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Report
Soil Investigation
Calistoga Hotel
1506/1522 Lincoln Avenue
Calistoga, California

Prepared for
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INTRODUCTION

This report presents the results of our soil investigation for a proposed new hotel and parking garage to be constructed in Calistoga, California. The site is located at 1506 and 1522 Lincoln Avenue.

The proposed hotel will consist of a two- and/or three-story, wood-frame structure with a concrete slab-on-grade floor. A two-level parking garage with a lower-level finish floor elevation about 6 feet below the existing ground surface is being considered behind the hotel at the eastern portion of the site. Retaining walls are anticipated at the perimeter of the below-grade garage. Foundation loads are not known at this time, but are assumed to vary between 4 and 15 kips per linear foot (klf) for continuous perimeter footings and 50 to 500 kips (k) for interior column footings.

The scope of our investigation, as outlined in our proposal dated March 20, 2014, was to explore subsurface conditions and perform engineering analyses to develop conclusions and recommendations concerning:

1. Proximity of the site to active faults.
2. Site preparation and grading.
3. Foundation support and design criteria.
4. Support of concrete slab-on-grade floors.
5. Retaining wall design criteria.
6. Preliminary flexible pavement thicknesses based on our experience with similar projects and soils.

7. Soil engineering drainage.

8. Supplemental soil engineering services.

WORK PERFORMED

We reviewed selected, published, geologic information in our files including:

1. Association of Bay Area Governments website (www.abag.ca.gov), Liquefaction Susceptibility Map, dated 2006.

2. Dwyer, M. D. and others, 1976, *Reconnaissance Photointerpretation Map of Landslides in 24 Selected 7.5-minute Quadrangles in Lake, Napa, Solano and Sonoma Counties, California*, U.S. Geological Survey Open File Map Sheet 14 of 25.

3. Federal Emergency Management Agency (FEMA) website, (www.fema.gov), 2011, Flood Insurance Rate Map.

4. The "Geologic Map of the Santa Rosa Quadrangle, California," by D.L. Wagner and E.J. Bortugno, California Division of Mines and Geology, 1982.

5. "Geologic Map and Map Database of Eastern Sonoma and Western Napa Counties, California," by R.W. Graymer, E. E. Brabb, D. L. Jones, R. S. Nicholson, and R. E. Stamski, Unites States Geological Survey, 2007.

6. The "Geology for Planning in Sonoma County" maps, Special Report 120, California Division of Mines and Geology, 1980.

7. "Historic Ground Failures in Northern California Associated with Earthquakes," Geological Survey Professional Paper 993, T. L. Youd and S. N. Hoose, US Department of the Interior, 1978, Plate 4.

8. The "Preliminary Geologic Map of the Calistoga 7.5' Quadrangle, Napa and Sonoma Counties California," by M.P. Delattre and C.I. Gutierrez, California Geological Survey, 2013.

On April 9, 10 and 11, 2014, we were on-site to observe surface conditions and explore subsurface conditions to the extent of 12 test borings at the approximate locations indicated on Plate 1. The borings were drilled to depths of about 21½ to 31 feet with truck-mounted auger equipment. Our field engineer located the borings, observed the drilling, logged the conditions encountered, and obtained samples for visual classification and laboratory testing. Relatively undisturbed samples were obtained with a 2½-inch (inside-diameter), split-spoon sampler driven with a 140-pound drop hammer. A 2-inch (outside-diameter) Standard Penetration, split-spoon sampler was used at selected depths where granular materials were encountered. The stroke during driving was about 30 inches. The blows required to drive the samplers were recorded and converted to equivalent Standard Penetration Test (SPT) blow counts for correlation with empirical data. Logs of the borings showing soil classifications, sample depths, and converted SPT blow counts are presented on Plates 2 through 13. The soils are classified in accordance with the Unified Soil Classification System, explained on Plate 14.

Selected samples were tested in our laboratory to determine moisture content, dry density, strength and consolidation characteristics and classification (percent free swell, percent passing No. 200 sieve, Atterberg Limits). The test results are shown on the logs with the strength data shown in the manner described by the Key to Test Data on Plate 14. Detailed results of the Atterberg Limits tests are shown on Plate 15. Detailed results of the consolidation tests are presented on Plates 16 through 19.

The boring locations indicated on Plate 1 are approximate and were established by measuring from existing surface features. The locations of the borings should be considered no more accurate than implied by the methods used to establish the data. The borings were backfilled with cement slurry at the completion of the exploration.

SURFACE AND SUBSURFACE CONDITIONS

The project site consists of two properties located on the old Calistoga Gliderport site east of Lincoln Avenue in Calistoga, California. Indian Springs Resort and Spa borders the project site to the north and the east extension of Fair Way forms the border on the south. A large, undeveloped lot borders the site to the east. The site is relatively flat, with an estimated difference in elevation across the properties of about 2 to 3 feet. At the time of our exploration, several existing buildings were located on the project site at the proposed hotel location, including a construction office, laundromat, museum and wood and metal storage sheds. The ground surface at the west portion of the site serves as asphalt-paved and gravel-surfaced driveway and parking for the on-site buildings and neighboring Indian Springs Resort and Spa. At the northeast portion of the site where the garage is proposed to be located, the ground surface consists of a moderate to light growth of grass and weeds.

The borings and laboratory tests indicate that the site is underlain by discontinuous layers of clayey, silty, sandy and gravelly soils to the depths explored. In general, the upper soils in Test Borings 1, 3 through 9, and 12 consist of sandy clay and silty gravel fill materials that extend to depths of about 1 to 3 feet below the existing ground surface. However, in Boring 12, positioned in the southeast corner of the site (see Plate 1), fill materials were encountered to a depth of about 14 feet. Natural soils were encountered at the ground surface in Test Borings 2, 10 and 11 and below the fill in the remaining borings. The natural upper soils consist of soft to medium stiff sandy clays and silts that extend to depths of about 8 to 10 feet below the existing ground surface. These soils exhibit low to moderate strength and were observed to be porous to depths of about 1 and 2 feet in Borings 2 and 10, respectively. The laboratory test results indicate the existing granular fills in Test Borings 3 through 9 and 14 and upper natural sandy soils are low in expansion potential. That is, the soils would tend to undergo low strength and volume changes with seasonal variation in moisture content. Laboratory tests indicate that the upper natural sandy clays and silts exhibit a moderate to high expansion potential. In general, from depths of about 10 feet to 20 feet below the existing ground surface, soft to medium stiff plastic silts were observed with discontinuous lenses of loose silty sands. The silts and sands are judged to be compressible under the anticipated loading conditions. Below the silt and sands, dense clayey gravels and hard sandy silts were observed for the remaining depth of the borings and are considered relatively incompressible under anticipated loading conditions.

Groundwater was observed in all of the test borings during our exploration. Water levels were recorded at depths that generally varied between about 7½ to 9 feet below the existing ground surface. In Test Borings 1 through 3, 5 and 11 at the northeast portion of the site, hot groundwater was noted. We believe that groundwater conditions vary seasonally, and water levels could rise and fall several feet annually. Determination of a precise groundwater location, or the presence of a perched water condition, is beyond the scope of this investigation.

CONCLUSIONS

Based on our field exploration, laboratory tests, engineering analyses and experience with similar subsurface conditions at nearby sites, we conclude that, from a soil engineering standpoint, the site can be used for the proposed construction. The most significant soil engineering factors that must be considered in design and construction are:

1. The presence of existing fills.
2. Weak and expansive upper natural soils.
3. Potentially high groundwater levels with respect to the proposed below-grade construction.
4. Relatively deep silt deposits that exhibit low strength and high compressibility.
5. Locally occurring loose to medium dense sands that could be subject to liquefaction and/or densification during earthquake shaking.

We have not been provided with any documentation to indicate that the existing fill materials were properly placed and compacted under soil engineering observation and testing services. Therefore, we must conclude that the materials could be subject to total and/or differential settlements. Also, our experience indicates that weak, porous soils can similarly undergo considerable strength loss and settlement when loaded in a saturated condition. Where evaporation of moisture is inhibited by footings, slabs or fill, eventual saturation of the underlying soils can occur. Therefore, we conclude that the existing fill and weak, natural upper soils are not suitable for new fill, footing or slab support in their present condition.

Where encountered, it will be necessary to remove (overexcavate) the existing fill and the weak upper soils and replace the materials as properly compacted fill, or extend foundation elements below the fill and weak, compressible soils.

Expansive soils can shrink and swell with seasonal changes in moisture content and can heave and/or distress lightly loaded footings or slabs. Therefore, for footing and slab-on-grade floor support, it will be necessary to verify that expansive soils, if present, have not dried and cracked. Also, where expansive soils are encountered, it would be necessary to moisture condition the soils to cause pre-swelling and then cover the materials with a moisture protecting and confining blanket of approved on-site or imported nonexpansive fill.

Because of potentially high groundwater levels, dewatering measures could be needed to facilitate the proposed below-grade construction. As discussed earlier, groundwater was measured in our test borings at depths of approximately 7½ to 9 feet below the existing ground surface. However, we anticipate that in winter months the groundwater could possibly rise to as shallow as about 4 feet below the existing ground surface.

Finish floor elevation of the garage is currently proposed at about 6 feet below the existing ground surface. We conclude that design measures should be considered to mitigate potential detrimental effects from high groundwater. Such measures could include installation of retaining wall backdrain systems, subsurface drainage facilities beneath the concrete floor slab, installation of sumps, waterproofing of building elements where migration of moisture would be detrimental, and/or designing of structural elements to resist hydrostatic pressures.

A major consideration for foundation support of the proposed structures is the presence of underlying soft silt deposits that exhibit low strength and high compressibility. Significant settlements of overlying improvements could occur from consolidation of the silts under new loading conditions. The amount and rate of settlement are influenced by several factors, including past loading history, thickness and weight of planned new fills, new building loads and variations in the thickness and compressibility of the silt deposits. Generally, maximum settlements will occur in areas of heaviest structural loads overlying the thickest silt deposits. The laboratory consolidation tests indicates that the silt deposits appear to be overconsolidated. That is, the current vertical effective stress in the field is lower than effective stress levels that have occurred historically. The consolidation tests also indicates that the settlement of the soft silts under new loading conditions would occur relatively rapidly, such that about half of the consolidation settlement would be anticipated to occur during construction.

We have calculated allowable bearing capacities for the soils within the upper 28 to 30 feet of the existing ground surface and computed anticipated consolidation settlements of foundations under various assumed loading conditions. The settlements summarized below were calculated for continuous perimeter and interior column footings bottomed on natural soils at finished floor elevations at the existing ground surface (hotel) and 6 feet below the existing grade (parking garage).

Table 1: Calculated Settlements of Hotel Footings at Existing Grade

Continuous Wall Footings		Column Pad Footing	
Assumed Design Load*	Calculated Total Settlement	Assumed Design Load*	Calculated Total Settlement
4 klf	2 inches	150 k	3 inches
10 klf	3½ inches	300 k	3½ inches
15 klf	4 inches	500 k	4 inches

*Assumed loading condition for a two-story structure

Table 2: Calculated Settlements of Garage Footings at 6 feet Below Grade

Continuous Wall Footings		Column Pad Footing	
Assumed Design Load*	Calculated Total Settlement	Assumed Design Load*	Calculated Total Settlement
4 klf	2½ inches	50 k	1 inch
10 klf	3 inches	100 k	1½ inches
15 klf	3½ inches	200 k	2 inches

*Assumed loading condition for a one-story parking structure

We have also considered the use of a mat slab foundation and have performed a similar settlement analysis assuming uniform dead loads of 600 and 1,000 pounds per square foot (psf).

For a mat slab foundation bottomed on natural soils at the existing ground surface and at a depth of 6 feet below the existing grade, we calculated the following consolidation settlements.

Table 3: Estimated Settlements for Mat Slab Foundation at Existing Grade (Hotel)

Location in Structure	Calculated Settlement, Uniform Dead Load = 600 psf	Calculated Settlement, Uniform Dead Load = 1,000 psf
Center	3 inches	4½ inches
Edge of Slab at Center	1½ inches	2½ inches
Corner	1 inch	1½ inches

Table 4: Estimated Settlements for Mat Slab Foundation at 6 feet Below Grade (Parking)

Location in Structure	Calculated Settlement, Uniform Dead Load = 600 psf	Calculated Settlement, Uniform Dead Load = 1,000
Center	2½ inches	3½ inches
Edge of Slab at Center	1 inch	2 inches
Corner	1/2-inch	1 inch

The magnitude of total and differential settlements could be reduced by underlying the structures with a layer of properly compacted fill. However, because of the presence of groundwater near planned pad grade of the garage, we understand that providing a layer of compacted fill is only being considered at the hotel. For a mat slab foundation underlain by 3 feet of compacted fill, we have calculated the following settlements for assumed loading conditions:

Table 5: Estimated Settlements for Mat Slab Foundation at Existing Grade Underlain by 3 feet of Compacted Fill

Location in Structure	Calculated Settlement, Uniform Dead Load = 600 psf	Calculated Settlement, Uniform Dead Load = 1,000 psf
Center	2 inches	3½ inches
Edge of Slab at Center	1 inches	2 inches
Corner	1/2- inch	1 inch

Liquefaction, a loss in shear strength, and densification, a reduction in void ratio, are phenomena associated with loose sandy soil deposits subjected to ground shaking. Whether such phenomena will actually occur depends on complex factors such as earthquake intensity, duration of earthquake shaking and underlying soil conditions. We have analyzed the soil data from our borings at the site in accordance with the "*Simplified Procedure for Evaluating Soil Liquefaction Potential*" by H. B. Seed and I. M. Idriss, published in the Journal of the Soil Mechanics and Foundation Division of the American Society of Civil Engineers (ASCE), dated September 1971, and subsequent papers by Seed and others, published in 1985. Our analysis indicates that the natural, loose sandy soils in Test Borings 2, 4, 7, and 8 below the groundwater level would be classified as likely to liquefy.

We have calculated the magnitude of settlements of the soils in the test borings that were indicated to be likely to liquefy based on the procedures detailed in "Soil Liquefaction During Earthquakes" by Idriss and R.W. Boulanger, published by the Earthquake Engineering Research Institute in 2008. Using a 6.8 moment magnitude (M_w) earthquake event with a peak ground acceleration of 0.515g and the SPT blowcount data from our test borings, we calculated the approximate vertical displacement of the sand layers designated as likely to liquefy. Our analysis indicates that about 1 to 3½ inches of differential settlement from liquefaction should be anticipated during the design earthquake event. The following table summarizes the results of our analysis.

Table 6: Calculated Liquefaction-Induced Settlements During Earthquake Event

Test Boring	Approximate Depth Below Existing Ground of Liquefiable Layer	Estimated Differential Settlement
2	7½ to 9 feet	1 inch
4	13 to 14½ feet	1 inch
7	13½ to 20 feet	3½ inches
8	13 to 14 feet	1/4-inch
8	15 to 20½ feet	2 inches

If liquefaction or densification were to occur, we believe that there is a high risk of potential damage to the structure from differential settlement. Therefore, we judge that the risk of liquefaction and densification should be considered in the design process. Subsequent sections of this report are intended to satisfactorily reduce the risk of distress to the structure should liquefaction or densification occur, or to improve the on-site soils such that the risk of liquefaction could be considered low.

We have considered several options for reducing the risk of distress to the structure from both consolidation and liquefaction settlements, including site grading techniques, grid or mat slab foundations, ground improvement and deep foundations. Because of the difficulty associated with site grading below the groundwater table, the possibility of caving soils and the likelihood of a highly unstable excavation bottom unable to support construction equipment, we judge that the most appropriate methods for mitigating distress to the structures from consolidation and liquefaction settlements would be to: 1) support the structures on driven pile or drilled pier foundations; or, 2) underlie a mat slab foundation or structural floor slab with a grid pattern of Rammed Aggregate Piers or stone columns. Because of the presence of relatively high groundwater and granular soils subject to caving, installation of drilled piers would likely be difficult. Accordingly, the balance of this report is oriented toward the use of driven piles or shallow foundations with ground modification to mitigate settlement/liquefaction. We could provide specific recommendations for other ground modification techniques and/or alternative foundation systems, if desired.

SEISMIC DESIGN PARAMETERS

The geologic maps reviewed did not indicate the presence of active faults at the site, and the properties are not located within a presently designated Alquist-Priolo Earthquake Fault Zone. Therefore, we judge that there is little risk of fault-related ground rupture at the site during earthquakes. In a seismically active region such as Northern California, there is always some possibility for future faulting at any site. However, historical occurrences of surface faulting have generally closely followed the trace of the more recently active faults. The closest faults generally considered active are the Maacama fault zone located approximately 5½ miles to the west, the Rodgers Creek fault zone located approximately 9 miles to the southwest, the Collayomi fault zone located approximately 13½ miles to the northwest, the West Napa fault zone located approximately 15 miles to the southeast, the Concord-Green Valley fault zone approximately 18 miles to the southeast and the San Andreas Fault located approximately 21 miles to the southwest.

Strong ground shaking will occur during earthquakes. The intensity at the site will depend on the distance to the earthquake epicenter, depth and magnitude of the shock, and the response characteristics of the materials beneath the site. Because of the proximity of active faults in the region and the potential for strong ground shaking, it will be necessary to design and construct the project in strict accordance with current standards for earthquake-resistant construction.

We have determined seismic ground motion values in accordance with procedures outlined in Section 1613 of the 2013 California Building Code (CBC). Mapped acceleration parameters (S_S and S_1) were obtained by inputting approximate site coordinates (latitude and longitude) into earthquake ground motion software developed by the United States Geological Survey. Based on our review of available geologic maps and our knowledge of the subsurface conditions, we judge that the site can be classified as Site Class D, as described in Chapter 20 of ASCE 7-10. Using corresponding values of site coefficients for Site Class D and procedures outlined in the CBC, the mapped acceleration parameters were adjusted to yield the design spectral response acceleration parameters S_{DS} and S_{D1}. The following earthquake design data summarize the results of the procedures outlined above.

Table 7: 2013 CBC Ground Motion Parameters

Site Class: D	
Mapped Spectral Response Accelerations	
S_s	1.500 g
S_1	0.600 g
Design Spectral Response Accelerations	
S_{DS}	1.000 g
S_{D1}	0.600 g

RECOMMENDATIONS

Site Grading

The site should be cleared of designated buildings and foundations, debris and dense growths of grass and vegetation. Designated trees should be removed and the roots excavated.

The resultant voids from tree removal should be backfilled as subsequently described. The ground surface then should be stripped of the upper soils containing root growth and organic matter, where necessary. We anticipate that the depth of stripping needed in these areas will average about 3 inches. The strippings should be removed from the site, stockpiled for reuse as topsoil, or mixed with at least five parts of soil and used as fill at least 10 feet away from structure, walkway and paved areas.

Wells, septic tanks or other underground obstructions encountered during grading should be removed or abandoned in-place. The resultant voids should be backfilled with soil or granular material that is properly compacted, as subsequently discussed, or capped with concrete. The method of removal/abandonment and void backfilling should be determined by the appropriate governing agency or the soil engineer.

After clearing and stripping, excavations can be performed as necessary. Any existing fills or weak upper natural soils encountered after excavation to planned pad grade level within building and paved roadway and parking areas should be removed (overexcavated) for their full depth. Overexcavations in such areas should extend at least 5 feet beyond the building perimeter, 3 feet beyond the edge of building foundations, 3 feet beyond adjacent exterior concrete slabs that abut the building and 3 feet beyond planned paved areas. The depth of the overexcavation should be adjusted, as needed, so as to provide space for a blanket of at least 30 inches of approved nonexpansive fill over upper expansive soils, where encountered.

Areas to receive fill should be scarified to a depth of at least 6 inches, moisture conditioned to near optimum (at least 4 percentage points above optimum for on-site expansive clayey soils and as necessary to close any shrinkage cracks for their full depth) and compacted to at least 90 percent relative compaction.[1] Approved on-site or imported fill of low expansion potential then should be spread in 8-inch-thick loose lifts, moisture conditioned and similarly compacted.

We anticipate that, with the exception of organic matter and rocks or hard fragments larger than 6 inches in diameter, excavated gravel fill materials will be suitable for reuse as compacted fill. Expansive silty and clayey soils should not be reused as fill within the upper portions of planned pad grade elevation where concrete slabs are proposed. Imported fill materials, if used, should be low in expansion potential, free of organic matter, rocks or hard fragments larger than 4 inches in diameter, and have a Plasticity Index of 15 or less. The material proposed for use as imported fill of low expansion potential should be tested and approved by the soil engineer prior to importation to the site.

For grading performed during mid-summer or early fall prior to winter rains, we judge that the materials may exhibit sufficient strength such that the ground surface exposed after excavations may be relatively stable under the weight of the grading equipment. For grading in the spring or early summer, we believe that materials would likely be saturated, such that the soils could tend to break down and become unstable under the weight of the earth-moving equipment, resulting in more than normal effort to satisfactorily excavate and/or compact the

1 Relative compaction refers to the in-place dry density of fill expressed as a percentage of maximum dry density of the same material determined in accordance with the ASTM D 1557 laboratory compaction test procedure. Optimum moisture content refers to the moisture content at maximum dry density.

materials. The need for overexcavation to remove unstable soils, the use of track-mounted equipment for excavation, imported granular working pads, geotextile fabrics, dewatering systems, lime-treating techniques, or other measures could be required to complete the building pad grading. Accordingly, we suggest that the possible need for such measures be accounted for in the contract documents.

Finished slopes should be trimmed to expose firm material and should be no steeper than two horizontal to one vertical (2:1). Slopes over 3 feet high should be planted with fast-growing, deep-rooted ground cover to help reduce erosion.

Driven Pile Foundations

The structures can be supported on driven pile foundations gaining support from skin friction, or end bearing in the dense gravel layer. We believe that either 12- or 14-inch-square, prestressed concrete or steel H-beam piles could be used. If driven piles are used for foundation support, no ground improvement measures to upgrade the on-site compressible or liquefiable soils would be necessary, provided the floor slabs are designed to structurally span between foundation elements.

We are recommending relatively conservative pile design criteria because of variation in subsurface conditions and earthquake hazards. We are recommending that a condition of no support be assumed for the first 20 feet below the existing ground surface. For planning purposes, piles should extend through the compressible and potentially liquefiable soils and bottom at least 5 feet into the underlying dense gravels to a minimum depth of 30 feet regardless of their structural load. Piles can be designed using an average allowable skin friction value of 1,200 psf commencing at a depth of 20 feet below the existing ground surface. To resist uplift forces, pull-out capacity of piles can be assumed to be one-half their downward capacity. Resistance to lateral loads can be obtained from a passive earth pressure of 300 pcf (triangular distribution) acting on the face of piles and pile caps. Passive pressure should be neglected in the upper 12 inches unless confined by pavement or slab.

Piles should be spaced so that the clear distance between piles is at least 3 pile diameters (or widths), and all piles should be interconnected in at least one direction with pile caps, grade or tie beams. The vertical capacity and lateral resistance will be reduced for piles arranged in groups. If pile groups are used, we can provide revised recommendations for the vertical capacity and lateral resistance based on the planned foundation configuration.

Piles should be driven with a hammer having a minimum rated energy of 42,000 foot-pounds per blow for 12-inch-square and 57,000 foot-pounds per blow for 14-inch-square concrete piles. During driving operations, the pile and hammer should be held firmly in proper alignment. Piles closest to the center of the building area should be driven first. All piles heaved by subsequent driving of adjacent piles should be redriven.

Initially, pile lengths should be determined for skin friction support. However, refusal blow counts may be reached when driving piles into the dense gravels. Therefore, prior to ordering the majority of the foundation piles, the contractor should drive 8 to 10, or more, indicator piles located at the corners and near the center of the planned structure. The indicator piles should be at least 10 feet longer than the design lengths and can be incorporated into the structure provided they are not damaged.

The driving records for installation of the indicator piles should be evaluated to determine whether the remaining foundation piles (production piles) will gain support from skin friction or end bearing in the dense gravels. The pile lengths then should be revised, if needed, prior to the contractor ordering the production piles. Acceptance criteria should also be determined prior to installation of the production piles.

Refusal blow counts will depend upon the design capacity of the piles, the pile size, the rated energy of the pile driving hammer and the materials in which the piles are driven. A preliminary refusal blow count value of about 10 blows per inch for the last few inches of the pile driving can be used. However, the actual refusal blow count criteria should be determined during and after the driving of the indicator piles.

Ground vibration caused by pile driving will be noticeable. Existing structures and utilities in the vicinity of the site should be checked before commencing pile driving and then during and after driving so that any damage can be monitored and repaired.

We recommend that the bid documents require a lump-sum price for the number and length of piles shown on the foundation plans. This price should include all costs in connection with the work, including mobilization, driving, and cutting off, based on the lengths indicated on the drawings. In addition to the lump-sum price, the contractor should quote unit prices as follows:

1. Unit price per pile for additional piles, based on the design length.

2. Unit cost per foot of additional pile length in excess of the design length; the unit price should be no more than two-thirds greater than the base unit cost.

3. Unit cost per foot for reduction in the length of piles. The unit price for reduction in the lengths should be not less than 50 percent of the base unit price. In the event of the elimination of piles, the reduction should be this unit cost times the stated length.

Payment should be made for length of pile remaining in place from the tip to the cut-off elevation shown on the plans. No payment should be made for any rejected pile or for any portion of piles remaining above the cut-off elevation.

We can provide additional consultation to assist in the preparation of the pile foundation specifications. Appropriate concrete and reinforcing steel specifications should be provided by the structural engineer.

Mat Slab Foundations

Provided ground modifications methods are implemented to satisfactorily reduce the risk of settlement and liquefaction, mat slabs can be used for foundation support of the proposed structure. We should be consulted to provide specific recommendations for design of mat slabs when ground modification method has been selected and more detailed information is available for analysis.

Mat slabs should be at least 12 inches thick with a thickened edge at the perimeter for stiffening. The thickened edge should be at least 12 inches wide and extend to at least 8 inches below the planned bottom of the slab. For design, an allowable bearing value of 600 psf can be used. A subgrade modulus (k) of 50 psi/in could be used for on-site soils.

Where subjected to heavy wheel or storage loads, the slabs should be thickened and reinforced to accommodate the increased loading. Actual slab thickness and reinforcing used should be determined by the structural design engineer or architect based on anticipated use and performance. Prior to placing the reinforcing or slab rock, the subgrade soils should be thoroughly moisture conditioned and be smooth, firm and uniform. Slab-on-grade subgrade should not be allowed to dry prior to concrete placement.

Slabs should be underlain by a capillary moisture break and cushion layer consisting of at least 4 inches of free-draining, crushed rock or gravel (drainrock) at least 1/4-inch and no larger than 3/4-inch in size. Crushed rock should be used where the slabs would be subjected to wheel loads such as forklifts or trucks.

Moisture vapor will condense on the underside of slabs. Where migration of moisture vapor through slabs is detrimental, a 10-mil moisture vapor retarder conforming to ASTM E1745 Class C should be provided between the supporting base material and the slabs. Two inches of moist, clean sand could be placed on top of the membrane to aid in curing and to help provide puncture protection. However, the actual use of sand should be determined by the architect or design engineer. The use of a less permeable and stronger membrane should be considered if sand is not to be placed for puncture protection, or where the flooring manufacturer requires a vapor barrier. Concrete design and curing specifications should recognize the potential adverse affects associated with placement of concrete directly on the membrane.

Retaining Walls

Retaining walls that are free to rotate and support a level (and up to 3:1) backslope should be designed to resist an active equivalent fluid pressure of 45 pcf acting in a triangular pressure distribution. Where the backfill slope is steeper than 3:l, the pressure should be increased to 60 pcf. If the wall is constrained at the top and cannot tilt, the design pressures for level and sloping backfill should be increased to 60 and 75 pcf, respectively. Where retaining wall backfill is subject to vehicular traffic, the walls should be designed to resist an added surcharge pressure equivalent to 1½ feet of additional backfill. Where an imaginary 1½:1 line is projected down from adjacent foundations intersects a retaining wall, the portion of retaining wall below the intersection should be designed for an additional horizontal surcharge load of 100 psf.

Because of the potential for groundwater to rise to within about 4 feet of the ground surface, we recommend the portion of the retaining walls below a depth of 4 feet be designed to resist hydrostatic and soil pressures. Cantilever walls should be designed to resist an active equivalent fluid pressures discussed above to a depth of 4 feet. The pressure should be increased to 90 pcf below 4 feet to account for hydrostatic pressure. If the retaining walls are constrained at the top and cannot tilt, the additional hydrostatic pressure should be increased to 110 pcf.

If not designed to resist hydrostatic pressures, retaining walls should be fully backdrained. The backdrains should consist of 4-inch-diameter, perforated rigid plastic pipe sloped to drain to outlets by gravity and clean free-draining, crushed rock or gravel (drainrock). The crushed rock or gravel should extend to within 12 inches of the surface. The drainrock should consist of Class 2 Permeable Material in accordance with the latest edition of the

Caltrans Standard Specifications. As an alternative, any clean, washed, durable rock product containing less than 1 percent soil fines by weight could be used if the rock is covered and separated from the soil bank by a nonwoven, geotextile fabric (such as Mirafi 140N or equivalent) weighing at least 4 ounces per square yard. The upper 12 inches should be backfilled with compacted soil to inhibit surface water infiltration unless capped with a concrete slab. The ground surface behind retaining walls should be sloped to drain. Where migration of moisture through walls would be detrimental, the walls should be waterproofed.

As outlined in the 2013 CBC, it may be necessary to design retaining walls to resist additional lateral soil loads imposed during seismic shaking. Accordingly, based on the Mononobe-Okabe Method, we have computed the following dynamic component of total thrust induced on the wall for varying backslope inclinations.

Table 8: Dynamic Lateral Soil Loads on Retaining Walls at Various Backfill Inclinations

Backslope Inclination (β)	Dynamic Component* of Total Thrust (lbs/ft)
$0 \leq \beta \leq 8:1$	$7\ H^2$
$8:1 < \beta \leq 4:1$	$11\ H^2$
$4:1 < \beta \leq 3:1$	$18\ H^2$

* The dynamic component of total thrust should be applied as a line load at a height of 0.6H above the base of the retaining wall; where H is height of the retaining wall.

Flexible Pavements

For planning purposes, based on our experience with similar projects and soils, we recommend the following minimum pavement sections for driveways and parking areas:

Table 9: Recommended Minimum Pavement Sections for Driveway and Parking Areas

Material	Parking Areas	Driveway Areas
Class II Aggregate Base	6 inches	8 inches
Asphalt Concrete	2½ inches	2½ inches

Such pavements would be suitable for auto and light pickup truck traffic. Where heavier loads are anticipated, the pavement thickness should be increased to at least 3 inches of asphalt and about 12 to 16 inches of aggregate base, depending on anticipated loading. We can provide specific recommendations, if desired. The flexible pavement materials and methods used should conform to the quality requirements of the State of California, Caltrans Standard Specifications, current edition, and the requirements of the County of Napa.

Prior to subgrade preparation, underground utilities in the paved areas should be installed and properly backfilled. Pavement subgrades should be prepared by scarifying to a depth of at least 6 inches, moisture conditioning to slightly above optimum and compacting to at least 95 percent relative compaction. Finished subgrade should be smooth, firm, uniform and nonyielding. Approved aggregate base materials should be spread in layers, moisture conditioned and compacted to at least 95 percent relative compaction. The aggregate base surface should also be firm and nonyielding.

Geotechnical Drainage

Ponding water will cause softening of the site soils and could be detrimental to foundations. It is important that the ground surface adjacent to structures be sloped to drain away from foundations. Roofs should be provided with gutters, and the downspouts should be connected to nonperforated, rigid plastic pipelines with water-tight joints that discharge into planned drainage facilities.

We recommend that good, positive surface drainage away from and around the structures be provided. A gradient of at least 1/4-inch per foot extending at least 4 feet out and careful attention to fine (finish) grading around the structures should be provided. Loose or poorly compacted materials should not be allowed adjacent to foundations.

To provide an outlet for water that could accumulate in the underslab rock and reduce the risk of future moisture migration up through concrete floor slabs, a system of perforated plastic pipes could be embedded in the grade below the underslab rock. The underslab subdrain system should be designed so as to drain each bay created by interior and/or perimeter foundations. The underslab subdrain system should be connected to a nonperforated outlet pipe that extends through or beneath the perimeter foundation to a suitable discharge point. A typical cross-section of our recommended underslab subdrain is shown on the attached Plate 20. We should provide additional consultation concerning the actual configuration and location of the underslab subdrains during final design, if the use of underslab subdrains is desired. Roof downspouts and surface drains must be maintained entirely separate from underslab subdrains.

Supplemental Soil Engineering Services

We should review the final grading and foundation plans for conformance with the intent of our recommendations. During site grading operations, we should provide intermittent soil engineering observation and testing to determine the conditions encountered and modify our recommendations, if warranted. Field and laboratory tests should be performed to ascertain that the specified moisture content and degree of compaction are being attained.

We should observe footing excavations to verify that the conditions are as anticipated and to modify our recommendations, if warranted. Concrete placement and reinforcing should be checked as stipulated on the project plans or as required by the Building Department. It is our understanding that approval from the Building Department must be obtained prior to the placement of concrete in foundation elements.

The soil engineer should observe driving of indicator and production foundation piles, if used, to verify that the suitable bearing materials are penetrated and to modify our recommendations, if needed.

LIMITATIONS

We have performed the investigation and prepared this report in accordance with generally accepted standards of the soil engineering profession. No warranty, either express or implied, is given. This scope of work is limited to evaluating the physical properties of earth materials considered typical of geotechnical engineering practice and does not include other concerns such as soil chemistry, corrosion potential, mold, and soil and/or groundwater contamination.

Subsurface conditions are complex and may differ from those indicated by surface features or encountered at test boring locations. Therefore, variations in subsurface conditions not indicated on the logs could be encountered.

If the project is revised or if conditions different from those described in this report are encountered during construction, we should be notified immediately so that we can take timely action to modify our recommendations, if warranted.

Supplemental services as recommended herein are performed on an as-requested basis. Such services are in addition to this soil investigation, and are charged for on an hourly basis in accordance with our Standard Schedule of Charges. We accept no responsibility for items we are not notified to check, or for use and/or interpretation by others of the information contained herein.

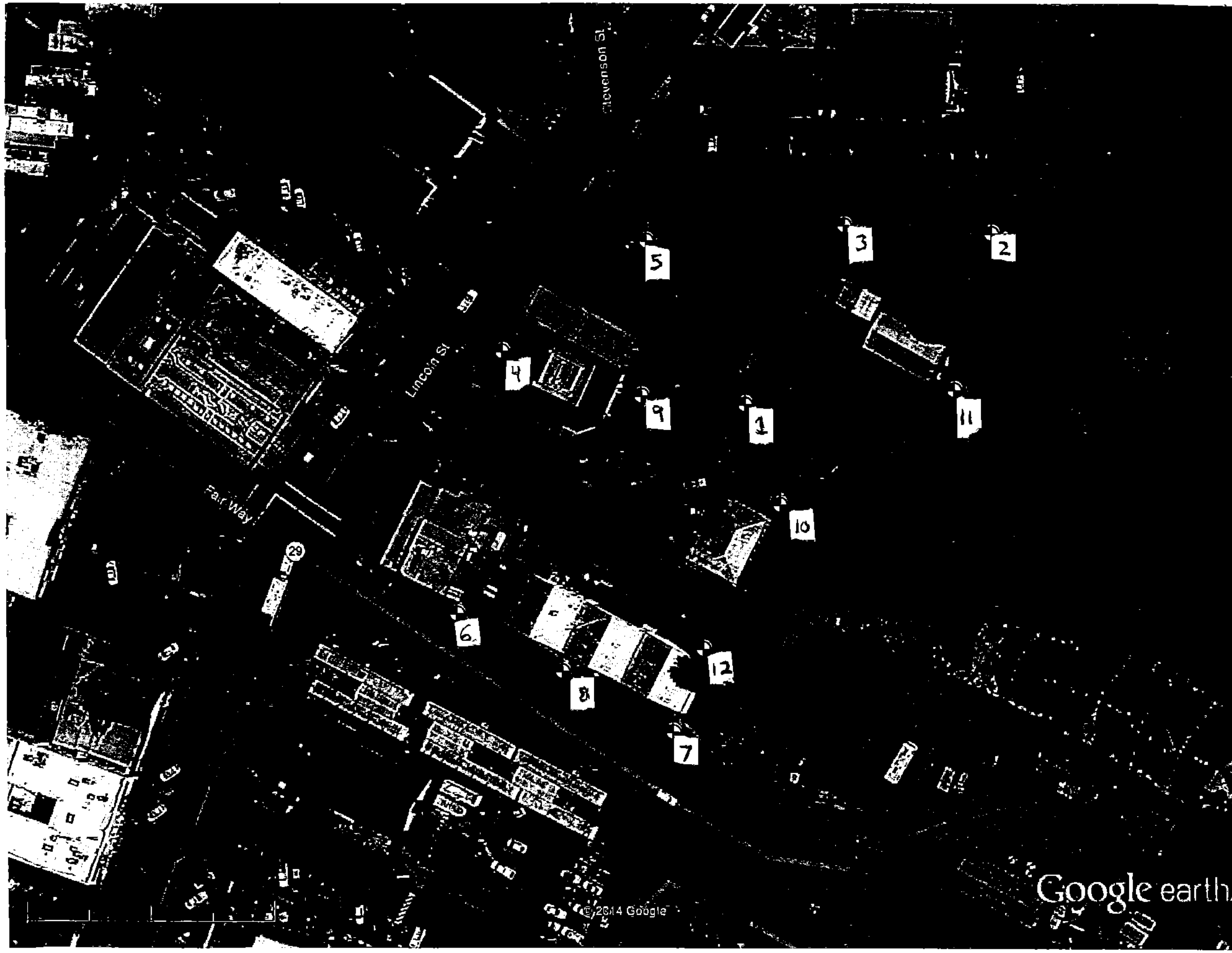
Site conditions and standards of practice change. Therefore, we should be notified to update this report if construction is not performed within 24 months.

LIST OF PLATES

DISTRIBUTION

Copies submitted:	2	Danny Merchant 1522 Lincoln Avenue Calistoga, CA 94515
	3	Paul Coates Construction, Inc. P.O. Box 1006 Calistoga, CA 94515

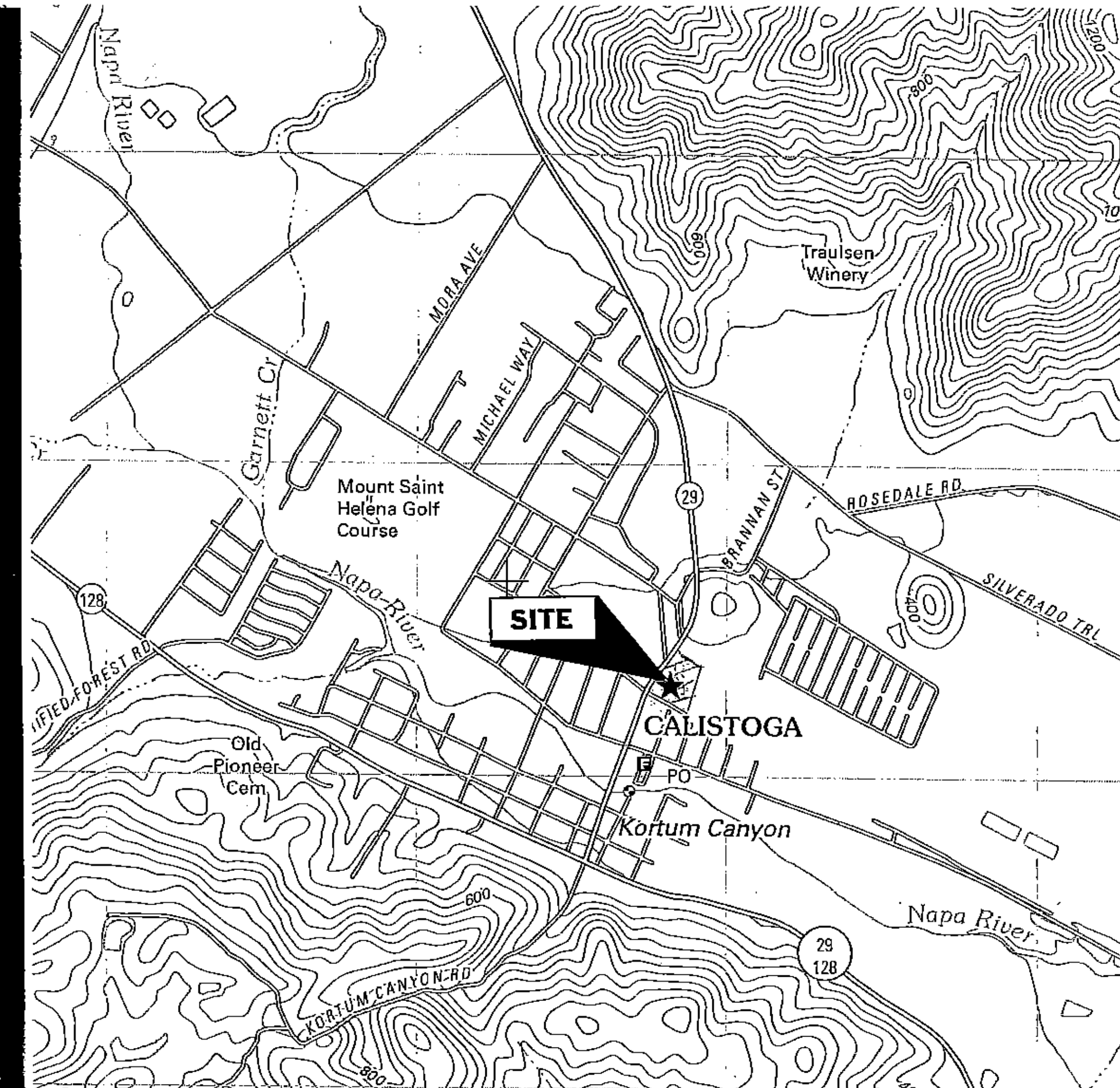
JM/JKR:nay/ra/Job No. 644.1.1



feet 500

N

Test Boring Location Plan



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KILOMETERS 1 0.5 0 1 2

METERS 1000 500 0 1000 2000

MILES 1 0.5 0 1

FEET 1000 0 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000

CONTOUR INTERVAL 40 FEET
NORTH AMERICAN VERTICAL DATUM OF 1988

N

Site Vicinity Map

⌖ Approximate Test Boring Location

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 644.1.1 Date: 06/24/14 Appr: *[signature]*	TEST BORING LOCATION PLAN AND SITE VICINITY MAP CALISTOGA HOTEL 1506/1522 LINCOLN STREET CALISTOGA, CALIFORNIA	PLATE 1

groundwater first encountered at time of drilling

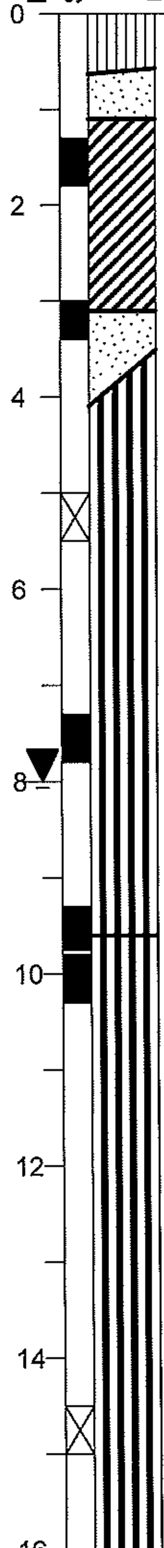
groundwater at time of backfilling

LOG OF BORING 1

Equipment 3.25" HOLLOW STEM

Elevation ____________ Date 4-9-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				0	DARK BROWN SANDY SILT (ML), medium stiff, moist, with occasional roots (Fill)
					YELLOW-BROWN SILTY SAND (SP), dense, dry, with abundant angular gravel (Fill)
LL = 75 PL = 27 PI = 26 Percent Free Swell = 75	8	27.6	94	2	DARK GRAY SANDY CLAY (CH), medium stiff, moist, plastic
Percent Free Swell = 50	5				GRAY-BROWN SILTY SAND (SP), loose, wet
				4	DARK GRAY-BROWN SANDY SILT (MH), medium stiff, dry to moist, plastic
	7				
				6	
	8	44.5	75	8	becomes stiff
UC(P) = 2300		51.6	69		
UC(P) = 2800 UC(P) = 1900	5	53.7	68	10	BLUE-GRAY COARSE SANDY SILT (MH), medium stiff, saturated, plastic
				12	
				14	
Percent Passing No. 200 Sieve = 60.9	7				
				16	



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Job No: 644.1.1

Date: 6-11-14

Appr: JM

LOG OF BORING 1

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **2a**

*Converted to Standard Penetration Blow Counts

∇ groundwater first encountered at time of drilling

$\blacktriangledown$ groundwater at time of backfilling

LOG OF BORING 1

Equipment 3.25" HOLLOW STEM

Elevation ______ Date 4-9-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				16	
				18	
UC = 680		76.9	54		DARK GRAY SANDY SILT (MH), stiff, moist
UC(P) = 2100 UC(P) = 4000	7	59.8	63	20	
				22	DARK BLUE-GRAY CLAYEY GRAVEL (GC), very dense, moist, with sand
				24	
	50+	17.1			
				26	
				28	
Percent Passing No. 200 Sieve = 13.7	50+				
				30	Rhyolite of Sonoma Volcanics Group, closely fractured, very hard, very strong
	50+				

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Date: 6-11-14

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LOG OF BORING 1

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **2b**

*Converted to Standard Penetration Blow Counts

▽ groundwater first encountered at time of drilling

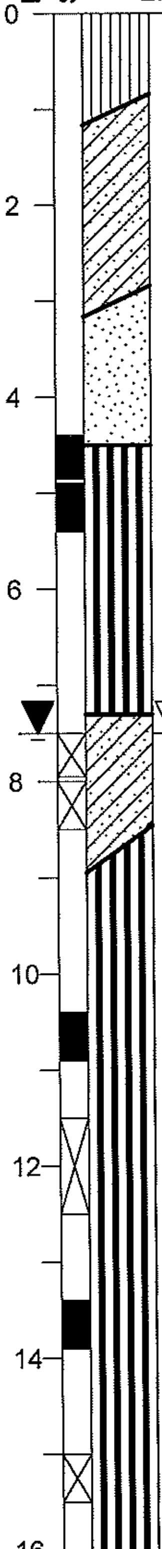
▼ groundwater at time of backfilling

LOG OF BORING 2

Equipment: 3.25" HOLLOW STEM

Elevation: ______ Date: 4-9-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				0	DARK BROWN SANDY SILT (ML), medium stiff, saturated, with roots (topsoil)
				1	GRAY-BROWN VERY CLAYEY SAND (SC), loose, saturated with gravel
				3	GRAY-BROWN CLAYEY SAND (SP), loose, wet
				4.5	DARK GRAY-BROWN SANDY SILT (MH), soft, saturated, plastic
UC(P) = 1300	2	45.5	72	5	
				7.3	MOTTLED ORANGE AND GRAY-BROWN CLAYEY SAND (SC), loose, saturated
		27.5		7.7	
	2	41.1		8.2	
				9	MOTTLED BLUE AND DARK GRAY SANDY SILT (MH), soft, saturated
UC(P) = 1700	3	56.5	66	10.5	
LL = 35 PL = 32 PI = 29 Percent Free Swell = 35	8			11.7	
				12.2	becomes blue-gray in color
	8	53.3	67	13.5	
	5	62.6		15	



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LOG OF BORING 2

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **3a**

*Converted to Standard Penetration Blow Counts

groundwater first encountered at time of drilling

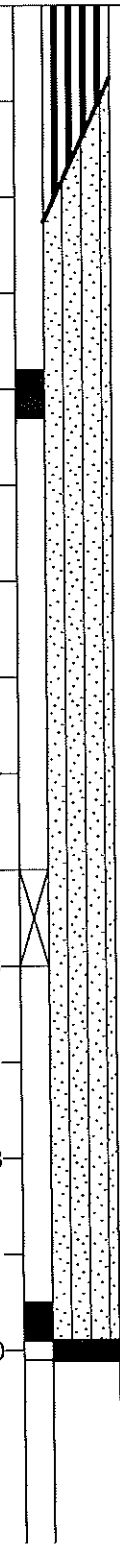
groundwater at time of backfilling

LOG OF BORING 2

Equipment: 3.25" HOLLOW STEM

Elevation: ______ Date: 4-9-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				16	
				18	MOTTLED BROWN AND BLUE-GRAY SILTY SAND (SM), very dense, moist, with subrounded gravel
	50+	22.7	101	20	cobble in tip of sampler
				22	
				24	
	50+			26	with some angular gravels
				28	
Percent Passing No. 200 Sieve = 10.0	50+			30	becomes cemented sands and gravels



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Job No: 644.1.1
Date: 6-11-14
Appr: JM

LOG OF BORING 2
CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **3b**

*Converted to Standard Penetration Blow Counts

▽ groundwater first encountered at time of drilling

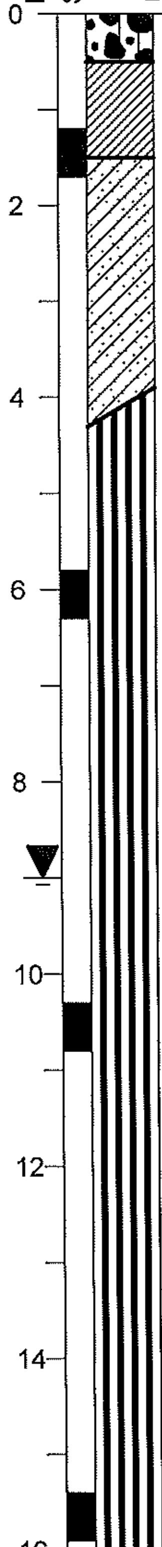
▼ groundwater at time of backfilling

LOG OF BORING 3

Equipment 3.25" HOLLOW STEM

Elevation ______ Date 4-9-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)
	9			~1.5
UC(P) = 1900	5	51.7	69	~6
TxUU = 510 (1500)	6	57.8	65	~10.5
	4	53.4	69	~15.7



DARK BROWN SILTY GRAVEL (GP-GM), dense, moist (roadbase)

MOTTLED YELLOW-BROWN AND DARK BROWN SANDY CLAY (CL), stiff, moist

GRAY-BROWN VERY CLAYEY SAND (SC), loose, wet, with gravel

MOTTLED ORANGE-BROWN AND DARK GRAY-BROWN SANDY SILT (MH), medium stiff, wet, plastic

becomes orange-brown

becomes blue-gray in color

free water on sampler

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 644.1.1 Date: 6-11-14 Appr: Jm	LOG OF BORING 3 CALISTOGA HOTEL CALISTOGA, CALIFORNIA	PLATE **4a**

*Converted to Standard Penetration Blow Counts

groundwater first encountered at time of drilling

groundwater at time of backfilling

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Sample	LOG OF BORING 3 Equipment 3.25" HOLLOW STEM Elevation ____ Date 4-9-14
				16		
				18		
				20		
	49			22	X	BLUE-GRAY CLAYEY GRAVEL (GC), very dense, moist

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 644.1.1 Date: 6-11-14 Appr: Jm	LOG OF BORING 3 CALISTOGA HOTEL CALISTOGA, CALIFORNIA	PLATE 4b

*Converted to Standard Penetration Blow Counts

▽ groundwater first encountered at time of drilling

▼ groundwater at time of backfilling

LOG OF BORING 4

Equipment 3.25" HOLLOW STEM

Elevation ______ Date 4-10-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				0	2 inches of asphalt concrete
					DARK BROWN SILTY GRAVEL (GP-GM), dense, moist (roadbase)
					DARK BROWN SANDY CLAY (CL), stiff, saturated, with occasional gravel
	12			2	LIGHT GRAY-BROWN CLAYEY SAND (SC), medium, moist
				4	
				6	DARK GRAY-BROWN SANDY SILT (MH), stiff, moist, plastic
CONSOL	9	47.0	75	8	
				10	
				12	becomes blue-gray in color
	8	42.2	77		BLUE-GRAY SILTY FINE SAND (SM), loose, saturated
Percent Passing No. 200 Sieve = 28.6				14	
Percent Passing No. 200 Sieve = 52.7	5				BLUE-GRAY VERY SAND SILT (MH), medium stiff, saturated, soft plastic
				16	

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CONSULTING GEOTECHNICAL ENGINEERS

Job No: 644.1.1

Date: 6-11-14

Appr: Jm

LOG OF BORING 4

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **5a**

*Converted to Standard Penetration Blow Counts

▽ groundwater first encountered at time of drilling

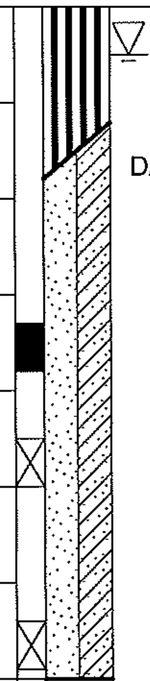
▼ groundwater at time of backfilling

LOG OF BORING 4

Equipment: 3.25" HOLLOW STEM

Elevation: ______ Date: 4-10-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				16	▽
				18	DARK GRAY CLAYEY COARSE SAND (SP-SC), medium dense, saturated, with rounded gravel
Percent Passing No. 200 Sieve = 9.3	14	24.3	99	20	
Percent Passing No. 200 Sieve = 8.3	19				
				22	
	20				



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Date: 6-11-14

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LOG OF BORING 4

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **5b**

*Converted to Standard Penetration Blow Counts

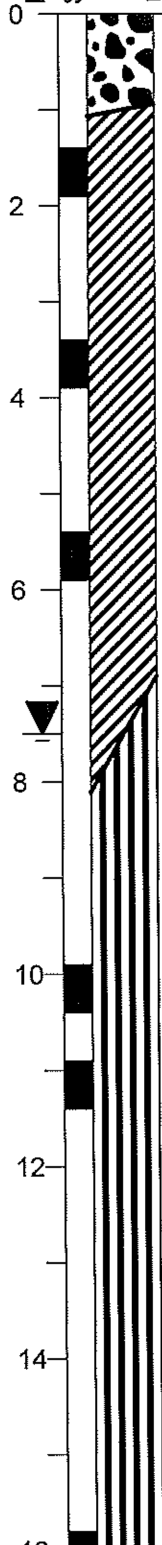
▽ groundwater first encountered at time of drilling

▼ groundwater at time of backfilling

LOG OF BORING 5

Equipment 3.25" HOLLOW STEM

Elevation ______ Date 4-10-14



Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				0	DARK BROWN SANDY ROUNDED FINE GRAVEL (GP), dense, dry (Fill)
					DARK BROWN SANDY CLAY (CH), stiff, wet, plastic, with gravel and debris
Percent Free Swell = 90	9				
				2	
					becomes dark olive-brown, medium stiff, with coarse sand
UC(P) = 1700	6	31.9	88		
				4	
UC(P) = 2600 UC(P) = 1500	5	36.8	83		becomes brown in color
				6	
					BLUE-GRAY SANDY SILT (MH), stiff, wet, plastic
				8	
UC(P) = 2500	7	47.2	73	10	
					becomes blue in color with less sand
	9	48.1	73		
				12	
				14	
				16	

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Job No: 644.1.1

Date: 6-11-14

Appr: Jm

LOG OF BORING 5

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **6a**

*Converted to Standard Penetration Blow Counts

▽ groundwater first encountered at time of drilling

▼ groundwater at time of backfilling

LOG OF BORING 5

Equipment 3.25" HOLLOW STEM

Elevation ______ Date 4-10-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
CONSOL	7			16	becomes dark gray in color
				18	
				20	becomes very stiff
		48.5	72		
	15	37.2	82		
				22	▽
					BLUE SILTY SUBANGULAR GRAVEL (GM), very dense, saturated, with sand
	45			24	
					sample discarded (disturbed)

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Job No: 644.1.1

Date: 6-11-14

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LOG OF BORING 5

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **6b**

*Converted to Standard Penetration Blow Counts

groundwater first encountered at time of drilling

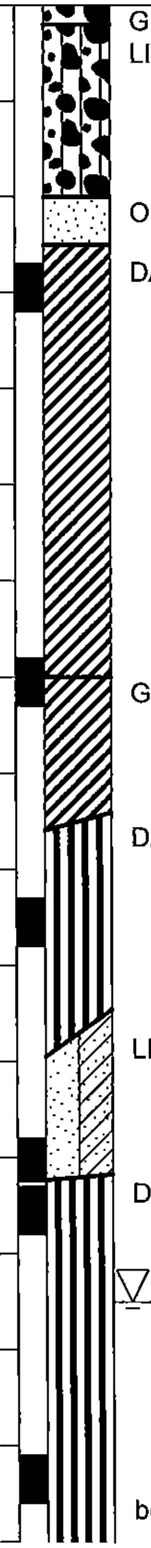
groundwater at time of backfilling

LOG OF BORING 6

Equipment 3.25" HOLLOW STEM

Elevation ______ Date 4-10-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				0	GRAY-BROWN GRAVEL (GP), loose, dry
					LIGHT BROWN SILTY GRAVEL (GM), dense, dry (Fill)
				2	ORANGE-BROWN SILTY FINE SAND (SP), loose, saturated
Percent Free Swell = 70	4	29.7	91		DARK GRAY SANDY CLAY (CH), medium stiff, saturated, plastic, with debris in upper 6 inches
				4	
				6	
UC(P) = 1700 UC(P) = 2600	6	36.7	83		GRAY-BROWN VERY SANDY CLAY (CH), stiff, moist
				8	DARK BLUE-GRAY SILT (MH), medium stiff, wet, plastic
TxUU = 740 (2000)	6	43.6	76		
				10	LENS OF CLAYEY SAND (SP-SC), medium loose, saturated
		46.5	73	12	
	7	43.8	76		DARK GRAY SANDY SILT (MH), medium stiff, moist, plastic
				14	
UC(P) = 2700	11	51.4	71		
				16	becomes stiff



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Job No: 644.1.1

Date: 6-11-14

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LOG OF BORING 6

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **7a**

*Converted to Standard Penetration Blow Counts

▽ groundwater first encountered at time of drilling

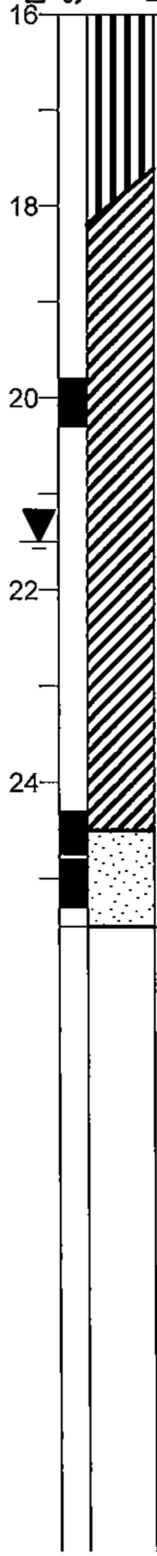
▼ groundwater at time of backfilling

LOG OF BORING 6

Equipment 3.25" HOLLOW STEM

Elevation ______ Date 4-10-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				16	
				18	DARK GRAY SANDY CLAY (CH), stiff, saturated, plastic
	13	54.1	68	20	
				22	
				24	
Percent Passing No. 200 Sieve = 16.4	26	29.8	92		BLUE-GRAY SILTY SAND (SM), medium dense, moist, cemented



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CONSULTING GEOTECHNICAL ENGINEERS

Job No: 644.1.1

Date: 6-11-14

Appr: Jm

LOG OF BORING 6

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **7b**

*Converted to Standard Penetration Blow Counts

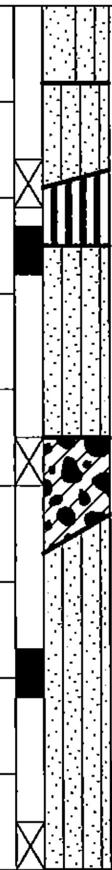
▽ groundwater first encountered at time of drilling

▼ groundwater at time of backfilling

LOG OF BORING 7

Equipment: 3.25" HOLLOW STEM

Elevation: ______ Date: 4-10-14



Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				16	
Percent Passing No. 200 Sieve = 20.3					BLUE-GRAY SILTY FINE SAND (SM), loose, saturated
	6				
				18	LENS OF BLUE-GRAY SANDY SILT (MH),
Percent Passing No. 200 Sieve = 14.0	8	44.0	77		BLUE-GRAY SILTY COARSE SAND (SM), loose, saturated
				20	
	16	21.2			BLUE SILTY GRAVEL (GM), medium dense, saturated
					MOTTLED BLUE AND DARK BROWN SILTY SAND (SM), medium dense, moist, with subangular gravel
				22	
UC(P) = 4500+	21	40.9	80		
				24	
	34	20.2			becomes dense, cemented

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Job No: 644.1.1

Date: 6-11-14

Appr: Jm

LOG OF BORING 7

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **8b**

*Converted to Standard Penetration Blow Counts

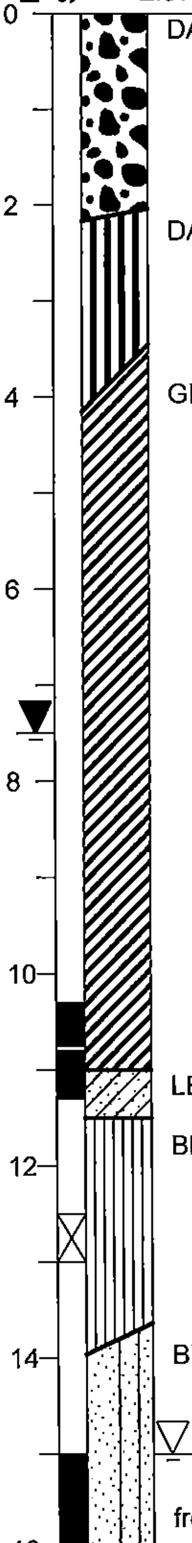
▽ groundwater first encountered at time of drilling

▼ groundwater at time of backfilling

LOG OF BORING 7

Equipment 3.25" HOLLOW STEM

Elevation ______ Date 4-10-14



Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				0	DARK BROWN SANDY GRAVEL (GP), dense, moist, with cobbles and debris (Fill)
				2	DARK BROWN SANDY SILT (MH), stiff, wet, plastic
				4	GRAY-BROWN SANDY CLAY (CH), logged from auger cuttings (object lodged in sampler)
				6	
				8	
				10	
UC(P) = 2700		49.0	71		
UC(P) = 2000	7	44.5	74		LENS OF BROWN VERY CLAYEY SAND (SC), loose, saturated, slightly plastic
					BLUE-GRAY SANDY SILT (MH), medium stiff, moist
				12	
	7				
				14	BLUE-GRAY SILTY SAND (SP-SM), loose, saturated
Percent Passing No. 200 Sieve = 6.9	4				free water on sampler
				16	

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Job No: 644.1.1

Date: 6-11-14

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LOG OF BORING 7

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **8a**

*Converted to Standard Penetration Blow Counts

▽ groundwater first encountered at time of drilling

▼ groundwater at time of backfilling

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				0	LIGHT BROWN CLAYEY GRAVEL (GP-GC), medium dense, dry (Fill)
				2	DARK BROWN SANDY CLAY (CH), medium stiff, wet, plastic
	8				DARK GRAY-BROWN VERY SANDY SILT (MH), stiff, moist
UC(P) = 3000	9			8	same as above, sample discarded
				10	
				12	
					LENS OF DARK GRAY COARSE SAND (SP-SM), loose, saturated
Percent Passing No. 200 Sieve = 9.1	4			14	BLUE-GRAY SILT (MH), soft, wet
				16	

LOG OF BORING 8

Equipment 3.25" HOLLOW STEM

Elevation

Date 4-11-14

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Job No: 644.1.1

Date: 6-11-14

Appr: Jm

LOG OF BORING 8

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **9a**

*Converted to Standard Penetration Blow Counts

▽ groundwater first encountered at time of drilling

▼ groundwater at time of backfilling

LOG OF BORING 8

Equipment: 3.25" HOLLOW STEM

Elevation: ______ Date: 4-11-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Sample	Description
				16		
				18		DARK GRAY SILTY COARSE SAND (SP), loose, saturated
	7				X	no sample recovered
				20		
						BLUE VERY SANDY SILT (MH), very stiff, moist, plastic
	24				X	
				22		

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LOG OF BORING 8

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **9b**

*Converted to Standard Penetration Blow Counts

▽ groundwater first encountered at time of drilling

▼ groundwater at time of backfilling

LOG OF BORING 9

Equipment: 3.25" HOLLOW STEM

Elevation: ______ Date: 4-11-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				0	2 inches asphalt concrete
					DARK BROWN CLAYEY GRAVEL (GC), medium dense, moist (roadbase) — Fill
					DARK GRAY SANDY SILT (MH), medium stiff, saturated, plastic
				2	
				4	LENS OF LIGHT BROWN CLAYEY SAND (SC), loose, saturated
TxUU = 330 (500) UC(P) = 1550	2	40.4	78		DARK GRAY SANDY CLAYEY SILT (MH), medium stiff, saturated, plastic
				6	
					BLUE-GRAY SILTY COARSE SAND (SM), medium dense, dry to moist, with fine gravel
				8	
	16				
				10	
					LENS OF BLUE COARSE SAND (SP), loose, saturated
				12	
		22.0	102		DARK BLUE-GRAY CLAYEY SAND (SP-SC), medium dense, moist
TxUU = 4620 (4000)	17				
				14	
				16	

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LOG OF BORING 9

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **10a**

*Converted to Standard Penetration Blow Counts

∇ groundwater first encountered at time of drilling

▼ groundwater at time of backfilling

Laboratory Test Results or Remarks

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Sample	
				16		
				18		
				20		
	19					BLUE FINE SANDY SILT (MH), very stiff, moist
	29					
				22		becomes cemented

LOG OF BORING 9

Equipment 3.25" HOLLOW STEM

Elevation ______ Date 4-11-14

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LOG OF BORING 9

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **10b**

*Converted to Standard Penetration Blow Counts

▽ groundwater first encountered at time of drilling

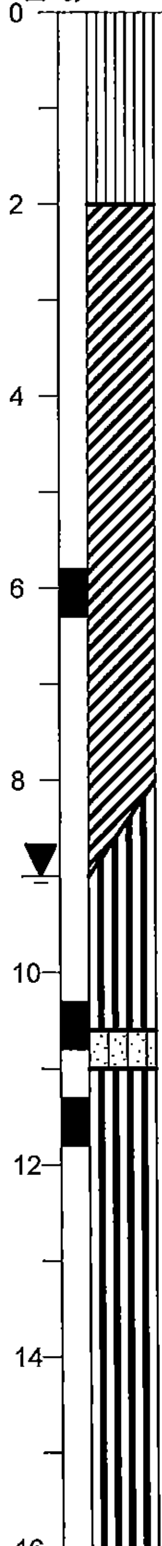
▼ groundwater at time of backfilling

LOG OF BORING 10

Equipment 3.25" HOLLOW STEM

Elevation ______ Date 4-11-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				0	LIGHT BROWN SANDY SILT (ML), medium stiff, moist, with roots, porous (topsoil)
				2	DARK GRAY-BROWN VERY SANDY CLAY (CH), medium stiff, wet, plastic
				4	
UC = 1530	7	48.2	73	6	
				8	
					BLUE SANDY SILT (MH), stiff, dry to moist, with very occasional subangular gravel, plastic
				10	
TxUU = 8010 (5000)	12	32.9	89		VERY THIN LENS OF SILTY SAND (SM)
					BLUE FINE SANDY SILT (MH), stiff, dry to moist, with very occasional subangular gravel
TxUU = 710 (7000)	9				
				12	
				14	
				16	



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Job No: 644.1.1

Date: 6-11-14

Appr: JM

LOG OF BORING 10

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **11a**

*Converted to Standard Penetration Blow Counts

▽ groundwater first encountered at time of drilling

▼ groundwater at time of backfilling

LOG OF BORING 10

Equipment 3.25" HOLLOW STEM

Elevation ________ Date 4-11-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Sample	Description
				16		
						BLACK SILT (MH), medium stiff, wet, plastic
				18		
		94.3	49			
CONSOL	6					
				20		
						BLUE-GRAY COARSE SANDY SILT (MH), very stiff, wet
	16					

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Date: 6-11-14

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LOG OF BORING 10

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE 11b

*Converted to Standard Penetration Blow Counts

LOG OF BORING 11

Equipment: 3.25" HOLLOW STEM
Elevation: ______ Date: 4-11-14

▽ groundwater first encountered at time of drilling
▼ groundwater at time of backfilling

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				0	DARK GRAY VERY SANDY CLAY (CH), soft, saturated, with gravel
	3			2	becomes light gray in color
				4	MOTTLED DARK GRAY AND BROWN GRAVELLY CLAY (CH), medium stiff, wet, plastic
	5			6	LENS OF LIGHT BROWN VERY CLAYEY COARSE SAND (SC), loose, wet
					LIGHT GRAY-BROWN SANDY CLAY (CH), stiff, wet, plastic
				8 ▼	
				10	LENS OF LIGHT BROWN CLAYEY SAND (SC)
					BLUE-GRAY FINE SANDY SILT (MH), soft, moist
CONSOL	3	54.6	68		
				12	
				14	
UC(P) = 2200		86.2	50		becomes less sandy, slightly plastic
UC(P) = 4500+	7	44.7	74	16	BLUE SILTY COARSE SAND (SM), loose, wet

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Date: 6-11-14
Appr: Jm

LOG OF BORING 11
CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **12a**

*Converted to Standard Penetration Blow Counts

groundwater first encountered at time of drilling

groundwater at time of backfilling

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Sample	
				16		
				18		
						BLUE SANDY SILT (MH), very stiff, dry
UC(P) = 4500+				20		
	43					LIGHT BLUE-GRAY CLAYEY GRAVEL (GC), dense, moist

LOG OF BORING 11

Equipment 3.25" HOLLOW STEM

Elevation ______ Date 4-11-14

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Date: 6-11-14

Appr: Jm

LOG OF BORING 11

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **12b**

*Converted to Standard Penetration Blow Counts

▽ groundwater first encountered at time of drilling

▼ groundwater at time of backfilling

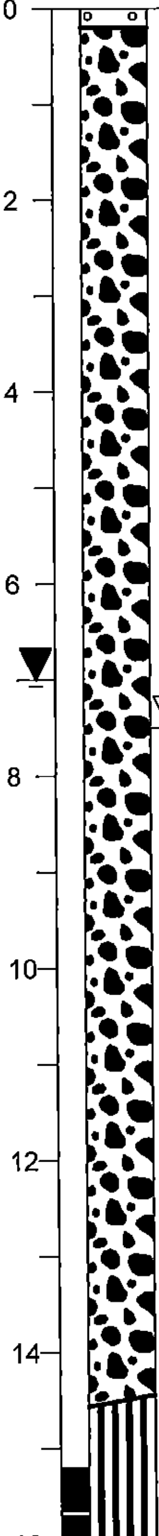
Laboratory Test Results or Remarks

LOG OF BORING 12

Equipment 3.25" HOLLOW STEM

Elevation ______ Date 4-11-14

Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
			0	2 inches crush gravel (loose road fill)
				GRAY-BROWN ANGULAR GRAVEL (GP), medium dense, moist
			2	
			4	
			6	no sample recovered
			8	
			10	no sample recovered
			12	
			14	BLUE-GRAY VERY SANDY SILT (MH), stiff, wet
8	52.6	69	16	



Fill

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Job No: 644.1.1

Date: 6-11-14

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LOG OF BORING 12

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **13a**

*Converted to Standard Penetration Blow Counts

▽ groundwater first encountered at time of drilling

▼ groundwater at time of backfilling

Laboratory Test Results or Remarks

Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
			16	
14				
			18	
			20	
16				BLUE-GRAY SILTY GRAVEL (GM), medium dense, moist to wet

LOG OF BORING 12

Equipment 3.25" HOLLOW STEM

Elevation ______ Date 4-11-14

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Job No: 644.1.1

Date: 6-11-14

Appr: gm

LOG OF BORING 12

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE **13b**

*Converted to Standard Penetration Blow Counts

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			TYPICAL NAMES	
COARSE GRAINED SOILS MORE THAN HALF IS LARGER THAN No. 200 SIEVE	GRAVEL MORE THAN HALF OF COARSE FRACTION IS LARGER THAN No. 4 SIEVE SIZE	CLEAN GRAVEL WITH LESS THAN 5% FINES	GW	WELL GRADED GRAVEL, GRAVEL-SAND MIXTURE
			GP	POORLY GRADED GRAVEL, GRAVEL-SAND MIXTURE
		GRAVEL WITH OVER 12% FINES	GM	SILTY GRAVEL, GRAVEL-SAND-SILT MIXTURE
			GC	CLAYEY GRAVEL, GRAVEL-SAND-CLAY MIXTURE
	SAND MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN No. 4 SIEVE SIZE	CLEAN SAND WITH LESS THAN 5% FINES	SW	WELL GRADED SAND, GRAVELLY SAND
			SP	POORLY GRADED SAND, GRAVELLY SAND
		SAND WITH OVER 12% FINES	SM	SILTY SAND, GRAVEL-SAND-SILT MIXTURE
			SC	CLAYEY SAND, GRAVEL-SAND-CLAY MIXTURE
FINE GRAINED SOILS MORE THAN HALF IS SMALLER THAN No. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILT, ROCK FLOUR, SANDY OR CLAYEY SILT WITH LOW PLASTICITY
			CL	INORGANIC CLAY OF LOW TO MEDIUM PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAY (LEAN)
			OL	ORGANIC CLAY AND ORGANIC SILTY CLAY OF LOW PLASTICITY
	SILT AND CLAY LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILT, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOIL, ELASTIC SILT
			CH	INORGANIC CLAY OF HIGH PLASTICITY, GRAVELLY, SANDY OR SILTY CLAY (FAT)
			OH	ORGANIC CLAY OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

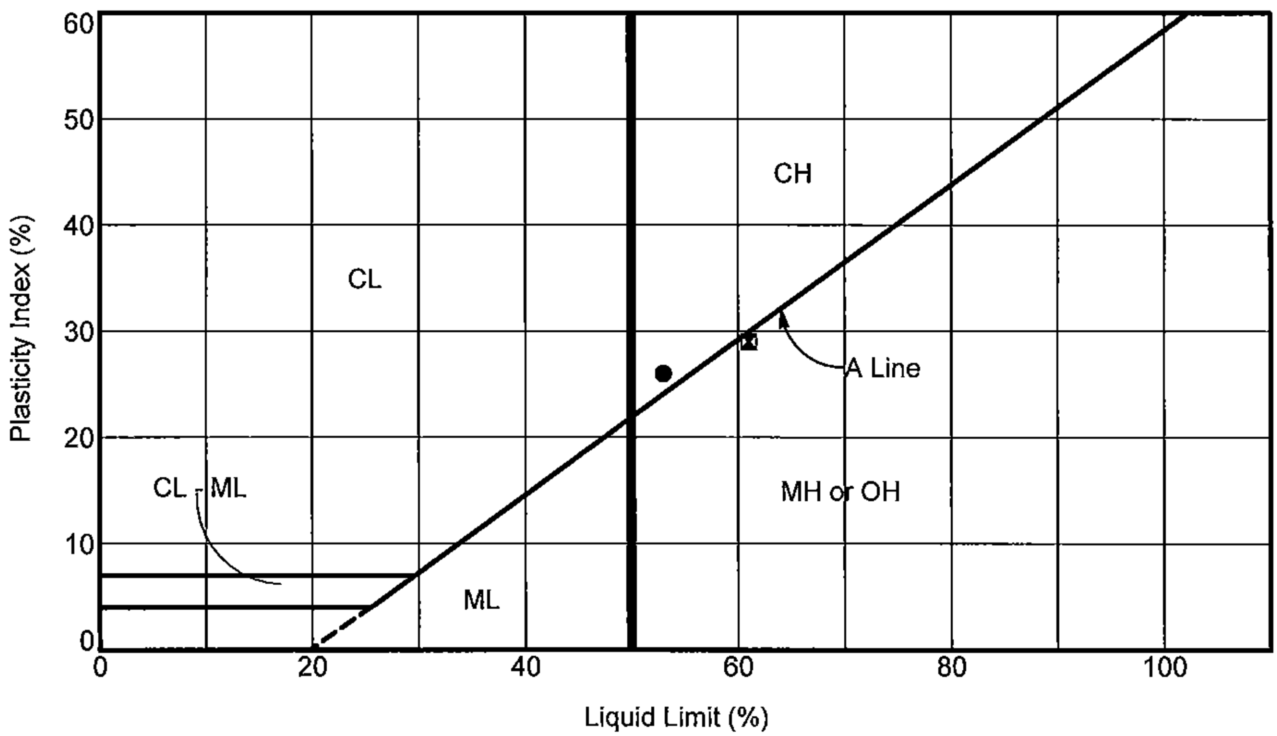
KEY TO TEST DATA

				Shear Strength, psf	Confining Pressure, psf
EI	Expansion Index	TxUU	Unconsolidated Undrained Triaxial	320	(2600)
Consol	Consolidation	TxCU	Consolidated Undrained Triaxial	320	(2600)
LL	Liquid Limit (in %)	DSCD	Consolidated Drained Direct Shear	2750	(2000)
PL	Plastic Limit (in %)	FVS	Field Vane Shear	470	
PI	Plasticity Index	LVS	Laboratory Vane Shear	700	
SA	Sieve Analysis	UC	Unconfined Compression	2000	*
G_s	Specific Gravity	UC(P)	Laboratory Penetrometer	700	*
■	"Undisturbed" Sample				
⊠	Bulk Sample				

Notes: (1) All strength tests on 2.8" or 2.4" diameter samples unless otherwise indicated.

* Compressive Strength

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 644.1.1 Date: 6-11-14 Appr: Jm	SOIL CLASSIFICATION CHART AND KEY TO TEST DATA CALISTOGA HOTEL CALISTOGA, CALIFORNIA	PLATE 14



ASTM D 4318-98

Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Free Swell (%)
●	**BLACK SANDY CLAY (CH)** **Test Boring 1 at 1.3 feet**	**53**	**27**	**26**	**75**
⊠	**MOTTLED GRAY-BROWN GRAVELLY SILTY FINE SAND (SM)** **Test Boring 2 at 11.5 feet**	**61**	**32**	**29**	**35**

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Job No: 644.1.1

Date: 6-11-14

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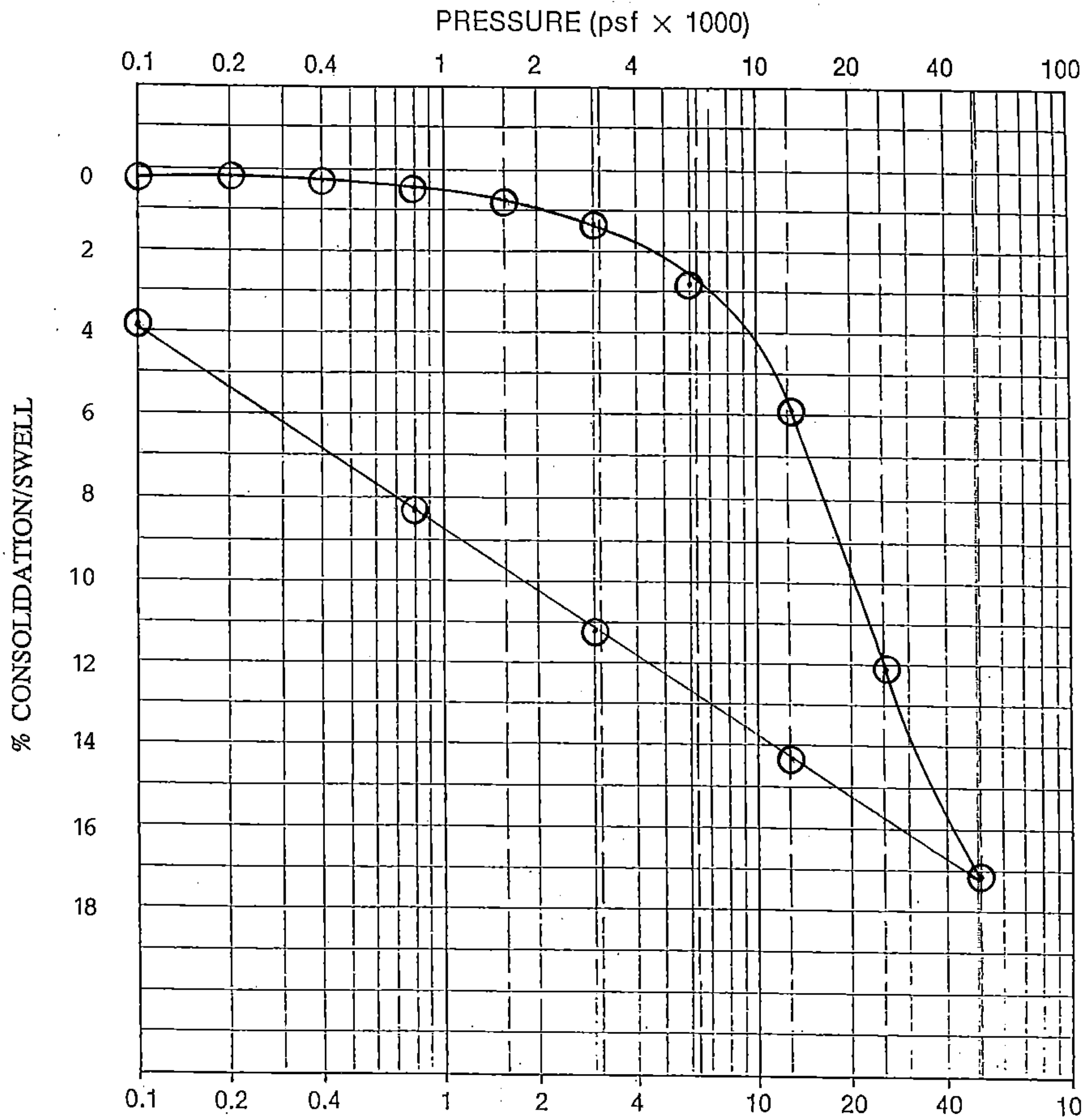
ATTERBERG LIMITS TEST RESULTS

CALISTOGA HOTEL
CALISTOGA, CALIFORNIA

PLATE
15



Reference: ASTM D 4546

Type of Specimen Undisturbed		Condition		Before Test		After Test	
Diameter (in.) 2.43	Height (in.) 0.80	Water Content	w_o	47.0 %	w_f	45.4 %	
Overburden Press., P_o psf		Void Ratio	e_o	1.390	e_f	1.298	
Preconsol. Press., P_c (CASSA) 10000 psf		Saturation	S_o	96 %	S_f	100 %	
Compression Index, C_c 0.206		Dry Density	γ_d	74.5 pcf	γ_d	77.5 pcf	
LL	PL	PI		Gs 2.85			
Classification DARK GRAY-BROWN SANDY SILT (MH)			Source Test Pit 4 at 7.4 feet				

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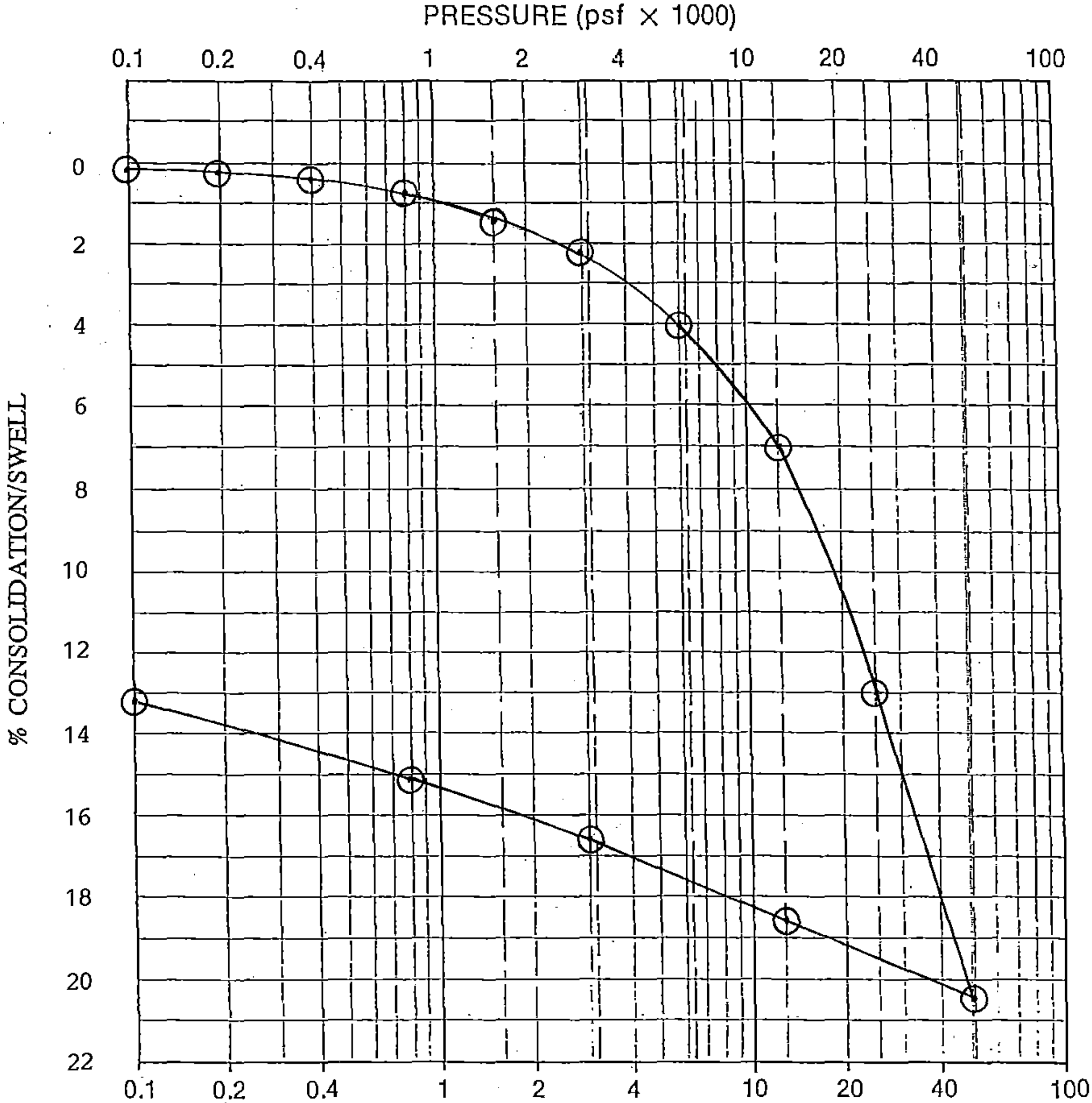

Reference: ASTM D 4546

Type of Specimen Undisturbed		Condition		Before Test		After Test
Diameter (in.) 2.43	Height (in.) 0.80	Water Content	w_o	54.6 %	w_f	46.1 %
Overburden Press., P_o psf		Void Ratio	e_o	1.714	e_f	1.358
Preconsol. Press., P_c (CASSA) 10000 psf		Saturation	S_o	94 %	S_f	100 %
Compression Index, C_c .244		Dry Density	γ_d	67.9 pcf	γ_d	78.1 pcf
LL	PL	PI		Gs 2.95		
Classification DARK GRAY SANDY SILT (MH)			Source Test Pit 5 at 15.8 feet			

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Date: 07-17-14

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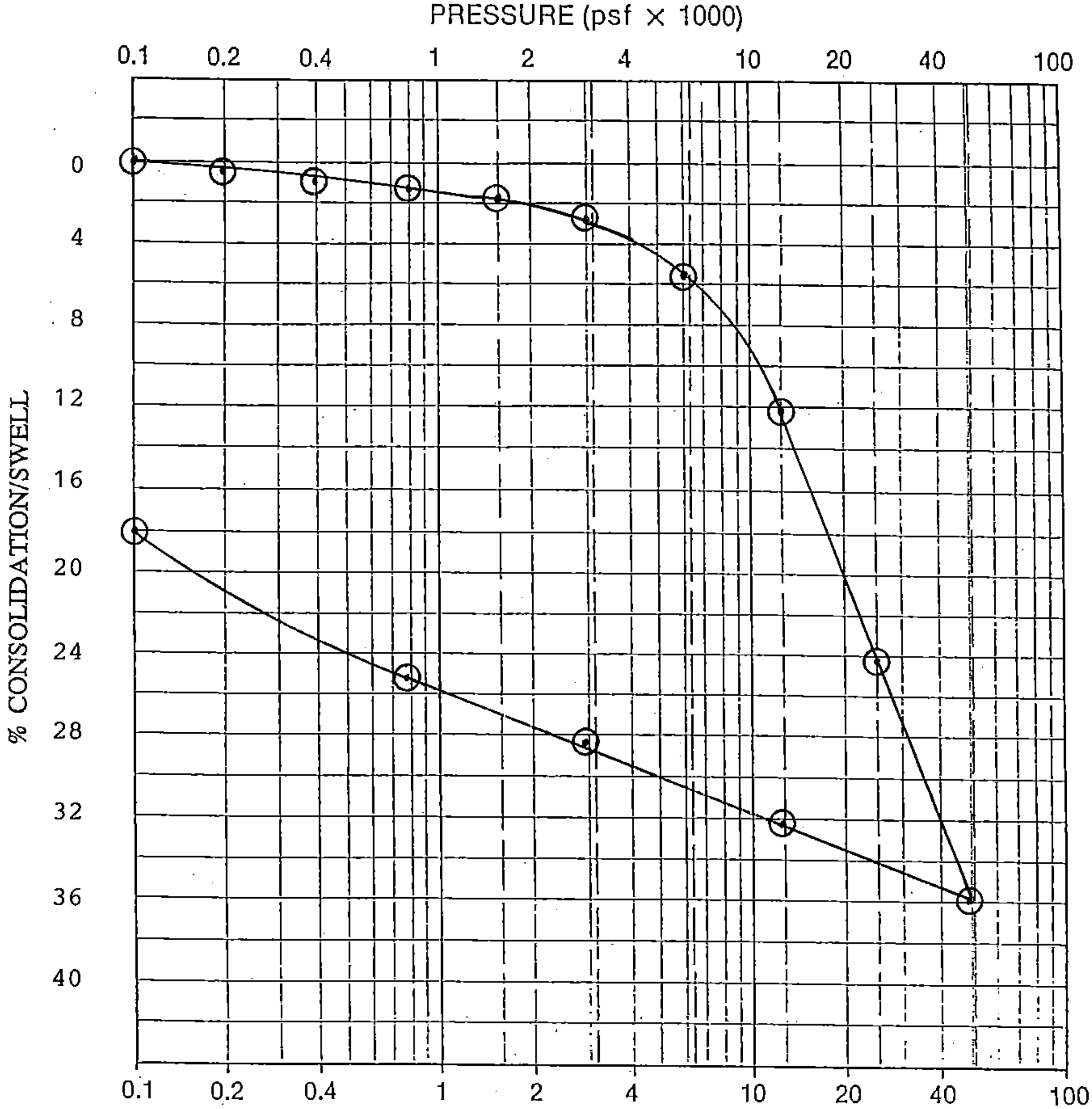
CONSOLIDATION/SWELL
TEST REPORT

CALISTOGA HOTEL
1506/1522 LINCOLN STREET
CALISTOGA, CALIFORNIA

PLATE

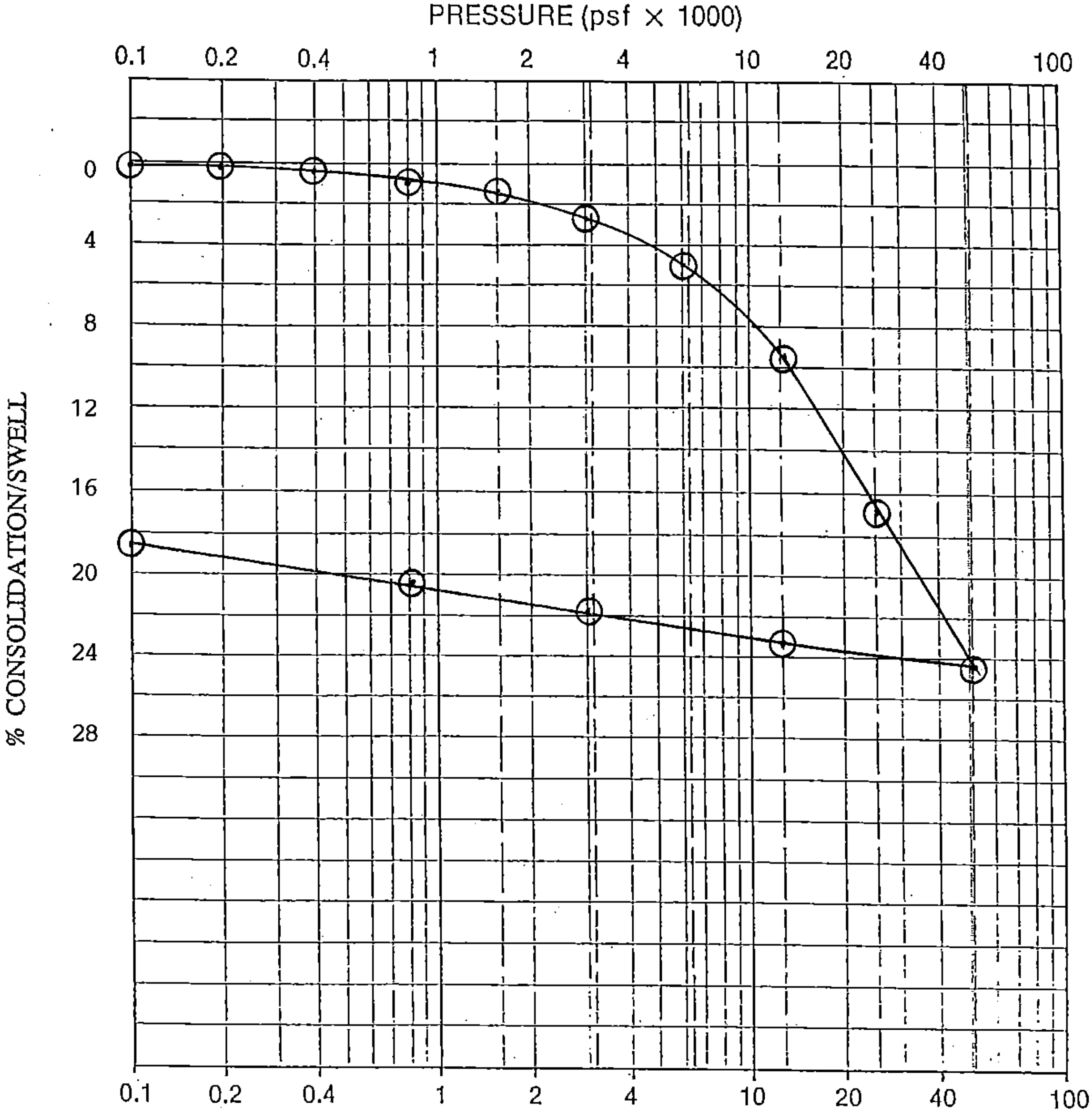
17



Reference: ASTM D 4546

Type of Specimen Undisturbed		Condition		Before Test		After Test
Diameter (in.) 2.43	Height (in.) 0.80	Water Content	w_o	94.5 %	w_f	71.8 %
Overburden Press., P_o psf		Void Ratio	e_o	2.751	e_f	2.074
Preconsol. Press., P_c (CASSA) 9000 psf		Saturation	S_o	100 %	S_f	100 %
Compression Index, C_c 0.390		Dry Density	γ_d	48.3 pcf	γ_d	58.9 pcf
LL	PL	PI			Gs	2.90
Classification BLACK SILT (MH)			Source	Test Pit 10 at 19.3 ft		

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 644.1.1 Date: 07-17-14 Appr: jm	CONSOLIDATION/SWELL TEST REPORT CALISTOGA HOTEL 1506/1522 LINCOLN STREET CALISTOGA, CALIFORNIA	PLATE 18



Reference: ASTM D 4546

Type of Specimen Undisturbed		Condition		Before Test		After Test	
Diameter (in.) 2.43	Height (in.) 0.80	Water Content	w_o	54.6 %	w_f	40.3 %
Overburden Press., P_o psf		Void Ratio	e_o	1.648	e_f	1.154
Preconsol. Press., P_c (CASSA) 10,000 psf		Saturation	S_o	95 %	S_f	100 %
Compression Index, C_c 0.241		Dry Density	γ_d	67.7 pcf	γ_d	83.2 pcf
LL	PL	PI		Gs 2.87		
Classification BLUE SANDY SILT (MH)		Source 11 at 10.8 feet				

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Date: 07-17-14

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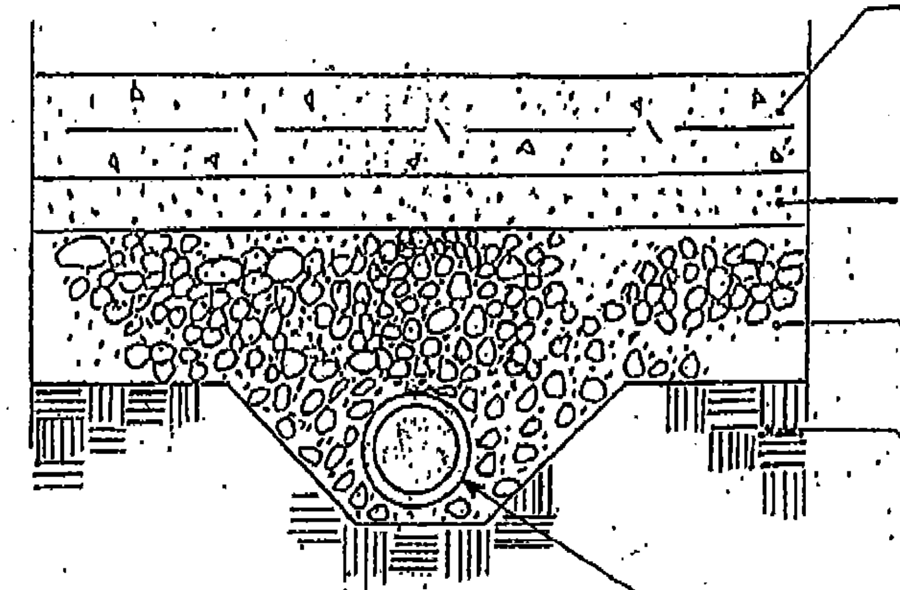
CONSOLIDATION/SWELL TEST REPORT

CALISTOGA HOTEL
1506/1522 LINCOLN STREET
CALISTOGA, CALIFORNIA

PLATE 19



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