Geotechnical Report

New Retail/Warehouse Building
Crescent City Harbor
Assessor's Parcel Number 117-020-016

January 23, 2014

Prepared For:
CIDA Inc.

Prepared By:
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Project No. 7934.00

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1.0 INTRODUCTION

In accordance with our Engineering Service Agreement, dated November 19, 2013, LACO Associates (LACO) has prepared this Geotechnical Report in support of the design and construction of a new approximately 10,000-square-foot retail/warehouse building at the subject property. The subject property (Site) is identified as Assessor's Parcel Number 117-020-016, and is located near the intersection of Starfish and Citizens Dock Road in Crescent City, California (Figure 1). The planned new building is anticipated to be a one-story metal and/or wood frame structure with a concrete slab-on-grade foundation. As we understand, CIDA Inc. (Client) is assuming that deep foundations or shallow foundation on engineered fill will be used to mitigate a known liquefaction hazard that exists for the area.

Our scope of services for this project was limited to:

- Review existing published geologic maps pertinent to the site and available unpublished soils and geologic reports
- Obtain boring permits with the Del Norte County Environmental Health Department
- Mark site and notify USA North
- Field exploration program utilizing Cone Penetration Testing (CPT)
- Prepare this 2013 California Building Code (CBC) compliant Geotechnical Soils Report documenting the results of the exploration with recommendations to support design and construction of the proposed building. The report also includes pavement design recommendations and quantitative liquefaction analysis, as stated as requirements in the RFP dated November 19, 2013.

Our scope of services did not include an environmental assessment for the presence of hazardous materials.

1.1 Previous Geotechnical Explorations

Previous geotechnical explorations reviewed by LACO for sites within the project vicinity include the following;

- GeoDesign Inc. completed a geotechnical exploration and report in December 2004, for proposed improvements at the Crescent City Waste Water Treatment Plant. The geotechnical exploration consisted of the installation of 11 geotechnical borings advanced to depths ranging from 7 to 94 feet below ground surface (bgs) and a seismic refraction survey.
- Treadwell & Rollo completed a geotechnical exploration and report in June 2011, for the rehabilitation of Crescent City Harbor from tsunami damages. The geotechnical exploration consisted of six geotechnical borings advanced to depths ranging from 28 to 51 feet bgs. A supplemental geotechnical report was prepared for the site in October 2011.
- LACO Associates performed a geotechnical exploration and report in February 2012, to support design and construction of a pedestrian promenade and restroom at Crescent City Harbor. The geotechnical exploration consisted of five geotechnical borings advanced to depths ranging from 17 to 31.5 feet bgs.
- SHN Consulting Engineers & Geologists, Inc., performed a geotechnical exploration and report in January 2013, for the proposed Visitor Center at the intersection of Highway 101 and Citizens Dock Road in Crescent City, California. The geotechnical exploration consisted of the installation of six borings (4 CPT, 2 continuous-core) and four backhoe pits.

2.0 LIMITATIONS

This Report has been prepared for the exclusive use of CIDA Inc. (Client), their contractors and consultants, and appropriate public authorities for specific application to Client's proposed development of the site. The extent and accuracy of LACO's exploration and report are consistent with the standard of care of other geoscience professionals practicing in the area at this time. A brochure prepared by Association of Firms Practicing in the Geosciences (ASFE) has been included as Attachment 1 of this Report. We recommend that all individuals reading this Report also read this brochure to gain an understanding of the scope and accuracy that can be reasonably expected from this investigation.

Data generated for this Report represents information gathered at that time and at the indicated locations. Subsurface conditions may change with time and under anthropologic influences. As such, the recommendations included in this Report are based, in part, on assumptions about subsurface conditions that may only be checked through observations and/or testing during subsequent project earthwork and foundation installation operations. Accordingly, the validity of these recommendations is contingent upon review of the subsurface conditions exposed during construction in order to check that they are consistent with those characterized in this Report. Upon request, LACO can discuss the extent of (and fee for) observations and tests required to check the validity of the recommendations presented herein.

LACO disclaims any and all liability for any errors, omissions, or inaccuracies in the information and data presented in this Report and/or any consequences arising therefrom, whether attributable to inadvertence or otherwise. LACO makes no representations or warranties of any kind including, but not limited to, any implied warranties with respect to the accuracy or interpretations of the data furnished. This Report is valid solely for the purpose, site, and project described in this document. Any alteration, unauthorized distribution, or deviation from this description will invalidate this Report. LACO also assumes no responsibility for any third-party reliance on the data presented. Additionally, the data presented should not be utilized by any third party to represent data for any other time or location.

3.0 FIELD EXPLORATION

3.1 Methods

To assess the in-situ soil conditions at the subject site, LACO performed subsurface exploration on August 2, 2013, consisting of Cone Penetration Test (CPT) and continuous core borings at locations denoted on Figure 2. CPT borings near the eastern edge of the proposed building were met with refusal on concrete debris within 2.5 feet of the ground surface. The continuous core boring CC-2 was located adjacent to CPT-2 to visually compare the soils to those interpreted by the CPT data. The continuous core borings CC-3 and CC-4 were installed in lieu of CPT borings, due to refusal of CPT equipment on shallow concrete debris. Boring CC-5 was installed in the proposed parking lot area southwest of the proposed building to characterize shallow soils within the parking lot.

Continuous core borings were logged in the field, in accordance with the Unified Soil Classification System (USCS) ASTM D2488 (Visual-Manual Procedure), by a LACO Staff Geologist. A computer-generated log of subsurface conditions was generated for each CPT boring. Boring logs and CPT logs for this exploration are provided as Attachments 2 and 3, respectively.

4.0 SITE AND SUBSURFACE CONDITIONS

4.1 Topography and Site Conditions

The project site is adjacent to the southern edge of the Crescent City Harbor inner boat basin, extending from Marine Way to Starfish Way on the southerly side of Citizens Dock Road. The topography at the Site is gently sloped with a westerly grade toward the ocean. The closest slopes to the site are approximately 200 feet northwesterly in the inner boat basin slopes. Slopes within the inner boat basin descend at gradients greater than 1H:1V in the harbor waters and are covered with rock slope protection.

The site is currently mainly vacant and covered with grass and gravel. A small restaurant in a mobile trailer occupies the northeastern edge of the site. A representative of the existing Englund Marine facility reported that the site was previously developed with a building.

4.2 Geologic Setting

Based on a review of the site and published geologic maps (CDMG 1987), the undisturbed native soils beneath the site consist of loose to dense sand (beach sand and Battery Formation) overlying stiff siltstone/mudstone "bedrock" (St. George Formation). A veneer of the fill soils placed during construction of the harbor covers the native soils. Where explored, the fill soils were encountered to a depth of 15 feet bgs and contained concrete debris near the eastern edge of the proposed building.

The Battery Formation is a Pleistocene-age terrace that is composed of marine nearshore sand and sand dune deposits over an abrasion platform cut into the St. George Formation.

The St. George Formation is primarily composed of marine-deposited grey siltstone and shale, with thin beds of sand and scattered pebbles. Based on soils observed in borings CPT-1, CPT-2, CC-2, and CC-4, siltstone interpreted to be St. George Formation is located approximately 28 feet bgs.

4.3 Seismic Setting

This project site is located within a seismically-active region in which large earthquakes are expected to occur during the economic life span (50 years) of the development. North of the Mendocino Triple Junction, the regional tectonic framework is controlled by the Cascadia Subduction Zone (CSZ), wherein oceanic crust of the Juan de Fuca/Gorda plate is being actively subducted beneath the leading edge of the North American plate. The CSZ in its entirety extends from the Mendocino Triple Junction to British Columbia. Plate convergence along the Gorda segment of the CSZ is occurring at a rate of approximately 30 to 40 millimeters per year (mm/yr) (Heaton & Kanamori 1984). Rupture along the entire CSZ boundary may produce an earthquake with a maximum moment magnitude (Mw) of 9.0 or greater (Satake 2003).

The project site is located in proximity to the late Quaternary-aged Big Lagoon Bald Mountain fault, which is a north-northwest trending thrust fault. Currently, the Big Lagoon Bald Mountain fault is not recognized by the State of California as being active within the past 11,000 years (CGS 2007). The Trinidad fault is the closest recognized active fault, located about 75 kilometers (km) to the south-southwest of the project site (CDMG 1983). The Trinidad fault is a northwest-striking, northeast dipping, low-angle thrust fault. The upper-bound earthquake considered likely to occur on the Trinidad fault has an estimated Mw of 7.3 (ICBO 1998).

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

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

4.4 Site Soils

Review of the subsurface exploration results previously conducted in the vicinity of the Site (GeoDesign 2004; Treadwell & Rollo 2011; LACO 2012; SHN 2013) and the subsurface data obtained during our current exploration indicate that the shallow soils underlying the Site primarily consist of sand and silty sand fills (to a maximum depth of 15 feet bgs) overlying poorly-graded marine sands and siltstone rock to the maximum depth explored (~30 feet).

LACO has not received any information documenting the construction of the fills; therefore, we are considering them non-structural fill. Concrete debris was encountered in three of the borings (CC-2 through CC-4). Additional debris may be present at other locations within the proposed development area.

4.5 Groundwater Conditions

Due to the proximity to the ocean and low elevation of the site, the groundwater elevation is likely tidally influenced. All four boring locations recorded saturated conditions at a depth of approximately 5 feet bgs. Previous geotechnical exploration adjacent to the Site recorded groundwater at depths ranging from 3.5 to 12 feet bgs (LACO 2012; SHN 2013). Based on the information provided above, groundwater should be anticipated within 5 feet of the ground surface.

5.0 GEOLOGIC AND SOIL HAZARDS

Potential geologic and soil hazards assessed for the subject Site include seismic ground shaking, surface fault rupture, liquefaction and related phenomena, settlement, flooding and high groundwater, tsunami inundation, and swelling or shrinking soils. The assessments for these potential hazards are presented below.

5.1 Seismic Ground Shaking

As noted in Section 4.3 of this report, the project site is situated within a seismically active area proximal to multiple seismic sources capable of generating moderate to strong ground motions. Given the proximity of significant active faults (the Cascadia Subduction Zone to the west and the Trinidad fault to the south), as well as other active faults within and offshore of northern California. The risk is high that the site will experience strong ground shaking during the economic life span of the proposed development.

Site-specific spectral response accelerations are presented in the subsequent recommendations section of this report (Section 5.3, Table 2).

5.2 Surface Fault Rupture

The closest recognized active faults to the site are the Trinidad fault and the Cascadia Subduction zone, located approximately 42 miles south (offshore segment) and 56 miles west, respectively. The project site is not located within an Alquist-Priolo earthquake fault hazard zone.

Based on the distance between the site and the closest active faults, and the lack of evidence indicating active faults traverse the site, the risk of surface fault rupture to occur within the proposed development area is estimated to be low.

5.3 Liquefaction

CPT boring data was utilized to perform quantitative analysis of the liquefaction potential and related dynamic settlement of the Site using the liquefaction analysis program CLiq Version 1.5.1.26 by Geologismiki. The calculations assumed a magnitude 7.3 earthquake with a peak acceleration of 0.623g (ASCE 7-10 Equation 11.8-1). Table 1 presents the method and seismic parameters used in the liquefaction analysis.

Table 1 - Liquefaction Analysis Input Parameters

Calculation Method [1]	NCEER 1998
Maximum Moment Magnitude [2]	7.3
Maximum Ground Acceleration [3]	0.623
Soil Aging Correction Factor (Kdr) [4]	1.52

Notes: 1. NCEER = Northwestern Center for Engineering Education Research
2. Adapted from Mw of Trinidad fault as described in Section 4.3
3. Maximum ground acceleration equal to calculated using ASCE 7-10 Equation 11.8-1
4. Soil aging factor only applied to Pleistocene age deposits (~15 – 30 feet bgs) using Hayati et al. (2008)

The calculation method used for the liquefaction analysis compares the Cyclic Stress Ratio (CSR) to the Cyclic Resistance Ratio (CRR), which is a comparison of the seismic driving force to the resistance provided by each soil layer. The CRR is divided by the CSR to find the Factor of Safety (FS), which is used to interpret the potential for the Site to liquefy. When the CSR exceeds the CRR (FS<1), the soil is considered to have a high liquefaction potential.

Our liquefaction analysis, based on the date presented in Table 2 and soil data from borings CPT-1 through CPT-2, indicates the Site has a high liquefaction risk. Possible dynamic settlement and lateral spreading as a consequence of liquefaction occurring at the Site was also determined using the CLiq software. Table 2 presents the CLiq software analysis results for liquefaction potential, dynamic settlement, and lateral spreading at the Site. The output from the CLiq software analysis is included in Attachment 4.

Table 2 - Liquefaction and Related Movement Analysis Results

Boring Location	Liquefaction Potential	Potentially Liquefiable Soil Depth (feet bgs)	Estimated Dynamic Settlement (inches)	Estimated Lateral Displacement (inches)
CPT-1	High	16 - 28	1.3	7
CPT-2	High	15 - 29	2.7	15

These results are further supported by CDMG Special Publication 115 Map S-3 (CDMG 1995), which show the vicinity to be near an area of moderate to high liquefaction potential. Therefore, from a quantitative and qualitative standpoint, we determine that the risk of liquefaction to occur at the Site to be high.

5.4 Static Settlement

The soils at the Site are primarily composed of loose to dense granular material. Generally, the soils exposed in our borings were relatively uniform. However, a thick fill soil containing concrete debris were observed within the borings.

Using the CPT data and an assumed 24-inch square footing with a bearing pressure of 2,000 pounds per square foot, static settlement for a shallow foundation founded 24 inches below the existing grade is anticipated to be less than one half of an inch.

5.5 Slope Instability / Landsliding

Geomorphic mapping of the area by the State of California indicates that there are no active or dormant landslides in the immediate vicinity of the site (CDMG 1983). The closest slopes to the Site are the descending fill slopes that are covered with RSP, located over 150 feet to the West of the Site.

Performing a quantitative slope instability analysis of the descending slopes along the barrier is specifically excluded from our scope of services for this project. However, Treadwell & Rollo performed a quantitative slope instability evaluation of similar slopes within adjacent harbor development areas, and concluded that the slopes were relatively stable under static condition, but potentially unstable under seismic conditions. In the absence of a site-specific slope instability analysis, LACO assumes that the risk static slope instability along the descending slopes is low.

5.6 Flooding, Tsunami, and High Groundwater

Flooding

The Del Norte County Flood Insurance Rate Map (Panel 06015C0331E, effective September 26, 2008) indicates that the Site is within flood hazard "Zone X" defined as areas being outside of the 0.2 percent annual chance floodplain. Therefore, based on the currently available published data, the risk of future flooding from a 100-year storm event, with the potential to adversely affect the new development should be considered low to moderate.

Tsunami

The most recent tsunami hazard maps published by the State of California (Sister Rocks Quadrangle, CGS 2009) indicate the site is within a predicated tsunami inundation zone. The site was inundated during the 1964 tsunami.

On the basis of the mapping by the state and historical tsunami occurrence for the area, the risk of tsunami inundation at the site is considered very high.

High Groundwater

As noted above, groundwater at the Site should be considered within 5 feet of the ground surface. Therefore, the risk of encountering groundwater in relatively shallow utility trenches or other required earthwork excavations is high.

5.7 Soil Swelling or Shrinkage Potential

Expansion potential represents a significant structural hazard to buildings founded on plastic clay soils that can undergo volume change where site conditions cause a seasonal fluctuation in soil moisture. Due to the presence of primarily non-plastic granular soils (see boring logs in Attachment 2), the risk of expansive soil movement (shrink or swell) at this site is considered negligible.

6.0 DISCUSSION AND CONCLUSIONS

Based on the results of this exploration and evaluation, we conclude construction of the proposed development is feasible, provided the recommendations of this report are incorporated into the project design and construction. Further, we judge the project will be subject to the following main engineering geologic/geotechnical considerations:

- Strong seismic ground shaking
- Potential liquefaction and resulting dynamic settlement and lateral spreading of underlying soils
- Potential tsunami inundation
- Presence of shallow groundwater levels during construction phase
- Presence of concrete and debris within the shallow subsurface, presumably from previous developments on the site

The level of mitigation to reduce the consequences resulting from the dynamic settlement and liquefaction hazards associated with strong earthquake ground shaking is at the discretion of the developer. Mitigation for a liquefaction hazard can range from minor structural improvements to extensive site preparation and specialized foundation design. In the following sections we provide recommendations for both end-bearing pile foundations and shallow foundation system options. Pile foundations should be used if Client determines that the potential dynamic settlement (estimated to be up to 2.7 inches) is not acceptable.

7.0 RECOMMENDATIONS

7.1 Foundation

Discussion

As noted above, the site is underlain by deep fill soils that may experience liquefaction and both static and dynamic settlement. Additionally, the site is located within an area that has been inundated by tsunami. A seismic event capable of inducing liquefaction and dynamic settlement will likely result in a tsunami that will inundate the site and cause significant damage to buildings within the inundation area. A deep foundation system designed to mitigate liquefaction and dynamic settlement may not necessarily ensure continued use of the building following liquefaction because of the risk of damage associated with a tsunami.

Given the risk of damage associated with tsunami inundation and the intended use of the proposed building as a warehouse/commercial structure, a shallow foundation system may be appropriate for this site if the stakeholders can accept the settlement related risks associated with a shallow foundation system.

LACO recommends two foundation design alternatives depending on the risk tolerances of the project stakeholeders:

- Option 1 is a shallow foundation design consisting of a structural mat slab supported on a 2.0-foot thick (minimum, below the base of the slab) section of controlled (structural) fill reinforced with woven geotextile.
- Option 2 is a reinforced concrete mat foundation supported on a deep foundation to reduce the risk of slab deformation, settling, and/or tilting during a liquefaction event.

The intent behind the structural mat slab foundation is to reduce the potential for excessive differential and total structural settlement associated with settlement of the fill soils following a liquefaction event. Utilization of a deep pile or pier foundation is intended to minimize settlements and preserve the functionality and utility of the structure following seismically-induced liquefaction.

In either option, flexible utility lines and utility line connections are recommended where underground utilities enter the building.

Structural Mat Foundation on Structural Fill (Option 1)

To mitigate the hazards from settlement and liquefaction-induced structural damage, a structural mat slab foundation supported on a reinforced structural fill may be utilized. Isolated foundation elements supporting structural loads should be tied together with grade beams or the structural slab to reduce the magnitude of differential dynamic settlement and the potential for structural collapse.

Due to the presence of deep fill soils, the structural fill beneath the mat slab should be reinforced with geogrid (Tensar TX1200, or equivalent). The structural fill under the rigid mat foundation should be a minimum of 24 inches thick as measured from the base of the rigid mat, and should extend a minimum of 5 feet beyond the rigid mat exterior.

Foundations bearing in the above-recommended reinforced fill can be designed for: (1) allowable bearing pressure of 2,000 pounds per square foot (psf) for static loads; (2) an allowable lateral bearing pressure of 150 pounds per cubic foot per foot of footing depth below the lowest adjacent soil grade; and (3) an allowable coefficient of friction of 0.25 for granular bearing soils at the base of the footings. From experience with similar materials and published values (Das 2009), we recommend a subgrade modulus of 150 pci.

Resistance to lateral forces may be computed using friction along or passive pressure against foundation elements. Friction between the undersurface of concrete footings and the supporting soil is available, as well as passive pressure acting against the sides of foundations. In computations, if friction and passive pressures are combined, the lesser value should be reduced by 50 percent.

Footing concrete should generally be placed neat against a firm soil surface that is relatively free of loose debris material. If backfill against formed footings is required, the backfill should be a structural fill material that is placed and compacted in accordance with the recommendations contained in this report.

Be advised that this type of foundation design may not preserve the function and utility of the structure following a liquefaction event as well as a deep foundation system (Option 2).

Mat Foundation Supported on Piers/Piles (Option 2)

To increase the potential for continued use following a liquefaction event, support the foundation with either prestressed, precast concrete piles or timber piles that are tied together with grade beams and gain support from the siltstone rock located at a depth of approximately 28 feet bgs. The mat foundation should be designed to span between the supporting piles without relying on any support from the subgrade soils. Pile design should be based solely upon end-bearing capacity; the contribution of the side friction to the overall pile axial load capacity should be neglected.

Allowable end-bearing capacities for driven piles ranging from 12 to 24 inches square at an expected refusal embedment depth of 30 feet bgs (2 feet into the siltstone rock located beneath the Site) are presented in Table 3. Calculations were performed using the Table 1 soil properties in Kulhawy's equation for toe-bearing resistance in sandy soils (Kulhawy et al. 1983).

Table 3 - Allowable End-Bearing Capacities
(Estimated using Kulhawy et al, 1983)

Square Pile Size (inches)	Allowable End-Bearing Capacity Per Pile (kips)
12	48
18	110
24	195

Note: A Factor of Safety of 3.5 is incorporated into the end-bearing values presented above

Design stresses of the piles should not exceed those presented in Table 1810.3.2.6 of the 2013 California Building Code. Piles should be spaced no closer than three times the width of each pile, measured center-to-center. Buckling capacity of the piles shall be determined by the engineer without relying on resistance from the potentially liquefiable soils.

Criteria for driven pile refusal will depend on pile size and design capacity, and on the Contractor's equipment. Refusal criteria should be established just prior to driving when these factors are known. Indicator piles should be driven at pre-selected locations to aid the Contractor in selecting his production pile lengths. LACO should provide consultation during the selection of locations, which should be near test borings to allow correlation of driving data with known subsurface conditions. Some variation in driving conditions should be expected, which could result in some pile cut-off and or deepened pile caps. Since the existing fill may contain obstructions, which could affect pile driving and alignment, the Contractor should consider pre-drilling, or spudding through, the existing fill. The pre-drilled hole diameter should not exceed 80 percent of the diagonal width of the pile.

Specific lateral load analysis and recommendations can be provided as an Addendum to this report if a deep foundation system is selected for use at the Site.

Where continued use of the development is desired following a liquefaction event, flatwork areas outside of the deep foundation supported structure should be designed to accommodate settlements and/or allow for repair.

7.2 Moisture Control for Concrete Slab Foundations

All concrete slabs intended for habitable space should be underlain by at least 4 inches of clean, ¾-inch, drain rock (slab base rock) to act as a capillary moisture break. To reduce the possibility of moisture migration through the floor slab, a 15-mil plastic membrane (vapor retarder) such as Stego Wrap (or equivalent) should be placed on the compacted base rock. To help protect the membrane against puncture during steel and concrete placement, and to provide for a more uniform curing of the concrete, the membrane should be covered with at least 2 inches of clean sand. These recommendations are intended to reduce the potential for moisture to infiltrate through the concrete. Flooring consultants and/or flooring manufacturers should be consulted for slab design where slab finishes require stringent moisture control.

7.3 Seismic Design Parameters

Based on the Site conditions encountered within the geotechnical borings, we have classified the Site as Site Class F consisting of "soils requiring site response analysis" (ASCE 7-10 – Table 20.3-1). However, the Site Class Definition Standards (ASCE 7- 20.3, 2010) provide an exemption to the requirement for a site response analysis for structures having fundamental periods of vibration equal to, or less than, 0.5 seconds. Since the structure is proposed to be less than three stories high, we assume the structure will have a fundamental period of less than 0.5 seconds. As such, the redefined Seismic Design Category for the Site is Class E, which consists of a "soft soil profile".

The design spectral response accelerations S_S, S_1, Fa, Fv, S_{MS}, S_{M1}, S_{DS}, and S_{D1} were determined using the USGS U.S. Seismic Design Map application (version 3.1.0, July 11, 2013), and based on the American Society of Civil Engineers (ASCE) Standard 7-10, Minimum Design Loads for Buildings and Other Structures analysis option. Calculated values are presented in Table 4.

Table 4 – Summary of Seismic Design Factors

Site Class	F_a	Fv	S_s	S_1	S_{MS}	S_{M1}	S_{DS}	S_{D1}
E	0.9	2.4	1.407	0.682	1.267	1.637	0.844	1.091

*Latitude and longitude are 41.7478° north and -124.1821° west.

These design spectral response accelerations are further defined as follows:

- Fa — Short period coefficient to modify 0.2-second period of mapped spectral response accelerations for Site Class E.
- Fv — Long period coefficient to modify 1.0-second period of mapped spectral response accelerations for Site Class E.
- Ss — Mapped spectral response acceleration, 5 percent damped, at 0.2-second period for Site Class B (%g).
- S_1 — Mapped spectral response acceleration, 5 percent damped, at 1.0-second period for Site Class B (%g).
- S_{MS} — Maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-second for Site Class effects (%g).
- S_{M1} — Maximum considered earthquake spectral response acceleration, 5 percent damped, at 1.0-second period for Site Class effects (%g).
- S_{DS} — Design spectral response acceleration, 5 percent damped, at 0.2-second period (%g).
- S_{D1} — Design spectral response acceleration, 5 percent damped, at 1.0-second period (%g).

7.4 Retaining Walls

Retaining walls, where needed, will be subjected to lateral loads from the adjacent soil. Where walls are unrestrained and free to deflect at the top, they may be designed for "active" soil pressures. If walls are restrained from movement at the top, soil pressures will approach "at-rest" pressures. To design for the lateral earth loads, we recommend using a friction angle of 30 degrees and a moist unit weight of 130 pounds per cubic foot to calculate soil pressures. Walls that have a drainage system constructed as recommended below, can be designed for the drained wall pressures, otherwise, undrained walls should be designed for the drained pressures plus hydrostatic water forces. In addition, if vehicle surcharges are anticipated adjacent to the walls, equivalent 2 feet of retained height should be added to the actual retained height during design.

Walls designed using the pressures presented above, should be constructed with a back drainage system consisting of a 1-foot-wide zone of drain rock extending from the base of the wall to at least 3 feet below the top of the wall backfill. The wall backfill can consist of either native soil or imported granular material; the upper 12 inches (minimum) of the wall backfill should consist of compacted native soil to reduce the potential for surface water to infiltrate into the granular backfill or back drain. A 4-inch-diameter, perforated, rigid PVC drainage pipe should be installed at the base of the wall back drain. The pipe should be laid to drain by gravity to a suitable drainage swale or site storm drain system. Rock for the back drain should meet the requirements of the Caltrans Standard Specifications (Section 68) for Class 2 Permeable Material or, alternatively, consist of clean, free-draining, ¾-inch gravel. The permeable backdrain material should be separated from the adjacent soils by a layer of non-woven filter fabric (Mirafi 140 or equivalent).

In lieu of the 12-inch-wide back drain, a prefabricated wall drain board (Tensar DCF100 or equivalent) may be used.

Resistance to the wall sliding can be calculated using friction between the base of the foundation and the underlying soil, and passive resistance on the sides of walls and footings. Recommendations for calculating lateral resistance, and for designing wall foundations, are presented in the Shallow Foundations section, above.

Section 1803.5.12 of the 2010 CBC requires a determination of lateral pressures on retaining walls due to earthquake motions for structures in Seismic Design Categories D, E, and F. We understand some jurisdictions are not requiring seismic loads to be applied to isolated retaining structures that are not connected to buildings. The seismic lateral force presented herein, if needed, was estimated using Mononobe-Okabe analysis (1929). Using a pseudo-static horizontal ground acceleration of 0.623g (ASCE Equation 11.8-1), the seismic lateral force equal to an equivalent fluid density of 70 pcf (rectangular distribution) should be used. In contrast to the static force, which is assumed to have a triangular distribution with resultant at a height of H/3 above the base of the wall, the resultant of the seismic lateral pressure should be assumed to act at a height of 0.6H above the base of the wall.

7.5 Flexible Pavement Design

The pavement structural section should be selected by the project design team to withstand the anticipated traffic loads over the design life of the pavement. A flexible pavement system may be used for this site consisting of Asphalt Concrete (AC) placed over compacted State of California Department of Transportation (CalTrans) Class 2 Aggregate Base (AB) which, in turn, rests on a properly prepared subgrade soil.

Resistance (R-) Value

Due to the presence of deep fills and the potential for lateral variation within the fills, an R-Value test was not conducted for this project. To be conservative and account for the potential for fine grain soils within the fill, we recommend that an R-value of 25 pounds per square inch (psi) exudation pressure be used for flexible and rigid pavement design at the Site.

Pavement Thicknesses

Our thickness recommendations presented herein are based on the assumption that the pavement subgrade soils will consist of the on-site fill soils with a design R-value of 25. Due to the potential for lateral variation within fill soils, exposed subgrade soils should be reviewed during construction to verify that the recommended R-value of 25 is appropriate. In some situations, it may be feasible to increase the R-value and decrease the thickness of the recommended pavement sections.

We selected a Traffic Index (T.I.) range of 5.5 to 7.0 (5 to 50 three axel trucks per day for a 20-year design life). The Caltrans Flexible Pavement Design Method was used to provide the recommended pavement sections presented in Table 5. These pavement section thicknesses and corresponding T.I.s should be checked by the project Civil Engineer for their applicability prior to final design and use.

Table 5 – Recommended Pavement Sections

Trucks Per Day	Traffic Index	AC	AB
5	5.5	2.0	10.0
10	6.0	3.0	10.0
25	6.5	3.0	11.0
50	7.0	3.0	13.0

AC = Type B Asphalt Concrete; Minimum thickness recommended = 2.0 inches
AB = Class 2 Aggregate Base (Minimum R-Value = 78)

Pavement Subgrade Preparation

Areas to receive pavement should be prepared per Sections 8.2 and 8.4 of this Report. However, the upper 6 inches of the subgrade should be scarified and recompacted to a minimum of 95 percent relative compaction per CalTrans Test Methods Cal 216 and 231. Following preparation of the pavement subgrade, the surface should be proof rolled with a loaded 10 yard dump truck prior to placement and compaction of aggregate base to check that the surface is firm and unyielding.

Pavement Structural Fill and Compaction Standard

Aggregate Base (AB) used within the pavement sections should be compacted to 95 percent relative compaction per CalTrans Test Methods Cal 216 and 231. Unless directed otherwise by the project Civil Engineer or local codes, structural fill below the AB should be compacted to at least 90 percent relative compaction, except for the upper 6 inches of subgrade which should be compacted to a minimum of 95 percent relative compaction. For convenience, compaction testing may be performed using ASTM methods (D-1557) in lieu of CalTrans methods provided the specified relative compaction noted in the preceding paragraphs are adhered to.

7.6 Rigid Pavement Design

A rigid Portland Cement Concrete (PCC) pavement section can be used in lieu of a flexible pavement section for added resistance to heavy vehicular loads. PCC pavement sections presented below are based on Portland Cement Association (PCA) design procedures using a computer program titled PCAPAV 2.10 and the design parameters listed in Table 6. These assumptions should be reviewed by the project design team to evaluate their suitability for this project. Changes in the assumptions will affect the corresponding pavement section design thickness.

- Modulus of Subgrade Reaction = 150 pounds per cubic inch (pci)
- Modulus of Rupture of Concrete = 410 psi
- Aggregate Interlock Joints (No Dowels)
- No Concrete Shoulders
- 20-year Design Life
- Load Safety Factor = 1.0 & Light Axle Wheel Load Category

Table 6 – Rigid Pavement Thicknesses

Average Daily Truck Traffic	Portland Cement Concrete (inches)
5	7.5
10	8.0
25	8.0
50	8.5

PCC pavement section thicknesses provided above are further contingent on the following:

- Subgrade soils should be scarified to a minimum depth of 6 inches below the finished subgrade elevation; moisture conditioned at, or within, 2 percent of the optimum moisture, and compacted to at least 95 percent relative compaction.
- Aggregate base (if used) should be compacted to at least 95 percent relative compaction.
- Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils are not allowed to become wet.
- PCC should have a minimum 28-day compressive strength of 3,000 psi. The concrete slump should be between 3 and 4 inches. The concrete should be properly cured in accordance with PCA recommended procedures, and vehicular automobile traffic should not be allowed on the pavement for three days or seven days for truck traffic.
- To help offset plastic shrinkage, concrete pavement may be reinforced with at least No. 3 bars at 24 inches on-center each way or 6 by 6-W2.0 by W2.0 wire mesh located within the middle one-third of the slab. Actual reinforcement needs for shrinkage should be determined by the project Engineer.
- Construction joint spacing (in feet) should not exceed twice the slab thickness in inches (e.g., 12 by 12 feet for a 6-inch slab thickness) with a maximum spacing of 15 feet. Joints should be laid out to form square panels. When not practical, rectangular panels can be laid out if the long dimension is no more than one and a half times the short dimension. The actual joint pattern should be determined by the project Engineer.
- Generally, control joints should have a depth of at least one-fourth the slab thickness (e.g., 1-inch-deep for a 4-inch-thick slab). The actual joint depth should be determined by the project Engineer.
- Unless otherwise recommended by the project Engineer, isolation (expansion) joints should extend the full depth of the slab and should be used only to isolate fixed objects abutting or within paved areas.
- Unless otherwise recommended by the project Engineer, thickened edges should be used along outside edges of concrete pavements. The edge thickness should be at least 2 inches greater than the concrete pavement thickness and taper to the actual concrete pavement thickness 36 inches inward from the edge. Integral curbs may be used in lieu of thickened edges.

7.7 Site Preparation

The proposed building area was reportedly previously developed. Any existing asphalt concrete pavement, concrete foundations, building rubble, sod, topsoil, and/or other debris encountered at, or below the existing ground surface, should be removed from the proposed building and adjacent flatwork areas. All earthwork, including, but not limited to, site clearing, grubbing, and stripping should be conducted during dry-weather conditions, as wet-weather construction could result in excessive rutting and/or mixing of debris materials with the underlying soils.

7.8 Cut and Fill Slopes

The current development plans do not include permanent un-retained cut or fill slopes. In the event that un-retained cut and/or fill slopes greater than 3 feet high are required, the slopes should be constructed in accordance with the Current Building Code.

7.9 Subgrade Preparation

Areas to receive fill should be cleared of any existing asphalt concrete pavement, concrete foundations, building rubble, sod, topsoil, and any other debris. The subgrade surface should be sloped at 10 percent or less. Vertical sides or steps may be necessary in some situations to achieve the required maximum slope. The exposed subgrade should be prepared as follows:

1. Scarify and recompact the upper 6 inches to a minimum of 90 percent of the maximum relative dry density as determined by ASTM D1557 method; and
2. Proof roll under the supervision of the Geotechnical Engineer or their representative. Proof rolling should be conducted with a fully-loaded, 10-yard dump truck with a minimum rear axle load of 8 tons or equivalent. The subgrade surface should provide a firm and unyielding surface under the load of the dump truck. Unsuitable soils identified during proof rolling should either be removed and replaced or addressed through supplemental recommendations from the Geotechnical Engineer.

7.10 Structural Fill

Structural fill materials used to support foundations, floor slabs, sidewalks, and pavements should be composed of non-expansive, low-plasticity material free of organic material, debris, and other deleterious material. Structural fills should be placed on a prepared subgrade as specified above. The material should contain no rocks larger than 3 inches in greatest dimension, nor more than 15 percent larger than 2 inches. Additionally, the material should meet the following specifications:

Plasticity index:	<15 percent
Liquid Limit:	<40 percent
Percent passing No. 200 sieve:	50 maximum, 5 minimum

Compaction Standard

Unless directed otherwise by the project Engineer or their representative, structural fill should be compacted to a minimum of 90 percent of the maximum relative dry density as determined by the ASTM D1557 method. A qualified Field Technician should be present to observe fill placement and perform field density tests per ASTM D-6938 at random locations throughout each lift to verify that the specified compaction is being achieved by the contractor. The structural fill should be placed on a prepared subgrade as specified above in loose lifts less than 8 inches thick.

7.11 Utility Trenches

Utility trench excavations should anticipate encountering saturated soils at depths less than 5 feet bgs. Utility lines should be designed to accommodate the saturated conditions. Additionally, trench dewatering may be necessary. Where trenches closely parallel a footing and the trench bottom is within a two horizontal to one vertical plane, projected outward and downward from any structural element, concrete slurry should be utilized to backfill that portion of the trench below this plane. The use of slurry backfill is not required where a narrow trench crosses a footing at or near a right angle.

7.12 Drainage

The Site should be graded to provide positive drainage away from foundations. A minimum gradient of 3 percent should be maintained for all hardscaped areas. A 5 percent gradient should be maintained for landscaped areas within 10 feet of a structure. The grading or landscaping design and construction should be such that no water is allowed to pond on the Site, nor to migrate beneath any structure. Runoff from hardscaped areas, roofs, patios, and other impermeable surfaces should be contained, controlled, and collected, and tight-lined to the storm drainage system.

7.13 Observation and Testing

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

- Monitor site grading and inspect exposed subgrade prior to placement of structural fills and/or pavement sections;
- Observe foundation excavations prior to placement of any forms or reinforcing steel; and
- Monitor the placement of structural fill, and test all structural fill to verify the required relative compaction is achieved.

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FIGURES

Project New Retail\Warehouse Bldg

By BED

Client CIDA Inc.

Date 1/20/2014

Proj. No. 7934.00

Figure 1

SITE VICINTY MAP

Source: lat 41.747446 long -124.182364 . Google Earth Image dated April 24, 2010. Accessed January 20, 2014.

LACO			
Project	New Retail\Warehouse Bldg	By	BED
Client	CIDA Inc.	Date	1/20/2014
Proj. No.	7934.00	Figure	2

SITE MAP

CPT-1 = Boring location and number

All locations area approximate

1" = 75'

Source: lat 41.747446 long -124.182364 . Google Earth Image dated April 24, 2010. Accessed January 20, 2014.

ATTACHMENT 1

ASFE Brochure

Important Information about Your
Geotechnical Engineering Report

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

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

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

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

Read the Full Report

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

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

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

ASFE
THE BEST PEOPLE ON EARTH

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

ATTACHMENT 2

Boring Logs

BORING CC-2

LACO

CLIENT CIDA Inc.
PROJECT NAME New Retail/Warehouse Building
PROJECT NUMBER 7934.00
PROJECT LOCATION Crescent City Harbor, CA
DATE STARTED 12/20/13 **COMPLETED** 12/20/13
GROUND ELEVATION **HOLE SIZE** inches
DRILLING CONTRACTOR Fisch Drilling
GROUND WATER LEVELS:
DRILLING METHOD GeoProbe 6600 DT
▽ **AT TIME OF DRILLING** 5.00 feet
LOGGED BY JMW **CHECKED BY** MRL
AT END OF DRILLING ---
NOTES Boring installed adjacent to CPT-2

DEPTH (ft)	SAMPLE TYPE NUMBER	GRAPHIC LOG	MATERIAL DESCRIPTION
0			
		0.5	FILL: Poorly graded sand with gravel and silt, coarse angular gravel, moist brown
		0.7	CONCRETE DEBRIS
			POORLY GRADED SAND WITH SILT: Yellow brown, loose to medium dense, moist, fill (?)
5			▽ Becomes gray, saturated
10			Increase in shell fraction
		14.0	GRAVEL LENS
15		15.0	POORLY GRADED SAND WITH SILT: Gray to yellow brown, loose to medium dense, saturated
		17.5	SILTY SAND: Yellow brown, medium dense, saturated
20		21.0	POORLY GRADED MEDIUM SAND: Brown, loose, saturated
25		26.5	POORLY GRADED SAND WITH SILT: Gray, loose to medium dense, saturated, abundant shell fragments, fine grained sand
		28.0	SILTSTONE ROCK: St. George Formation (?)
		29.0	

Bottom of borehole at 29.0 feet.

BORING CC-3

LACO

CLIENT	CIDA Inc.	**PROJECT NAME**	New Retail/Warehouse Building
PROJECT NUMBER	7934.00	**PROJECT LOCATION**	Crescent City Harbor, CA
DATE STARTED	12/20/13 **COMPLETED** 12/20/13	**GROUND ELEVATION**	**HOLE SIZE** inches
DRILLING CONTRACTOR	Fisch Drilling	**GROUND WATER LEVELS:**	
DRILLING METHOD	GeoProbe 6600 DT	▽ **AT TIME OF DRILLING**	5.40 feet
LOGGED BY	JMW **CHECKED BY** MRL	**AT END OF DRILLING**	---
NOTES			

DEPTH (ft)	SAMPLE TYPE NUMBER	GRAPHIC LOG	MATERIAL DESCRIPTION
0		0.5	TOPSOIL
		0.9	MIXED FILL: Coarse angular gravel, sand, silt and clay
		2.0	CONCRETE DEBRIS
		3.9	MIXED FILL: Coarse angular gravel, sand, silt and clay, moist
5			POORLY GRADED SAND WITH SILT: Yellow brown, loose to medium dense, moist, fill (?) ▽ Becomes gray, saturated
10		11.5	
		12.0	Halt in same at 12 feet bgs due to heaving sands

Bottom of borehole at 12.0 feet.

BORING CC-4

LACO

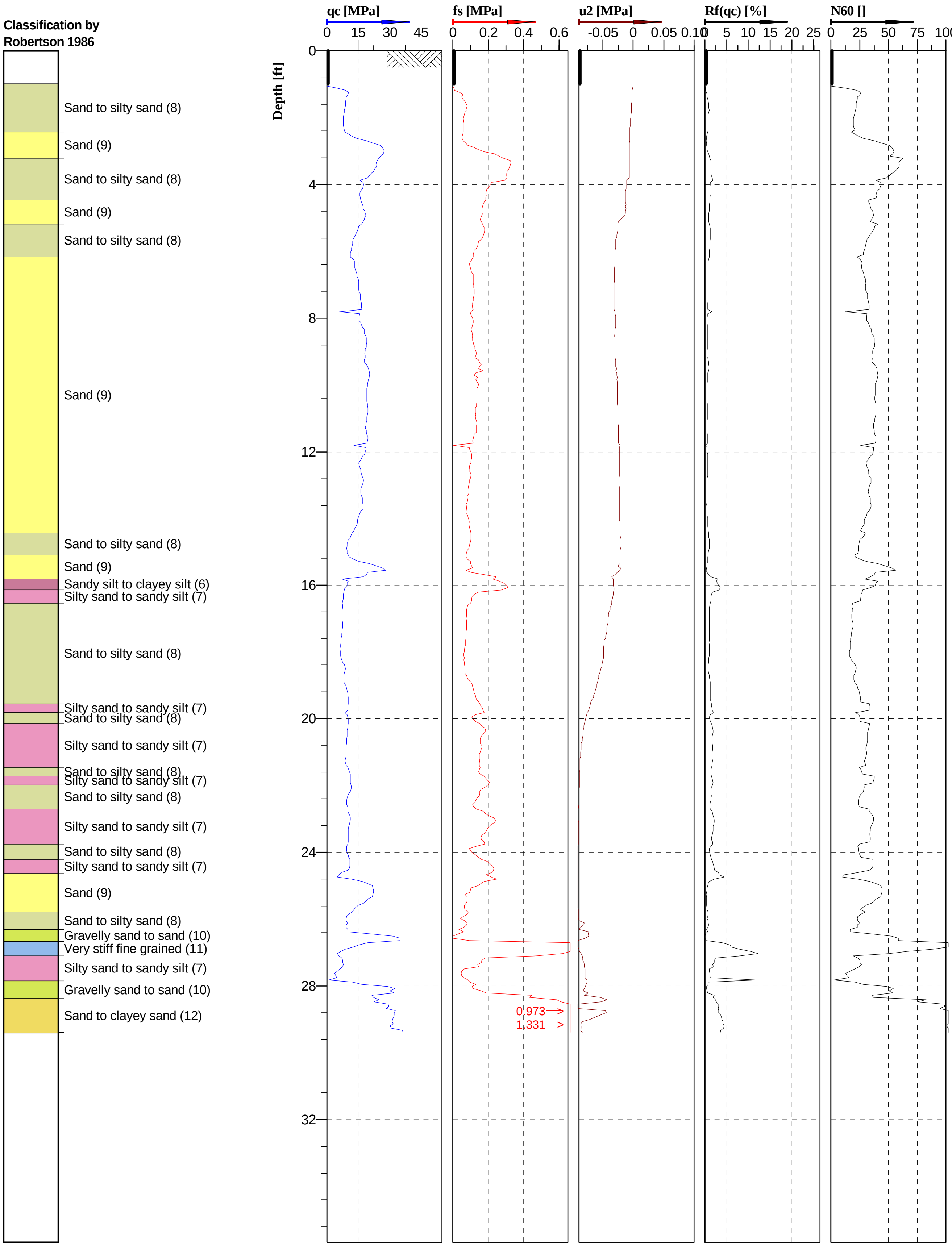
CLIENT CIDA Inc.
PROJECT NAME New Retail/Warehouse Building
PROJECT NUMBER 7934.00
PROJECT LOCATION Crescent City Harbor, CA
DATE STARTED 12/20/13 **COMPLETED** 12/20/13
GROUND ELEVATION **HOLE SIZE** inches
DRILLING CONTRACTOR Fisch Drilling
GROUND WATER LEVELS:
DRILLING METHOD GeoProbe 6600 DT
∇ **AT TIME OF DRILLING** 5.00 feet
LOGGED BY JMW **CHECKED BY** MRL
AT END OF DRILLING ---
NOTES

DEPTH (ft)	SAMPLE TYPE NUMBER	GRAPHIC LOG	MATERIAL DESCRIPTION
0			
0.5			TOPSOIL
			CONCRETE DEBRIS
3.0			POORLY GRADED SAND: Yellow brown, loose to medium dense, moist, fill (?)
5			∇ Becomes gray, saturated
10			Increase in shell fraction
15			
15.5			GRAVEL LENS
16.0			POORLY GRADED SAND WITH SILT: Yellow brown, medium dense, saturated
20			
23.5			POORLY GRADED MEDIUM SAND: Yellow brown, medium dense, saturated, slight oxidation
25			
26.0			SILTY SAND WITH GRAVEL: Brown, medium dense to dense, saturated, abundant shell fragments
28.0			
28.5			SILTSTONE ROCK: St. George Formation (?)

Bottom of borehole at 28.5 feet.

ATTACHMENT 3

CPT Logs

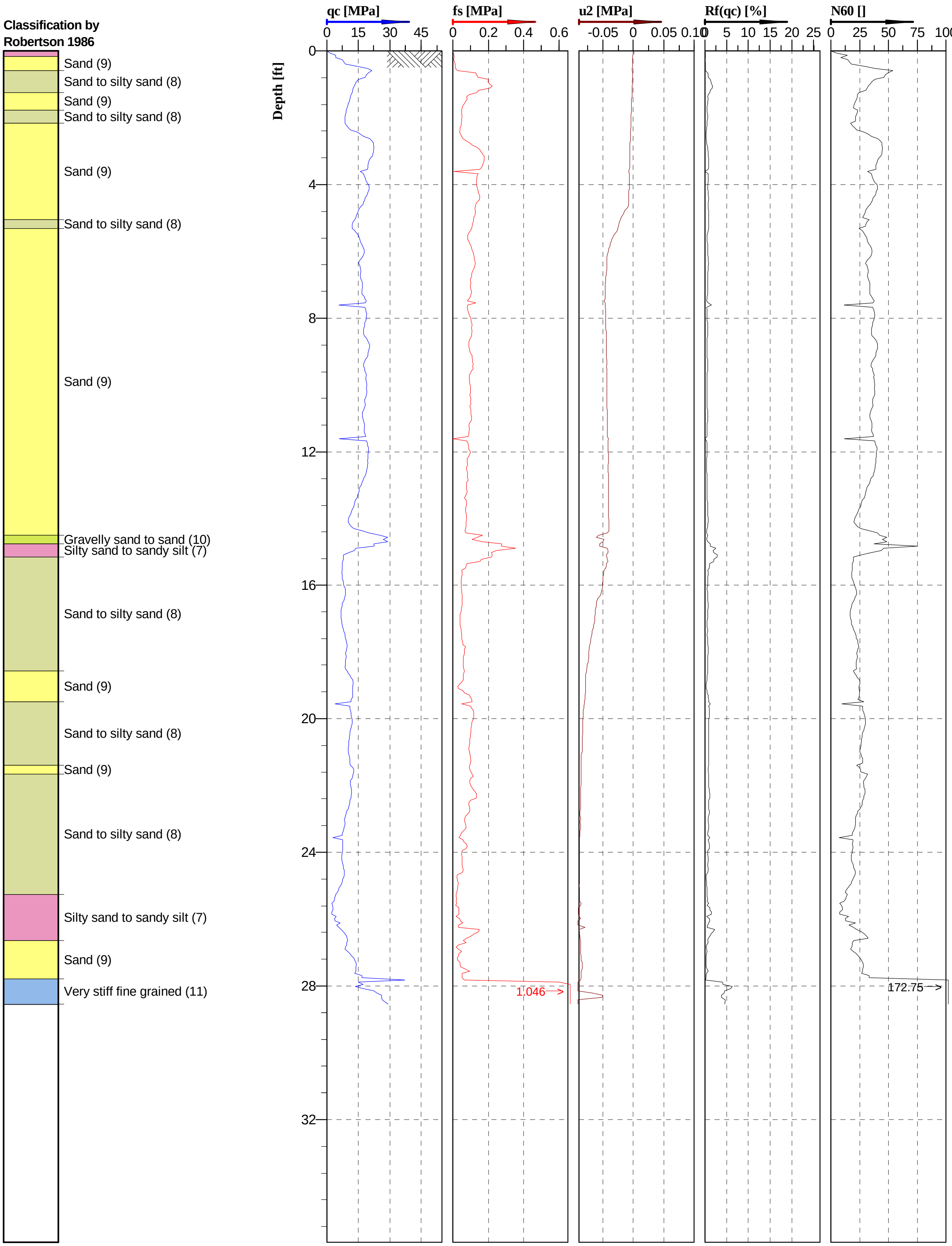


Fisch Drilling

U2

Cone No: 4057
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Crescent City, Ca	Position: X: 0.00 ft, Y: 0.00 ft	Ground level: 0.00	Test no: CPT1
Project ID: G4002	Client: LACO	Date: 12/20/2013	Scale: 1 : 50
Project: England Marine		Page: 1/1	Fig:
		File: laco cresent city harbor1.cpd	



U2

Fisch Drilling

Cone No: 4057
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Crescent City, CA	Position: X: 0.00 ft, Y: 0.00 ft	Ground level: 0.00	Test no: CPT2
Project ID: G4002	Client: LACO	Date: 12/20/2013	Scale: 1 : 50
Project: England Marine		Page: 1/1	Fig:
		File: laco cresent city harbor2.cpt	

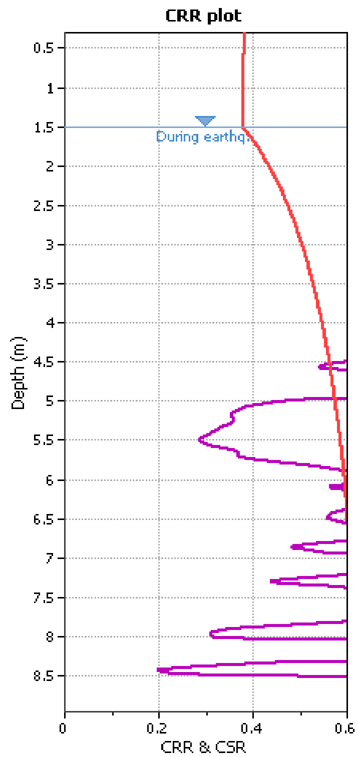
ATTACHMENT 4

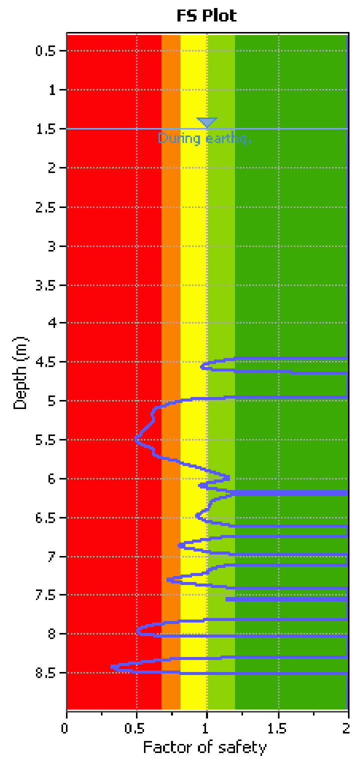
Liquefaction Analysis

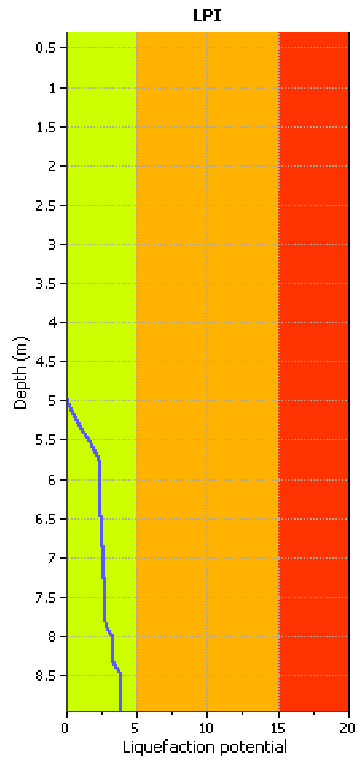
LACO

Project	New Retail/Warehouse Building	By	MRL
Client	CIDA Inc.	Date	1/5/2013
Proj. No.	7934	Page	1 of 2

Liquefaction Analysis (CPT-1)

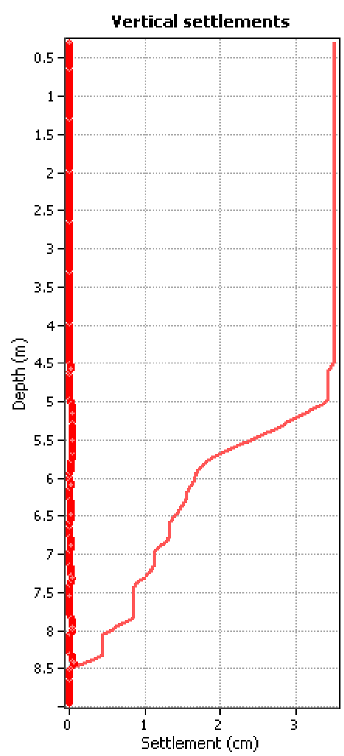


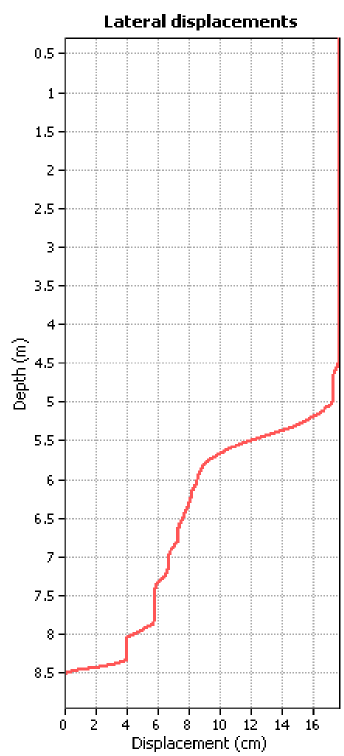




Cliq (version 1.7.5.27)

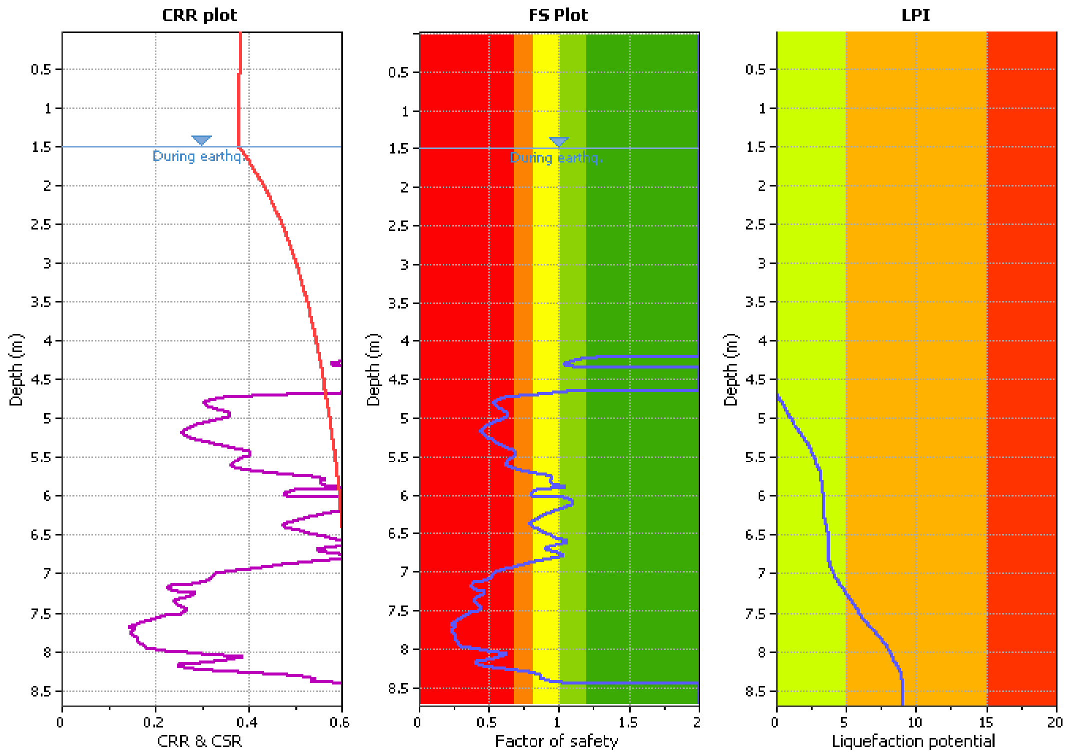
PGA	0.62 g
E.Q. Magnitude	7.3
CRR Aging Factor	1.5





Project	New Retail/Warehouse Building	By	MRL
Client	CIDA Inc.	Date	1/5/2013
Proj. No.	7934	Page	2 of 2

Liquefaction Analysis (CPT-2)



Cliq (version 1.7.5.27)

PGA	0.62 g
E.Q. Magnitude	7.3
CRR Aging Factor	1.5