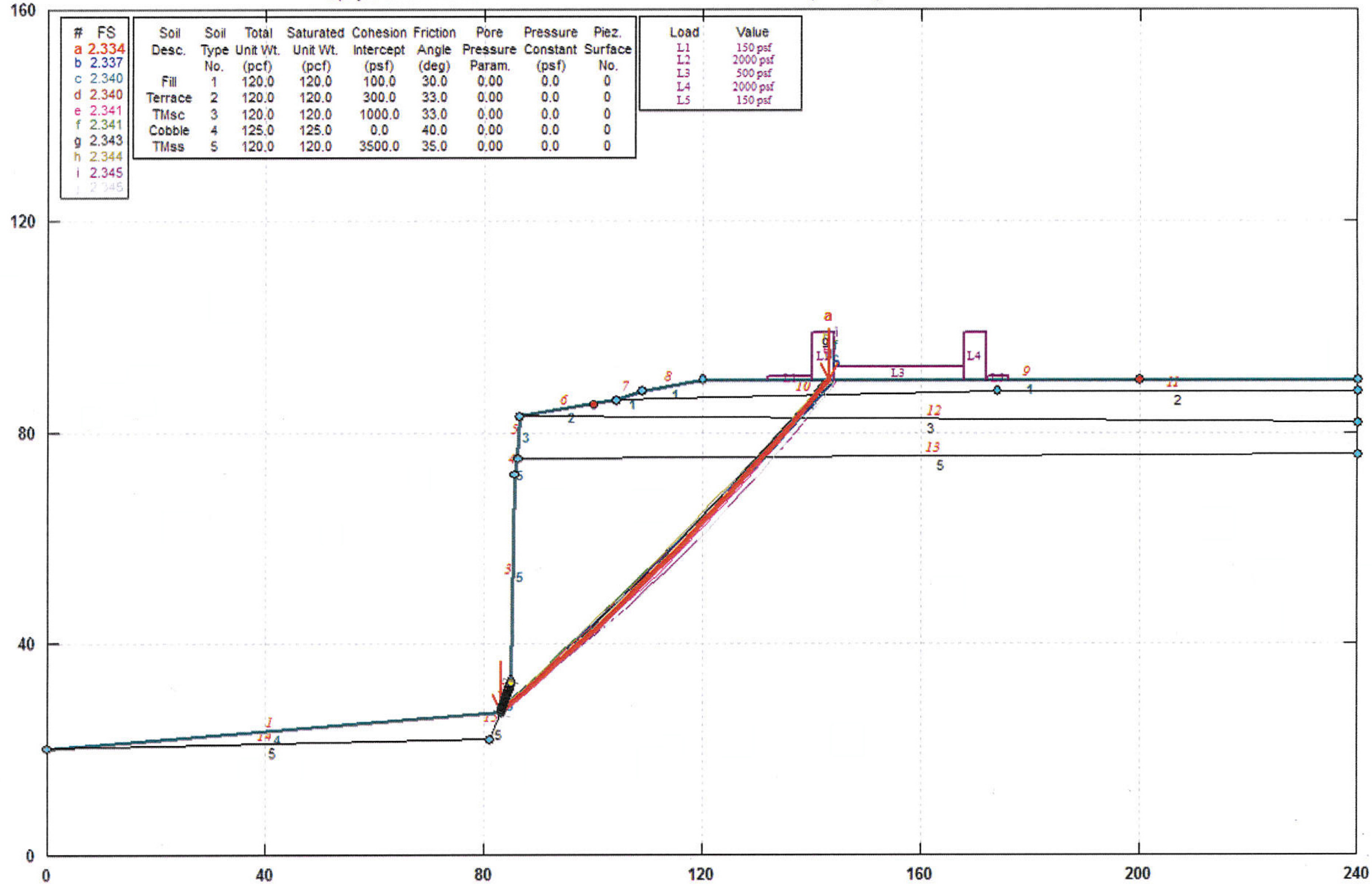

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	120.0	120.0	100.0	30.0	0.00	0.0	0
Terrace	2	120.0	120.0	300.0	33.0	0.00	0.0	0
TMsc	3	120.0	120.0	1000.0	33.0	0.00	0.0	0
Cobble	4	125.0	125.0	0.0	40.0	0.00	0.0	0
TMss	5	120.0	120.0	3500.0	35.0	0.00	0.0	0

GSTABL7 v.2 FSmin=2.334
Safety Factors Are Calculated By The Modified Bishop Method

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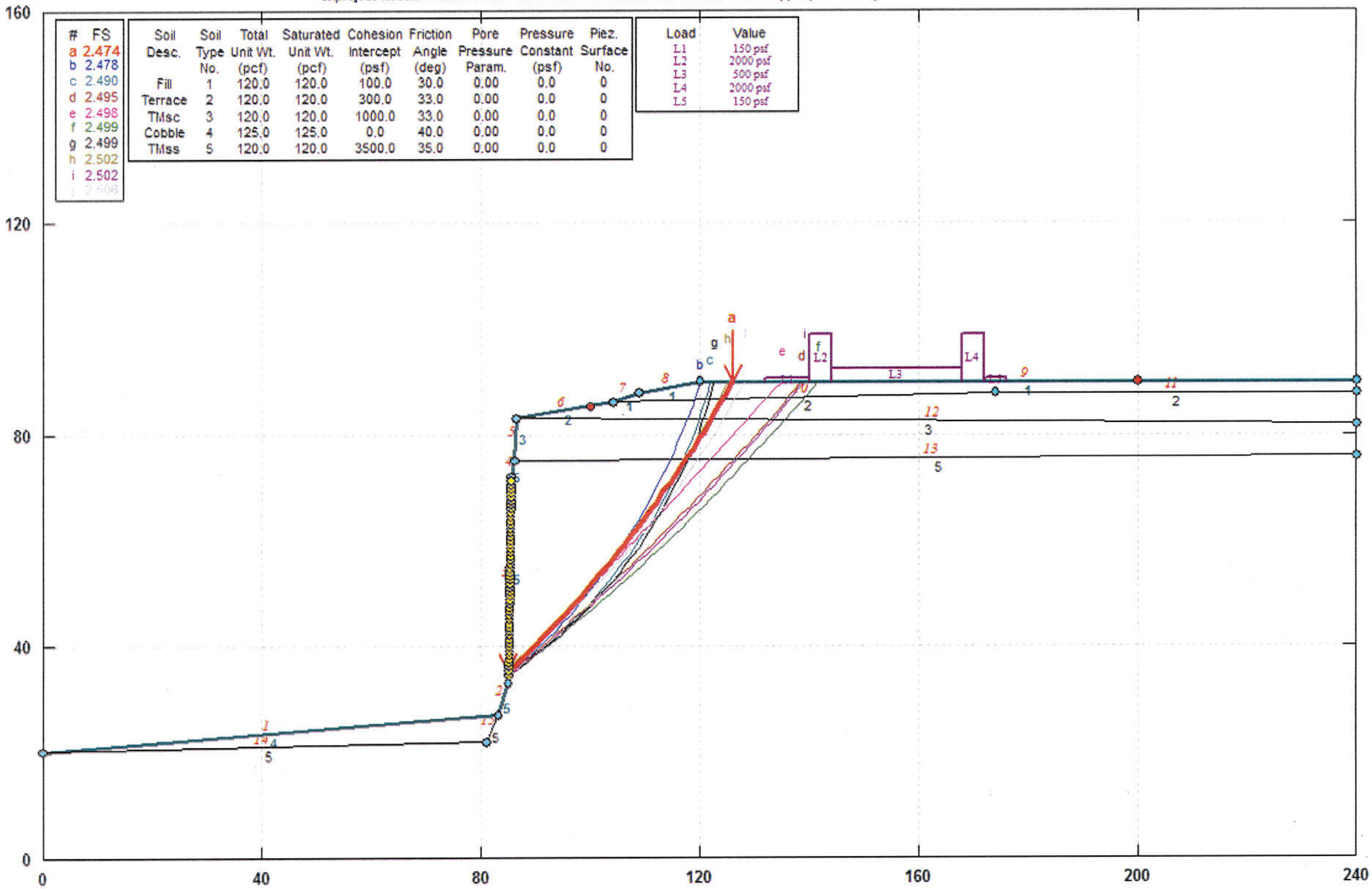

GSTABL7 v.2 FSmin=2.474
Safety Factors Are Calculated By The Modified Bishop Method

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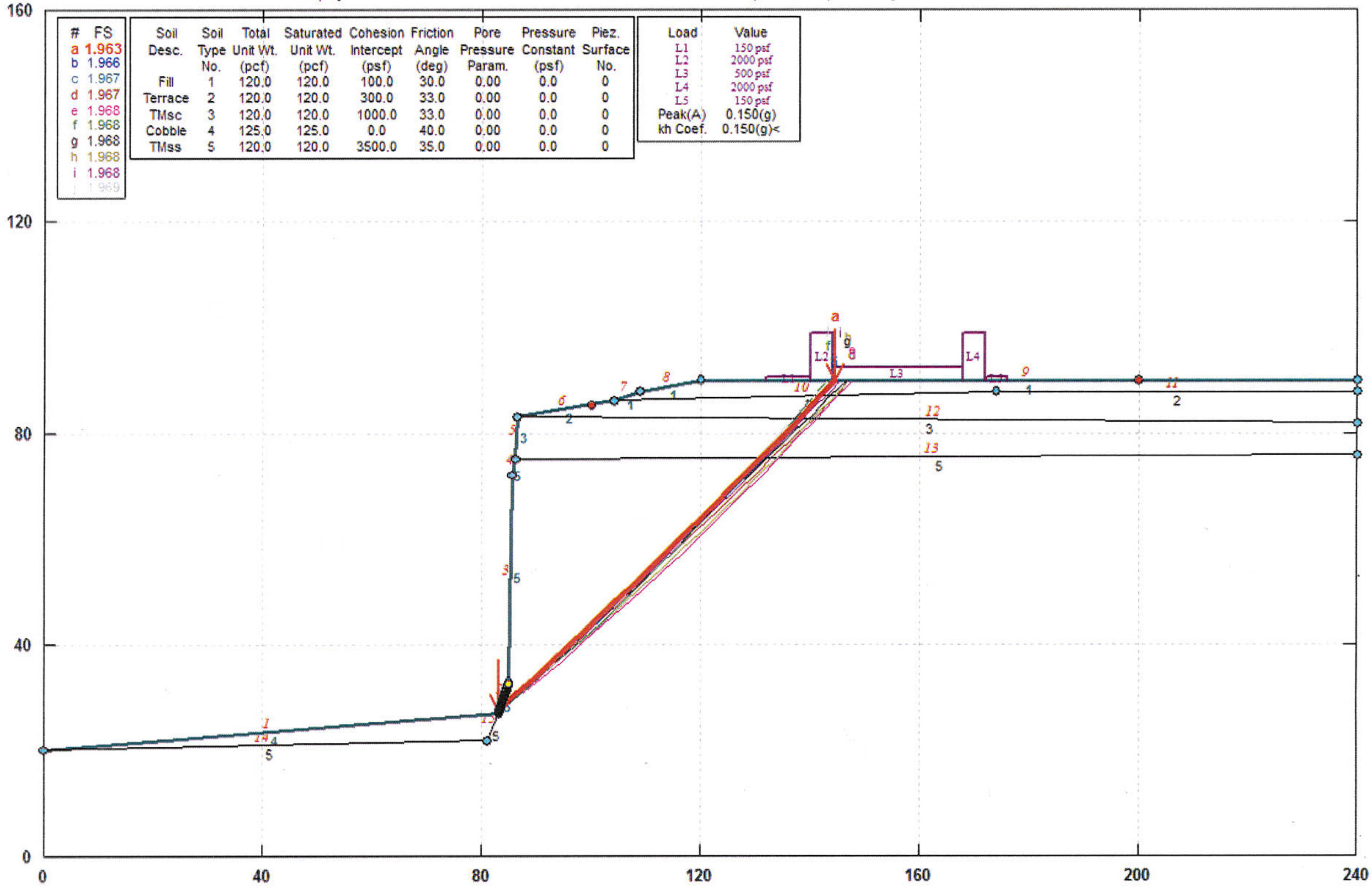

GSTABL7 v.2 FSmin=1.963
Safety Factors Are Calculated By The Modified Bishop Method

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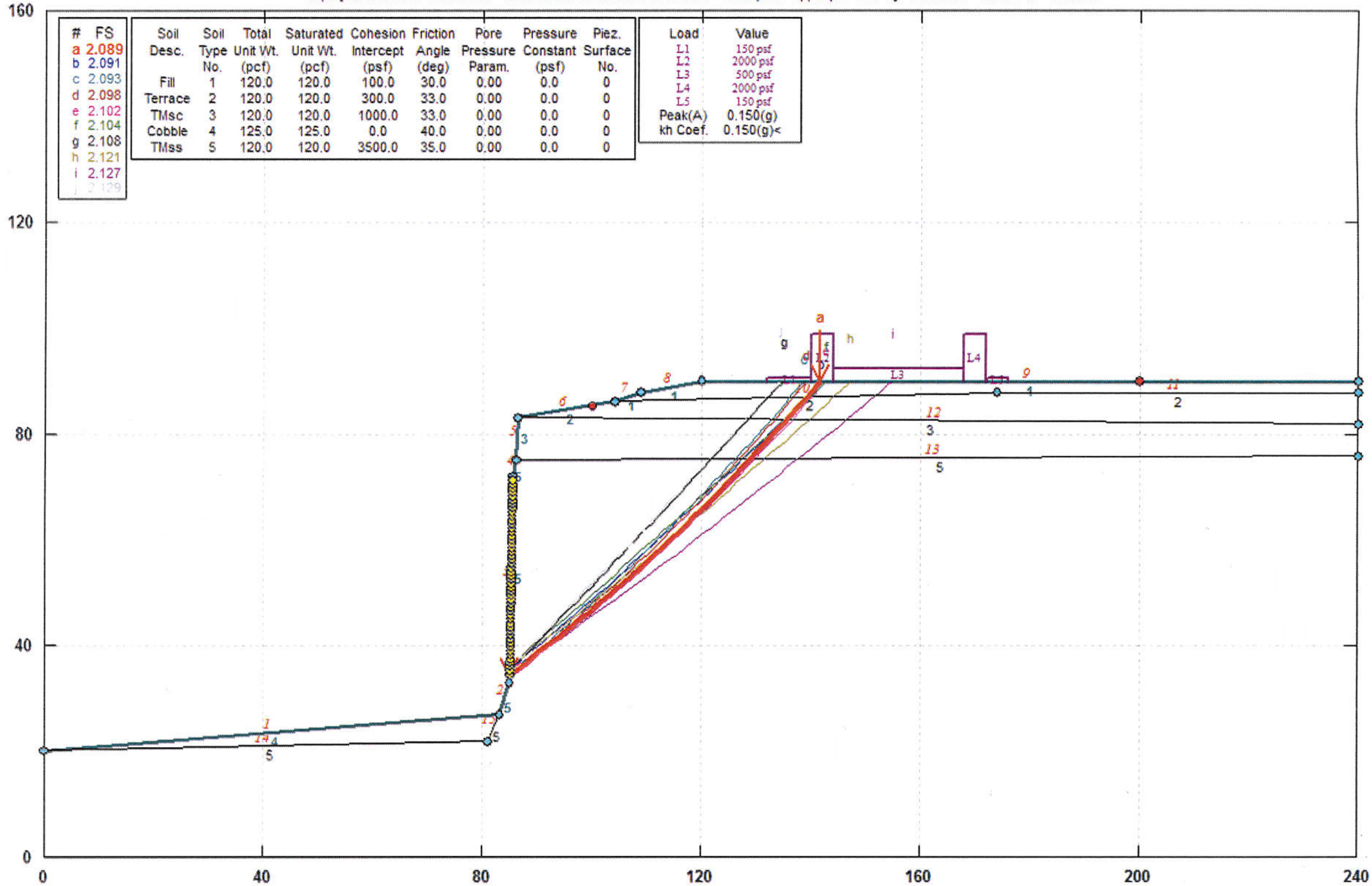

GSTABL7 v.2 FSmin=2.089

Safety Factors Are Calculated By The Modified Bishop Method

APPENDIX D

SPECIFICATIONS FOR ENGINEERED FILL

TerraCosta
Consulting Group

APPENDIX D
SPECIFICATIONS FOR ENGINEERED FILL

These specifications present the usual and minimum requirements for grading operations performed under observation and testing of TerraCosta Consulting Group, Inc.

No deviation from these specifications should be allowed, except where specifically superseded in the preliminary geology and soils report, or in other written communication signed by the Geotechnical Engineer or Engineering Geologist.

I. GENERAL

A. The Geotechnical Engineer and Engineering Geologist are the Owner's or Builder's representative on the project. For the purpose of these specifications, observation and testing by the Geotechnical Engineer includes that observation and testing performed by any person or persons employed by, and responsible to, the licensed Geotechnical Engineer signing the soils report.

B. The Contractor under the observation of the Geotechnical Engineer shall conduct, all clearing, site preparation, or earthwork performed on the project.

C. It is the Contractor's responsibility to prepare the ground surface to receive the fills and to place, spread, mix, water, and compact the fill in accordance with the specifications of the Geotechnical Engineer. The Contractor shall also remove all material considered unsuitable for use in the engineered fill by the Geotechnical Engineer.

D. It is also the Contractor's responsibility to have suitable and sufficient compaction equipment on the job-site to handle the amount of fill being placed. If necessary, excavation equipment will be shut down to permit completion of compaction. Sufficient watering apparatus will also be provided by the Contractor, with due consideration for the fill material, rate of placement, and time of year.

E. The Geotechnical Engineer and Engineering Geologist will issue a final report summarizing their observations, test results, and comments regarding the Contractor's conformance with these specifications.

II. SITE PREPARATION

A. In areas to be graded, all vegetation and deleterious material such as rubbish and any construction debris from previous structures shall be disposed of off site. This removal must be concluded prior to placing fill.

B. The Civil Engineer shall locate all sewage disposal systems and large structures on the site or on the grading plan to the best of his knowledge prior to preparing the ground surface.

C. Soil, alluvium, or rock materials determined by the Geotechnical Engineer as being unsuitable for placement in compacted fills shall be removed and wasted from the site. The Geotechnical Engineer is to approve any material incorporated as a part of a compacted fill.

D. After the ground surface to receive fill has been cleared, it shall be scarified, disced, or bladed by the Contractor until it is uniform and free from ruts, hollows, hummocks or other uneven features that may prevent uniform compaction.

The scarified ground surface shall then be brought to optimum moisture, mixed as required, and compacted as specified. If the scarified zone is greater than 12 inches in depth, the excess shall be removed and placed in lifts on the order of 6 to 8 inches, depending upon material type and available construction equipment.

Prior to placing fill, the ground surface to receive fill shall be inspected, tested, and approved by the Geotechnical Engineer.

E. Any abandoned building, foundations, or underground structures, such as pipelines, or others not located prior to grading, are to be removed or treated in a manner prescribed by the Geotechnical Engineer.

III. <u>COMPACTED FILLS</u>

A. Any material imported or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable by the Geotechnical Engineer. Roots, tree branches, and other matter missed during clearing shall be removed from the fill.

B. Rock fragments less than 6 inches in diameter may be utilized in the fill provided:

1. They are not placed in concentrated pockets.

2. There is a sufficient percentage of fine-grained material to surround the rocks.

3. The distribution of the rocks is to be observed by the Geotechnical Engineer.

C. Rocks greater than 12 inches in diameter shall be taken off site.

D. Material that is spongy, subject to decay, or otherwise considered unsuitable shall not be used in the compacted fill.

E. Representative samples of materials to be utilized as compacted fill shall be analyzed in the laboratory by the Geotechnical Engineer to determine their physical properties. If any material other than that previously tested is encountered during grading, the appropriate analysis of this material shall be conducted by the Geotechnical Engineer as soon as possible.

F. Material used in the compacting process shall be evenly spread, watered or dried, processed and compacted in thin lifts to obtain a uniformly dense layer. Lift thickness shall be on the order of 6 to 8 inches. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Engineer.

G. If the moisture content or relative compaction varies from that required by the Geotechnical Engineer, the Contractor shall rework the fill until it is approved by the Geotechnical Engineer.

H. Each layer shall be compacted to 90 percent (90%) of the maximum density in compliance with the testing method specified by the controlling governmental agency. (In general, ASTM D 1557 will be used.)

IV. <u>GRADING CONTROL</u>

A. Inspection of the fill placement shall be provided by the Geotechnical Engineer during the progress of grading.

B. In general, density tests should be made at intervals not exceeding 2 feet of fill height. An adequate number of field density tests determined by the Geotechnical Engineer shall be made to verify that the required compaction is being achieved. The number of tests will vary depending on the soil conditions and the size of the job.

C. Density tests should also be made on the surface of the soils to receive fill as required by the Geotechnical Engineer.

D. All cleanout, processed ground to receive fill, key excavations, subdrains and rock disposal must be inspected and approved by the

Geotechnical Engineer (and often by the governing authorities) prior to placing any fill. It shall be the Contractor's responsibility to notify the Geotechnical Engineer and governing authorities when such areas are ready for inspection.

V. CONSTRUCTION CONSIDERATIONS

A. Erosion control measures, when necessary, shall be provided by the Contractor during grading prior to the completion and construction of permanent drainage controls.

B. Upon completion of grading and termination of observations by the Geotechnical Engineer, no further filling or excavating, including that necessary for footings, foundations, large tree wells, retaining walls, or other features shall be performed without the approval of the Geotechnical Engineer or Engineering Geologist.

C. Care shall be taken by the Contractor during final grading to preserve any berms, drainage terraces, interceptor swales, or other devices of a permanent nature on or adjacent to the property.

VI. ON-PAD UTILITY TRENCH BACKFILL RECOMMENDATIONS

A. SHALLOW TRENCHES: (Maximum Trench Depth of 2 Feet). Use soils approved by the Geotechnical Engineer. The soils should be compacted to 90 percent of the maximum dry density, as determined by ASTM Test Method D 1557, and tested by the Geotechnical Engineer. Compaction by flooding or jetting will be permitted only when, in the opinion of the Geotechnical Engineer, the backfill materials have a Sand Equivalent of at least 30 and the foundation materials will not soften or be damaged by the applied water.

B. DEEP TRENCHES: (Depth of Trench Greater than 2 Feet). The soils should be compacted to 90 percent of the maximum density, as determined by ASTM Test Method D 1557, and tested by the Geotechnical Engineer. The backfill placement method should consist of mechanically compacting the backfill soils throughout the trench depth.

If trench depth extends 5 feet, placement/compaction method should be reviewed by the Geotechnical Engineer. Contractor should exercise, and is responsible for, necessary and required safety precautions in all trenching operations.

C. TRENCHES UNDER VEHICLE PAVEMENTS: A minimum of 3 feet of fill should be placed over conduit, apply criteria B, above.

D. TRENCHES NEAR FOOTINGS: Approved backfill soils must be mechanically compacted to 90 percent of the maximum density, as determined by ASTM Test Method D 1557, and tested by the Geotechnical Engineer. The general backfill technique will be in accordance with the applicable criteria stated in A, above.

E. REPORTING: If the Geotechnical Engineer will be providing a written opinion as to adequacy of soil compaction and trench backfill, the entire operation should be performed under the Geotechnical Engineer's observation and testing.