DRAFT
GEOTECHNICAL INVESTIGATION
LAKEPORT COURTHOUSE
675 Lakeport Boulevard
Lakeport,California

Prepared For:

Mark Cavagnero Associates
1045 Sansome Street, Suite 200
San Francisco, California 94111

Prepared By:

Langan Treadwell Rollo
501 14th Street, 3rd Floor
Oakland, California 94612

DRAFT

Elena M. Ayers, PE, GE
Senior Project Manager

DRAFT

Richard D. Rodgers, PE, GE
Managing Principal

5 March 2014
Project No. 731563902

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GEOTECHNICAL INVESTIGATION
LAKEPORT COURTHOUSE
675 LAKEPORT BOULEVARD
Lakeport, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Langan Treadwell Rollo, for the planned Lakeport Courthouse at 675 Lakeport Boulevard in Lakeport, California. This investigation was performed in accordance with our proposal dated 20 January 2015. Previously, we performed a geotechnical investigation for the project and submitted the results in a report dated 10 February 2012. Since that time, the location of the building has been modified and additional information was requested for design of the building foundations. This report supersedes the 2012 report.

The site is irregularly shaped and is bound by Lakeport Boulevard on the north, retail buildings and parking lots on the east, the Lake County Chamber of Commerce visitor center and vista point on the west, and undeveloped property and businesses on the south, as shown on Figure 1. The western shoreline of Clear Lake is approximately 1/2 mile to the east. The site has maximum plan dimensions of approximately 520 by 560 feet, and is currently vegetated with low weeds and grass. The ground surface elevation at the site ranges from about 1343 to 1413 feet.[1] The western two-thirds of the site is relatively level, with ground surface elevations generally between approximately 1392 and 1395 feet, except near the western boundary, where the site slopes up to Elevation 1413 feet. The eastern one-third of the site slopes down toward the north and east at a maximum inclination of about 1.8:1 (horizontal to vertical) to approximate Elevation 1343 feet.

We understand the courthouse will be two stories. The lower level will be cut into the north and east slopes with a finished floor elevation at Elevation 1380 feet. The upper level will have a finished floor at Elevation 1394 feet. A parking lot will be located south of the courthouse.

1 Elevations discussed in this report are based on National Geodetic Vertical Datum of 1929.

Additional improvements will include a new access road from Lakeport Boulevard, a driveway to access lower-level of the building from the north side of the courthouse, an equipment enclosure, hardscaping, and landscaping. Retaining walls will be required to support portions of the eastern and northern edges of the building and the north side of the driveway. The approximate locations of the planned improvements are shown on Figure 2.

Based on information provided by the project structural engineer, Forell/Elsesser Engineers, Inc., we anticipate dead plus live column loads will be on the order of 376 kips if the building is framed using steel or 548 kips for concrete construction.

2.0 SCOPE OF SERVICES

Our scope of services, as outlined in our proposal dated 20 January 2015, consisted of further exploring the subsurface conditions at the site and performing supplemental engineering analyses to develop geotechnical conclusions and recommendations regarding:

- soil, rock, and groundwater conditions at the site
- site seismicity and seismic hazards
- site geology and geologic hazards
- presence of naturally-occurring asbestos in bedrock
- the most appropriate foundation type(s) for the proposed courthouse
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of building settlement, including total and differential settlements
- excavation
- cut slopes and temporary shoring
- basement and retaining walls

- concrete flatwork and flexible pavement
- site grading, including criteria for fill quality, fill placement, and compaction
- slope stability
- subgrade preparation and moisture protection for floor slabs
- corrosion potential of near-surface soil
- underground utilities
- seismic design parameters in accordance with the 2013 California Building Code
- construction considerations.

3.0 FIELD INVESTIGATION

3.1 Previous Investigation

In 2011, we investigated the site by drilling six borings and excavating three test pits at the site. The approximate locations of the borings and test pits are presented on Figure 2. Prior to performing the field investigation permits were obtained from Lake County Health Services Department and Lake County Air Quality Management District, and Underground Service Alert was notified to check that the locations of exploratory points were clear of existing utilities.

The borings, designated B-1 through B-6, were drilled on 28 and 29 November 2011 by Clear Heart Drilling of Santa Rosa, California using a truck-mounted drill rig equipped with hollow-stem augers. Three of borings, B-1 through B-3, were drilled at the location of the planned courthouse to depths ranging from about 40-1/2 to 60-1/2 feet below the existing ground surface (bgs). The remaining three borings, B-4 through B-6, were drilled in the planned parking lot to depths ranging from 5-1/2 to 6-1/2 feet bgs. The test pits, designated TP-1 through TP-3, were excavated on 28 and 29 November 2011 using a backhoe by Ryan Villanueva Construction of Lakeport, California. The test pits were excavated to depths of approximately 2-1/2 to 17 feet bgs. Our geologists logged the borings and test pits and obtained representative samples of the soil and rock encountered for classification and laboratory testing. The boring logs are presented in Appendix A on Figures A-1 through A-6. The test pit logs are presented in Appendix A on Figures A-7 through A-9. The soil and rock encountered during our investigation were classified in accordance with the classification systems presented on Figures A-10 and A-11, respectively.

Soil samples were obtained during drilling of the borings using the following sampler types:

- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch-outside diameter and a 1.5-inch-inside diameter, without liners
- Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch-outside diameter and a 2.5-inch-inside diameter lined with brass or stainless steel tubes with an inside diameter of 2.43 inches.

The samplers were driven with a 140-pound automatic hammer falling 30 inches. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers every six inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The driving of samplers was discontinued if the observed (recorded) blow count was 50 for six inches or less of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy and are shown on the boring logs. The blow counts used for this conversion were: 1) the last two blow counts if the sampler was driven more than 12 inches, 2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and 3) the only blow count if the sampler was driven six inches or less.

Upon completion of the field investigation, the boreholes were backfilled with cement grout in accordance with Lake County requirements. Soil cuttings generated from the borings were scattered onsite adjacent to each borehole. The test pits were backfilled with the excavated material, which was tamped in place using the backhoe bucket. The disturbed soil surfaces were misted with water and covered with hay to control dust.

3.2 Supplemental Investigation

To further evaluate the depths of bedrock and develop bedrock elevation contours, we retained Norcal Geophysical Consultants Incorporated (NCGI) to perform six seismic refraction surveys at the site. At one of the seismic lines, a multichannel analysis of surface waves (MASW) evaluation was also performed to measure shear wave velocities of the subsurface strata. The locations of the seismic lines were determined at the site by our geologist and are shown on Figure 2. The surveys were performed on 28 and 29 January 2015. The methodology and results of the surveys are presented in the NGCI report in Appendix B.

4.0 LABORATORY TESTING

4.1 Geotechnical Laboratory Testing

The soil and rock samples obtained from the borings and test pits were re-examined in our office to confirm the field classifications and to select representative samples for geotechnical laboratory testing. Soil samples were tested to measure moisture content, Atterberg limits, resistance value (R-value), and corrosion potential. The geotechnical laboratory test results are presented on the boring logs and in Appendix C.

4.2 Analytical Laboratory Testing for Asbestos

Four samples of fill, soil, and serpentinite-type rock collected from the test pits were submitted to an analytical laboratory for evaluation of naturally-occurring asbestos content. The test results are presented in Appendix D. The samples were analyzed using the Polarized Light Microscopy method, with sample preparation in accordance with California Air Resources Board Method 435, to evaluate the presence and quantity of asbestos (particularly chrysotile-type fibers) for the purpose of disposal. The laboratory results indicated that asbestos fibers were detected in one of the samples; however, the concentration was less than 0.25 percent chrysotile fibers by weight, as shown in Appendix D. Serpentinite material with less than 0.25 percent chrysotile fibers may be disposed offsite or used onsite as backfill without restriction.

5.0 SITE AND SUBSURFACE CONDITIONS

5.1 Site Conditions

The site is located on the northeast flank of a northwest-southeast trending, serpentinite bedrock ridge. The site is characterized by relatively steep, north-, east- and south-facing slopes throughout most of the site, with relatively level topography within the vicinity of the planned parking area and adjacent portions of the new courthouse, as shown on Figure 2. Based on subsurface information and observations made during the 2011 field investigation, it appears that previous grading activities have resulted in an extensive cut/fill pad at the top of the site. Slopes associated with the fill prism underlying the pad extend radially from the pad from the northeast to the south, with inclinations of approximately 1.8:1 (horizontal to vertical). A cut at the same approximate inclination was excavated into the slope below the Lakeport Community Center property, located immediately west of the planned site improvements. Steep cuts were also made downslope to the north of the planned development, most likely in association with Lakeport Boulevard construction. Along the eastern and southern edges of the site, cuts were graded at the base of the fill prism to create an unpaved access road from Lakeport Boulevard to the top of the fill pad. It appears that the access road is supported on the outboard edge by fill throughout its length. A new access road is depicted as being roughly within the same alignment of the existing road, as shown on Figure 2.

5.2 Site Geology and Subsurface Conditions

According to published geologic maps of the area (Regional Geologic Map, Figure 3), the site is underlain at depth by serpentinite bedrock materials of the Franciscan Assemblage. An engineering geologic map of the site is shown on Figure 4. Our generalized interpretations of the subsurface conditions at the site are depicted on Figures 5 and 6, Idealized Subsurface Profiles A-A' and B-B', respectively.

As much as 18 feet of fill overlying serpentinite bedrock was encountered in boring B-2, located on the northeastern crest of the fill pad. Fill up to 15-1/2 feet thick was encountered in test pit TP-2, located approximately 50 feet downslope of boring B-2. A small wedge of fill was identified in boring B-5 underlying the southwestern section of the pad, within the vicinity of the planned parking lot. Fill in this area is at least six feet thick; drilling was not advanced to bedrock in this boring. The fill is generally comprised of cobble- to boulder-sized serpentinite clasts, loose to dense clayey gravel to gravel with sand, stiff to very stiff clay with variable sand and gravel content, and hard sandy silt with gravel. Approximately two to three feet of fill, consisting of sandy to silty clay with gravel, appears to have been placed on the pad to the west of the main fill prism, likely to construct a level pad. Based on the results of an Atterberg limits test, the fill at the site has a high expansion potential.[2]

In general, the cut and fill slopes at the site appear to be in good condition. However, the existence of a buried topsoil layer under the fill in test pit TP-1 indicates that it is unlikely that the fill was placed in accordance with accepted engineering standards. During our site visit to conduct subsurface exploration activities, we noted several areas of topographic depressions on the fill pad, potentially resulting from fill settlement.

The fill is underlain by bedrock that consists of serpentinite. The condition of the serpentinite bedrock encountered during the field investigation was observed to be variable throughout the site and within the individual borings and test pits. Bedrock conditions are characterized as ranging from soft and deeply weathered to very hard with little weathering, with areas intact (few fractures) to intensely crushed. Bedrock was well-exposed in site cuts. The approximate depth to the top of the bedrock, as measured from the existing ground surface in our borings and test pits, and the corresponding elevation are summarized in Table 1. Bedrock was not

[2] Highly expansive soil undergoes large volume changes with changes in moisture content.

encountered in borings B-5 and B-6. Top of bedrock contours based on the results of the Norcal seismic refraction surveys are presented on Figure 7.

TABLE 1

Approximate Depths and Elevations of Bedrock

Boring/ Test Pit No.	Approximate Depth to Bedrock (feet bgs)	Approximate Bedrock Elevation (feet)
B-1	2.75	1388
B-2	18	1376
B-3	17.5	1378
B-4	2.5	1390
TP-1	1.5	1368
TP-2	16	1365
TP-3	1	1350

Groundwater was encountered in borings B-1 and B-3 at approximately 60 feet below ground surface, corresponding to Elevations 1331 feet and 1335 feet, respectively. The groundwater level at the site is expected to vary with seasonal rainfall.

6.0 REGIONAL GEOLOGY

The site is approximately 1/2 mile west of Clear Lake. The property is located within the Geysers-Clear Lake geologic region, within the northern California Coast Ranges geomorphic province. The Geysers-Clear Lake region lies within the Maacamas Mountains, between the San Andreas fault system to the southwest and the Coast Range thrust system to the northeast. The Coast Range thrust fault system offsets accretionary wedge rocks of the Franciscan assemblage from rocks of the Great Valley Sequence. The regional geology of the site vicinity is shown on Figure 3.

The Franciscan assemblage is a heterogeneous assemblage that consists largely of dismembered sequences of greywacke, shale, and lesser amounts of mafic volcanic rocks, thinly-bedded chert, and limestone. These rocks also occur with serpentinite and tectonic pods of blueschist in localized areas. The assemblage also contains many areas of sheared heterogenous mixes of these rocks, classified as mélange. The sedimentary and volcanic Franciscan rocks were formed in a marine environment, as attested by the abundance of foraminifers in the limestone and by radiolarians in the chert. Most of these rocks are probably Late Jurassic and Cretaceous in age (Bailey and others, 1964), but some of the chert and associated volcanic rocks are as old as Early Jurassic (Irwin and others, 1977; Blome and Irwin, 1983). In the northern Coast Ranges, some of the rocks assigned to the coastal belt of the Franciscan assemblage are as young as late Tertiary and are thought to have accreted to North America during post-middle Miocene time (McLaughlin and others, 1982). The Franciscan assemblage consists of mélange units and less disturbed sedimentary, meta-sedimentary, and meta-volcanic rocks that were scraped off the subducting plate in the Jurassic and Cretaceous time.

The Great Valley sequence consists of interbedded marine mudstone, sandstone, and conglomerate that range from Late Jurassic to Cretaceous in age (Bailey and others, 1964). It crops out as thick, monotonously bedded sections of strata that generally are markedly less deformed and more coherent than sedimentary sections of the Franciscan and also have greater lateral continuity. Where most fully developed, such as along the west side of the northern Great Valley, the aggregate stratigraphic thickness of Great Valley sequence is at least 12 kilometers (km). The strata normally lie depositionally on Coast Range ophiolite, except where disrupted by faults, but at the north end and along the east side of the Great Valley they overlie the Nevadan and older basement terranes of the Klamath Mountains and Sierra Nevada. This enormous thickness of clastic detrital material probably represents submarine fans and turbidity deposits that formed as a result of rapid erosion of the ancestral Klamath Mountains and Sierra Nevada.

Overlying the Franciscan assemblage within the site vicinity are localized younger deposits comprised of the early Holocene to late Pliocene (approximately 10,000 to 2.25 million years old) Clear Lake Volcanic rocks. The Clear Lake Volcanics are mostly silica-rich volcanic rocks (such as obsidian) located in and around Clear Lake, but also include some basaltic rocks. For the past million years or so, the main center of volcanic activity has been south and east of Clear Lake. Interbedded with the Clear Lake Volcanics is a Pliocene-Pleistocene sequence of lake and stream bed deposits up to approximately 2 km thick.

7.0 REGIONAL SEISMICITY AND FAULTING

The western margin of California is recognized by geologists and seismologists as one of the most active seismic regions in the United States. The three major faults that pass through the region, trending northwest-southeast, have produced approximately 12 earthquakes per century strong enough to cause structural damage. The faults causing such earthquakes are part of the San Andreas and Coast Range thrust fault systems. The major active fault systems in the vicinity of the project site are the Collayomi, Maacama-Garberville, Bartlett Springs and Huntington Creek-Berryessa fault zones. These and other faults of the region are shown on Figure 8. For each of the active faults within 100 kilometers of the particular site, the distance from the site and estimated mean characteristic Moment magnitude[3] event [2007 Working Group on California Earthquake Probabilities (WGCEP) (2008) and Cao et al. (2003)] are summarized in Table 2.

TABLE 2

Regional Faults and Seismicity

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude
Collayomi	6.8	Southeast	6.70
Maacama-Garberville	15	West	7.40
Bartlett Springs	24	Northeast	7.30
Hunting Creek-Berryessa	38	East	7.10
Rodgers Creek	52	South	7.07

3 Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude
Total Hayward-Rodgers Creek	52	South	7.33
Great Valley 2	55	East	6.50
N. San Andreas - North Coast	55	Southwest	7.51
N. San Andreas (1906 event)	55	Southwest	8.05
Great Valley 3, Mysterious Ridge	56	East	7.10
Great Valley 1	62	East	6.80
N. San Andreas - Offshore	81	West	7.37
Great Valley 4a, Trout Creek	81	East	6.60
West Napa	84	Southeast	6.70

Figure 8 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through December 2000. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 9) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, M_w, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake of 17 October 1989, in the Santa Cruz Mountains with a M_w of 6.9, approximately 240 kilometers from the site.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake (M_w = 6.2). The most recent earthquake felt in the vicinity of the site occurred on 24 August 2014 and was located on the West Napa Fault, approximately 100 kilometers southeast of the site, with a M_W of 6.0.

The 2007 WGCEP at the U.S. Geologic Survey (USGS) predicted a 30-year probability of a Magnitude 6.7 or greater earthquake on one of the active faults in the San Francisco Bay Area to be about 63 percent. The Hayward-Rodgers Creek and North San Andreas faults are estimated to have 30-year probabilities of a magnitude 6.7 or greater earthquake of 31 percent and 21 percent, respectively (WGCEP, 2008).

In addition to the active faults listed in Table 2, the site is mapped as being located within close proximity of two potentially active fault traces, as discussed in a geological hazards screening evaluation performed by Fugro-William Lettis & Associates (FWLA), dated 19 May 2010. The West Margin fault is located approximately 0.8 miles to the west of the site and is considered to be active within the Quaternary period, 1.8 million years ago to present). The western trace of the Big Valley fault is mapped approximately 700 feet east of the site. Portions of this fault located east/southeast of the site exhibited displacement within the Late Quaternary period (about 700,000 years ago to present). Based on our review of the Lake County General Plan Background Report, dated February 2003, we understand that Lake County considers faults with Quaternary displacement as potentially active. These faults are not considered to be potential seismic sources for large earthquakes; however fault rupture on these faults could occur as sympathetic movement during a large earthquake on one of the other fault traces in the region.

8.0 DISCUSSION AND CONCLUSIONS

On the basis of the results of our subsurface investigation and geologic reconnaissance, we conclude that from a geotechnical engineering standpoint, the site can be developed as planned. The primary geotechnical concerns for the project include:

- the presence of variable subsurface conditions, including shallow bedrock in the western portion of the site, highly expansive soil, and up to 18 feet of fill in the eastern portion of the site
- support of the planned courthouse on the existing fill
- proper design and construction of below-grade and/or retaining walls to support the existing fill slopes, new fill, and rock.

These and other geotechnical concerns, and their impact on foundation design, excavation, and construction, are discussed in the following sections.

8.1 Seismic and Geologic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the project site. Very strong shaking during an earthquake can result in ground deformation associated with seismically-induced slope instability, soil liquefaction[4], lateral spreading[5], and cyclic densification[6]. Soil most susceptible to liquefaction, lateral spreading, and cyclic densification is loose, clean, uniformly graded sand and silt of low plasticity that is relatively free of clay.

We conclude the primary geologic hazards that may affect the site are the potential for strong to very strong shaking associated with a large-magnitude earthquake on a major active fault in the region and ground deformation associated with sympathetic movement of a nearby

4 Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

5 Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

6 Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing differential settlement.

potentially-active fault during such an event. These and other geologic hazards are discussed in the following sections.

8.1.1 Strong Ground Shaking

The intensity of the earthquake ground motion at the site will depend upon the type of source fault (i.e. reverse, strike-slip), distance of the earthquake epicenter, magnitude and duration, as well as site geologic conditions. We conclude that the site will be subjected to strong to very strong ground shaking from a major earthquake on at least one of the nearby active faults during the design life of courthouse.

8.1.2 Surface Fault Rupture

Historically, ground surface ruptures closely follow the traces of geologically young faults. The property is not mapped as being within an Alquist-Priolo Zone and no known active or potentially active faults exist on the site. In their fault rupture hazard evaluation, FWLA concluded a moderate potential for fault rupture exists for the site, likely associated with the potentially active, western trace of the Big Valley fault or a potentially unknown, active fault trace.

Based on our review of the FWLA report, and the California Fault Activity Map (Figure 7) and associated report (Jennings and Bryant, 2010), we understand that ground ruptures were mapped approximately one mile southeast of the site on the Big Valley fault following the 1906 earthquake, possibly as a result of sympathetic fault movement with the San Andreas fault.

We did not observe evidence for faulting in the borings or test pits; however, our field investigation did not include a specific geologic hazards evaluation for fault rupture potential, which would include continuous fault trenching and/or seismic refraction surveys across the entire site.

On the basis of our review of the regional geologic map of the area, it appears that the serpentinite outcrops that penetrate up through the overlying younger lake and terrace deposits within this area are part of a north to northwest trending, steeply dipping bed of serpentinite. The serpentinite all appears to be located west of the western trace of the Big Valley fault, and the eastern edge of the serpentinite may actually lie in faulted contact (along the western trace of the Big Valley Fault) with the underlying basement rock beneath the Tertiary lake deposits. Thus, areas such as our site which appears to be entirely underlain by serpentinite would be located west of the western trace of the Big Valley fault.

On the basis of our not observing any fault features in our test pits or borings, and our observations of continuity of bedrock (serpentinite) across the site, we conclude that the potential for surface fault rupture at the site is low, but not negligible. We recommend that our geologist observe the foundation excavations for the building during construction to confirm our conclusions that that no active faulting is observed beneath the structure.

8.1.3 Liquefaction and Lateral Spreading

Groundwater was encountered at approximately 60 feet bgs in bedrock, between Elevations 1331 and 1335 feet. Based on our observations of the subsurface conditions, we conclude that the potential for seismically-induced liquefaction and liquefaction-induced ground failures such as lateral spreading at the site is very low.

8.1.4 Cyclic Densification

Seismically-induced compaction or cyclic densification of non-saturated cohesionless soil (sand, silt, and gravel above the groundwater table) caused by earthquake vibrations may result in settlement. Approximately 2-1/2 to 3-1/2 feet of loose gravel with sand and medium dense gravel with clay were encountered above the groundwater in borings B-2 and B-3. We compute that shallow foundations and surface improvements bearing within these non-saturated granular layers may settle as much as 1/4 inch due to strong shaking from a large earthquake, with a possibility of abrupt differential settlements of as much as 1/4 inch.

8.1.5 Landslides and Slope Stability

On the basis of our observations, we conclude the existing fill slopes at the site are stable and the potential for deep-seated landslides to develop at the site is low. However, we conclude there is a moderate potential for sloughing or raveling of the fill on the surfaces of the slopes, especially when subjected to prolonged wet weather. Where not retained by new walls, a possibility exists that the fill slopes may creep. The risks associated with these hazards can be reduced by flattening slopes, implementing proper drainage control, and maintaining vegetation on the slopes.

We anticipate site grades will generally be maintained in their current condition, except where retaining walls are planned and where a cut on the order of 15 feet will be excavated into the slope to accommodate the lower level of the courthouse. We conclude the planned development should not adversely affect the stability of the slopes, provided the proposed grading, fill placement, retaining walls, and drainage are designed and constructed in accordance with our recommendations.

8.1.6 Subsidence

Subsidence typically occurs as a result of subsurface fluid extraction (e.g. groundwater, petroleum) or compression of soft, geologically young sediments from vertical loads. Groundwater extraction for municipal and agricultural use has the potential to cause ground subsidence. The groundwater at the site was encountered within bedrock. Based on our observations, we judge the potential for subsidence at the site due to groundwater extraction to be low. We expect that subsidence resulting from future extraction of groundwater would be negligible.

8.1.7 Expansive Soil

Expansive soils are those that shrink or swell significantly with changes in moisture content. The clay content and porosity of the soil also influence the change in volume. The shrinking and swelling caused by expansive clay-rich soil often results in damage to overlying structures. Based on the field observation and test results, it appears that fill materials encountered on the pad are highly expansive with a plasticity index (PI) of 32.

8.1.8 Flood Inundation

Our review of Lake County Special Flood Hazard Area Maps and FEMA Digital Flood Insurance Rate Maps indicate that the site is not located within an area subject to flooding.

8.1.9 Seiches

Seiches are large waves that occur within enclosed bodies of water as a result of ground shaking caused by seismic activity. Seiches can cause damage by flooding caused by wave run-up on the shore, or if they overtop a dam or berm. The site is located approximately 1/2 mile inland of the western shore of Clear Lake, with an elevation difference of approximately 14 feet between the lake and lowest point of the property. The elevation difference between the lake and the proposed development at the top of the site is 51 feet; consequently, we conclude that the potential for damage to site improvements as a result of a seiche from Clear Lake is negligible.

8.2 Corrosion Potential

We performed corrosivity tests on soil samples collected from boring B-3 at depths of 3 and 16 feet bgs. The soil samples were tested in accordance with Caltrans and ASTM protocols by Environmental Technical Services (ETS) of Petaluma, California. The corrosivity test results are presented on Figure C-4 in Appendix C.

8.3 Settlement of Existing and New Fill

As much as 18 feet of fill is present at the site, and we anticipate on the order of 5 to 10 feet of new engineered fill will be placed at the northeast corner of the building pad and for the planned driveway, where retaining walls are planned. It is not known whether the existing fill at the site was placed in a controlled manner. SPT blowcounts recorded during our field investigation indicate the fill is generally stiff to very stiff (for clays and silts) and loose to dense (for gravels), as discussed in Section 5.2. Based on the extent and variability of the fill at the site, as well as topographic depressions observed on the fill pad, we conclude that settlement of the existing fill may occur under new loads.

We estimate that near-surface site improvements supported on fill may experience erratic settlements on the order of 1-1/2 percent of the total thickness of existing fill and on the order of 1/2 percent of the total thickness of proposed fill, resulting in settlements of about 3-1/4 inches for the 18 feet of existing fill and between about ¼ and ¾ inch for the 5 to 10 feet of planned engineered fill.

8.4 Foundation Support and Settlement

The proposed building location is underlain by:

- variable subsurface conditions, with as much as 18 feet of existing hetergeneous fill at the eastern portion of the site and bedrock depths ranging from about 3 to 15 feet bgs within the planned building footprint
- highly expansive near-surface fill.

Expansive soil is subject to high volume changes during seasonal fluctuations in moisture content, which can cause cracking of foundations and floor slabs. \The detrimental effects of near-surface expansive soil can be mitigated by moisture-conditioning the expansive soil below slabs, placing non-expansive fill below slabs, supporting foundations below the zone of severe moisture change, and/or designing foundations to resist the movements associated with the volume changes.

The variable depth to bedrock and thickness of existing fill within the building footprint can result in differential settlement of soil underlying the planned building; the settlement is expected to be erratic. To reduce the potential for differential movement of foundations resulting from fill settlement and expansive soil, we conclude foundations for the proposed courthouse should gain support in the bedrock underlying the fill. Where rock is encountered at or near the subgrade level, the structure can be supported on spread footings. Where shallow rock is encountered on the lower portions of the existing slopes at the northern and eastern edges of the building (below the existing fill prism), we conclude spread footings can be used provided that adequate vertical and lateral support on the slopes can be achieved. Where bedrock depth or slope renders footings impractical, drilled piers bearing in rock may be used to support the structure. We anticipate that footings and drilled piers bottomed in rock will settle less than an inch.

Approximate top of bedrock contours were developed using the results of our field investigation and our supplemental investigation and are shown on Figure 7. Additional investigation consisting of exploratory pits, borings, or piers can be performed during the initial stages of construction to further confirm the depths to bedrock. It is therefore important that the foundation design and construction documents allow for switching from one foundation type to the other as field conditions dictate.

Where the northern and eastern edges of the building will extend over the existing fill slopes, we have assumed that drilled piers or footings installed on the slope will be capped with a continuous grade beam supporting a formed wall backfilled with engineered fill to support the building slab. Footings behind retaining walls will need to be deepened below the zone of influence of the wall, or drilled piers be used, to reduce the potential for surcharging the wall.

8.5 Floor Slabs

The floor slab will be underlain by bedrock or fill consisting of very stiff sandy clay, hard sandy silt, or medium dense clayey gravel, and we conclude the floor slab will need to be designed as a structural slab to span between footings and piers and not rely on the ground for support. For the upper level floor slab, if movement of water vapor through the slab is undesirable, a capillary moisture break and water vapor retarder (recommended in Section 9.3) can be installed beneath the slab to reduce water vapor transmission through the slab. We conclude the lower level floor slab will need to be waterproofed.

8.6 Excavation and Shoring

We understand the lower level of the courthouse will be cut into the fill slope with a finished floor elevation at 1380 feet, approximately 15 feet below the existing grade at the top of the slope. Additional excavations are planned to be cut into the existing bedrock and fill slopes to construct the driveway along the northern side of the courthouse; these excavations will be up to approximately 6 feet deep. The excavations at the site will need to be permanently retained.

The soil to be excavated consists predominantly of clay, sand, silt, and gravel, which can be excavated with conventional earth-moving equipment such as loaders and backhoes. We anticipate that bedrock will be encountered within the excavations, especially at the western portion of the site outside the zones of existing fill. Where bedrock is present within the planned depth of excavation, the contractor will need to select equipment that is capable of excavating and removing rock from the site. Excavations deeper than five feet that will be entered by workers should be shored or sloped in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926).

If there is insufficient space to slope the sides of the excavations, shoring will be required. Considering the anticipated excavation depths and the expected soil/rock conditions, we conclude that soldier-pile-and-lagging shoring systems are suitable for this project. A soldier-pile-and-lagging system consists of steel soldier beams placed in vertical predrilled holes that are backfilled with concrete and wood lagging between the soldier beams as the excavation proceeds.

Depending on the height of the shoring system, lateral restraint such as tiebacks may be required. Tiebacks will extend significant distances into the soil and rock behind the wall, and if they will be incorporated into a permanent retention system, use of deep foundations, utilities, and trees may need to be restricted or used cautiously in areas behind the wall. For permanent retention systems, double-corrosion protection will be required for tiebacks and all other system components.

9.0 RECOMMENDATIONS

Our recommendations regarding earthwork, foundations, basement and retaining walls, pavement design, and other geotechnical aspects of this project are presented in this section.

9.1 Earthwork

9.1.1 Site Preparation

Any vegetation and organic topsoil should be stripped in areas to receive new fill or site improvements. Voids resulting from demolition activities should be properly backfilled with engineered fill as described in Section 9.1.3. Topsoil with an organic content greater than three percent should not be reused as compacted fill; however, this material may be stockpiled onsite and reused in landscaped areas if approved by the project architect.

9.1.2 Subgrade Preparation

In areas to receive fill or near-surface site improvements, the exposed subgrade soil should be properly scarified, moisture-conditioned, and recompacted. Expansive subgrade soil should be scarified to a depth of at least eight inches, moisture-conditioned to at least three percent above optimum moisture content, and compacted to at least 90 percent relative compaction.

Where lean clay, granular soil, or rock with a low to moderate expansion potential (defined as material with a plasticity index less than 25) is exposed during the subgrade preparation process, the scarified surface should be moisture-conditioned to above the optimum moisture content and compacted to at least 90 percent relative compaction. The soil subgrade should be kept moist prior to placing new fills, pavements, or near-surface improvements. An exception to this general procedure occurs within the proposed pavement areas, where the upper six inches of low to moderately expansive pavement subgrade soil should be compacted to at least 95 percent relative compaction.

If areas of weak soil are encountered during subgrade preparation, we recommend the areas be repaired by either: 1) removing and replacing the weak soil with engineered fill, 2) over-excavating the weak material and filling the excavation with a reinforcing geotextile (Mirafi 500X or equivalent) overlain by granular fill, or 3) using lime- or cement-based admixtures to strengthen the weak soil.

9.1.3 General Fill Placement and Compaction

We anticipate fill placement during construction of the planned courthouse will consist primarily of backfill behind and around retaining walls and for utility trenches. The soil excavated during construction will be acceptable for use as general site fill and backfill provided it is free of organic material, is non-hazardous, and contains no rocks or lumps larger than three inches in greatest dimension. If the onsite expansive clay is to be used as fill or backfill, it should be moisture-conditioned to at least three percent above optimum moisture content, placed in lifts not exceeding eight inches in uncompacted thickness, and compacted to between 88 and 92 percent relative compaction for fill thickness equal to or less than five feet and 92 percent relative compaction for fill thickness greater than five feet. Granular soil used as fill should be moisture-conditioned to above optimum moisture content, placed in horizontal lifts not exceeding eight inches in uncompacted thickness, and compacted to at least 90 percent relative compaction for fill thickness equal to or less than five feet and 95 percent compaction for fill thickness greater than five feet. Clean sand or gravel (defined as soil with less than

10 percent fines by weight) used as backfill should be compacted to at least 95 percent relative compaction.

All fill material should be submitted to the Geotechnical Engineer for approval at least 72 hours before it is to be used on site. Where imported fill is required, the grading subcontractor should provide analytical test results or other suitable environmental documentation at least three days before use at the site indicating that the proposed fill material is free of hazardous materials. If this data is not provided, up to two weeks may be required to perform any required analytical testing on proposed import soil.

9.1.4 Fill Slopes

Where fill is planned along existing slopes, such as behind and around new retaining walls, the fill should be keyed and benched into the slope to reduce the potential for differential settlement and movement of the fill. Prior to placement of fill, the exposed subgrade should be scarified, moisture-conditioned, and compacted as previously discussed in Section 9.1.2. If the final fill surface will be sloped, we recommend the fill slope be overbuilt by placing and compacting horizontal lifts of fill as described in Section 9.1.3. Subsequently, the fill slope should be cut back to achieve the proper slope inclination.

We recommend that fill slopes be designed to have a maximum slope inclination of 2:1 (horizontal to vertical). At the toe of the proposed fill slope, a keyway should be installed to interconnect the new fill material into the existing strata. The keyway should be at least five feet wide at the base and extend at least two feet into competent soil or rock or at least 15 percent of the overall slope height, whichever is greater. The side slopes of the keyways should not be steeper than 1:1.

Where new fill is placed over existing slopes that are steeper than 5:1, the fill should be benched as the fill operation proceeds upslope. These benches will provide horizontal surfaces for the placement and compaction of the fill and reduce the effects of downward creeping of the soil. Benches should be a maximum of five feet high and should expose competent soil or rock along the base of the bench.

The face of fill slopes should be planted with deep-rooted vegetation and covered by an erosion control blanket to reduce the potential for surface erosion. We recommend using a biodegradable erosion control blanket (North American Green SC150 or equivalent erosion control material that is acceptable to the Geotechnical Engineer) on the slope face that has been disturbed by grading. The biodegrable erosion control blanket should be installed in accordance with the manufacturer's specifications.

To limit the concentration of surface water on slopes, areas upslope of the cut or fill slope should be graded to drain away from these slopes. As an alternative, V-ditches or curbs and gutters should be placed at the crest of these slopes to capture and control surface water and re-direct it away from the slope.

9.1.5 Cut Slopes

We recommend that temporary cut slopes in fill or native soil over five feet high be graded no steeper than 1:1. Temporary cuts in bedrock may be made vertical; however, the height of any vertical segment should not exceed six feet unless shoring is used. If poor rock quality or adverse bedding is present, cuts in rock should be flattened and/or retained using temporary shoring. The safety of workers and equipment in or near excavations is the responsibility of the contractor. The contractor should be familiar with the most recent OSHA Trench and Excavation Safety standards.

If cut slopes will be permanent, the fill and native soil should be graded no steeper than 2.5:1 (horizontal to vertical). Unretained cuts in bedrock may be graded as steep as 1:1, depending on the rock fracturing, hardness, and weathering. If poor rock quality or adverse bedding is present, rock slopes should be flattened and/or retained using rock bolts.

We should review plans for temporary and permanent cut slopes prior to construction. During construction, we should observe cut slopes to verify the inclinations are appropriate for the conditions encountered. It is the responsibility of the contractor to maintain safe and stable slopes during construction. During wet weather, runoff should be prevented from running across slopes and from entering excavations.

9.1.6 Utility Trenches

Excavations for utility trenches in clay, sand, silt, and gravel can be readily made with a backhoe. Where bedrock is present within utility trenches, the contractor should select equipment that is capable of excavating and removing rock. All trenches should conform to the current CAL-OSHA requirements for slopes, shoring, and other safety concerns.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After the pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with sand or fine gravel, which should be mechanically tamped. Backfill for utility trenches is also considered fill, and should be placed and compacted according to the recommendations previously presented. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

Where utility trenches enter the building pad, an impermeable plug consisting of lean concrete, at least five feet in length, should be installed where the trenches enter the building footprint. Furthermore, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The plug should extend from the bottom of the trench to the subgrade elevation. The purpose of these recommendations is to reduce the potential for water to become trapped in trenches beneath the building or pavements. This trapped water can cause heaving of soils beneath slabs and softening of subgrade soil beneath pavements.

9.2 Foundation Support

We recommend the proposed courthouse be supported on spread footings where bedrock is encountered at or near the subgrade level, and on drilled piers extending into bedrock where bedrock is too deep to be practically reached by the footings. The following sections present our recommendations for footing and pier foundations.

9.2.1 Spread Footings

Where it is practical to reach bedrock by excavating for the footings (we estimate this to be a depth of up to about 5 feet), the proposed structure can be supported on spread footings. Footings should be embedded at least three feet below the lowest adjacent grade where fill or soil are present and a minimum of one foot into bedrock. Footings bearing on bedrock may be designed for a maximum allowable bearing pressure of 10,000 pounds per square foot (psf) for dead plus live loads, which can be increased by one-third for total loads, including wind and/or seismic loads. These values include factors of safety of at least 2.0 and 1.5 for dead plus live loads and total loads, respectively.

To design footings using the modulus of subgrade reaction method, we recommend a modulus of 240 kips per cubic foot (kcf) be used. This modulus is representative of the anticipated settlement under the building loads provided.

Lateral loads on footings can be resisted by a combination of passive resistance acting against the vertical faces of the footings and friction along the bases of the footings. Passive resistance may be calculated using uniform pressures of 1,800 psf for fill and 6,000 psf for bedrock. The upper foot of soil or rock should be ignored unless it is confined by slabs or pavement. Frictional resistance at the base of the footings should be computed using a friction coefficient of 0.4. These values include a factor of safety of about 1.5. Passive resistance should not be used for foundation elements on the existing slope unless the face of the footing is at least 7 feet from the slope face, measured horizontally.

Uplift loads may be resisted by the weight of the footing and any overlying soil. If footings are inadequate to provide the necessary uplift resistance, drilled piers or tiedowns may be used. Recommendations for design of drilled piers are provided in the following section; recommendations for tiedowns can be provided upon request.

The footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. If disturbed, highly weathered, or decomposed bedrock is encountered at the bottom of footing excavations, the excavations should be deepened to expose more competent bedrock, as determined by the geotechnical engineer. We should check foundation excavations prior to placement of reinforcing steel to confirm suitable bearing material is present.

If overexcavation is required to reach bedrock or to remove unsuitable rock, the overexcavation may be backfilled to the design bottom of footing using lean concrete. The lean concrete should have a minimum unconfined compressive strength of 50 pounds per square inch.

9.2.2 Drilled Piers

Drilled piers bottomed in bedrock should be designed to derive their axial capacity from end bearing and skin friction. To compute the axial compressive capacity of drilled piers, we recommend using an allowable end bearing of 17,000 psf (provided the bottoms of the pier shafts can be cleaned) and allowable skin friction values of 375 psf for dead plus live loads in fill and 1,200 psf for dead plus live loads in bedrock. The allowable skin friction values may also be used to resist temporary uplift loads. For temporary compressive total loads, including wind and/or seismic loads, these values can be increased by one third. For design of the drilled piers using the subgrade modulus method, we recommend using spring constants of 255 kips/inch for 22-inch-diameter piers and 395 kips/inch for 30-inch-diameter piers. Piers installed in a group should be spaced at least three diameters on center.

Piers will provide lateral resistance from passive pressure acting on the upper portion of the piers and from their structural rigidity. Lateral resistance of piers will depend on the pier diameter, pier head condition (restrained or unrestrained), allowable deflection of the pier top, and the bending moment resistance of the piers. We have performed lateral load analyses for isolated, 22- and 30-inch-diameter piers for a deflection of 0.5 inch at the pier head. We assumed a cracked section at the pier head and used 30 percent of the elastic modulus for concrete in our analyses, based on discussion with the project structural engineer. In addition, we assumed that the pier head is at the ground surface and considered both a level ground surface and a ground surface inclined at approximately 1.8:1 (horizontal to vertical) for piers on the existing fill slope. The results of our analyses are presented in Tables 3 and 4 for level and sloped ground surface conditions, respectively. Plots of deflection and bending moment versus depth are presented on Figures 10 and 11.

TABLE 3

Results of Lateral Load Analyses
Drilled Pier, Level Ground Surface

Pile Diameter (inches)	Pile Top Condition	Pile Head Deflection (inches)	Applied Lateral Load (kips)	Computed Maximum Bending Moment (kip-feet)	Depth to Maximum Bending Moment (feet)
22	Unrestrained	0.5	24.7	78.1	5.8
22	Restrained	0.5	50.4	196	0
30	Unrestrained	0.5	41.6	163	7.3
30	Restrained	0.5	83.3	411	0

TABLE 4

Results of Lateral Load Analyses
Drilled Pier, Ground Surface Sloped at 1.8:1 (Horizontal to Vertical)

Pile Diameter (inches)	Pile Top Condition	Pile Head Deflection (inches)	Applied Lateral Load (kips)	Computed Maximum Bending Moment (kip-feet)	Depth to Maximum Bending Moment (feet)
22	Unrestrained	0.5	17.9	64.4	6.2
22	Restrained	0.5	37.0	160	0
30	Unrestrained	0.5	30.4	134	8.1
30	Restrained	0.5	61.4	337	0

The lateral resistances tabulated in Tables 3 and 4 are for piers with a spacing of at least six pier diameters. If piers are installed in a group of two with a spacing of three pier diameters, the lateral capacities should be reduced by 15 percent. However, the design bending moments should be taken as the same as those for single piers. If larger pier groups are needed to support the building, we should be contacted to provide the reduction factors for these groups.

Additional lateral load resistance can be obtained by passive resistance acting against the face of pier caps and grade beams. To calculate passive resistance, we recommend using an allowable uniform pressure of 1,800 psf in fill. The upper foot of soil should be ignored unless it is confined by slabs or pavement. Passive resistance should not be used for foundation elements on the existing slope unless the face of the footing is at least 7 feet from the slope face, measured horizontally.

Drilled piers should be installed by a qualified contractor with demonstrated experience in this type of foundation. It is likely that pier shafts will need to be cased during construction to prevent caving and to allow for inspection of the bottoms. Any water present at the bottom of the pier should be removed by pumping. Loose soil and rock encountered at the bottom of the pier should also be removed; if proper clean-out is not possible, the piers will need to be deepened and their end-bearing capacity ignored. Steel and concrete placement should start immediately upon completion of inspection and clean-out.

9.3 Concrete Floor Slabs

The floor slab will be underlain by fill, and we anticipate settlement of the fill will occur. Therefore, the floor slab should be designed to span between footings or piers and not rely on the ground for support. The subgrade soil should be scarified, moisture-conditioned, and recompacted to reduce the potential for detrimental effects of highly expansive soil, as discussed in Section 9.1.2. If the previously compacted soil subgrade is disturbed during foundation and utility excavation, the subgrade should be scarified, moisture-conditioned, and rerolled to provide a firm, unyielding surface prior to construction of the floor slab.

Because it will be below the ground surface, we recommend the lower level floor of the building be waterproofed. For the upper level of the building, where moisture on the floor slab is undesirable, we recommend installing a capillary moisture break and water vapor retarder beneath the floor to reduce water vapor transmission through floor slabs. A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder should be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 5.

TABLE 5

Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6
Sand	
No. 4	100
No. 200	0 – 5

The sand overlying the membrane should be moist at the time concrete is placed; however, there should be no free water present in the sand. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio – less than 0.50. If approved by the project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the w/c ratio of the concrete does not exceed 0.45 and water is not added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

9.4 Temporary Shoring

If the planned excavations cannot be sloped because of space limitations, shoring will be required to retain the excavation sides. We estimate excavations for the planned courthouse may be as deep as about 15 feet. If the shoring will be used as part of a permanent retention system, all system components should be double-corrosion protected and the shoring design should incorporate a factor of safety consistent with permanent structures.

Cantilevered shoring should be designed for an active earth pressure defined as an equivalent fluid weight of 42 pounds per cubic foot (pcf). This value is considered appropriate for an active condition, which assumes that some movement of the supported soil is tolerable. If movement of the soil is not acceptable, an at-rest pressure of 63 pcf should be considered. For shoring consisting of soldier beams and lagging, the active and at-rest earth pressures should be assumed to act over the full width of the shoring above the excavation and over one soldier beam width below the excavation. The foregoing earth pressures assume the ground surface at the top of the shoring wall will be level; if sloping ground surface conditions are anticipated, we should be contacted to provide additional recommendations.

If traffic is anticipated within a distance equal to the shoring depth, a uniform surcharge load of 100 pounds per square foot (psf) acting on the upper 10 feet should be used in the design. An increase in lateral design pressure for the shoring may be required where heavy construction equipment or stockpiled materials will be within a distance equal to the shoring depth. The increase in pressure should be determined after the surcharge loads are known. If this condition exists, we should be consulted and the additional pressure increment can be computed on a case-by-case basis.

Passive resistance can be computed using a uniform pressure of 1,800 psf plus an equivalent fluid weight of 80 pcf. This passive pressure value includes a factor of safety of about 1.5 for temporary shoring design. For beams spaced at least three shaft diameters, center-to-center, the passive resistances can be assumed to act over three soldier beam[7] widths.

The shoring designer should evaluate the required penetration depth of the soldier piles. The soldier piles should have sufficient axial capacity to support the vertical load component of the tiebacks and the vertical load acting on the piles, if any. To compute the axial capacity of the piles, we recommend using an allowable friction of 500 psf on the perimeter of the piles below the excavation level, which includes a factor of safety of 1.5. Vertical support from end bearing is neglected.

Where excavation depths exceed approximately 12 feet, tiebacks or internal bracing will likely be required. Figure 12 presents the lateral pressures we recommend for design of a tied-back or internally-braced soldier beam and lagging wall. Design criteria for tiebacks are also presented on Figure 12. As shown, tiebacks should derive their load-carrying capacity from the soil behind an imaginary line sloping upward from a point H/5 feet away from the bottom of the excavation at an angle of 60 degrees from horizontal, where H is the wall height in feet. The minimum stressing and bond lengths should be 15 feet each.

Tiebacks will generally be installed in fill consisting of cobble-to boulder-sized serpentinite clasts, loose to dense clayey gravel to gravel with sand, stiff to very stiff clay with variable sand and gravel content, and hard sandy silt with gravel. Allowable capacities of the tiebacks will depend upon the drilling method, shaft diameter, grout pressure, and workmanship. Because of the tendency of granular soil layers to cave, augers should not be used in these materials. We recommend a smooth-cased method (such as a Klemm rig) be used to install tiebacks in these materials. For estimating purposes, we recommend using the skin friction value for pressure-grouted tiebacks given on Figure 12.

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[7] The soldier beam width is defined as the diameter of the drilled hole for beams backfilled with structural concrete with an unconfined compressive strength of at least 50 pounds per square inch (psi).

The shoring designer should be responsible for determining the actual length of tieback required.

The determination should be based on the designer's familiarity with the installation method to be used. The computed bond length should be confirmed by a performance- and proof-testing program. The first two production tiebacks and two percent of the remaining tiebacks should be performance-tested to 1.5 times the design load for the proposed temporary shoring system. The remaining tiebacks should be confirmed by a proof-test to 1.25 times the design load for the proposed temporary shoring system. If any tiebacks fail to meet the proof-testing requirements, additional tiebacks should be added to compensate for the deficiency, as required by the shoring designer. We should review the shoring design prior to issuing bid documents for construction.

The movement of each tieback should be monitored with a free-standing, tripod-mounted dial gauge during proof and performance testing. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute readings is more than 0.04 inches, the load should be held for an additional 50 minutes. If the deflection is more than 0.08 inches between the 6- and 60-minute readings, the tieback design loading should be re-evaluated. If any tieback fails to meet the performance- and proof-testing requirements, additional tiebacks should be added to compensate for the deficiency, as directed by the shoring designer. After testing, the tiebacks should be loaded to the design load (less if specified by the shoring designer) and locked off. The tiebacks should be checked 24 hours after initial lock off to ensure that stress relaxation has not occurred. The bottom of the excavation should not extend more than two feet below a row of unsecured tiebacks.

The anticipated deflections of the shoring system should be estimated to check if they are acceptable. The shoring system should be sufficiently rigid to prevent detrimental movement of the temporary shoring and possible damage to improvements adjacent to the site. In our experience, the deflection of a properly designed shoring system should generally be held to one inch or less. The shoring system should be designed so that it does not conflict with nor damage planned project improvements, such as underground utilities or deep foundations.

The shoring system should be installed by an experienced shoring specialty contractor The contractor should be familiar with applicable local, state, and federal regulations for temporary shoring, including the current OSHA Excavation and Trench Safety Standards. The contractor should be solely responsible for the design of temporary shoring. We should review the final shoring plans to check that they are consistent with the recommendations presented in this report. In addition, we recommend a representative from our office observe the installation of the temporary shoring system as part of our special inspection services.

9.5 Basement and Retaining Walls

The below-grade walls and any retaining walls planned for the site should be designed to resist lateral pressures imposed by the soil and any adjacent surcharges. In addition, because the site is in a seismically active area, all below-grade walls and retaining walls should be designed to resist pressures associated with seismic forces. For walls free to deflect (unrestrained) and restrained walls, we recommend the lateral pressures be calculated using the parameters shown in Table 6. Restrained walls should be designed for the more critical of the static and seismic loading conditions.

TABLE 6

Lateral Earth Pressures
(Fully Drained Walls)

Loading Condition	Backfill Material	Unrestrained Walls	Restrained Walls
Static	Fill	Active pressure corresponding to equivalent fluid weight of 42 pcf for level backfill and 78 pcf for backfill sloped at 1.8H:1V	At-rest pressure corresponding to equivalent fluid weight of 63 pcf for level backfill and 85 pcf for backfill sloped at 1.8H:1V
Seismic	Fill	Active pressure plus an equivalent fluid weight of 5 pcf for seismic load	Active pressure plus an equivalent fluid weight of 5 pcf for seismic load
Static	Bedrock	Active pressure corresponding to equivalent fluid weight of 24 pcf for level rock behind wall and 32 pcf for rock sloped at 1.8H:1V	At-rest pressure corresponding to equivalent fluid weight of 41 pcf for level rock behind wall and 66 pcf for rock sloped at 1.8H:1V
Seismic	Bedrock	Active pressure plus an equivalent fluid weight of 5 pcf for seismic loading	Active pressure plus an equivalent fluid weight of 5 pcf for seismic loading

Lateral pressures from traffic or surcharges should be added to the static design pressures. If traffic loads are expected within 10 feet of the walls, an additional design load of 100 psf (rectangular distribution) should be applied over the full height of the wall. Footings adjacent to walls should be bottomed below an imaginary line drawn upward at an inclination of 1.5:1 (horizontal to vertical) from the base of the wall. Adjacent piers, if located within 10 feet of the wall, may impose a surcharge pressure on the wall. We should evaluate potential surcharge pressures if this occurs.

The recommended design pressures are for fully drained walls; hydrostatic pressures are not included. One acceptable method of backdraining below-grade walls is to place a prefabricated drainage panel against the back of the wall. Where shoring is used, the drainage panel may be attached to the shoring and the wall cast directly against it. The panel should extend down to a perforated PVC collector pipe at the base of the wall. The perforated pipe should be bedded on and covered by at least four inches of Class 2 permeable material (per Caltrans Standard Specifications) or by drain rock that is surrounded by filter fabric (Mirafi 140NC or equivalent). An acceptable alternative is to backdrain the wall with Caltrans Class 2 permeable material at least one foot wide, extending down to the base of the wall. A perforated PVC pipe should be placed at the bottom of the gravel, as described for the first alternative. The perforated collection pipe in either alternative should redirect the water to a solid pipe that is sloped to drain to a suitable outlet.

If moisture migration through the walls or effervescence is a concern, the walls should be waterproofed and water stops should be placed at all construction joints. Foundations for basement and retaining walls can be designed using the recommendations presented in Section 9.2. During placement of backfill behind basement and retaining walls, the walls should be braced, or hand compaction equipment should be used, to prevent unwanted surcharges on the walls or foundations (as determined by the structural engineer).

9.6 Asphalt Concrete Pavement Design

The State of California resistance value (R-value) method for flexible pavement design was used to develop recommendations for asphalt concrete pavement sections. We anticipate the final soil subgrade in areas to receive asphalt concrete pavement will generally consist of clay with varying amounts of sand and silt. Based on R-value test results, the clayey and silty soil at the site has approximate R-values ranging from 28 to 43. For our calculations, we used an R-value of 28.

We assumed traffic indices (TI) of 5.0, 6.0, and 7.0 for our calculations; these TIs should be confirmed by the project civil engineer. We can provide pavement section recommendations for other TIs upon request. Table 7 presents our recommendations for asphalt pavement sections.

TABLE 7

Asphaltic Concrete Pavement Section Design
Design R-Value of Subgrade Soil = 28

TI	Asphaltic Concrete (inches)	Class 2 Aggregate Base (inches)
5.0	3.0	6.0
6.0	3.5	8.0
7.0	4.0	10.0

Pavement components should conform to the current Caltrans Standard Specifications. The soil subgrade should be prepared as discussed in Section 9.1.2. The soil subgrade should be kept moist until it is covered with AB. Class 2 AB should be compacted to at least 95 percent relative compaction.

9.7 Concrete Flatwork

Exterior concrete flatwork that will not receive vehicular traffic (i.e., sidewalks) should be underlain by at least four inches of Class 2 AB compacted to at least 95 percent relative compaction. Prior to placement of the aggregate base, the upper six inches of subgrade soil should be scarified, moisture-conditioned to above the optimum moisture content (or at least three percent above the optimum moisture content for expansive soil), and compacted to at least 90 percent relative compaction. Within decorative concrete flatwork areas, 12 inches of aggregate base should be used beneath the exterior slabs to further reduce the potential for cracking due to shrinking and swelling of the underlying expansive soil. Thickening the slabs and adding reinforcement will also control cracking to some degree. The soil subgrade beneath the 12 inches of Class 2 AB should be prepared as discussed in Section 9.1.2.

9.8 Seismic Design

The closest active fault to the site is the Collayomi Fault, which is about 6.8 kilometers from the site. The foundation of the courthouse will bear on weak to moderately hard bedrock and we conclude that site class B (as defined by the 2013 CBC) is appropriate for the site on the basis of the results of the geophysical studies performed at the site. For design in accordance with the 2013 CBC, we recommend the following parameters be used:

- site class B
- site coefficient values F_a and F_v of 1.0 and 1.0, respectively
- mapped site class D short (S_s) and one-second (S_1) spectral acceleration values for the Risk Targeted Maximum Considered Earthquake (MCE_R) of 1.500g and 0.600g, respectively
- spectral acceleration values S_{Ms} and S_{M1} for the MCE_R of 1.500g and 0.600g, respectively
- spectral acceleration values for the Design Earthquake (DE) of S_{Ds} and S_{D1} of 1.000g and 0.400g, respectively.

10.0 ADDITIONAL GEOTECHNICAL SERVICES

Prior to construction, Langan Treadwell Rollo should review the project plans and specifications to check their conformance with the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing services during excavation, installation of temporary shoring, fill and backfill placement and compaction, subgrade preparation, permanent wall construction, and footing and drilled pier installation. These observations will allow us to compare the actual with the anticipated soil conditions and to check that the contractor's work conforms with the geotechnical aspects of the plans and specifications.

11.0 LIMITATIONS

The conclusions and recommendations presented in this report result from limited engineering studies based on our interpretation of the geotechnical conditions existing at the time of the investigation. Actual subsurface conditions may vary. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that described in this report, Langan Treadwell Rollo should be notified to make supplemental recommendations, if necessary.

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FIGURES

DRAFT

Reference: Google Earth Pro, 2011.

LAKEPORT COURTHOUSE **675 LAKEPORT BOULEVARD** Lakeport, California	**SITE LOCATION MAP**		
LANGAN TREADWELL ROLLO	Date 03/04/15	Project No. 731563902	Figure 1

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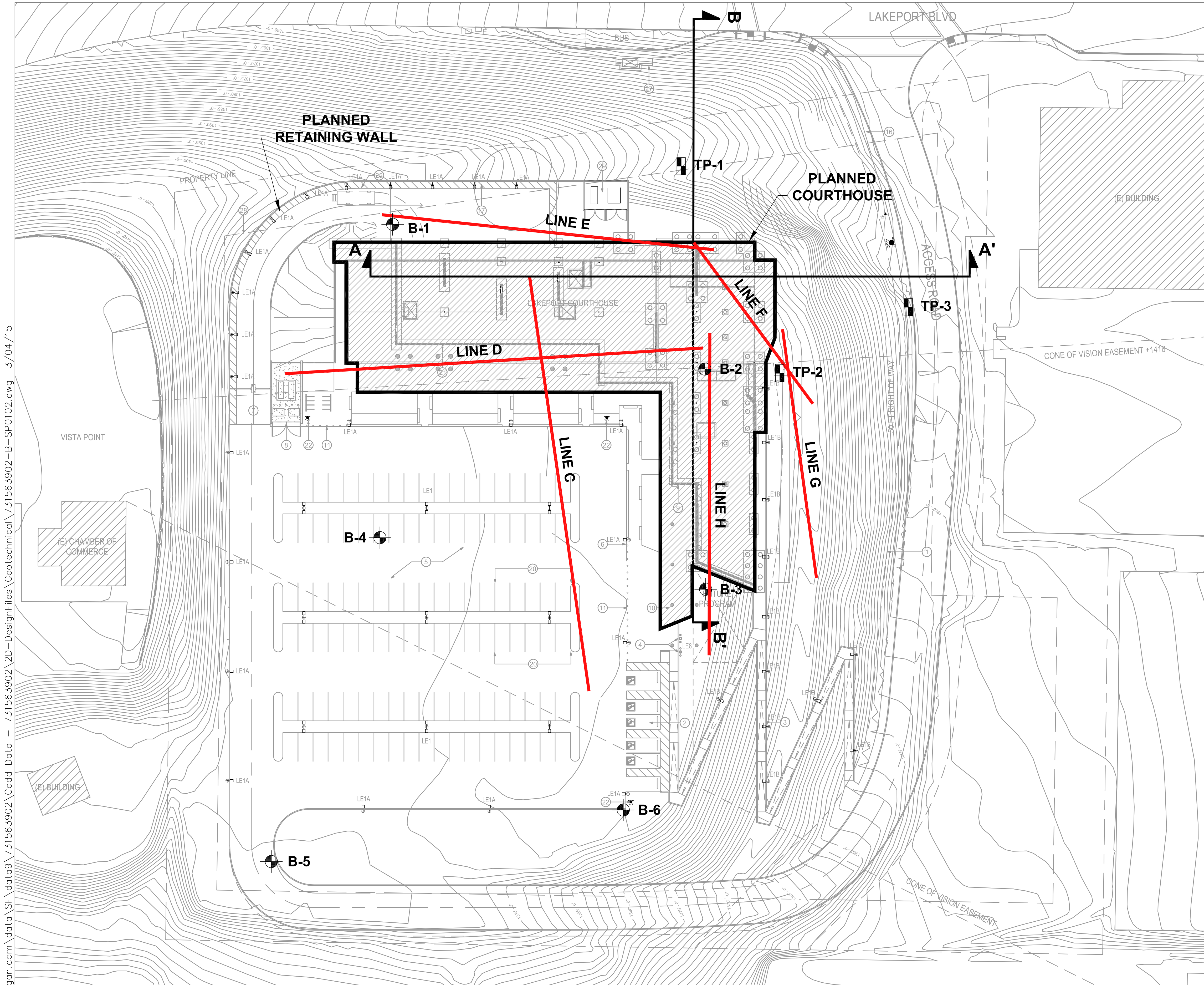

EXPLANATION

B-1 Approximate location of boring by Treadwell & Rollo, November 2011

TP-1 Approximate location of test pit by Treadwell & Rollo, November 2011

Seismic refraction line by Langan Treadwell Rollo, January 2015

A A' Idealized subsurface profile

0 60 Feet

Approximate scale

LAKEPORT COURTHOUSE
675 LAKEPORT BOULEVARD
Lakeport, California

SITE PLAN

Date 03/04/15	Project No. 731563901	Figure 2

LANGAN TREADWELL ROLLO

Reference: Base map from a drawing titled "Architectural Site Plan, Lakeport Courthouse, Lakeport, CA," by Mark Cavagnero Associates, 100% Schematic Design, dated 19 December 2011.

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Reference: MS-010, “Geology of Lakeport Quadrangle, Lake County, California,” California Division of Mines and Geology, 1;62,000, by James R. McNitt, 1967.

0 3,000 6,000 Feet

Approximate scale

N

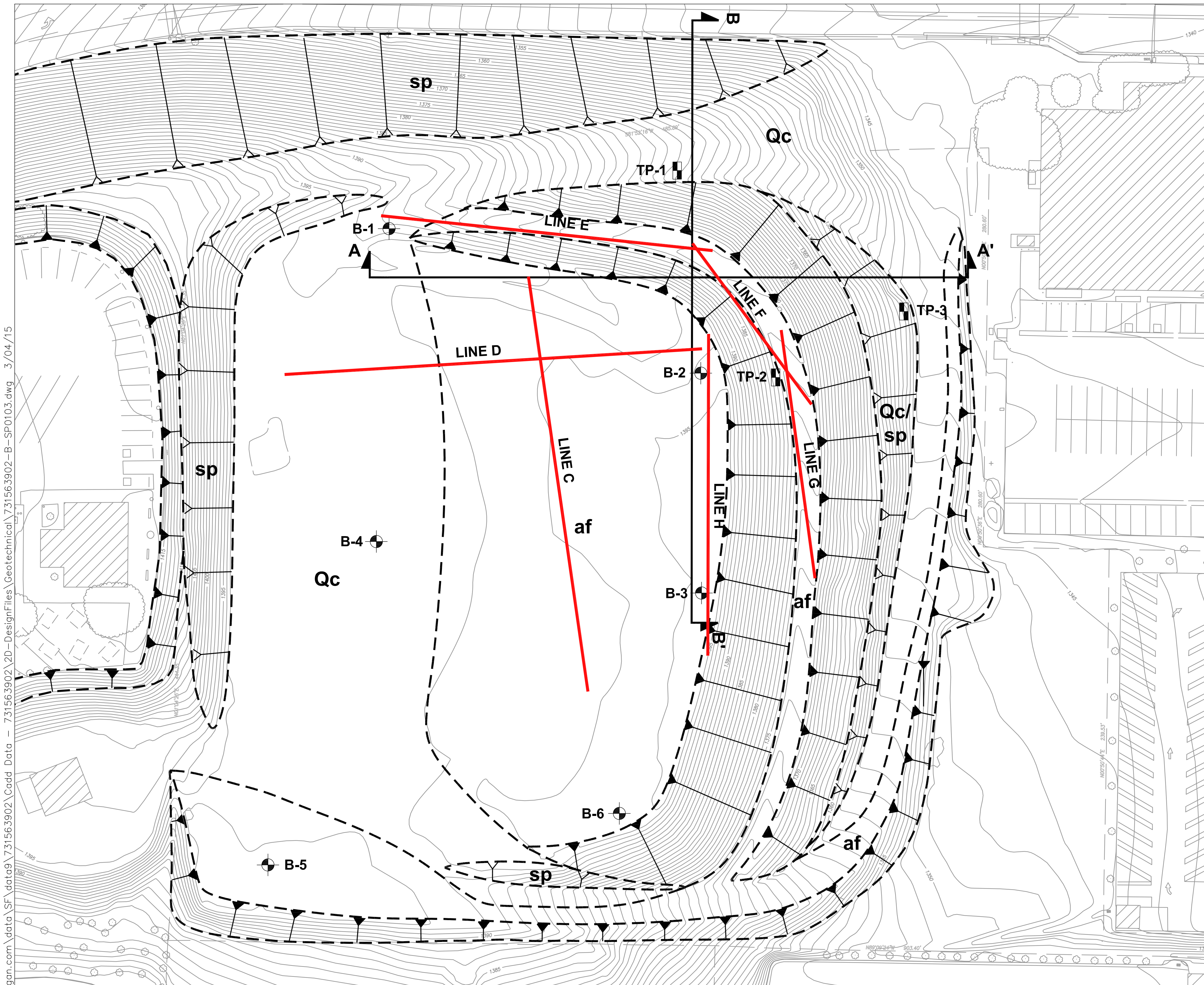
LAKEPORT COURTHOUSE
675 LAKEPORT BOULEVARD
Lakeport, California

LANGAN TREADWELL ROLLO

REGIONAL GEOLOGIC MAP

Date 03/04/15	Project No. 731563902	Figure 3

DRAFT



EXPLANATION

B-1	Approximate location of boring by Treadwell & Rollo, November 2011
TP-1	Approximate location of test pit by Treadwell & Rollo, November 2011
	Seismic refraction line by Langan Treadwell Rollo, January 2015
A A'	Idealized subsurface profile
af	Artificial fill
Qc	Colluvium/topsoil
sp	Serpentinite bedrock
	Geologic contact, dashed where approximate
	Fill slope
	Cut slope

0 60 Feet

Approximate scale

Reference: Base map from a drawing titled "Architectural Site Plan, Lakeport Courthouse, Lakeport, CA," by Mark Cavagnero Associates, 100% Schematic Design, dated 19 December 2011.

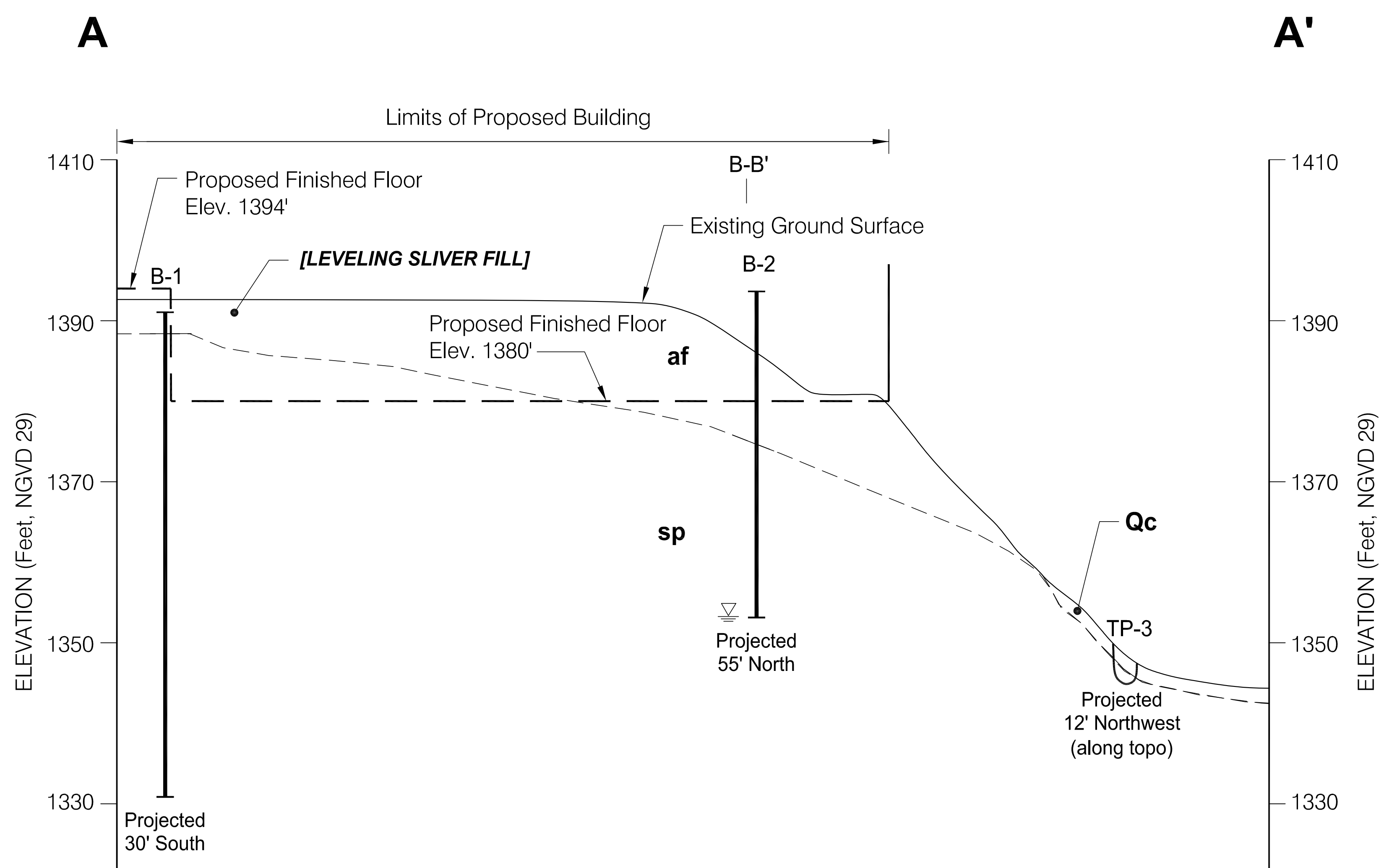
LAKEPORT COURTHOUSE
675 LAKEPORT BOULEVARD
Lakeport, California

ENGINEERING GEOLOGIC MAP

Date 03/04/15 | Project No. 731563902 | Figure 4

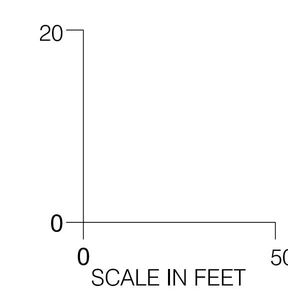
LANGAN TREADWELL ROLLO

DRAFT



EXPLANATION

af	Artificial fill (CLAYEY GRAVEL (GC), GRAVEL with SAND (GP), and GRAVEL with CLAY (GP-GC), loose to dense, SANDY CLAY (CL), SANDY CLAY with GRAVEL (CL), GRAVELLY CLAY (CL), CLAY with GRAVEL (CL), SANDY SILT (ML), and SANDY SILT with GRAVEL (MH), stiff to hard)
Qc	Colluvium/topsoil (SANDY CLAY (CL) and SANDY SILT (ML), stiff)
sp	Serpentinite bedrock
— — —	Geologic contact; solid where certain, dashed where approximate
B-2	Approximate location of boring by Treadwell & Rollo, November 2011
TP-3	Approximate location of test pit by Treadwell & Rollo, November 2011



Notes:
1. The above profile represents a generalized cross section interpreted from widely spaced borings. Earth materials may vary in type, strength, and other important properties between points of exploration.

LAKEPORT COURTHOUSE
675 LAKEPORT BOULEVARD
Lakeport, California

IDEALIZED SUBSURFACE PROFILE
A-A'

Date 03/04/15	Project No. 731563902	Figure 5

LANGAN TREADWELL ROLLO

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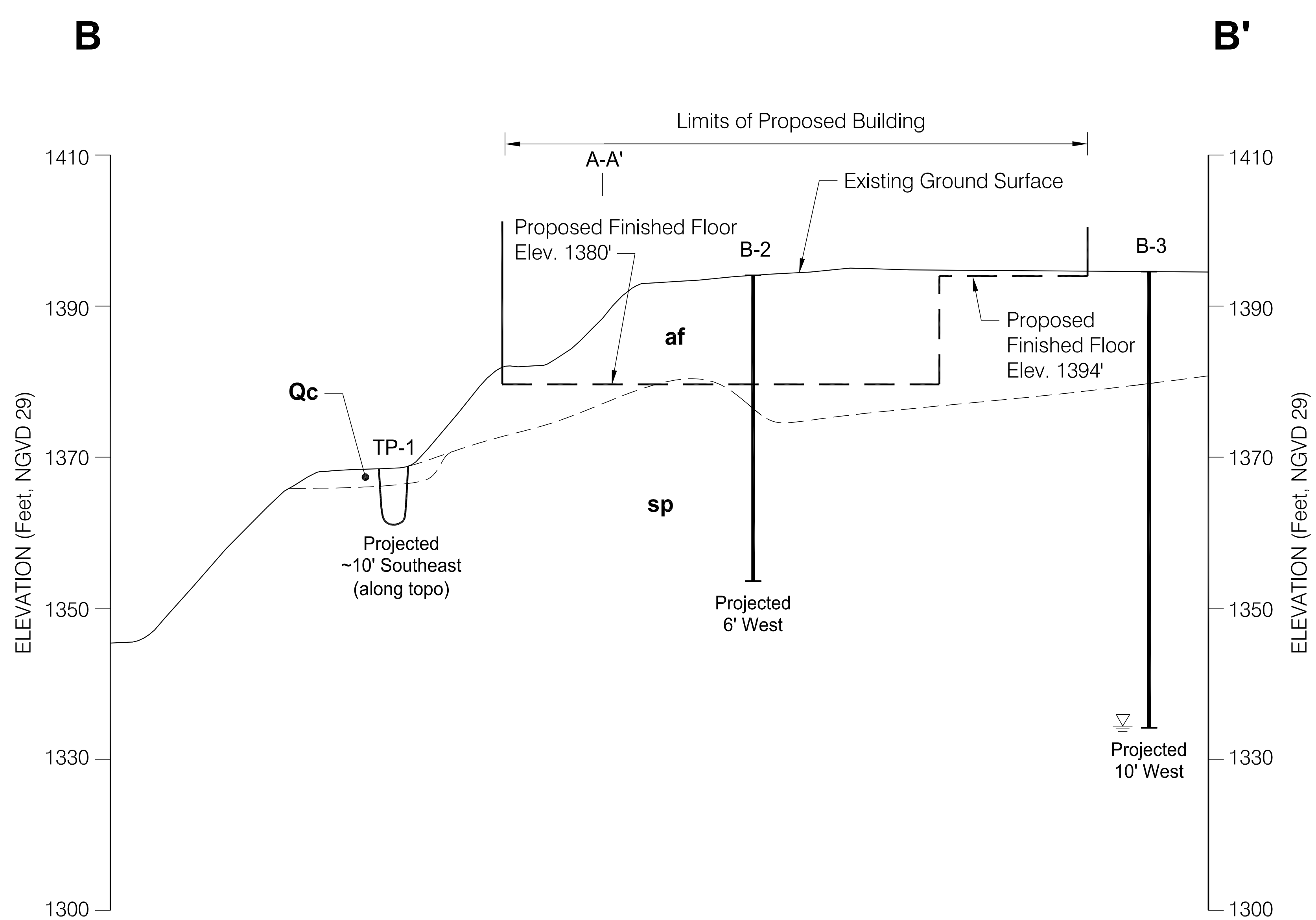
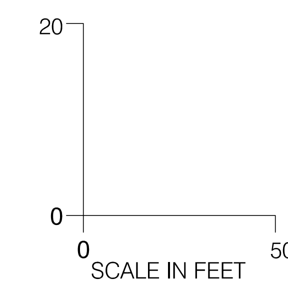


EXPLANATION

af	Artificial fill (CLAYEY GRAVEL (GC), GRAVEL with SAND (GP), and GRAVEL with CLAY (GP-GC), loose to dense, SANDY CLAY (CL), SANDY CLAY with GRAVEL (CL), GRAVELLY CLAY (CL), CLAY with GRAVEL (CL), SANDY SILT (ML), and SANDY SILT with GRAVEL (MH), stiff to hard)
Qc	Colluvium/topsoil (SANDY CLAY (CL) and SANDY SILT (ML), stiff)
sp	Serpentinite bedrock
	Geologic contact; solid where certain, dashed where approximate
B-2	Approximate location of boring by Treadwell & Rollo, November 2011
TP-1	Approximate location of test pit by Treadwell & Rollo, November 2011



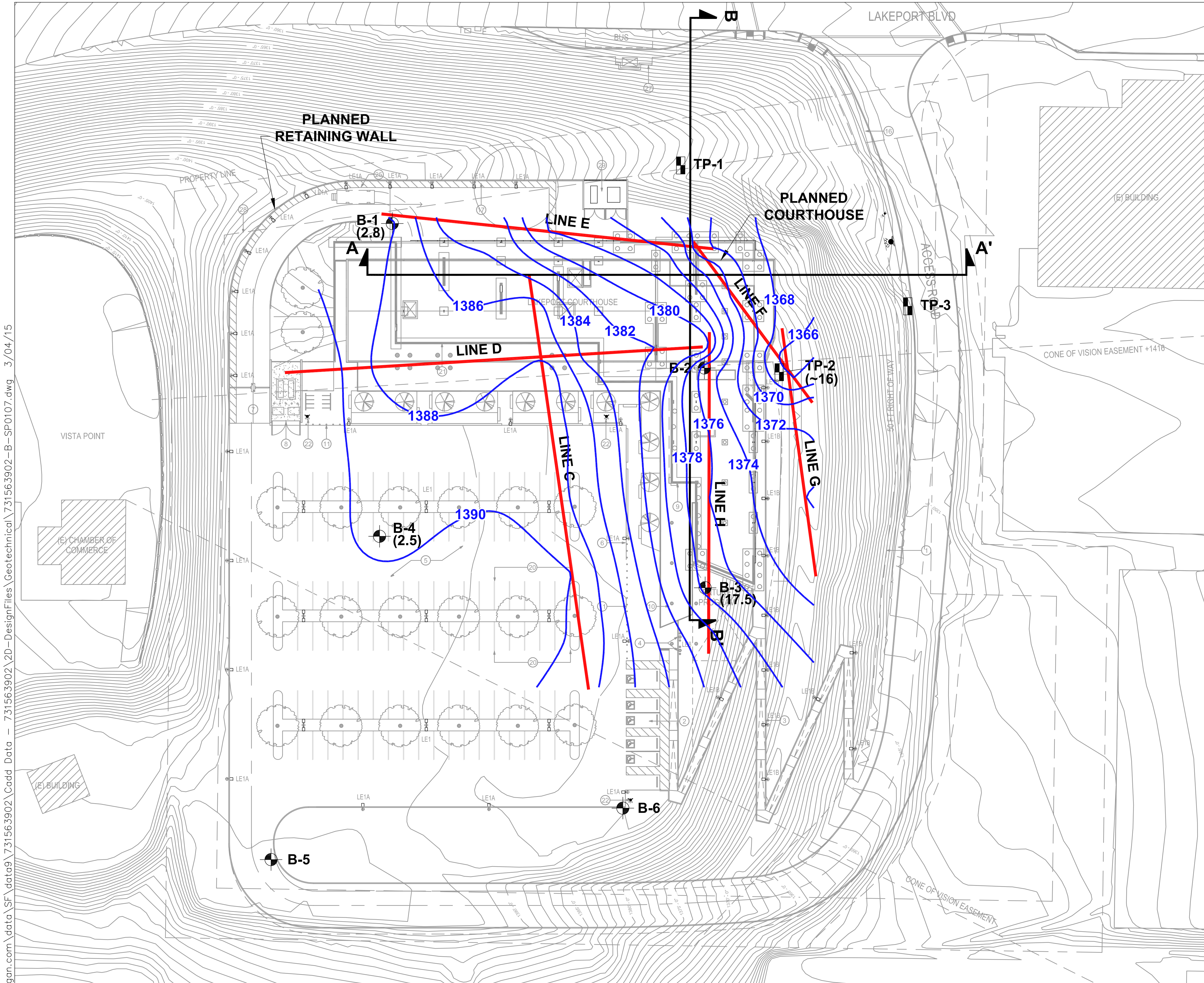
Notes:
1. The above profile represents a generalized cross section interpreted from widely spaced borings. Earth materials may vary in type, strength, and other important properties between points of exploration.

LAKEPORT COURTHOUSE
675 LAKEPORT BOULEVARD
Lakeport, California

IDEALIZED SUBSURFACE PROFILE B-B'

Date 03/04/15 | Project No. 731563902 | Figure 6

LANGAN TREADWELL ROLLO



EXPLANATION

B-1 — Approximate location of boring by Treadwell & Rollo, November 2011

TP-1 — Approximate location of test pit by Treadwell & Rollo, November 2011

Seismic refraction line by Langan Treadwell Rollo, January 2015

A — A' Idealized subsurface profile

—1390— Top of bedrock contour elevation (feet, NGVD 29 datum)

(2.5) Depth to bedrock (feet)

LAKEPORT COURTHOUSE
675 LAKEPORT BOULEVARD
Lakeport, California

TOP OF BEDROCK CONTOURS

Date 02/19/15	Project No. 731563902	Figure 7

LANGAN TREADWELL ROLLO

Reference: Base map from a drawing an electronic drawing titled "MCA 273 Lakeport Courthouse - COMPOSITE SITE PLAN 15_0218," by Mark Cavagnero Associates, dated --.

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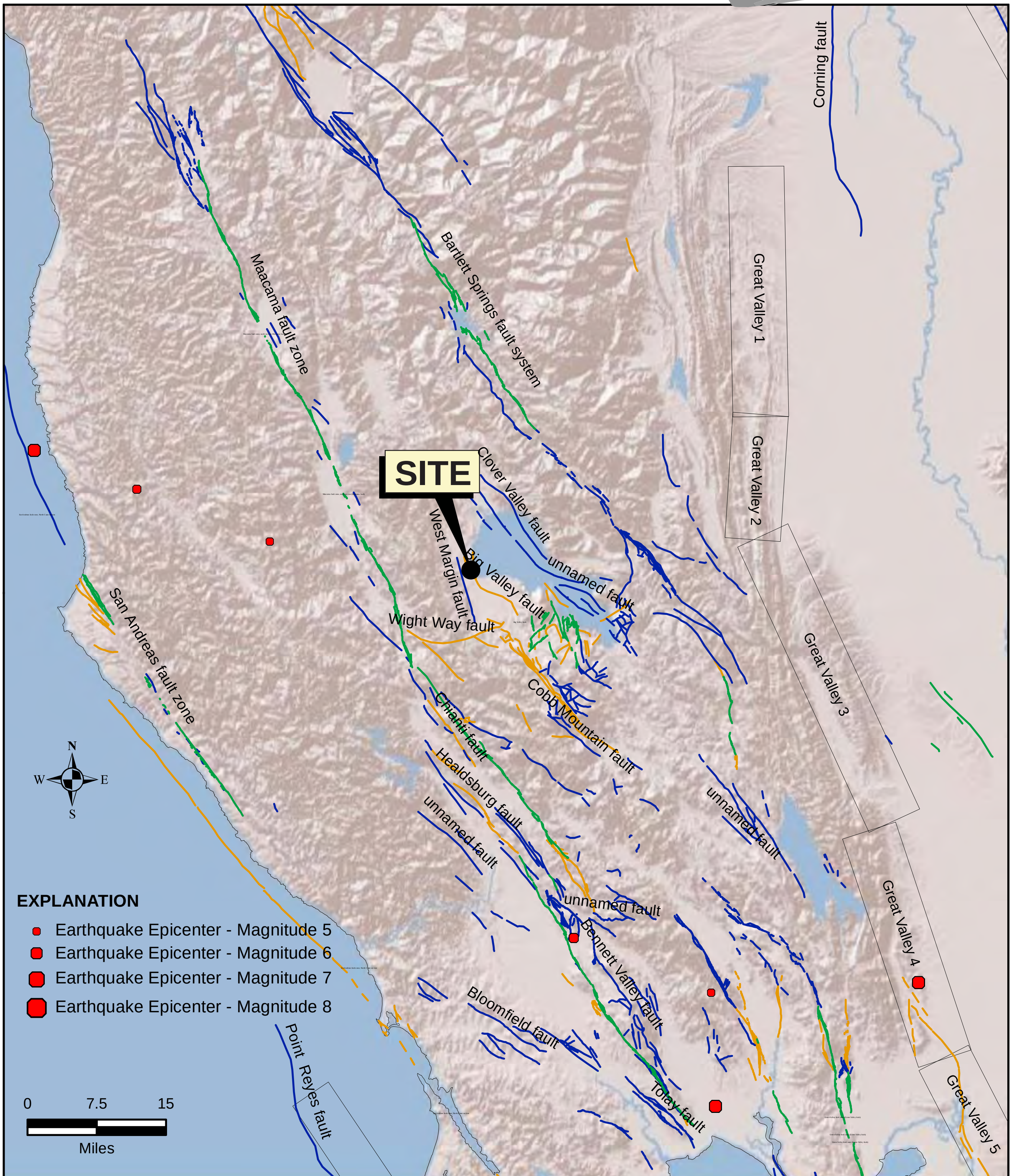


NOTES:
Digitized data for fault coordinates and earthquake catalog was developed by the California Geological Survey.
The historic earthquake catalog includes events from January 1800 to December 2000.

LAKEPORT COURTHOUSE **675 LAKEPORT BOULEVARD** Lakeport, California	**MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA**
LANGAN TREADWELL ROLLO	Date 03/04/15 | Project No. 731563902 | Figure 8

DRAFT

I **Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.**
Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.

II **Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.**
As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.

III **Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.**
Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.

IV **Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.**
Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.

V **Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.**
Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.

VI **Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.**
Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.

VII **Frightens everyone. General alarm, and everyone runs outdoors.**
People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.

VIII **General fright, and alarm approaches panic.**
Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.

IX **Panic is general.**
Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.

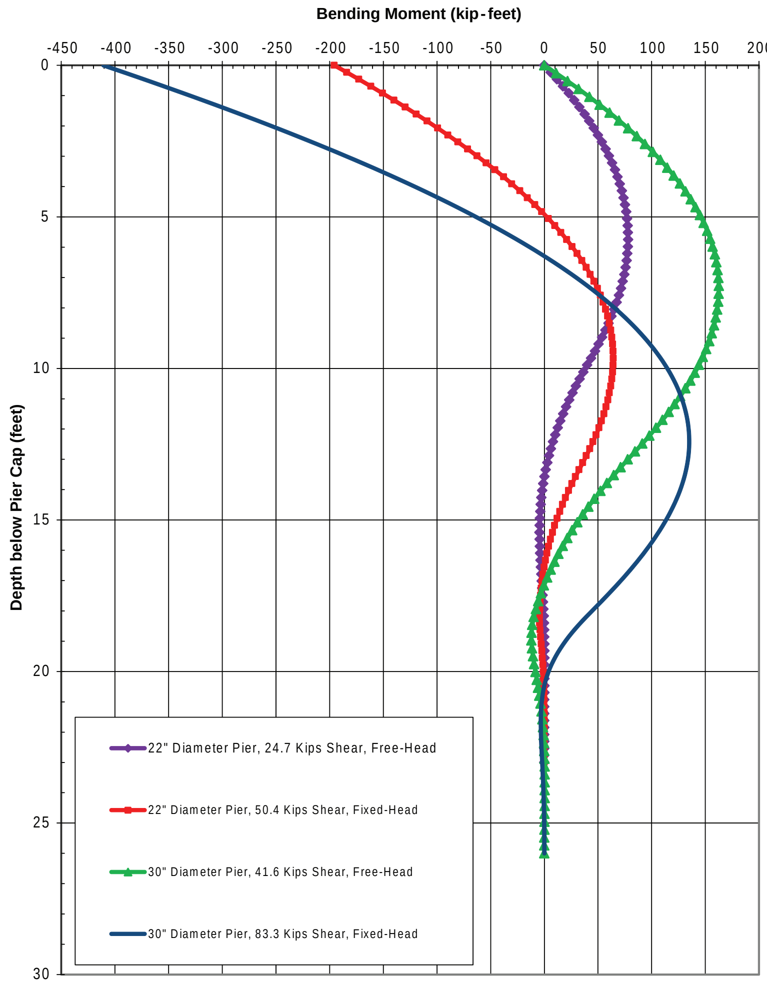
X **Panic is general.**
Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.

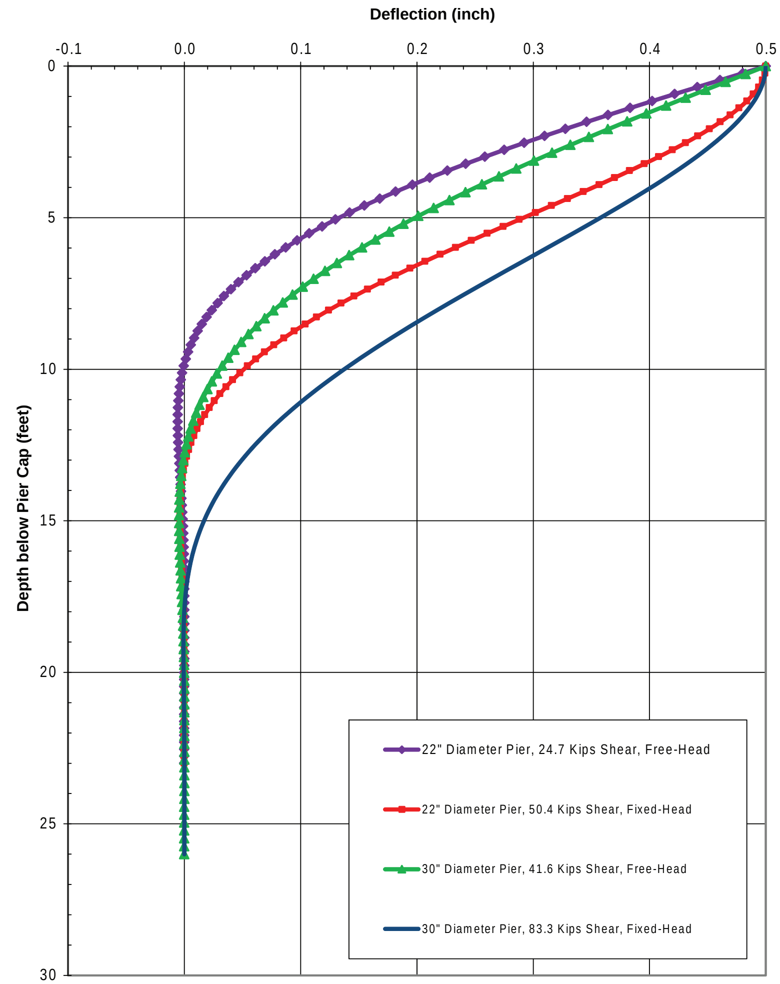
XI **Panic is general.**
Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.

XII **Panic is general.**
Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

LAKEPORT COURTHOUSE **675 LAKEPORT BOULEVARD** Lakeport, California	**MODIFIED MERCALLI INTENSITY SCALE**		
LANGAN TREADWELL ROLLO	Date 03/04/15	Project No. 731563902	Figure 9

DRAFT





Notes:
1. The profiles shown are for a single pier with an axial compressive load of 275 kips.
2. To account for group effects, the lateral load capacity of pier groups should be multiplied by a reduction factor. However, moment profile used to check individual piers in a group should be for the unfactored load.
3. Assumes there is no applied moment at the pier head.
4. Passive resistance of pier caps has not been included.

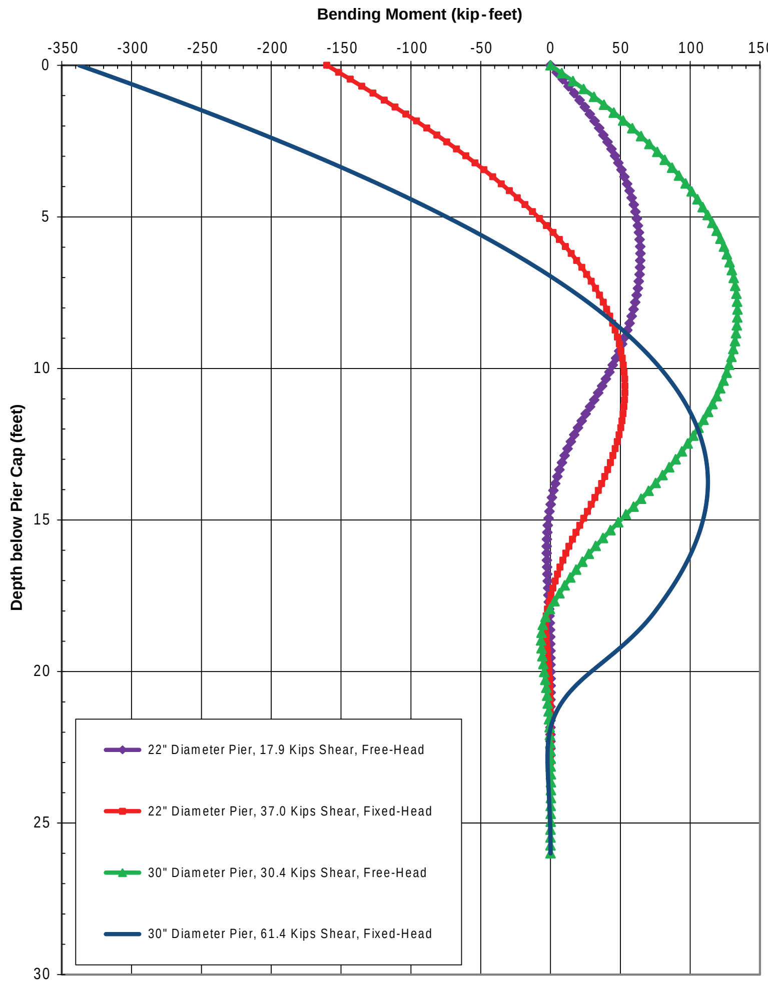
LAKEPORT COURTHOUSE
675 LAKEPORT BOULEVARD
Lakeport, California

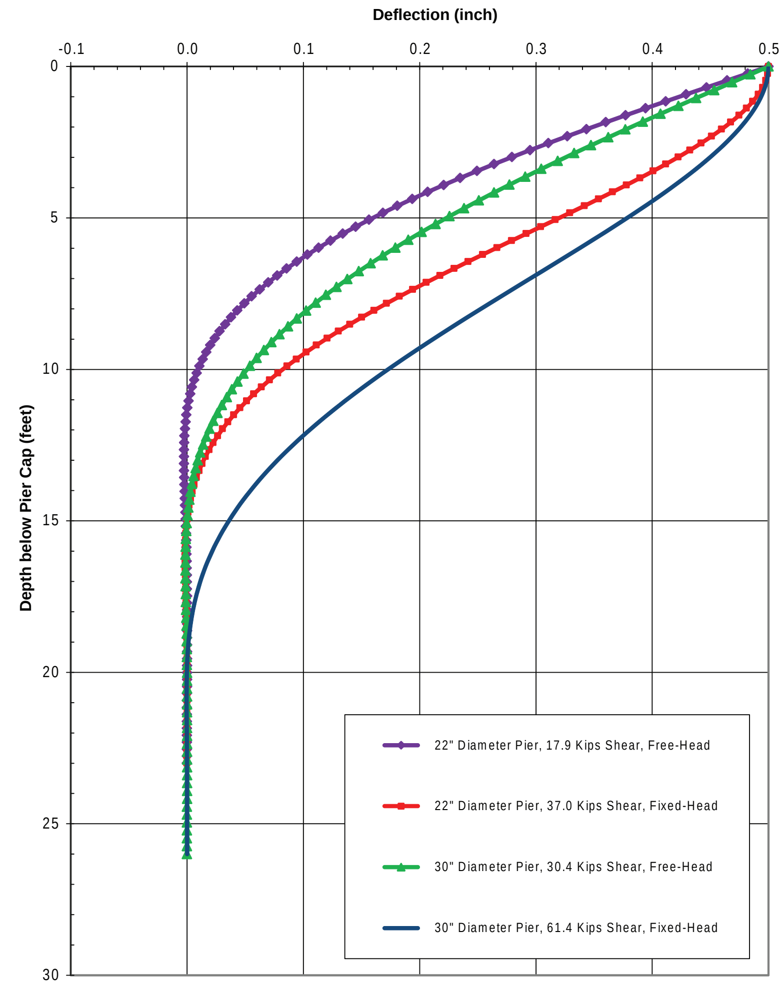
MOMENT AND DEFLECTION PROFILES
DRILLED PIER
LEVEL GROUND SURFACE

Date 03/04/15	Project No. 731563902	Figure 10

LANGAN TREADWELL ROLLO

DRAFT





Notes:
1. The profiles shown are for a single pier with an axial compressive load of 275 kips.
2. To account for group effects, the lateral load capacity of pier groups should be multiplied by a reduction factor. However, moment profile used to check individual piers in a group should be for the unfactored load.
3. Assumes there is no applied moment at the pier head.
4. Passive resistance of pier caps has not been included.

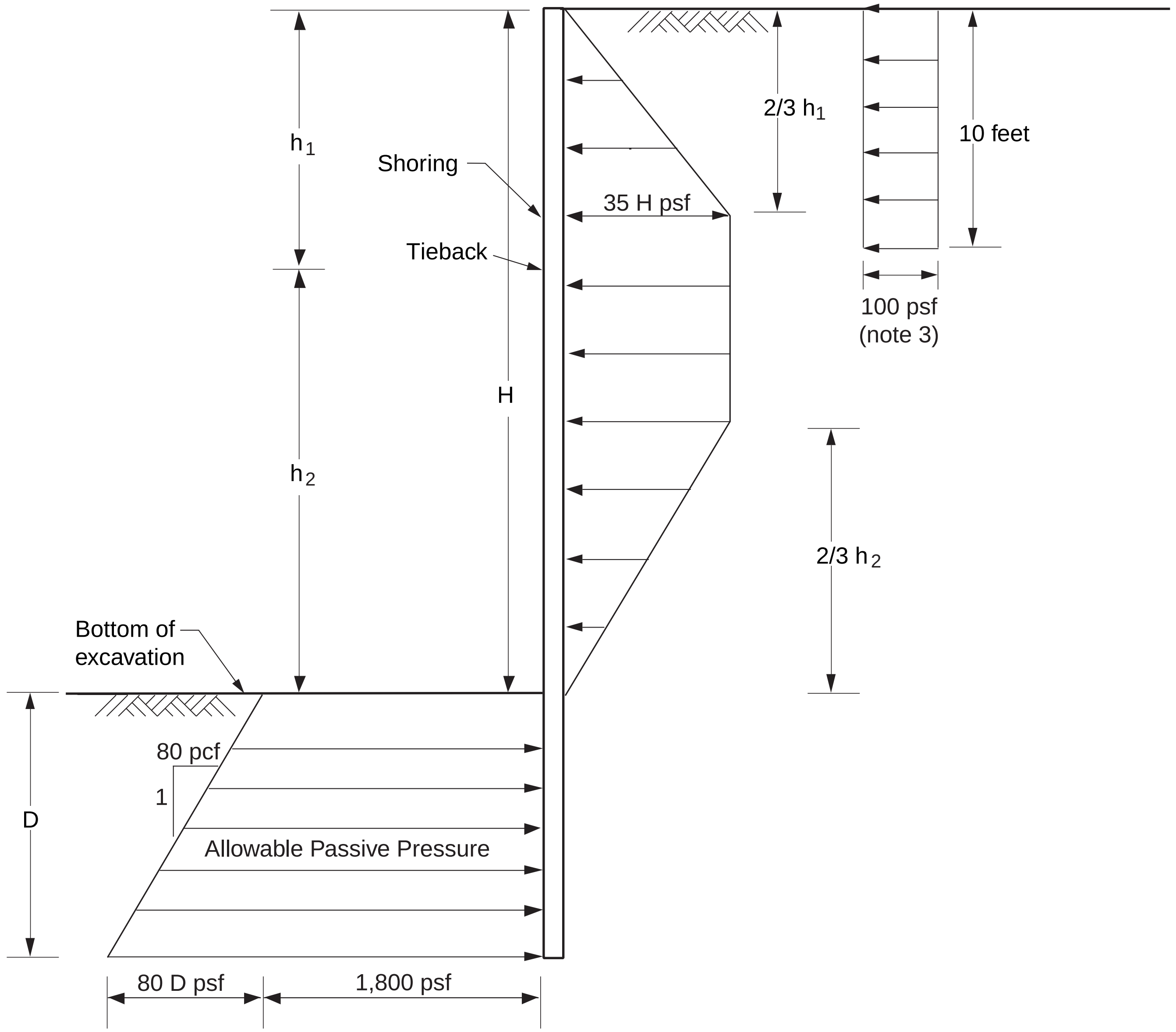
LAKEPORT COURTHOUSE
675 LAKEPORT BOULEVARD
Lakeport, California

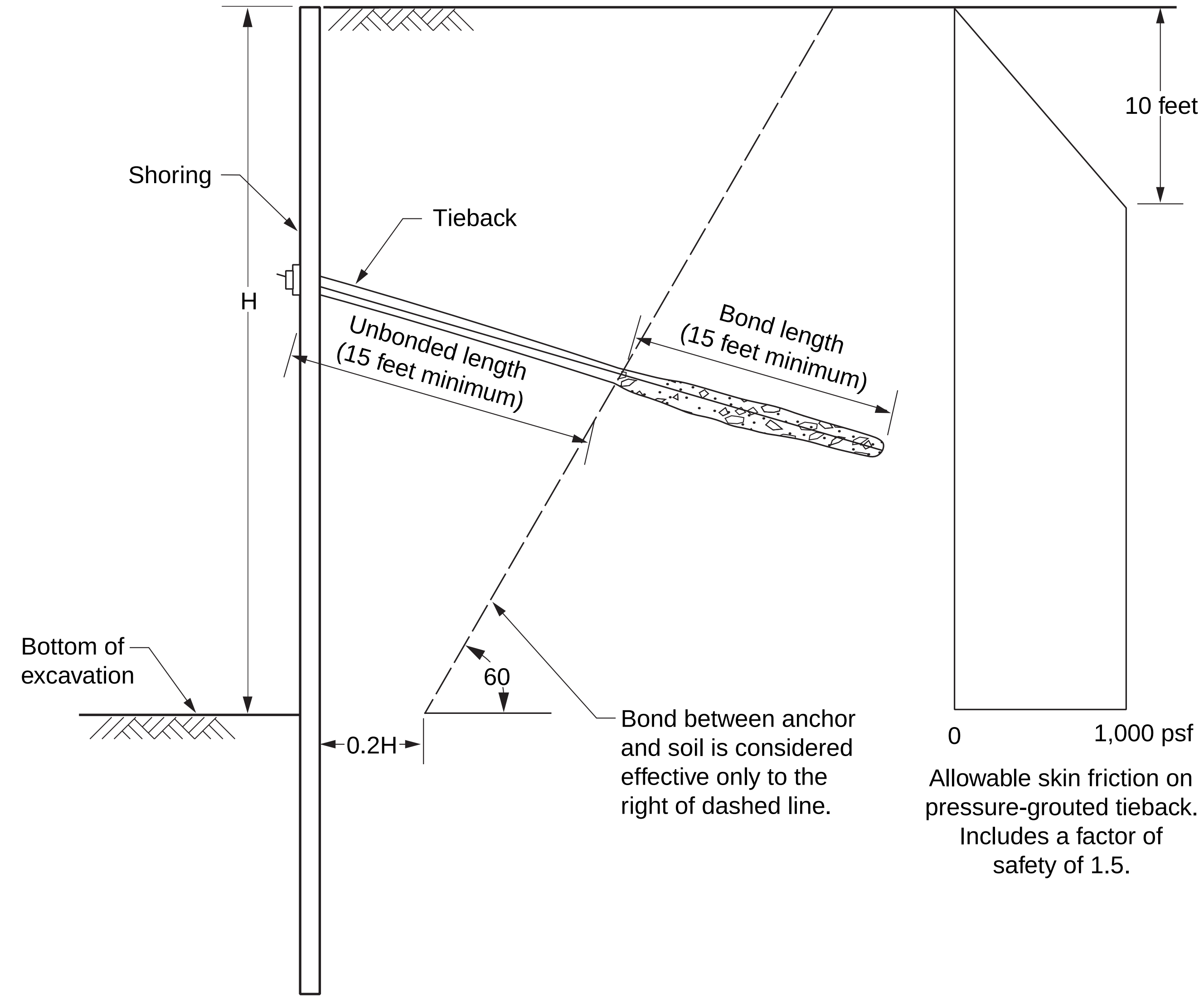
MOMENT AND DEFLECTION PROFILES
DRILLED PIER
SLOPED GROUND SURFACE

Date 03/04/15	Project No. 731563902	Figure 11

LANGAN TREADWELL ROLLO

DRAFT





Notes:
1. The above pressure diagram assumes that the shoring walls consist of pervious soldier-pile-and-lagging system.
2. Passive pressure values include a factor of safety of about 1.5 and can be applied over a width of three soldier pile diameters or pile spacing, whichever is smaller.
3. Pressure due to vehicle surcharge (heavy equipment should come no closer than 5 feet to face of excavation).
4. D and H in feet.

LAKEPORT COURTHOUSE
675 LAKEPORT BOULEVARD
Lakeport, California

TYPICAL LATERAL EARTH PRESSURES AND TIEBACK CRITERIA FOR TEMPORARY SHORING SYSTEM

Date 03/04/15	Project No. 731563902	Figure 12

LANGAN TREADWELL ROLLO

APPENDIX A

LOGS OF BORINGS AND TEST PITS

DRAFT

PROJECT:	LAKEPORT COURTHOUSE 675 LAKEPORT BOULEVARD Lakeport, California	Log of Boring B-1

Boring location: See Site Plan, Figure 2

Logged by: M. Mascorro

Date started: 11/29/11 Date finished: 11/29/11

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches Hammer type: Automatic

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 1391 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1–2	BULK				CL	SANDY CLAY with GRAVEL (CL) dark reddish-brown, moist, with roots, abundant serpentinite rock fragments LEVELING SLIVER FILL						
2.5	S&H		50/ 5"	35/ 5"								
3						SERPENTINITE BEDROCK olive-gray to black, very hard, weak to moderately strong, little weathered, moist						
5	S&H		50/ 3"	60/ 3"								
5.5	SPT		50/ 3"	60/ 3"								
7.5	SPT		50/ 2.5"	60/ 2.5"		dark green to black, very hard, fresh fracture surfaces						
10	SPT		50/ 3"	60/ 3"		green and yellow-brown to black, hard with fragments of moderately hard rock, weak, foliated, soapy fracture surfaces						
15	SPT		50/ 2"	60/ 2"		moisture on fracture surfaces, some oxidation, in foliated fragments						
18						increased moisture content in cuttings from 15 to 18 feet						
20	SPT		50/ 4"	60/ 4"		green-gray to black, very hard, weak to moderately strong, foliated						
25	SPT		50/ 2"	60/ 2"		black, moderately hard, moderately strong, blocky and foliated fracturing						
30												

LANGAN TREADWELL ROLLO

Project No.: 731563902

Figure: A-1a

TEST GEOTECH LOG 731563901 FOR 102.GPJ TR.GDT 3/4/15

DRAFT

PROJECT: **LAKEPORT COURTHOUSE**
675 LAKEPORT BOULEVARD
Lakeport, California

Log of Boring B-1

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
30	SPT		50/ 1"	50/ 1"		SERPENTINITE BEDROCK (continued) dark green to black, very hard, with thin veins of low hardness, foliated fracturing, primarily along vein planes						
31												
32												
33												
34												
35												
36												
37												
38												
39												
40	SPT		50/ 6"	60/6"		blue-green to black, low hardness to moderately hard, weak, soapy fracture surfaces, highly foliated						
41												
42												
43												
44												
45												
46												
47												
48												
49												
50	SPT		50/ 6"	60/ 6"		dark green to black, low hardness to very hard, friable to moderately strong, angular fracturing, fresh, polished fracture surfaces						
51												
52												
53												
54												
55												
56												
57												
58												
59												
60						▽						

LANGAN TREADWELL ROLLO

Project No.: 731563902

Figure: A-1b

DRAFT

PROJECT:	LAKEPORT COURTHOUSE 675 LAKEPORT BOULEVARD Lakeport, California	Log of Boring B-1 PAGE 3 OF 3

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	SPT		50/ 3"	50/ 3"		SERPENTINIITE BEDROCK (continued)						
61												
62												
63												
64												
65												
66												
67												
68												
69												
70												
71												
72												
73												
74												
75												
76												
77												
78												
79												
80												
81												
82												
83												
84												
85												
86												
87												
88												
89												
90												

Boring terminated at a depth of 60.25 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 60 feet below ground surface during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on National Geodetic Vertical Datum 1929.

LANGAN TREADWELL ROLLO

Project No.: 731563902

Figure: A-1c

TEST GEOTECH LOG 731563901 FOR 102.GPJ TR.GDT 3/4/15

DRAFT

PROJECT: **LAKEPORT COURTHOUSE**
675 LAKEPORT BOULEVARD
Lakeport, California

Log of Boring B-2

Boring location: See Site Plan, Figure 2

Logged by: M. Mascorro

Date started: 11/28/11 | Date finished: 11/28/11

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

LABORATORY TEST DATA

MATERIAL DESCRIPTION

Ground Surface Elevation: 1394 feet[2]

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–2				GC	CLAYEY GRAVEL (GC) dark brown, medium dense, moist, black to brown to bluish-green angular serpentinite gravel, abundant cobble- to boulder-sized clasts in fill	FILL (0–18 ft)						
3	S&H	14 18 21	27									
6	S&H	13 17 22	27		reddish-brown sandy clay, increased moisture content, serpentinite rock fragments friable to moderately strong and deeply weathered (with iron staining)							
8	S&H	10 20 22	29									
11	S&H	23 20 18	27									
15–16	S&H	9 6 5	8	GP	GRAVEL with SAND (GP) greenish-black to black gravel, olive sand, loose, moist							
18–19					SERPENTINITE BEDROCK olive-brown to dark gray, intensely fractured, soft to hard, weak to strong, moderately weathered							
21	S&H	22 43 50	65									
25	S&H	50/ 2"	35/ 2"		black to bluish-green, seam of soft deeply weathered (oxidized) rock							
25.5	SPT	50/ 2"	60/ 2"									
30												

LANGAN TREADWELL ROLLO

Project No.: 731563902 | Figure: A-2a

TEST GEOTECH LOG 731563901 FOR 102.GPJ TR.GDT 3/4/15

PROJECT: **LAKEPORT COURTHOUSE**
675 LAKEPORT BOULEVARD
Lakeport, California

Log of Boring B-2

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	SPT		50/ 5"	60/ 5"		SERPENTINITE BEDROCK (continued) increased rock hardness, fresh fractures, fractures into angular fragments						
31												
32												
33												
34												
35												
36												
37												
38												
39												
40	SPT		50/ 5"	60/ 5"		intensely crushed, soft to moderately hard, friable to weak, deeply weathered (oxidized fracture surfaces)						
41												
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 40.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on National Geodetic Vertical Datum 1929.

LANGAN TREADWELL ROLLO

Project No.: 731563902 | Figure: A-2b

DRAFT

PROJECT:	LAKEPORT COURTHOUSE 675 LAKEPORT BOULEVARD Lakeport, California	Log of Boring B-3

Boring location: See Site Plan, Figure 2

Logged by: M. Mascorro

Date started: 11/28/11 | Date finished: 11/28/11

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 1395 feet[2])	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–2				CL	SANDY CLAY with GRAVEL (CL) dark brown mottled with yellow, very stiff, moist						
3	S&H	17 20 23	30		very stiff to hard, decreased clay content, increased sand content, with abundant fragments of serpentinite Corrosion Test, see Figure B-4						
5–6	S&H	12 20 35	39	GP-GC	GRAVEL with CLAY (GP-GC) reddish-brown clay, olive-gray and brown serpentinite fragments, dense, moist						
8	S&H	16 10 7	12		dark gray serpentinite fragments, medium dense, decreased clay content						
10–11	S&H	4 7 8	11	CL	GRAVELLY CLAY (CL) reddish-brown clay, stiff, moist, gravel consists of serpentinite fragments						
15–16	S&H	6 9 11	14	CL	CLAY with GRAVEL (CL) dark reddish-brown, gray gravel mottled with reddish-orange iron staining, stiff, moist to wet, friable to strong angular serpentinite gravel increase in moisture content Corrosion Test, see Figure B-4						
18					SERPENTINITE BEDROCK mottled olive-gray and black, moderately hard, little to moderately weathered, weak to moderately strong, moderately foliated, polished fractured surfaces, moist						
20	S&H	50/ 2"	60/ 2"								
20.5	SPT	50/ 2"	60/ 2"								
25	SPT	50/ 2"	60/ 2"								

FILL (0 to approximately 17.5 feet)

LANGAN TREADWELL ROLLO

Project No.: 731563902 | Figure: A-3a

DRAFT

PROJECT:	LAKEPORT COURTHOUSE 675 LAKEPORT BOULEVARD Lakeport, California	Log of Boring B-3 PAGE 2 OF 3

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
30	SPT		50/ 4"	60/ 4"		SERPENTINITE BEDROCK (continued) black to dark green, soft to hard, friable to weak, moist						
31												
32												
33												
34												
35												
36												
37												
38												
39												
40	SPT		50/ 6"	60/ 6"		black, polished fractured surfaces, fresh, some slickenside, foliated, variable hardness and strength, moist						
41												
42												
43												
44												
45												
46												
47												
48												
49												
50	SPT		50/ 3"	60/ 3"								
51												
52												
53												
54												
55												
56												
57												
58												
59												
60						▽						

LANGAN TREADWELL ROLLO

Project No.: 731563902

Figure: A-3b

DRAFT

PROJECT:	LAKEPORT COURTHOUSE 675 LAKEPORT BOULEVARD Lakeport, California	Log of Boring B-3 PAGE 3 OF 3

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	SPT		50/ 5"	60/ 5"		SERPENTINITE BEDROCK (continued) hard, fresh, wet, foliated fracturing						
61												
62												
63												
64												
65												
66												
67												
68												
69												
70												
71												
72												
73												
74												
75												
76												
77												
78												
79												
80												
81												
82												
83												
84												
85												
86												
87												
88												
89												
90												

Boring terminated at a depth of 60.4 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 60 feet below ground surface during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevations based on National Geodetic Vertical Datum 1929.

LANGAN TREADWELL ROLLO

Project No.:	Figure:
731563902	A-3c

TEST GEOTECH LOG 731563901 FOR 102.GPJ TR.GDT 3/4/15

DRAFT

PROJECT:	LAKEPORT COURTHOUSE 675 LAKEPORT BOULEVARD Lakeport, California	Log of Boring B-4 PAGE 1 OF 1

Boring location:	See Site Plan, Figure 2
Date started:	11/29/11
Date finished:	11/29/11
Drilling method:	Hollow Stem Auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Automatic
Sampler:	Sprague & Henwood (S&H)
Logged by:	M. Mascorro

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 1392 feet[2]	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–2.5					CL	SANDY CLAY with GRAVEL (CL) dark reddish-brown, moist, abundant angular serpentinite gravel LEVELING SLIVER FILL						
3	S&H		50/ 6"	35/ 6"		SERPENTINITE BEDROCK olive and dark yellowish-brown to black, highly mottled, intensely crushed, soft to low hardness, very weak, weathered to soil-like consistency, seam of highly plastic red clay						
5	S&H		50/ 6"	35/ 6"								

Boring terminated at a depth of 5.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H blow counts for the last two increments were converted to SPT N-Values using a factor of 0.7, to account for sampler type and hammer energy.
[2] Elevations based on National Geodetic Vertical Datum 1929.

LANGAN TREADWELL ROLLO

Project No.: 731563902

Figure: A-4

TEST GEOTECH LOG 731563901 FOR 102.GPJ TR.GDT 3/4/15

DRAFT

PROJECT: **LAKEPORT COURTHOUSE**
675 LAKEPORT BOULEVARD
Lakeport, California

Log of Boring B-5

Boring location: See Site Plan, Figure 2

Date started: 11/28/11 | Date finished: 11/28/11

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic

Sampler: Sprague & Henwood (S&H)

Logged by: M. Mascorro

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 1393 feet[2])		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1	BULK				MH	SANDY SILT with GRAVEL (MH) dark reddish-brown, hard, moist, serpentinite cobbles yellowish-brown to dark greenish black, intensely crushed, soft to moderately hard, very weak, deeply weathered	FILL					10.1	
3	S&H		35 50/ 5"	35/ 5"		LL = 66, PI = 32, see Figure B-1 Resistance Value Test, see Figure B-2							
5	S&H		30 50/ 5"	35/ 5"		yellowish-brown							

Boring terminated at a depth of 5.9 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H blow counts for the last two increments were converted to SPT N-Values using a factor of 0.7, to account for sampler type and hammer energy.
[2] Elevations based on National Geodetic Vertical Datum 1929.

LANGAN TREADWELL ROLLO

Project No.: 731563902 | Figure: A-5

TEST GEOTECH LOG 731563901 FOR 102.GPJ TR.GDT 3/4/15

DRAFT

PROJECT:	LAKEPORT COURTHOUSE 675 LAKEPORT BOULEVARD Lakeport, California	Log of Boring B-6 PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Logged by: M. Mascorro

Date started: 11/28/11 | Date finished: 11/28/11

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic

Sampler: Sprague & Henwood (S&H)

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 1394.5 feet[2]		LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1	BULK				CL	SANDY CLAY with GRAVEL (CL) dark reddish-brown clay, stiff, moist, abundant yellowish-brown to greenish-brown and black serpentinite gravel and cobbles of variable strength, hardness, and weathering Resistance Value Test, see Figure B-3	FILL						
2													
3	S&H		11 9 6	11									
4													
5													
6	S&H		12 16 23	27		very stiff							
7													
8													
9													
10													
11													
12													
13													
14													
15													
16													
17													
18													
19													
20													
21													
22													
23													
24													
25													
26													
27													
28													
29													
30													

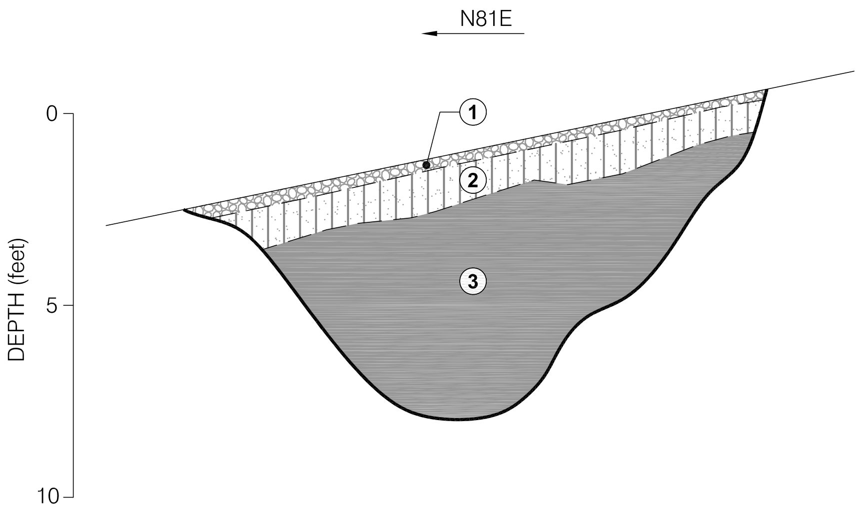
Boring terminated at a depth of 6.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H blow counts for the last two increments were converted to SPT N-Values using a factor of 0.7, to account for sampler type and hammer energy.
[2] Elevations based on National Geodetic Vertical Datum 1929.

LANGAN TREADWELL ROLLO

Project No.: 731563902 | Figure: A-6

DRAFT

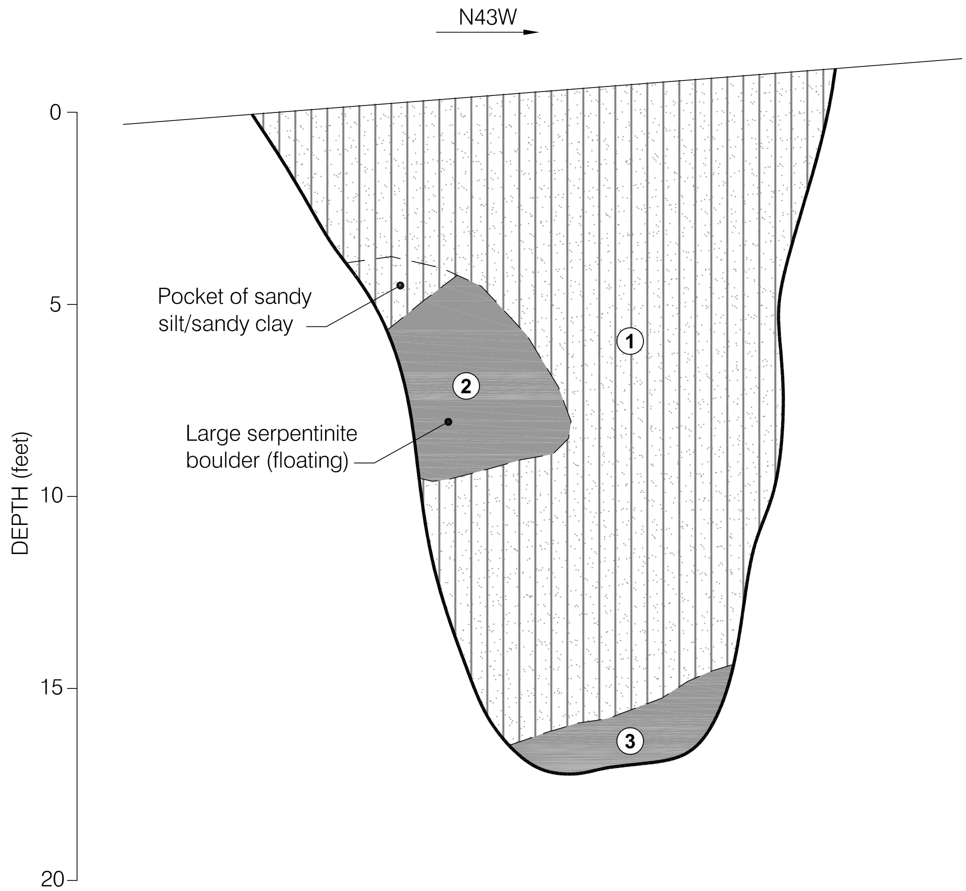


① GRAVEL (GP)
gray to greenish-gray, loose, angular to subangular, poorly sorted, 1/4-inch to 1 1/2-inch, scattered grass and organics *[FILL]*

② SANDY CLAY (CL)
very dark brown, stiff, moist, moderately plastic, poorly sorted, 20-25% fine- to coarse-grained subrounded to subangular sand and scattered gravel to 1/2-inch in diameter, scattered roots and organics *[BURIED TOPSOIL]*

③ SERPENTINITE
white to dark gray, very strong, slightly weathered, angular fractures, fractures lithified/cemented, hard, moist *[BEDROCK]*

LAKEPORT COURTHOUSE 675 LAKEPORT BOULEVARD Lakeport, California	LOG OF TEST PIT TP-1		
LANGAN TREADWELL ROLLO	Date 12/13/11	Project No. 731563902	Figure A-7



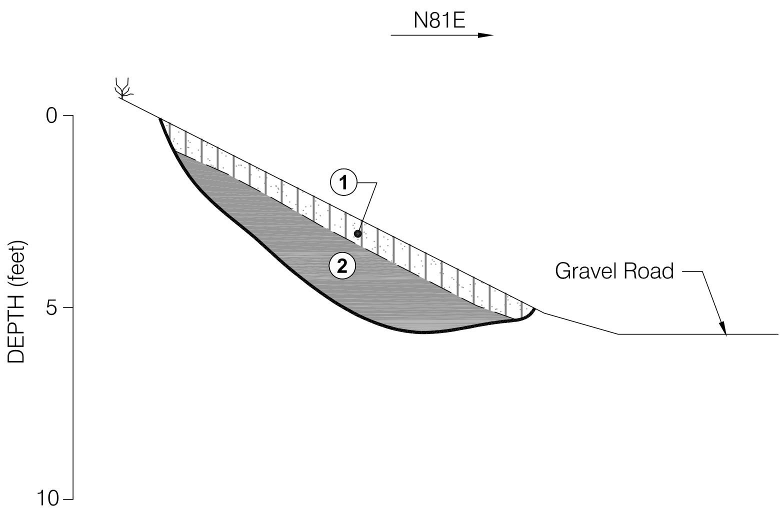
(1) SILTY SAND with GRAVEL (SM)
dark brown, medium dense to dense, moist, poorly sorted, fine- to coarse-grained, with 10-15% angular to subangular gravels to 1-inch in diameter, slightly to moderately oxidized *[FILL]*

(2) SERPENTINITE
white with brown oxidation staining, moderately weathered, hard, subrounded to subangular fractures, highly fractured, moist *[DISPLACED BEDROCK BOULDER]*

(3) SERPENTINITE
olive, olive-yellow, and black, slightly weathered, oxidation staining along fracture surfaces, moist, very hard, slightly fractured [*BEDROCK*]

LAKEPORT COURTHOUSE **675 LAKEPORT BOULEVARD** Lakeport, California	**LOG OF TEST PIT** **TP-2**
LANGAN TREADWELL ROLLO	Date 12/13/11 | Project No. 731563902 | Figure A-8

DRAFT



1. SANDY SILT (ML) to SANDY CLAY (CL)
   dark reddish-brown, moist, 10-20% very fine- to medium-grained sand with scattered gravel, scattered roots and decaying organics *[TOPSOIL]*

2. SERPENTINITE
   gray to black, slightly weathered, very hard, slightly fractured, moist, oxidized, angular fractures [*BEDROCK]*

LAKEPORT COURTHOUSE
675 LAKEPORT BOULEVARD
Lakeport, California

LANGAN TREADWELL ROLLO

LOG OF TEST PIT
TP-3

Date 12/13/11 | Project No. 731563902 | Figure A-9

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DRAFT

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size	**Gravels** (More than half of coarse fraction > no. 4 sieve size)	**GW**	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	**SW**	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	**Silts and Clays** LL = < 50	**ML**	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	**MH**	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		**PT**	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes: U.S. Standard Sieve Size	Range of Grain Sizes: Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

- Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered
- Classification sample taken with Standard Penetration Test sampler
- Undisturbed sample taken with thin-walled tube
- Disturbed sample
- Sampling attempted with no recovery
- Core sample
- Analytical laboratory sample
- Sample taken with Direct Push sampler

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
S&H	Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

LAKEPORT COURTHOUSE
675 LAKEPORT BOULEVARD
Lakeport, California

LANGAN TREADWELL ROLLO

CLASSIFICATION CHART

Date 12/13/11 | Project No. 731563902 | Figure A-10

DRAFT

I FRACTURING

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

II HARDNESS

1. **Soft** - reserved for plastic material alone.
2. **Low hardness** - can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** - can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. **Hard** - can be scratched with difficulty; scratch produced a little powder and is often faintly visible.
5. **Very hard** - cannot be scratched with knife blade; leaves a metallic streak.

III STRENGTH

1. **Plastic** or very low strength.
2. **Friable** - crumbles easily by rubbing with fingers.
3. **Weak** - an unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** - specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** - specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** - specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

IV WEATHERING - The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

D. Deep - moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.
M. Moderate - slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
L. Little - no megascopic decomposition of minerals; little of no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.
F. Fresh - unaffected by weathering agents. No disintegration of discoloration. Fractures usually less numerous than joints.

ADDITIONAL COMMENTS:

V CONSOLIDATION OF SEDIMENTARY ROCKS: usually determined from unweathered samples. Largely dependent on cementation.

U = unconsolidated
P = poorly consolidated
M = moderately consolidated
W = well consolidated

VI BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 ft.	very thick-bedded
Blocky	2.0 to 4.0 ft.	thick bedded
Slabby	0.2 to 2.0 ft.	thin bedded
Flaggy	0.05 to 0.2 ft.	very thin-bedded
Shaly or platy	0.01 to 0.05 ft.	laminated
Papery	less than 0.01	thinly laminated

APPENDIX B

RESULTS OF GEOPHYSICAL SURVEYS

DRAFT

NORCAL GEOPHYSICAL CONSULTANTS, INC.

February 24, 2015

Langan Treadwell Rollo
555 Montgomery Street, Suite 1300
San Francisco, California 94111

Subject: Seismic Refraction Investigation
675 Lakeport Boulevard
Lakeport, California

NORCAL Job No: 15-243.110

Attention: Ms. Marina Mascorro

This report presents the findings of a seismic refraction (SR) investigation performed by NORCAL Geophysical Consultants at the subject address. This investigation is in conjunction with the planned improvements to the site and the construction of the proposed Lakeport Courthouse. The survey was performed on January 28th and 29th, 2015 by NORCAL Professional Geophysicist David T. Hagin PGp 1033 and Staff Geophysicist Hunter S. Philson. Logistical support and safety information were provided onsite by Ms. Jane Elliot of Langan Treadwell Rollo.

1.0 SITE DESCRIPTION and PURPOSE

The site is composed of an approximate 320 X 280 ft empty building pad with large fill slopes on the northern and eastern sides, where a descending access road is located (Plate 1, base map supplied by Langan Treadwell Rollo). Based on the fill slopes and the surrounding terrain, we expect the fill to be thickest on the eastern portion of the pad. As the building pad is visible in an aerial photograph taken in 1993, it was constructed over 20 years ago. The site is accessed via a small gravel paved road off of Bevins Street. At the time of the survey the ground was dry and the weather fair.

The purpose of this investigation was to evaluate the shallow sub-surface conditions in the location of the planned structure by measuring the seismic p-wave velocity values using the seismic refraction (SR) method. These data are used to evaluate the thickness of the fill and possible underlying colluvium over serpentinite bedrock. Additionally, an MASW (Multichannel Analysis of Surface Waves) sounding was performed to measure s-wave velocities and aid in the evaluation of ground stiffness.

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2.0 METHODOLOGY

2.1 Seismic Refraction

The SR method is used to determine the compressional acoustic primary wave velocity (seismic velocity) of subsurface materials. The seismic velocity of fill, sediments, and rock are dependent on physical properties such as compaction, density, and induration (hardness). However, other factors such as bedding, fracturing, and saturation also affect seismic velocity. Typically, low velocities are indicative of loose, dry soils, poorly compacted fill material, poorly to semi-consolidated sediments, or alternatively, deeply weathered and/or highly fractured rock. Moderate velocities usually indicate dense and highly compacted or saturated sedimentary deposits or fill, and/or moderately weathered and fractured rock. High velocities typically represent slightly weathered to unweathered (fresh) rock with little fracturing. A more detailed description of the SR methodology is provided in Appendix A.

2.2 MASW

When seismic waves are generated at or near the ground surface, both body and surface waves are generated; these are commonly referred to as ground roll in seismic surveys. Surface waves have dispersion properties that body waves lack. By analyzing the dispersion of surface waves it is possible to obtain a near-surface s-wave velocity profile. A more detailed description of the MASW methodology is provided in Appendix B.

3.0 FIELD SURVEY AND DATA PROCESSING

3.1 Data Acquisition

The geophysical survey entailed the acquisition of six SR lines extending over the surface of the pad and along the descending access road near the area of the planned structure, as shown on Plate 1; the placement of the lines was determined by Langan Treadwell Rollo personnel. The seismic lines each consisted of a single geophone spread comprised of 24 geophones and 7 shot points distributed in a collinear array. The geophones were coupled to the ground surface at 5 to 10 foot intervals for total line lengths between 125 and 250 feet. The two end shot points were located one or one-half station beyond each end of the geophone spread and the remaining shot points were evenly spaced within the spread.

The MASW sounding was performed in the location of SR Line II. The sounding employed 24 geophones coupled to the ground at 6-ft intervals. Shot points were located at 12, 24 and 36 feet off of each end of the line.

3.2 Instrumentation

The SR data were recorded using a ***Geometrics Geode***, 24-bit digital seismic recording system and ***Oyo Geospace*** digital-grade geophones with a natural frequency of 10 Hz. We produced seismic energy at each shot point by striking an aluminum plate placed on the ground surface with a 16-pound sledge hammer. An accelerometer attached to the hammer transmitted a triggering pulse to the seismograph to begin recording each time the plate was struck. Several strikes were performed and stacked at each shot point to ensure an acceptable signal to noise ratio. The locations and elevations of the geophones and shot-points were determined using GPS locating and the topographic map supplied by Langan Treadwell Rollo.

3.3 Data Processing

The refraction data were processed in-house using ***SeisImager***, specialized software developed by Geometrics, Inc. of San Jose, California. We then used the program ***Surfer 12*** by Golden Software to graphically illustrate the subsurface distribution of seismic velocities. This consisted of generating a color-contoured seismic velocity cross-section (profile) for each seismic line, as shown on Plates 2, 3 and 4.

The MASW data were also processed in-house using ***SurfSeis 3***, dispersion-curve inversion software developed by the Kansas Geological Survey. The resulting model is a one dimensional sounding; depth intervals and their associated s-wave velocity values are presented in Table A.

4.0 RESULTS AND INTERPRETATIONS

The results of the seismic refraction survey are illustrated by the seismic velocity profiles shown on Plates 2, 3 and 4. The vertical axes represent elevation in feet (above mean sea level) and the horizontal axes represent survey stationing in feet (distance along the line). The profiles show the ground surface and color contours representing the distribution of seismic velocity values according to the color scale shown at the bottom of each plate.

4.1 Seismic Velocities

Low seismic velocity values of less than about 4,500 feet per second (ft/s) are generally interpreted to represent the overburden, consisting of fill and/or underlying colluvial material (brown, yellow). Moderate seismic velocity values ranging from 4,500 to 6,000 ft/s are interpreted to likely represent a transition zone to moderately weathered and/or fractured rock (green, blue). High seismic velocity values are greater than 6,000 ft/s; they are interpreted to represent less weathered and/or fractured rock (maroon). The maximum seismic velocity values measured were under 8,000 ft/s.

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4.2 Seismic Refraction Profiles

The SR profiles provide a general characterization of the fill/colluvium over bedrock. Inspection of the SR lines reveals undulating contours on many of the profiles, suggesting that the original ground surface may have been tortuous. On the building pad, SR line D suggests a wedge of fill material on the building pad thickening toward the east, as expected. Line C indicates only five or six feet of overburden, whereas Line H shows nearly 20 feet of overburden. The lines correlate well at the tie points and maximum velocity values are similar on all of the profiles.

On the access road, Lines E and F indicate a wedge of overburden that thickens toward the east to approximately 12 feet; however, Line G shows the low velocity wedge pinching out against higher velocities below at the southern end of the line. This is in agreement with the observation of a rising "knob" of bedrock visible in the cut/fill slope below the southern portion of the line (also visible in the aerial photographs). Again, the lines correlate well at the tie points and maximum velocity values are comparable on all of the profiles.

4.3 SR Limitations

It should be noted that the seismic refraction technique is based on the assumption that seismic velocity increases with depth. Any layers representing a decrease in velocity with depth, otherwise known as a velocity inversion, will not be defined and will result in the over-estimation of the depth of deeper, higher velocity layers. In addition, relatively thin layers might not be individually resolved and might, instead, be lumped together with other layers. Hard and soft zones within a given seismic layer will tend to be averaged into the velocity of that layer. Finally, there is not necessarily a one-to-one relationship between lithologic layers and seismic layers. It is entirely possible that two different types of material could have the same seismic velocity. Alternatively, a change in velocity can occur within a single lithologic unit. A more detailed discussion of the limitations with regard to the seismic refraction method is presented in Appendix A.

4.4 MASW Sounding

We acquired a single MASW sounding located at the center of Line H, where the SR profiles indicate that fill extends to a depth of approximately 20 feet. The results of the sounding are presented by Table A, providing depth intervals and their associated s-wave velocity values.

Table A

DEPTH INTERVAL (FT)	S-WAVE VELOCITY (FT/S)
0 – 2.5	1448
2.5 - 5	1444
5 - 10	1448
10 - 15	855
15 - 20	1158
20 - 25	2082
25 - 35	2564
35 - 45	2917
45 - 60	3425
60 - 75	5583

We interpret the sharp rise in the s-wave velocity values in the 20-25 ft depth interval to indicate the presence of bedrock; this correlates well with the results of Line II. The s-wave velocity values associated with the interpreted bedrock are greater than 2,000 ft/s.

5.0 STANDARD OF CARE

The scope of NORCAL's services for this project consisted of using geophysical methods to characterize the subsurface. The accuracy of our findings is subject to specific site conditions and limitations inherent to the techniques used. We performed our services in a manner consistent with the standard of care ordinarily exercised by members of the profession currently employing similar methods. No warranty, with respect to the performance of services or products delivered under this agreement, expressed or implied, is made by NORCAL.

Thank you for the opportunity to participate on this project.

Sincerely,

NORCAL Geophysical Consultants, Inc.

David Hagin

David T. Hagin
Professional Geophysicist PGp 1033

DTH/KGB/tt
Enclosures: Plates 1 through 4
Appendix A - Seismic Refraction Survey
Appendix B - MASW Survey

Appendix A

SEISMIC REFRACTION SURVEY

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Appendix A

SEISMIC REFRACTION (SR)

METHODOLOGY

The seismic refraction method provides information regarding the seismic velocity structure of the subsurface. An impulsive (mechanical or explosive) source is used to produce compressional (P) wave seismic energy. The P-waves propagate into the earth and are refracted along interfaces caused by an increase in velocity. A portion of the P-wave energy is refracted back to the surface where it is detected by sensors (geophones) that are coupled to the ground surface in a collinear array (spread). The detected signals are recorded on a multi-channel seismograph and are analyzed to determine the shot point-to-geophone travel times. These data can be used along with the corresponding shot point-to-geophone distances to determine the depth, thickness, and velocity of subsurface seismic layers.

The seismic refraction technique is based on several assumptions. Paramount among these are:

- seismic velocity increases with depth, and,
- the velocity of each seismic layer is uniform over the length of the given spread.

In cases where these assumptions do not hold, the accuracy of the technique decreases. For example, if a low velocity layer occurs between two layers of higher velocity, the low velocity layer will not be detected and the depth to the underlying high velocity layer will be erroneously large. Also, if the velocity of a seismic layer varies laterally within a spread, those variations will be interpreted as fluctuations in the elevation of the underlying seismic layer.

It should be noted that apparent velocities can be affected by the orientation of bedding planes with respect to the direction of the seismic profile. Apparent velocities of rock are typically slower when measured along lines oriented perpendicular to bedding planes of steeply dipping rock than those measured along lines oriented parallel.

INSTRUMENTATION

Data acquisition is initiated along each SR line by producing seismic energy using a mechanical source. Mechanical sources produce energy by impacting a metal strike plate on the ground surface with either a 12-16 pound sledge hammer or an elastic-band driven weight drop. The resulting seismic wave forms are recorded using a Geometrics 24-channel engineering seismograph and Mark Products geophones with a natural frequency of 10 Hz. The data are recorded on hard copy records (seismograms) as well as on computer disks for future processing. The seismograms display the amount of time it takes for a compression (P) wave to travel from a given shot point to each geophone in a spread.

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DATA ANALYSIS

The seismic data are downloaded to a computer and processed using the software ***Seisimager*** by Geometrics, Inc. This is an interactive program that is used to determine the shot point to geophone travel times, and to compute a 2D model based on those times. Once the travel times for a given line are determined, the programs time-term algorithm is used to compute a preliminary 2D seismic model. This model is then used as input for the programs tomographic routine. Using this procedure, the program divides the starting model into a network of cells and assigns velocities to those cells based on the starting model. The program then traces the refracted seismic travel paths through those cells and computes the associated travel times. It then compares the computed travel times with the measured times and adjusts the velocities of the appropriate cells to improve the fit. The software is programmed to continue this procedure for twenty iterations. Typically, at the end of the twenty iterations the travel times associated with the computed model match the observed travel times to an accuracy of one milli-second (mS) or better. Once a satisfactory model is computed, the software contours the model velocities to produce seismic velocity vs. depth and distance cross-sections (profiles).

LIMITATIONS

In general, there are limitations unique to the SR method. These limitations are primarily based on assumptions that are made by the data analysis routine. First, the data analysis routine assumes that the velocities along the length of each spread are uniform. If there are localized zones within each layer where the velocities are higher or lower than indicated, the analysis routine will interpret these zones as changes in the surface topography of the underlying layer. A zone of higher velocity material would be interpreted as a low in the surface of the underlying layer. Zones of lower velocity material would be interpreted as a high in the underlying layer.

Second, the data analysis routine assumes that the velocity of subsurface materials increase with depth. Therefore, if a layer exhibits velocities that are slower than those of the material above it, the slower layer will not be resolved. Also, a velocity layer may simply be too thin to be detected. Due to these and other limitations inherent to the SR method, the results of the SR survey should be considered only as approximations of the subsurface conditions. The actual conditions may vary locally.

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Appendix B

MASW SURVEY

Appendix B

1-D MULTI-CHANNEL ANALYSIS OF SURFACE WAVES (MASW)

Methodology

When seismic waves are generated at or near the ground surface, both body and surface waves are generated. Body waves consist of both compressional (P) and shear (S) waves. Surface waves (e.g., Rayleigh, Love, etc.) propagate at velocities that are proportional to shear wave velocity (Vs). If a vertical energy source is used, Rayleigh type surface waves are produced. These are commonly referred to as ground roll in seismic surveys. Rayleigh waves are retrograde elliptical and travel at approximately 0.9 times the velocity of S-waves.

MASW data are gathered in much the same way as high-resolution reflection data. Seismic energy - generated by vertical impacts on the ground surface - is detected by an array of closely spaced geophones. The primary differences are that the surface wave technique requires an energy source that is capable of producing ground roll and geophones that are capable of detecting low frequency (<10 Hz) signals.

Surface waves account for more than two-thirds of the energy produced by vertical seismic energy sources. As a result, surface waves are the most prominent signal on multi-channel seismic records. In addition, surface waves have dispersion properties that body waves lack. That is, different wavelengths have different penetration depths and, therefore, propagate at different velocities. By analyzing the dispersion of surface waves it is possible to obtain a near-surface S-wave velocity profile. Since s-wave velocity is directly proportional to shear modulus, this provides a direct indication in the variation of stiffness (or rigidity) of subsurface materials.

Data Acquisition and Analysis

The MASW data are recorded using a Geometrics Geode 24-channel seismograph and 24 8-Hz geophones. Typically, the geophones are distributed at 6-ft intervals along the seismic line, and shot points are located at 12, 24 and 36 feet off each end of the active geophone spread. Seismic energy is typically produced at each shot point using a 16-pound sledgehammer striking a metal plate on the ground surface: and excellent source of surface wave energy.

The surface wave measurements were converted to Vs versus depth models using a technique known as multi-channel analysis of surface waves (MASW). The raw seismic wave-traces (shot gathers) produced at the near and far offset shot points were input to the computer program ***SURFSEIS*** developed by the Kansas Geological Survey (Version 2.0, 2007). This program analyzes the data by identifying the ground-roll portion of the seismic wave traces, computing the frequency and velocity of the wavelets, and constructing a dispersion curve representing the variation in surface wave velocity versus frequency. The program then inverts the dispersion

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curve to compute a one-dimensional (1D) layered model indicating shear-wave velocity (Vs) versus depth beneath the center of the geophone array for each shot gather. In all cases the MASW modeling was iterated until the dispersion curve generated from the S-wave velocity model closely matched that calculated from the shot gathers. The 1D models inverted from all four shot gathers were then entered into a spread sheet which computed average Vs versus depth values. Since the inversion of the dispersion curve into a shear wave velocity profile is a non-unique process, the software will produce a shear wave profile containing 10 distinct subsurface velocity intervals at various depths.

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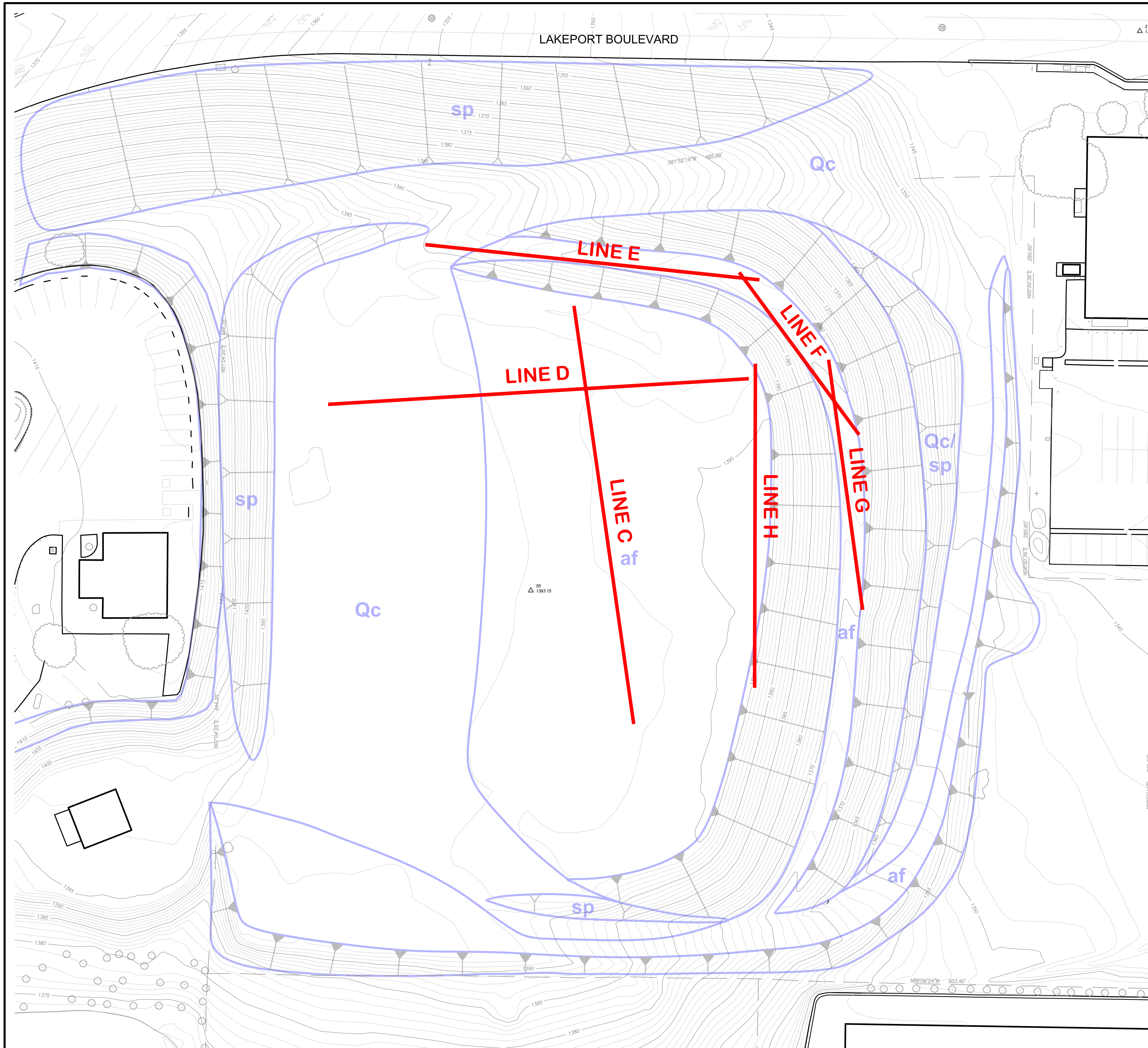
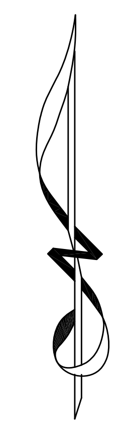


VICINITY MAP



SCALE

0 60 120

(1 inch = 60 feet)

LEGEND

	SEISMIC REFRACTION LINE
af	ARTIFICIAL FILL
Qc	COLLUVIUM/TOPSOIL
sp	SERPENTINE BEDROCK

NOTE: BASE MAP PROVIDED BY LANGAN TREADWELL ROLLO

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SITE LOCATION MAP
SEISMIC REFRACTION SURVEY
LAKEPORT COURTHOUSE

LOCATION: 675 LAKEPORT BOULEVARD, LAKEPORT, CALIFORNIA

CLIENT: LANGAN TREADWELL ROLLO

NORCAL GEOPHYSICAL CONSULTANTS INC.

JOB #: 15-243.110

DATE: FEB. 2015

DRAWN BY: G.RANDALL

APPROVED BY: DTH

PLATE 1

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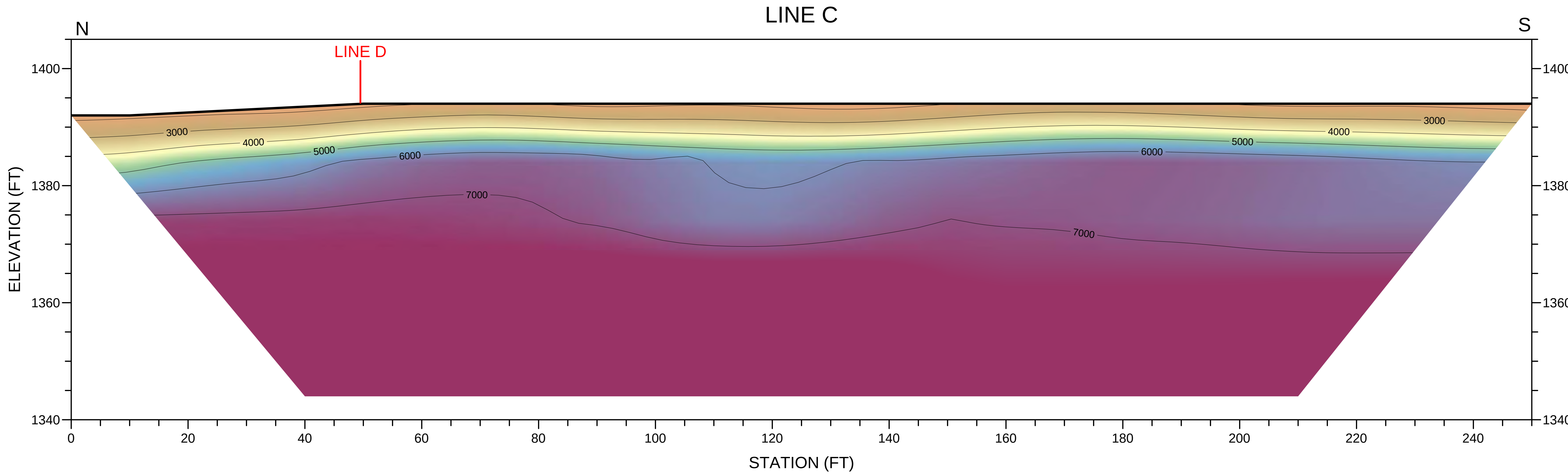

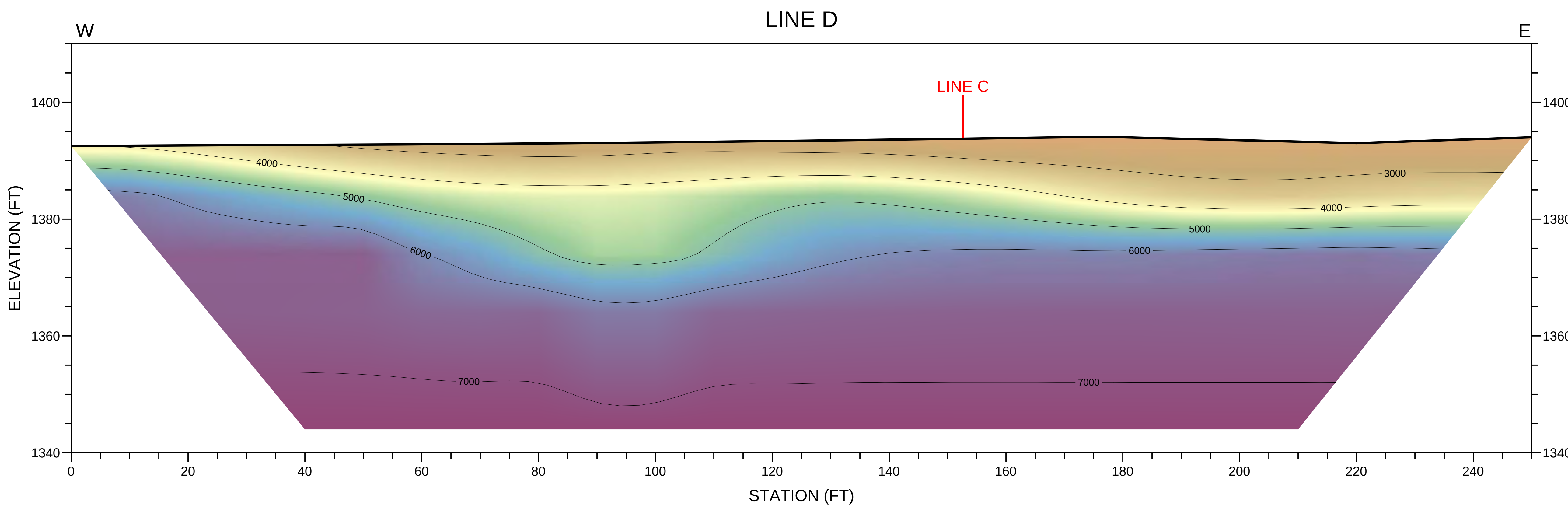

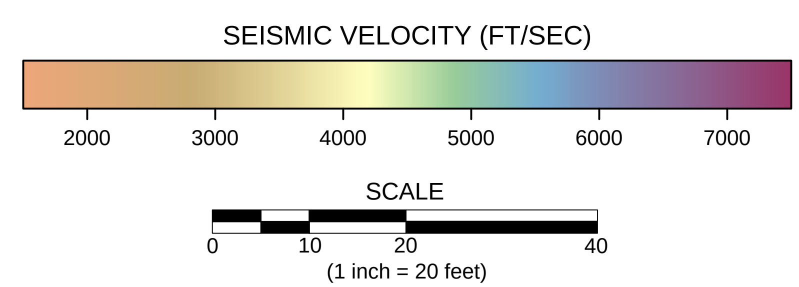

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SEISMIC REFRACTION PROFILES
LINES C & D
LAKEPORT COURTHOUSE

LOCATION: 675 LAKEPORT BOULEVARD, LAKEPORT, CALIFORNIA		
CLIENT: LANGAN TREADWELL ROLLO		PLATE 2
JOB #: 15-243.110	NORCAL GEOPHYSICAL CONSULTANTS INC.	
DATE: FEB. 2015	DRAWN BY: G.RANDALL	APPROVED BY: DTH

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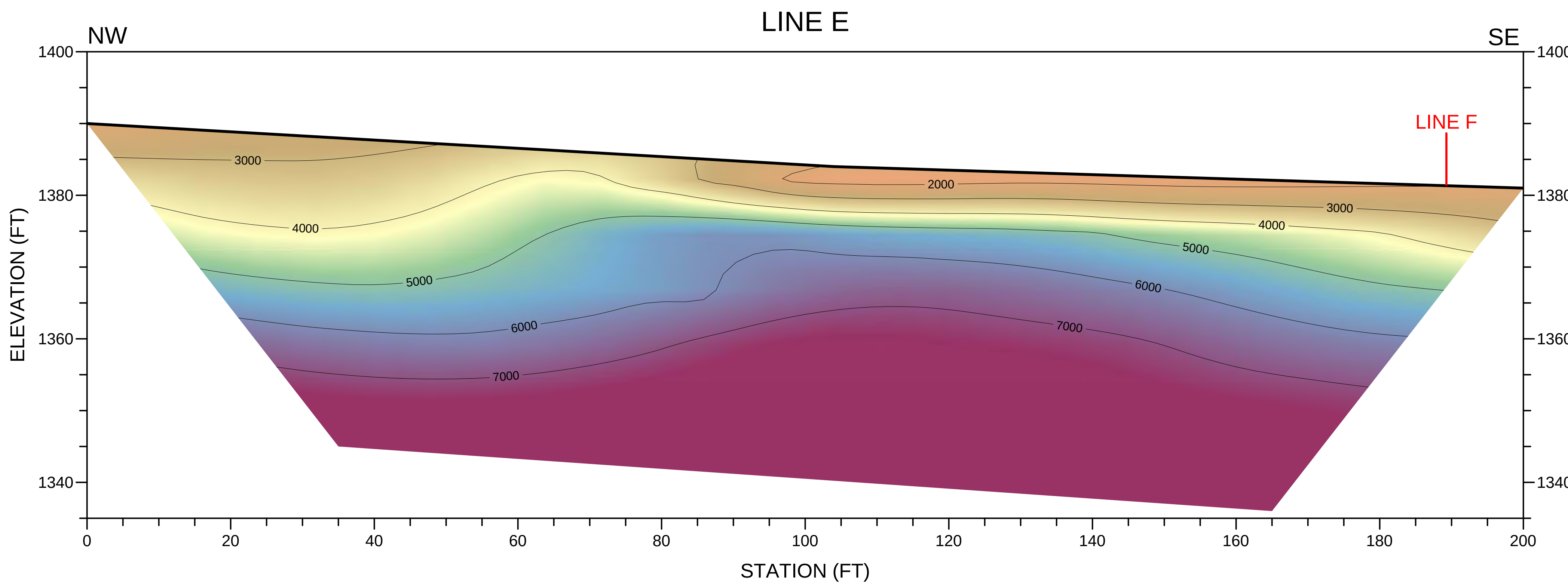

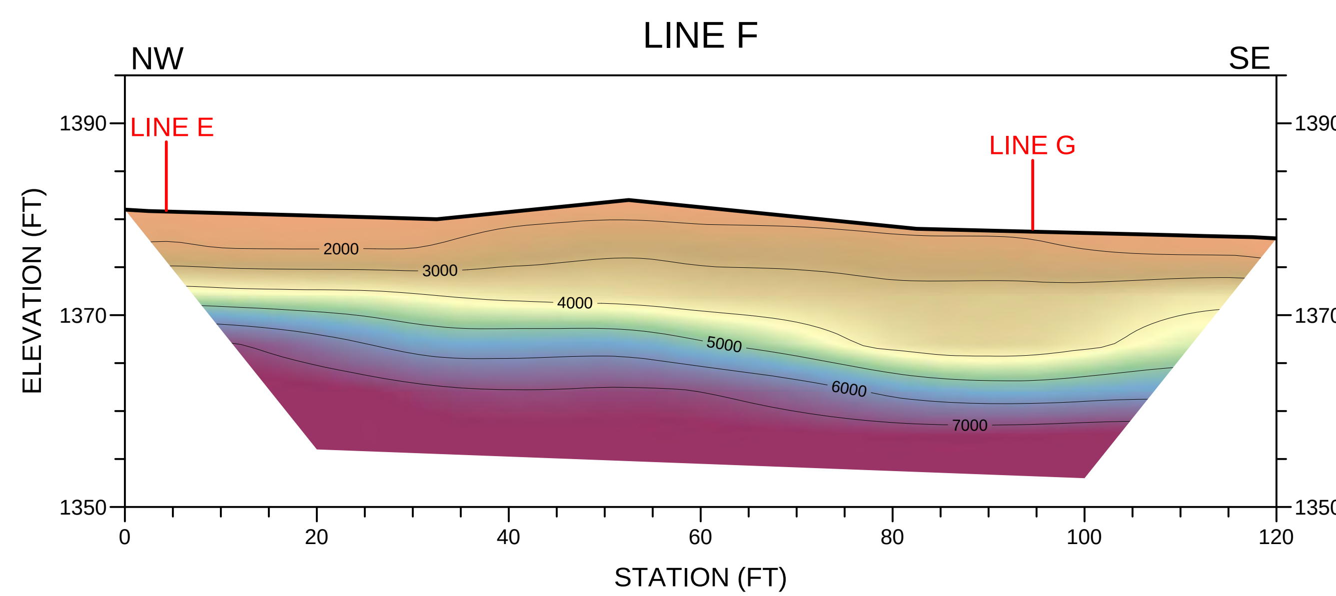

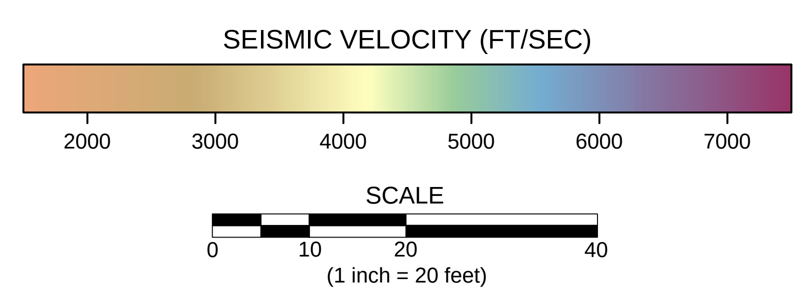

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SEISMIC REFRACTION PROFILES
LINES E & F
LAKEPORT COURTHOUSE

LOCATION: 675 LAKEPORT BOULEVARD, LAKEPORT, CALIFORNIA

CLIENT: LANGAN TREADWELL ROLLO

PLATE 3

JOB #: 15-243.110

NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE: FEB. 2015

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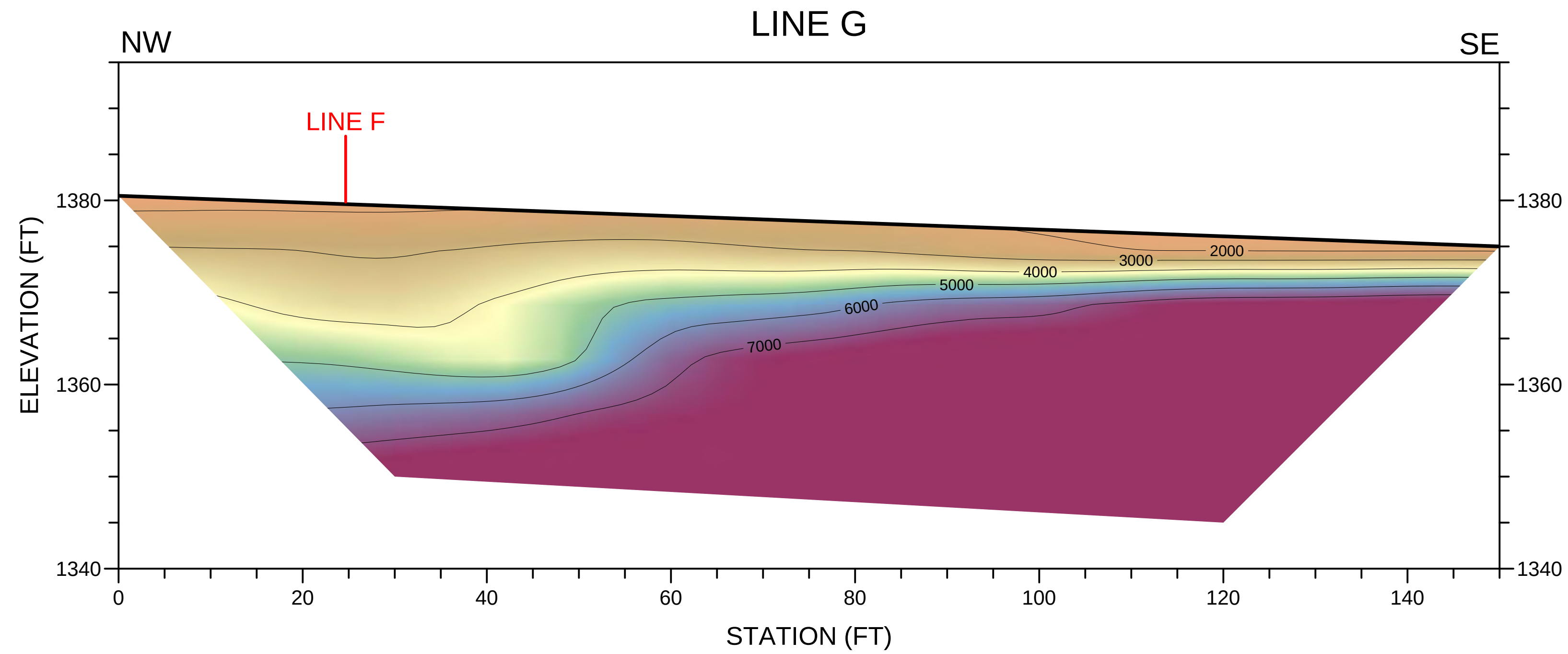

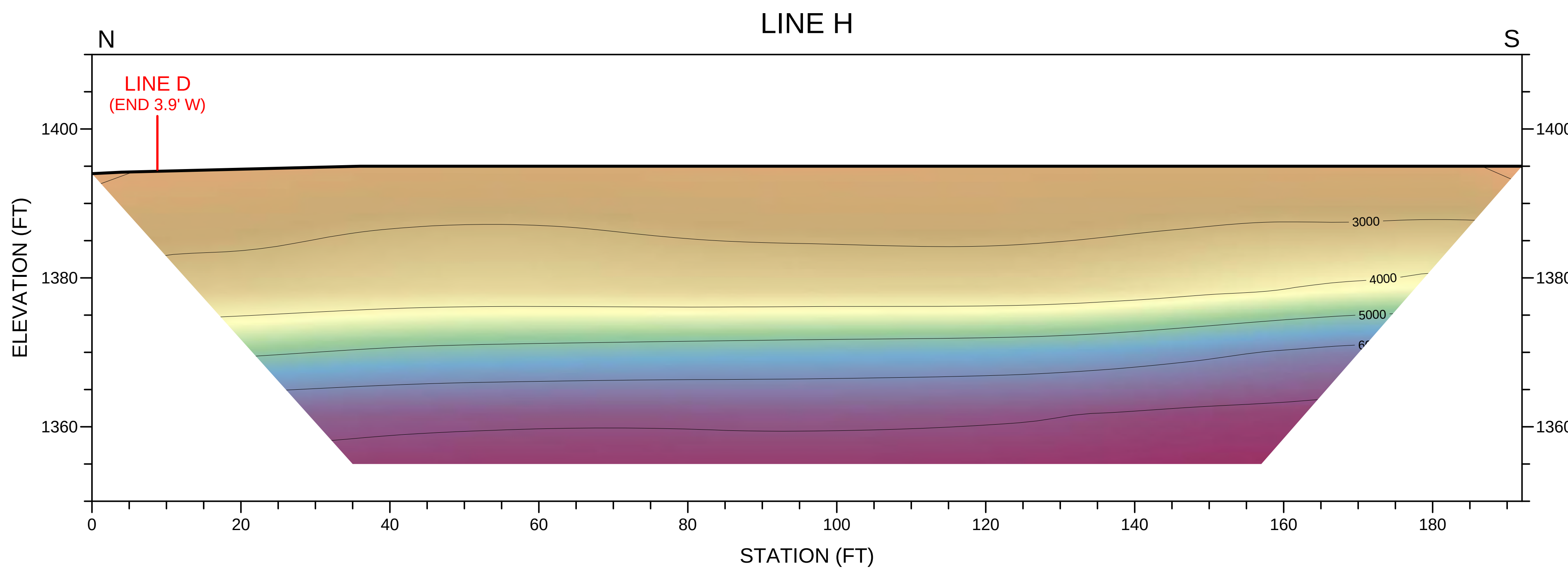

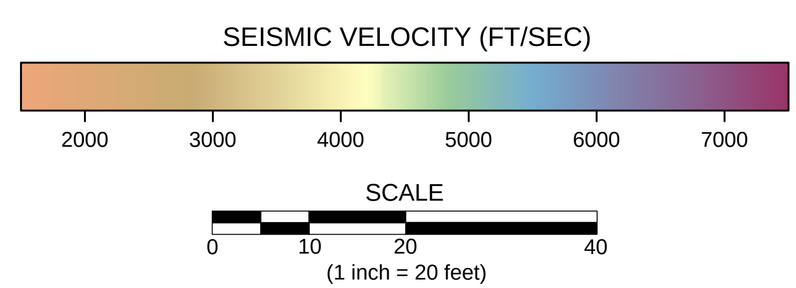

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SEISMIC REFRACTION PROFILES
LINES G & H
LAKEPORT COURTHOUSE

LOCATION: 675 LAKEPORT BOULEVARD, LAKEPORT, CALIFORNIA

CLIENT: LANGAN TREADWELL ROLLO

JOB #: 15-243.110

NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE: FEB. 2015

DRAWN BY: G.RANDALL

APPROVED BY: DTH

PLATE
4

APPENDIX C

GEOTECHNICAL LABORATORY TEST RESULTS

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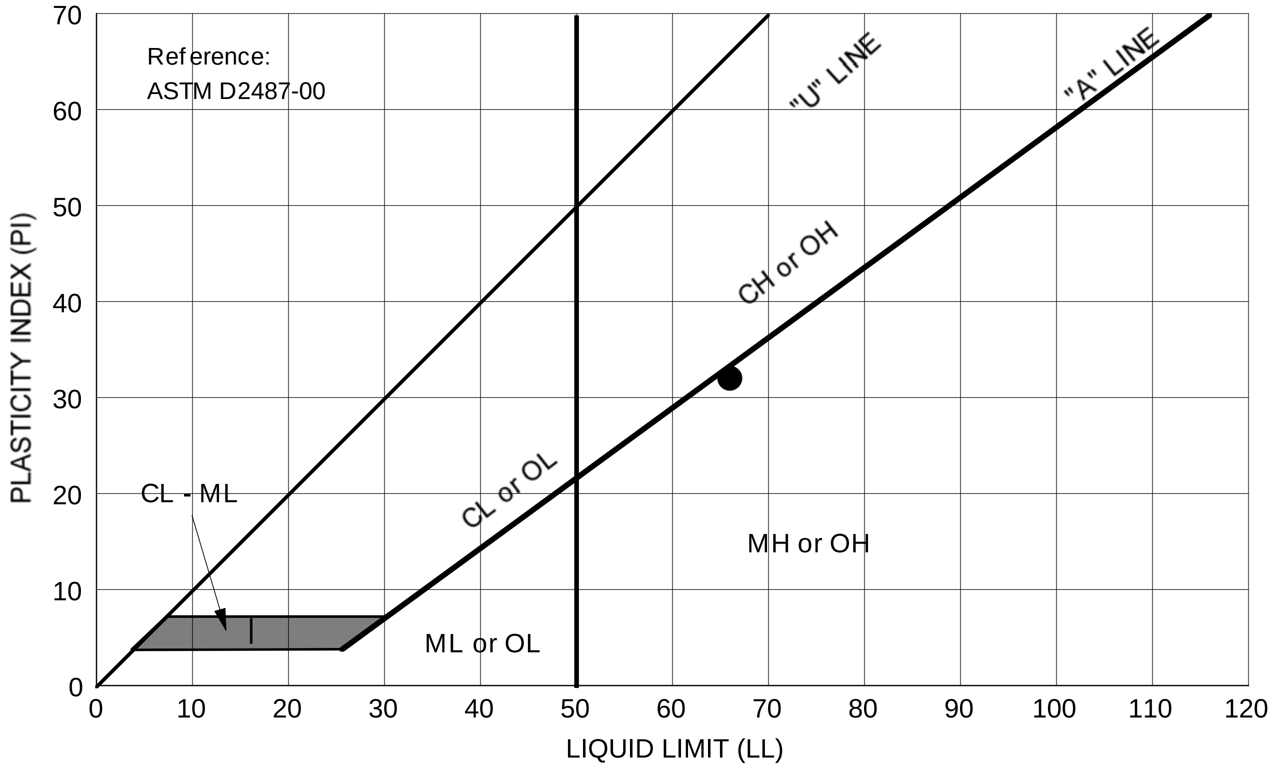

Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-5 at 0 to 5 feet	SANDY SILT with GRAVEL (MH), dark reddish-brown	10.1	66	32	--

LAKEPORT COURTHOUSE **675 LAKEPORT BOULEVARD** Lakeport, California	**PLASTICITY CHART**
LANGAN TREADWELL ROLLO	Date 03/04/15 | Project No. 731563902 | Figure C-1

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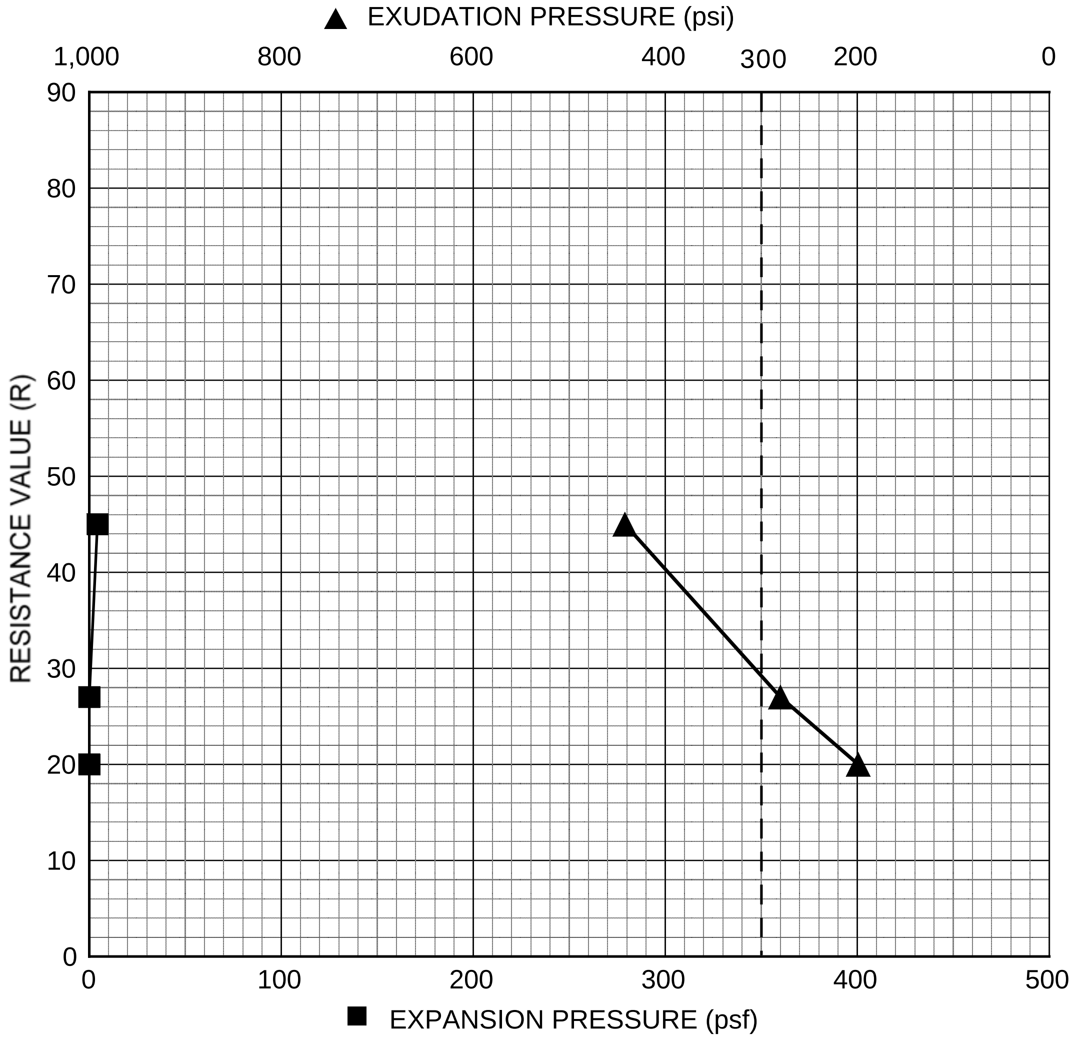


Specimen ID:	A	B	C	D
Water Content (%)	24.0	22.2	23.1	--
Dry Density (pcf)	95.2	98.0	96.2	--
Exudation Pressure (psi)	199	442	280	--
Expansion Pressure (psf)	0	4.3	0	--
Resistance Value (R)	20	45	27	--

Sample Source	Sample Description	Sand Equivalent	Expansion Pressure	R value
B-5 at 0 to 2.5 feet	SANDY SILT with GRAVEL (MH), dark reddish-brown	--	--	28

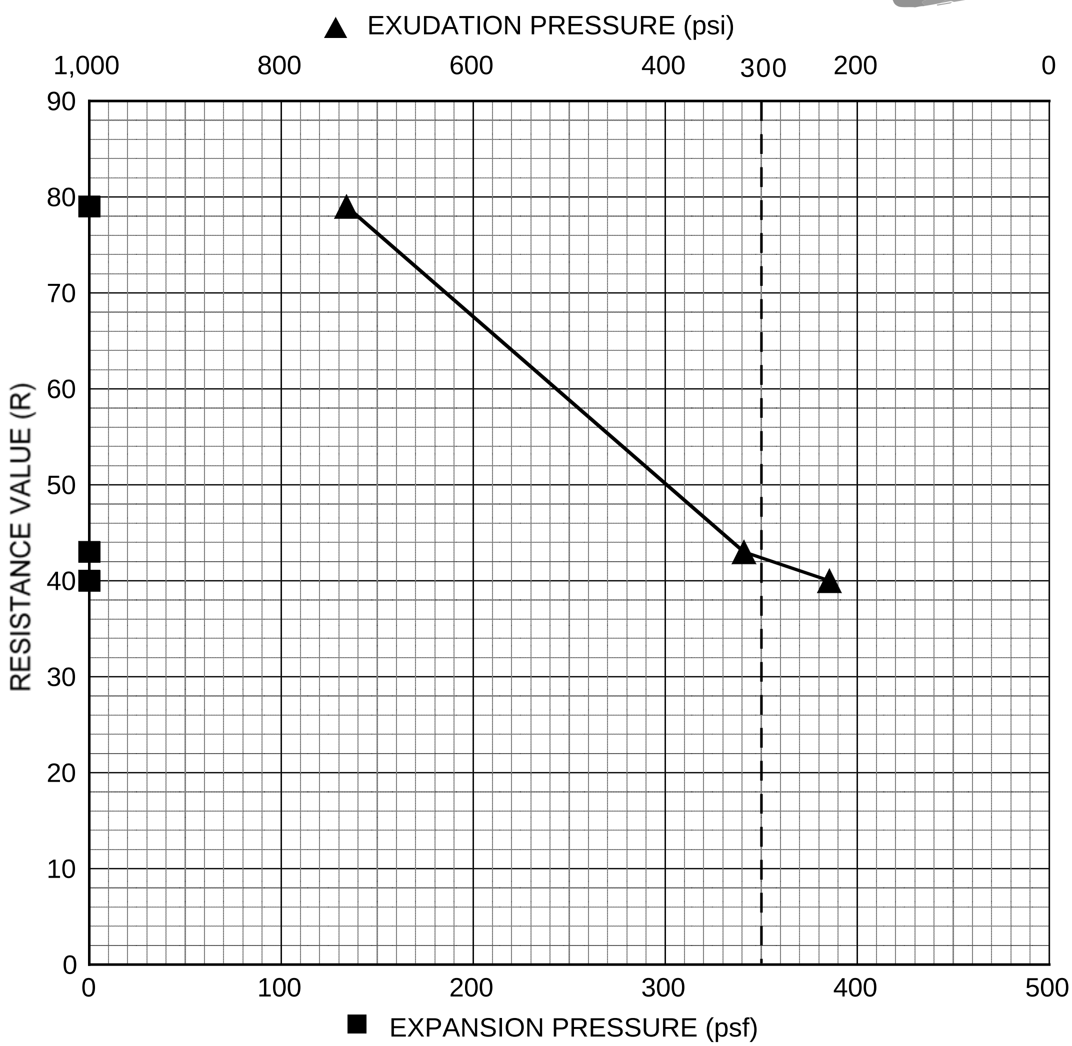
LAKEPORT COURTHOUSE
675 LAKEPORT BOULEVARD
Lakeport, California

LANGAN TREADWELL ROLLO

RESISTANCE VALUE TEST DATA

Date 03/04/15 | Project No. 731563902 | Figure C-2

DRAFT



Specimen ID:	A	B	C	D
Water Content (%)	16.3	18.0	18.5	--
Dry Density (pcf)	109.1	106.3	105.1	--
Exudation Pressure (psi)	732	318	229	--
Expansion Pressure (psf)	0	0	0	--
Resistance Value (R)	79	43	40	--

Sample Source	Sample Description	Sand Equivalent	Expansion Pressure	R value
B-6 at 0 to 2.5 feet	SANDY CLAY with GRAVEL (CL), dark reddish-brown	--	--	43

LAKEPORT COURTHOUSE
675 LAKEPORT BOULEVARD
Lakeport, California

LANGAN TREADWELL ROLLO

RESISTANCE VALUE TEST DATA

Date 03/04/15 | Project No. 731563902 | Figure C-3

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E T S

Environmental Technical Services

975 Transport Way, Suite 2
Petaluma, CA 94954
(707) 778-9605/FAX 778-9612
e-mail: entech@pacbell.net

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COMPANY: Treadwell & Rollo, 501 14th Street, 3rd Floor, Oakland, CA 94612			ANALYST(S)	SUPERVISOR
ATTN: Elena Ayers		DATE of	D. Salinas	D. Jacobson
JOB SITE: Lakeport Courthouse, Lakeport, California	DATE RECEIVED	COMPLETION	S. Santos	LAB DIRECTOR
JOB #: 731563901	12/7/2011	12/15/2011		G.S. Conrad PhD

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SOIL pH -log[H+]	NOMINAL RESISTIVITY ohm-cm	ELECTRICAL CONDUCTIVITY µmhos/cm	SULFATE SO4 ppm	CHLORIDE Cl ppm
04716-1	LPC1/L	B-3-1 @ 3.0'	**7.83**	**3,680**	**[272]**	**9**	**18**
04716-2	LPC2/L	B-3-10 @ 16.0'	**7.10**	**409**	**[2445]**	**111**	**36**
Method	Detection	Limits --->	---	1	0.1	1	1

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SALINITY ECe mmhos/cm	SOLUBLE SULFIDES (S=) ppm	SOLUBLE CYANIDES (CN=) ppm	REDOX mV	PERCENT MOISTURE %
04716-1	LPC1/L	B-3-1 @ 3.0'				**+281.3**	
04716-2	LPC2/L	B-3-10 @ 16.0'				**+296.8**	
Method	Detection	Limits --->	---	0.1	0.1	-400 -> +800	0.1

COMMENTS

Resistivities are well above and below 1,000 ohm-cm, i.e., fair and poor; soil reactions (i.e., pHs) are mildly to very mildly alkaline; sulfates and chlorides are low enough; soils are only mildly reduced. The standard CalTrans times to perforation for these soils are as follows: for LPC1 and 18 ga steel the time is >43 yrs, and for 12 ga it goes up to >93 yrs; and for LPC2 perf times are only <14 yrs for 18 ga, and just ≈30.5 yrs for 12 ga. For gray/ductile/mild steel & cast iron a calculated average pitting rate for LPC1 is at ≈0.045 mm/yr, thus pitting to 2 mm depth is >44 yrs, and to a 4 mm depth is >88 yrs; but for LPC2 the pitting rate is at 0.37 mm/yr putting the 2 mm depth time at ≈5.4 yrs, and the 4 mm depth time is <11 yrs. Chloride levels are very low thus there would be no adverse impact on steel reinforcement; likewise, sulfates are both low enough that there should be no significant adverse impact on concrete, cements, grouts and mortars. Soil redoxes do not appear to be an issue. In principle, the LPC2 soil could benefit from alkaline treatment in that raising its pH to the 7.5-8.5 range would increase the 18 ga time to 17 yrs, but this increase is quite minimal; and the improvement in pitting rate would be completely negligible. Therefore, this would not be a practical approach. On the other hand, metals longevity in these soils can be improved by upgrading (e.g. increased gauge or more resistant steels, etc.). Indeed, often times structural strength considerations will require much thicker steel than used in the presented examples such that perf & pitting times would be well beyond specified life span. On the other hand, cathodic protection along with coating or wrapping steel pipe can be of use where this is not true (requiring very different numbers and/or sizes of sacrificial anodes and little to no impressed current). Other alternatives include increased or specialized engineering fill, and/or use of plastic, fiberglass or concrete pipe, etc. Last, standard concrete mixes should be fine in both of these soils based on these results

\\\\NOTES: Methods are from following sources: extractions by Cal Trans protocols as per Cal Test 417 (SO4), 422 (Cl), and 532/643 (pH & resistivity); &/or by ASTM Vol. 4.08 & ASTM Vol. 11.01 (=EPA Methods of Chemical Analysis, or Standard Methods); pH - ASTM G 51; Spec. Cond. - ASTM D 1125; resistivity - ASTM G 57; redox - Pt probe/ISE; sulfate - extraction Title 22, detection ASTM D 516 (=EPA 375.4); chloride - extraction Title 22, detection ASTM D 512 (=EPA 325.3); sulfides - extraction by Title 22, and detection EPA 376.2 (= SMEWW 4500-S D); cyanides - extraction by Title 22, and detection by ASTM D 4374 (=EPA 335.2).

Figure C-4

APPENDIX D

ANALYTICAL LABORATORY TEST RESULTS

EMSL Analytical, Inc
2235 Polvorosa Ave , Suite 230, San Leandro, CA 94577
Phone: (510) 895-3675 Fax: (510) 895-3680 Email: sanleandrolab@emsl.com

DRAFT

Attn: **Elena Ayers**
Treadwell & Rollo
501 14th Street
3rd Floor
Oakland, Ca 94612

Fax: (510) 874-4507 Phone: (510) 874-4500

Project: **731563901 / Lakeport Courthouse, Lakeport, CA**

Customer ID: TREAD80
Customer PO: 731563901
Received: 12/07/11 9:00 AM
EMSL Order: 091113755

EMSL Proj:
Analysis Date: 12/20/2011

Test Report: PLM Analysis of Bulk Samples for Asbestos via EPA 600/R-93/116 Method with CARB 435 Prep (Milling) Level A for 0.25% Target Analytical Sensitivity

			Non-Asbestos				Asbestos
Sample	**Description**	**Appearance**	**%**	**Fibrous**	**%**	**Non-Fibrous**	**% Type**
1 *091113755-0001*	Test pit TP-1 : Serpentinite rock	Brown Non-Fibrous Homogeneous			100.00%	Non-fibrous (other)	**None Detected**
2 *091113755-0002*	Test pit TP-2 : Fill	Brown Non-Fibrous Homogeneous			100.00%	Non-fibrous (other)	**<0.25% Chrysotile**
3 *091113755-0003*	Test pit TP-3 : Soil	Brown Non-Fibrous Homogeneous			100.00%	Non-fibrous (other)	**None Detected**
4 *091113755-0004*	Test pit TP-3 : Serpentinite rock	Brown Non-Fibrous Homogeneous			100.00%	Non-fibrous (other)	**None Detected**

Initial report from 12/20/2011 16:51:21

Analyst(s)

Rui Cindy Geng (4)

Baojia Ke, Laboratory Manager
or other approved signatory

DISTRIBUTION

6 copies: Mr. Kang Kiang
Mark Cavagnero Associates
1045 Sansome Street, Suite 200
San Francisco, California 94111

QUALITY CONTROL REVIEWER:

DRAFT

Lori A. Simpson
Geotechnical Engineer