GEOTECHNICAL INVESTIGATION

THE AVALON INN
1201 NORTH MAIN STREET
FORT BRAGG, CALIFORNIA

Project Number 12122.01

December 4, 2015

Brunsing Associates, Inc.

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Project Number – 12122.01

prepared for

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TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF APPENDIX

1.0 INTRODUCTION

This report presents the results of the Geotechnical Investigation that Brunsing Associates, Inc. (BAI) performed for The Avalon Inn, lands of the former Hi-Seas Motel, located at 1201 North Main Street, Fort Bragg, California, Assessor Parcel Number APN 069-241-04 & 27. The site is near an ocean bluff on the west side of the intersection of Airport Road and Main Street, approximately three-tenths of a mile north of Pudding Creek, as shown on the Vicinity Map, Plate 1.

Based on correspondence with the property owner, we understand that the existing buildings have been removed and an Inn with Conference Center will be built. We understand from reviewing preliminary plans by Epikos Land Planning & Architecture, dated 8/17/2015 and 7/29/2015, that the planned new development will include three-story units, restaurant, event space, exterior slabs and paved parking areas. The proposed Inn will cover approximately 46,533 square feet and have concrete slab-on-grade. The existing structures and surrounding property are shown on the Topographic Map prepared by Forrest Francis, Land Surveyor, presented herein as our Existing Site Plan, Plate 2. The planned site improvements are shown on our Planned Site Plan, Plate 3.

The scope of our services, as outlined in our Service Agreement dated February 12, 2008, consisted of researching published geologic maps and BAI's previous file data on nearby projects, aerial photograph studies, field reconnaissance, subsurface investigation, laboratory testing, and engineering and geologic analyses, in order to provide conclusions and recommendations regarding:

- Geologic hazards, including bluff stability/retreat (erosion) rate and tsunami hazard;
- Classification of soil and rock types encountered;
- Suitable foundation type(s) with design criteria and estimated settlement behavior;
- Seismic design criteria per California Building Code (CBC), 2013 edition;
- Site grading;
- Support of concrete slabs-on-grade, as appropriate;
- Lateral earth pressures and drainage requirements for swimming pool or other subsurface walls, as appropriate;
- Support of driveway and parking areas, including pavement section design, as appropriate;
- Site drainage;
- The need for additional geotechnical services as appropriate.

2.0 INVESTIGATION AND LABORATORY TESTING

2.1 Published Map Research

As part of our investigation, we initially reviewed the following published geologic maps and references:

- California Division of Mines and Geology (CDMG), 1998, Uniform Building Code, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada: International Conference of Building Officials (ICBO).
- Eisner, R.K., Borrero, J.C., and Synolakis, C.E., 2001, Inundation Maps for the State of California. International Tsunami Symposium, ITS 2001 Proceedings, NHTMP Review Paper #4, 67-81.
- United States Geological Survey (USGS), 2015, Fort Bragg Quadrangle, 7.5-Minute Series.
- Kilbourne, R.T., 1983, Geology and Geomorphic Features Related to Landsliding, Fort Bragg 7.5 Minute Quadrangle, Mendocino County, California: California Division of Mines and Geology (CDMG), Open File Report 83-5.
- Jennings, C.W., 1960, Geologic Map of California, Ukiah Sheet: California Division of Mines and Geology (CDMG).
- Merrits, D., and Bull, W., 1989, "Interpreting Quaternary Uplift rates at The Mendocino Triple Junction from Uplifted Marine Terraces": Geological Society of America, Geology, v. 17, no. 11, p. 1020-1024.
- Miller, M., and Higdon, F., 2006, "City of Fort Bragg Tsunami Contingency Plan". Fort Bragg Police Department.
- State of California Seismic Safety Commission, 2005, "The Tsunami Threat to California: Findings and Recommendations on Tsunami Hazards and Risks." California Seismic Safety Commission.

2.2 Aerial Photograph Studies

Our investigation was augmented by studying aerial photographs dated June 30, 1963, June 23, 1981, and April 1, 2000. The photographs were each enlarged from the vendors' negatives, to an approximate scale of one inch equals 200 to 300 feet (varied by photograph). During our study, BAI determined relatively accurate photograph scales by comparing scaled measurements between various physical features in the site vicinity shown on the USGS 7.5-Minute Fort Bragg Quadrangle (such as the distance between driveway intersections along the highway) that are also shown on the photographs. We then compared the scaled map measurements with scaled distances of the same physical features on the photographs in order to calculate the photograph scales. The results of our photograph studies are presented in Section 5.2 of this report, Bluff Retreat.

In addition to reviewing aerial photographs, we also obtained oblique-angle aerial photographs from the California Coastal Records Project (www.californiacoastline.org). We qualitatively compared photographs of the site from 1979, 1987, 2002, and 2005. The 2005 photo is presented herein (with geology added, see section 2.3 below) as our Coastline Oblique Aerial Photograph on Plate 4.

2.3 Field Reconnaissance

BAI's Principal Engineering Geologist and Staff Geologist performed an initial site visit on February 8, 2008 to assess site conditions and mark test boring locations. Our Staff Geologist returned to the site on March 19, 2008 to perform a geologic reconnaissance. Our field reconnaissance consisted of examination of bedrock and soil exposed on the bluff face and interpretation of geomorphic expressions on the bluffs and terrace within the property and vicinity. During a low tide, we mapped the geology of the bluff face and terrace edge on the oblique aerial photograph (Plate 4). Site Photographs A through E on Plates 5 through 9, respectively, show the property and nearby bluffs from various vantage points. The locations from which the photographs were taken are shown on Plates 2 and 4.

2.4 Field Exploration

Our subsurface drilling exploration was conducted on February 20, 2008 and consisted of drilling, logging, and sampling seven exploratory test borings, borings B-1 through B-7. The approximate test boring locations are shown on Plates 2 and 3. Each of the test borings was advanced to practical drilling refusal with the exception of B-4, a shorter boring for pavement design purposes. In each of the other borings, practical drilling refusal was encountered within bedrock between approximately 10 to 16 feet below ground surface (bgs). The borings were advanced with a truck-mounted drill rig (B-53) utilizing 7-inch diameter hollow-stem flight auger equipment.

Our Staff Geologist made a descriptive log of each test boring and obtained relatively undisturbed tube samples of the soil and rock materials encountered for visual classification and laboratory testing. Relatively undisturbed samples were obtained using a 3.0-inch outside diameter Modified California (CA) split-barrel sampler and disturbed samples were obtained with a 2.5-inch outside diameter modified California (CM) and a 2-inch outside diameter Standard Penetration Test (SPT) sampler. Samplers were driven by a 140-pound drop hammer falling 30 inches per blow. Blows required to drive the CA and CM samplers were converted to SPT blow counts for correlation with empirical test data, using a conversion factor of 0.64 and 0.79, respectively. Selected samples were also obtained using a 2-inch outside diameter, SPT sampler containing 1.4-inch inside diameter liners. Blow counts are presented on the boring logs[1].

Graphic logs of the borings, showing the various soil types encountered and the depths of the samples taken are presented on Plates 10 through 16. The soils are classified in accordance with the Unified Soil Classification System outlined on Plate 17. The various soil and rock descriptive properties used are listed on Plates 18 and 19, respectively.

2.5 Laboratory Testing

Selected samples obtained during our subsurface exploration were tested in our laboratory to determine their pertinent geotechnical engineering characteristics. Laboratory testing consisted of moisture content-dry density, grain-size classification, Atterberg Limits, unconsolidated-undrained triaxial compression, direct shear, and resistance value tests. The test results are presented opposite the samples tested on the test boring logs (see Key to Test Data on Plate 17). In addition, grain size distribution test results are presented on Plate 20, Atterberg Limits test results are presented on Plate 21, triaxial compression test data are presented on Plate 22, direct

[1] SPT blow counts provide a relative measure of soil consistency and strength, and are utilized in our engineering analyses.

shear test results are presented on Plate 23, and resistance value test results are presented on Plate 24.

3.0 SITE CONDITIONS

The property is located on the west side of Main Street (Highway 1), just northwest of the intersection with Airport Road. A paved bike path (former lumber company haul road/railroad bed) forms the westerly boundary of the property. The parcel is within a gently-sloping, Pleistocene marine terrace platform that locally extends from the bluff edge to the east side of the highway. The previous buildings on the property consisted of a single-story motel with its long axis oriented roughly north-south, and a small detached garage near the southern property line. The buildings experienced fire damage and where in poor repair. Paved parking areas are located west of the previous main building (Plate 5). The previous leach fields are located between the previous motel and the rear (western) property line. A low wooden fence in poor repair is located approximately 50 feet east of the western property line.

The ground surface within the property is nearly level and supports a thick growth of seasonal grasses (Plate 6). A small drainage swale is located near the southwestern corner of the parcel, and drains toward a culvert west of the property. A seasonal pond is located in the northwestern corner of the property and spans into the northerly adjacent parcel. At the time of our reconnaissance there was a modest amount of water in the pond, and no drainage outlet was visible. Thick brambles and other shrubs grow near the pond and along the wooden fence.

A shallow drainage ditch runs east to west just inside the southerly property boundary. Buried drainage structures are shown in this area on the 1998 survey (Plate 2). A small pond is located near the southwestern corner of the existing garage building. At the time of our reconnaissance, it appeared that the pond was fed by drainage from pipes emanating from the southerly adjacent property. A 12-inch CMP shown on the 1998 survey of the property also appears to terminate in this area, although we did not observe the CMP.

West of the property, the nearly-level terrace extends other 200-plus feet to the bluff edge. The bluffs west of the property are roughly 20 feet in vertical height, with slope gradients that vary from moderately steep to near-vertical. The shape of the coastline at the bluff edge is variable, with locally recessed areas between small promontories. A number of shallow, active drainage channels cross the terraces and drain over the bluffs to the ocean. Seepage from the bluff face itself was also observed (See Plate 4). Vegetation on the terrace consists of a moderate to thick cover of seasonal grasses, flowering plants, and iceplant; vegetation is very thick along the drainage channels. Parts of the bluff face are vegetated, whereas other areas consist of bare rock and soil.

4.0 SITE GEOLOGY AND SOIL CONDITIONS

4.1 Regional Geologic and Seismic Setting

According to the published geologic references we reviewed for this investigation, the bedrock of this part of the Mendocino coast is comprised of well-consolidated sedimentary rocks of the Cretaceous-Tertiary Period coastal belt Franciscan Complex, such as sandstone and shale. The blufftop property occupies a near-level marine terrace underlain by the Franciscan Complex bedrock. The terrace was formed during the Pleistocene Epoch, when periods of glaciation caused sea level fluctuations, which created a series of steps, or terraces, cut into the coastal bedrock by wave erosion. Shallow marine sediments (Pleistocene terrace deposits) were deposited on the wave-cut, bedrock platforms while they were submerged beneath the ocean during interglacial sea-level high stands. Some of these marine deposits have been locally eroded as the terraces began to emerge from the ocean due to uplift associated with the San Andreas Fault Zone during the middle and late Pleistocene. Present sea levels were achieved about 5,000 to 7,000 years ago.

The seismicity and tectonics of the Mendocino coastal region are controlled by a network of generally northwest-trending strike-slip faults of the San Andreas Fault system. The active San Andreas Fault (north coast segment) is located offshore, approximately 7 miles west-southwest of the site. The active Maacama Fault (central segment) is located approximately 20.6 miles northeast of the site. Other, possibly active faults, such as the Pacific Star Winery Fault, are located a couple miles east-northeast of the site. Future, large magnitude earthquakes originating on these or other nearby faults are expected to cause strong ground shaking at the site.

4.2 Site Soil and Geologic Conditions

Surface soils at the site generally consist of about one to 2 ½ feet of brown silty sand topsoil that is very loose to medium dense and porous. Beneath the topsoil, the property is typically mantled by poorly-consolidated, Pleistocene Epoch marine terrace deposits. The terrace deposits consist of beach or shallow marine sediments that are typically comprised of sands with some silt, gravel, and clay, along with incorporated rock fragments eroded from the underlying bedrock platform. The terrace materials were deposited in lenses that are generally flat, with local undulations caused by the variable-energy nature of the depositional environment.

As observed in our test borings, the terrace deposits underlying the property are mostly comprised of loose to medium dense, saturated sands. Clayey or silty sands were observed in some of the borings, generally just under the topsoil, but most of the terrace deposits we observed are relatively devoid of fines (silt- and clay-sized particles). As observed in our borings, the terrace deposits generally extend to between 7 ½ and 15 feet below the ground surface (bgs), and appear to generally thicken toward the east. The terrace deposits we observed within the bluff face west of the property are up to about 10 feet thick and consist of reddish brown silty sands interbedded with silty sandy gravels.

Bedrock was encountered in our test borings at approximately 7 ½ to 15 feet bgs. As observed in our test borings, the bedrock underlying the property generally consists of dark gray sandstone that is moderately to intensely fractured, moderately hard to very hard, and deeply- to little-weathered. The upper few feet of the rock is generally deeply weathered to orange-brown in color. Weathering generally decreases with depth, and hardness increases. Practical drilling refusal was generally encountered after about one to three feet of penetration into the bedrock.

Unstabilized ground water levels were observed within our test borings ranged between about 2 ½ and 7 feet bgs. Shallow ground water can temporarily occur during, and immediately following wet weather periods.

As observed during our reconnaissance of the bluffs west of the property, the upper bedrock underlying the terrace deposits is comprised of orange-brown sandstone that is intensely fractured, low to moderate in hardness, and deeply weathered. The lower bluffs consist of dark gray sandstone that is intensely to closely fractured, hard, and little weathered. Many of the Franciscan rocks are poorly bedded. However, locally, the bedding orientation of the Franciscan rocks consists of a northwest to northeast trending strike with a moderately steep dip, about 40 degrees from horizontal, to the east. The bluffs as viewed from the beach are shown on Plates 7 through 9.

Two ancient (inactive) faults and associated narrow shear zones (a few feet wide) were observed within the bluffs (Plates 4 and 8). Some increased erosion has occurred along the faults due to the relatively weakened condition of the rocks and increased permeability along the fault plane. The faults do not appear to offset the overlying terrace deposits. No evidence of active faulting was observed by us nor is shown on the references we reviewed for this investigation. The active San Andreas fault is located approximately 7 miles offshore.

No sea caves were observed within the bluffs west of the property during our reconnaissance. A few, small open cracks (on the order of one foot high/deep) were observed along bedding planes or ancient faults.

A few areas of accelerated erosion, small rockfalls, and small landslide deposits were observed along the bluff edge (Plates 4 and 9). The slides we observed are relatively small and do not appear to involve the bedrock.

The beach exposed during our reconnaissance consists of large gravels and cobbles, and is strewn with large pieces of driftwood (Plate 9). The beach sands have been eroded away by winter waves. Plate 4 shows the same beach during October of 2005, illustrating the notable difference in seasonal beach deposits due to wave and current action.

5.0 DISCUSSIONS AND CONCLUSIONS

5.1 General

Based on the results of our reconnaissance and subsurface exploration, we conclude that the site is geologically suitable for the proposed redevelopment. The main geotechnical considerations affecting the proposed construction are bluff stability, bluff erosion/retreat rate, tsunami/storm wave hazard, potential soil liquefaction and densification, potential settlement, and strong seismic shaking from future earthquakes. These considerations and their possible mitigation measures are discussed below.

5.2 Effects of Sea Level Rise

The California Coastal Commission (CCC) adopted the March 2013 update to the State of California Sea-Level Rise Guidance Document prepared by the Ocean Protection Counsel (OPC). The OPC report is based upon a 2012 report prepared by the National Academy of

Sciences. The OPC report provided sea-level rise projections for the coast south of Cape Mendocino, as follows in Table 1:

Table 1: Sea Level Rise

Time Period	Sea Level Rise (Feet)
2000-2030	0.13 to 0.98
2000-2050	0.39 to 2.0
2000-2100	1.38 to 5.48

The CCC requires a 75-year lifespan for new, coastal construction or major remodel. According to recent projections, by 2090, the sea level will be as much as 58 inches higher than present.

Using the CCC's economic lifespan of a building of 75 years, we must consider the effects of sea level rise for a structure built circa 2015 through 2090. For this discussion, we will assume a linear rate of sea level rise (which may or may not be the case) in order to estimate a projected sea level rise of approximately 58 inches (4.8 feet) by 2090. Table 2 shows our estimated periodic increase in retreat rate as sea level rises.

Table 2: Bluff Retreat Rate

Years	Span (years)	Cumulative Sea Level Rise (inches)	Retreat Rate (inches per year)	Amount of Retreat (inches)
2015-2030	15	12"	3.0"/yr	45
2030-2050	20	24"	4.0"/yr	80
2050-2090	50	58"	5.0"/yr	250
				375" = 31.25'

Based upon historic aerial photographs and site observations, the current historic average bluff retreat rate appears to be about 3 inches per year. Eventually, as the bluff toe and sea cave interiors are continually subject to strong wave activity even during low tides, the retreat rate should increase to approximately five inches per year.

Table 1 sums up the amount of projected retreat using estimated retreat rates over a 75-year span from a time of 2015 construction. Cumulative sea level rise is from 2015. This results in a total bluff retreat of 31.25 feet.

5.3 Bluff Stability and Retreat

Our site reconnaissance and aerial photograph study indicate that the *average* bluff retreat rate for the bluffs within the State Park property west of the project site is approximately 6.5 to 7.5 inches per year. Eventually, the retreat rate will increase to 6-inches per year due to sea level rise. This will result in an estimated 31.25 feet of bluff loss over the next 75 years. There are no deep-seated or otherwise significant landslides on the bluff in the property vicinity. Since the erosion may not be uniform (some areas of erosion would be greater or less) and considering the possible effects of sea level rise, a safety factor is usually included in determining building setbacks. In this case, the site's western property line is approximately 217 feet east of the bluff edge (measured perpendicular to the previously existing buildings), a distance that includes a safety factor of approximately 6.9. Therefore, BAI concludes that no additional bluff setback within the property is required from a safety standpoint.

5.4 High Groundwater

Results of our investigation indicated that excavations may encounter temporary seepage and/or perched water. If excavations are performed during the rainy season (November through May), groundwater could be encountered within one to two feet of the surface, as measured during our field exploration. Near surface saturated soils should be anticipated at almost any time of the year. If dewatering is necessary, it can likely be accomplished by conventional pumping. However, installation of ground drain blankets, geotextile fabric, sumps, or perimeter dewatering wells (at the contractor's option) will facilitate dewatering, and provide a reasonably dry working pad for subsequent fill placement and compaction.

5.5 Loose/Porous Surface Soils

In unpaved and undeveloped areas, the upper, approximately one foot to 2.5 feet of surface soils at the site contain roots and have a weak, porous consistency. These soils are susceptible to collapse and consolidation under light to moderate loads and are not suitable for support of foundations or slabs-on-grade in their current condition. In addition, as observed in our borings, the majority of sandy deposits overlying the bedrock at the site are loose, and not suitable for support of shallow foundations. Recommendations for deepening of foundations below the weak soil zones or reinforcing the soils are presented in the Recommendations Section of this report.

5.6 Settlement

The terrace deposits encountered during our investigation are compressible for normal building loads and susceptible to liquefaction. However, if drilled piers are used, we estimate post-construction settlements for the foundations to be between ¼-inch and ½-inch. Concrete slabs should be placed on a minimum of five feet of compacted fill. Concrete slabs on compacted fill could still experience differential settlement on the order of about one inch, except for seismic loads, see Section 5.7 Soil Liquefaction and Densification for approximate settlement due to seismic loads. Also, see Section 5.7 Soil Liquefaction and Densification for additional settlement considerations. If a mat foundation, compacted aggregate piers or compacted stone column system is to be considered, BAI should be retained to provide estimated settlement behavior.

5.7 Seismicity and Faulting

As is typical of the Mendocino County area, the site will be subject to strong ground shaking during future, nearby, large magnitude earthquakes originating on the active San Andreas Fault, Maacama Fault, or possibly other, more distant fault systems. The intensity of ground shaking at the site will depend on the distance to the causative earthquake epicenter, the magnitude of the shock, and the response characteristics of the underlying earth materials. Generally, structures founded in supporting materials and designed in accordance with current building codes are well suited to resist the effects of ground shaking.

No evidence of recent faulting, including evidence of recent movement on the ancient faults observed in the bluffs, was observed by BAI or shown in the site vicinity on the published geologic maps that we reviewed for this investigation. The presence of ancient (pre-Pleistocene) faults should not impact the proposed development due to their inactivity. Therefore, the potential for fault rupture at the site is considered low.

5.8 Soil Liquefaction and Lateral Displacement

As discussed above, the site will be subject to strong ground shaking during future, nearby, large magnitude earthquakes originating on the active San Andreas Fault or possibly other fault systems. The potential for surface fault rupture at the site is considered low. However, the potential for liquefaction exists.[2]

To evaluate liquefaction potential, we performed laboratory testing and liquefaction analysis of the soils. The results of our analysis indicate liquefaction will occur at the site during a design earthquake. To quantify liquefaction induced vertical settlement, we performed an analysis based on procedures by Idriss and Boulanger. The results of our analysis are shown in Table 3 below. Liquefaction calculations were performed on the basis of a 7.7 maximum magnitude earthquake and peak ground acceleration of 0.4g for a design basis earthquake. We used a groundwater elevation of approximately 2 feet below ground surface in our liquefaction calculations.

Lateral spreading is generally caused by liquefaction adjacent to slopes. In these cases, the saturated soils move toward an unsupported face, such as a bluff, river channel bank or body of water. Our analysis indicates that there is a potential for lateral displacement, during a design earthquake event, estimated lateral displacements are shown in Table 3.

Table 3. Liquefaction Settlement and Lateral Displacement

Boring/CPT	Settlement (inches)	Lateral Displacement (inches)
B-1	<0.1	2.5
B-2	1.5	14.5
B-3	4.0	44.0
B-5	3.0	30.5
B-6	4.0	34.5
B-7	3.0	34.5

The liquefaction analysis results are presented in Appendix A.

To mitigate the concern of liquefaction, the planned structures should be supported on drilled piers penetrating the underlying supporting bedrock and a compacted fill pad for lateral resistance and slab-on-grade support. Recommendations for drilled piers are given in Section

[2] Liquefaction results in a loss of shear strength and potential soil volume reduction in saturated sandy, silty, silty/clayey, and coarse gravelly soils below the groundwater table from earthquake shaking. The occurrence of this phenomenon is dependent on many factors, including the intensity and duration of ground shaking, the soil age, density, particle size distribution, and position of the groundwater table.

6.0 of this report. As an alternative the terrace deposit can be reinforced by an aggregate pier or compacted stone column system.

5.9 Tsunami/Storm Waves

As typical of the Mendocino County coastal area, the site could be subject to large storm waves or tsunami waves. In February 1960, the Point Cabrillo Light House was damaged by an approximately 60 feet high storm wave (meteorological tsunami or meteotsunami). No such waves are recorded at the light house from 1909, the year it was built, to 1960. Nor have such large waves occurred since 1960.

Within the City of Fort Bragg, the generally-recognized "safe elevation level" with regard to tsunami events is approximately 60 feet above mean sea level. Since the property bluffs are approximately 20 to 30 feet in vertical height and the property elevation is approximately 40 feet above mean sea level, the property lies within a designated low-lying area.

Therefore, impact or inundation from a severe storm surge or tsunami event must be considered a risk for the site, albeit a relatively low risk. The overall height of the bluffs affords blufftop structures continued protection from storm surges, tsunamis and wave run-ups, except for extreme events, even considering the projected sea level rise.

Despite many large earthquakes around the Pacific Rim, including Chile, Alaska, the Aleutians, Kuril Island, Japan and Samoa, no tsunami waves have caused significant damage along the Mendocino County coast except in waterfront harbor areas. There are no records of "significantly-damaging" tsunami waves occurring in this area since Europeans first visited the area in the mid-1750's. No evidence of tsunami wave debris has been found on the coastal terraces in the site vicinity.

Tsunamis are caused by large-scale sea floor elevation changes resulting from earthquakes on thrust faults associated with tectonic subduction zones. Serval major earthquakes have occurred along the Pacific Rim subduction zones in recent times; however, no significant tsunami in the site vicinity, (except in harbor areas), has resulted from these earthquakes. There are several factors that minimize the tsunami potential for the Mendocino County coast:

- The San Andreas Fault is a strike slip fault. Earthquake fault rupture causes ground shifting relative to one side versus the other, but does not result in large, vertical uplift.
- The Point Reyes Fault is a thrust fault that could, theoretically, cause localized uplifting of the sea floor. Therefore, the Point Reyes Fault could be a possible, local source of tsunamis. However, the Point Reyes Fault is only exposed at the southwest end of Point Reyes. The mapped trace of the fault is only 5 miles long. Very little is known of the offshore extent (length or depth) of this fault. The age of the most recent rupture of Point Reyes Fault is uncertain; the fault offsets Cretaceous (66 to 145 Ma) and Miocene (5.3 to 23 Ma) bedrock formations. Therefore, the rupture potential of this fault is uncertain.
- The Mendocino Escarpment is a large, undersea ridge that extends west of Cape Mendocino. The ridge forms a partial wall that runs a few hundred miles to the west. According to Trenkwalder and Stover, the overall effect is that tsunami waves running south toward the escarpment tend to turn north "to impinge on Crescent City".

- In the area south of the Mendocino Escarpment, the ocean is deeper than in the region north of the escarpment. This effect causes a dispersion and reduction in tsunami wave energy impinging on the coast south of Cape Mendocino.

Based upon the above, BAI considers the tsunami potential at the site to be a relatively low risk.

6.0 RECOMMENDATIONS

6.1 Site Grading

6.1.1 Clearing and Stripping

Areas to be graded should be cleared of existing vegetation, rubbish, existing structures, and debris. After clearing, surface soils that contain organic matter should be stripped. In general, the depth of required stripping will be about 2 to 4 inches; deeper stripping and grubbing may be required to remove isolated concentrations of organic matter or roots. The cleared materials should be removed from the site; however, strippings can be stockpiled for later use in landscape areas.

6.1.2 Structural Area Preparation

As used in this report, "Structural Areas" refers to the building envelopes and the areas extending five feet beyond their perimeters, and to exterior concrete slabs and asphalt paved areas and the areas extending three feet beyond their edges.

Within building and exterior slab areas, existing weak soils should be removed to at least to a depth of 5 feet below soil subgrade to help minimize differential settlement. Deeper excavating may be necessary to remove isolated, very weak soils. Within asphalt-paved areas, existing weak soils should be removed for a depth of at least 2 feet below soil subgrade. Deeper excavations may be necessary to remove isolated, very weak soils, if encountered.

After the recommended excavations, BAI should observe the exposed soils. These soils should then be scarified to about six inches deep, moisture conditioned to at or near optimum moisture content and compacted to at least 90 percent relative compaction as determined by the ASTM D 1557 test procedure, latest edition. These moisture conditioning and compaction procedures should be observed by BAI to determine that the soil is properly moisture conditioned and the recommended compaction is achieved.

Within building and exterior slab areas geotextile stabilization fabric, such as Mirafi HP Series, or equivalent, should be used on the bottom of the excavation. Within pavement areas a geotextile stabilization fabric, such as Mirafi: 600X or equivalent, may be needed if the underlying soils are yielding under equipment loads.

Fill material, either imported or on-site, should be free of perishable matter and rocks greater than six inches in largest dimension, and should be approved by a representative of BAI before fill placement. We anticipate that the existing on-site soils to be excavated, in a "cleaned" condition (i.e., less any organics and debris) are satisfactory for reuse as compacted fill.

Imported fill for use in structural areas should be of relatively low expansion potential (i.e., Expansion Index of 30 or less).

Low-expansive engineered fill, on-site or imported, should be placed in thin lifts (six to eight inches depending on compaction equipment), moisture conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction to achieve planned grades.

6.2 Foundation Support

6.2.1 General

As encountered in our test borings, the site is underlain by approximately 7.5 to 15 feet of weak, relatively loose soils that are highly permeable, saturated by the high groundwater conditions at the time of our investigation and has a high potential for liquefaction. These soils are unsuitable for foundation support in their current state. Structure foundations and concrete slabs placed directly upon weak, porous or soils with a potential for liquefaction could undergo damaging differential settlement due to porous soil collapse or loss of shear strength due to liquefaction when loaded in a saturated condition. Foundation-supporting elements must either penetrate through these upper, weak soils, such as cast-in-place drilled piers or spread footings or mat foundation on top of reinforced soils. Our recommendations for drilled piers, spread footings and mat foundation elements are presented below.

6.2.2 Cast-In-Place Drilled Piers

Support for the proposed development can be obtained using a drilled, cast-in-place concrete pier and grade beam foundation system. Piers should be a minimum of 24 inches in diameter and spaced no closer than three pier diameters, center to center. The piers should penetrate a minimum of ten feet into bedrock or drilling refusal, as identified by BAI personnel. The topsoil and weak or soils with a liquefaction potential should be neglected for support (7.5 to 15 feet as observed in our test borings). The average pier depth is anticipated to range from about 18 to 25 feet below the existing ground surface, additional pier lengths should be determined by a structural engineer. Pier depth should be verified in the field by BAI personnel.

For static conditions skin friction can be obtained from the terrace deposits and the underlying bedrock. An allowable skin friction of 100 psf of shaft area in the terrace deposit and 700 psf of shaft area in the bedrock for dead plus live loads.

For dynamic conditions skin friction should be ignored in the terrace deposit, resulting in an allowable skin friction of 700psf of shaft area in the bedrock only for dead plus live loads. For total downward loads, including wind or seismic forces, the pier capacity can be increased by one-third. Uplift frictional capacity for piers should be limited to 2/3 of the allowable downward capacity.

If needed the upper five feet of soil subgraded could be compacted to provide resistance to lateral loads using passive earth pressure of 400psf plus 100psf per foot of depth (trapezoidal distribution). Resistance to lateral loads should be neglected within the weak/liquefaction potential soils. Resistance to lateral loads can be obtained on the bedrock using passive earth pressure of 550psf plus 100psf per foot of depth. Passive pressures can be projected over two pier diameters and should be limited to depths above 7 times the pier diameter.

When final pier depths have been achieved, as determined by BAI in the field, the bottoms of the pier holes should be cleaned of loose material. Final clean out of the pier holes should be observed by BAI. If necessary, pier holes should be dewatered prior to placement of reinforcing steel and concrete. Alternatively, concrete can be trimmed into place with an adequate head to displace water or slurry if groundwater has entered the pier hole. Concrete should not be placed by freefall in such a manner as to hit the sidewalls of the excavation.

Difficult drilling conditions should be anticipated in the hard bedrock. In addition, caving may occur, especially in the terrace sands, during the pier drilling operations. The foundation contractor should be prepared to temporarily case the pier holes, pulling the casing out as the concrete is poured.

6.2.3 Spread Footings

As an alternative, if compacted aggregate soil reinforcement is used, spread footing could be utilized. Spread footings would need to be designed in conjunction with the design of the soil reinforcement. BAI would need to review the soil reinforcement design and foundation system.

6.2.4 Mat

Satisfactory foundation support can be achieved by utilizing a rigid, reinforced concrete mat that is supported on a layer of uniformly compacted fill that is at least five (5) feet thick. A coefficient of subgrade reaction (K) equal to 180 pounds per square inch/inch can be used for the mat design. The mat should be designed to free span a distance of at least 8 feet within the body of the foundation areas, and at least 4 feet at the edges.

The foundation should be designed using an average allowable bearing pressure of 1,000 pounds per square foot (psf), with a localized maximum allowable bearing pressure below and immediately adjacent to columns, load-bearing walls, and edge of slab of 1,500 psf, for dead plus live loads. These allowable bearing pressures may be increased by one-third for short-term wind or seismic loads. The bottom of the mat, or the thickened portions, if used, should be at least 12 inches below lowest adjacent finished grade.

Resistance to lateral loads can be obtained from a combination of passive pressure against the faces of below grade portions of the foundation and friction across the foundation base. Passive pressure equal to 400 psf plus 200 psf per foot of depth below compacted soil subgrade (trapezoidal distribution) can be used. The upper 12 inches should be neglected where not confined by slab or pavement. A base friction coefficient of 0.30 times the net vertical dead load should be used.

Utility line connections at the edge of the mat should be flexible to resist breakage in the event that tilting of the mat or differential settlement occurs.

6.3 Seismic Design Criteria

The proposed structures should be designed and/or constructed to resist the effects of strong ground shaking (on the order of Modified Mercalli Intensity IX) in accordance with current building codes. The California Building Code, 2013 edition, indicates that the site classification for the property is Site Class F, due to the potential for liquefaction. BAI is anticipating that the fundamental period of vibration will be equal to or less than 0.5 seconds, which a site-response analysis is not required in accordance with ASCE 7-05. However, if the structural engineer determines that the fundamental period of vibration is greater than 0.5 seconds, BAI will need to re-evaluate the site and may need to perform a site response analysis. For design purposes BAI is using Site Class D with the following seismic design parameters for the site:

Site Class = D
Mapped Spectral Response Acceleration at 0.2 sec Ss = 1.500g
Mapped Spectral Response Acceleration at 1.0 sec S_1 = 0.620g
Design Spectral Response Acceleration at 0.2 sec S_{DS} = 1.000g
Design Spectral Response Acceleration at 1.0 sec S_{D1} = 0.620g
Seismic Design Category = D

6.4 Concrete Slab Floor Support

If a structural concrete slab is used (i.e., the slab is supported by and able to span between, interconnecting foundation elements without gaining support from underlying soil), then over-excavation of the near-surface weak soil zone is not required, unless upper 5 feet of subgrade needs to be compacted for resist lateral loads on piers. However, topsoils containing organics should be removed beneath the planned slab (as much as four inches to six inches in depth below existing ground surface).

The weak soils in their present condition are not suitable for slab support. Concrete slab-on-grade floors not supported by foundation elements should be supported on properly compacted fill soils placed in accordance with our recommendations previously presented in section 6.1 Site Grading. Note, concrete slabs supported on 5 feet of compacted fill underlined by stabilization fabric could still experience differential settlement due to densification and/or liquefaction settlement on the order of 2 to 3.5 inches.

During foundation and utility trench construction, previously compacted subgrade surfaces may be disturbed. Where this is the case, the subgrade should be moisture conditioned as necessary, and re-rolled to provide a firm, smooth, unyielding surface compacted to at least 90 percent relative compaction before construction of slabs-on-grade.

Concrete slab floors in contact with the ground surface should be underlain by at least four inches of clean, free-draining gravel or crushed rock, graded in size from 1-1/2 or 3/4 inches maximum to 1/4 inches minimum, to act as a capillary moisture break. An underslab drain should be constructed as shown on Plate 25. Where migration of moisture through the slab would be detrimental to its intended use, the installation of a vapor retarder membrane should be considered. The moisture/vapor retarder geomembrane, placed upon the gravel layer, should be at least 15 mils thick (i.e., Stego® Wrap 15-mil Class A, Carlisle RMB 400 15-mil Class A, or equivalent), installed in accordance with the manufacturer's specifications to prevent moisture migration through the seams. With a 15-mil minimum thickness membrane, the 2 inches of wetted sand typically placed upon the membrane may be omitted. Construction of moisture/vapor retarders does not guarantee the prevention of moisture moving through the slab. However, this provision should substantially reduce the potential for moisture-vapor problems on the slab and/or future mold and mildew problems.

6.5 Asphalt Pavement

From soil samples taken at the property and tested for this investigation, we have determined that the near surface soils have an R-value of 36, see Plate 24. For the pavement design at the planned Avalon Inn, we used an R-value of 35, assumed Traffic Index (T.I.) of 5 for driveway and a T.I. of 3 for parking areas, and Caltrans flexible pavement design procedures. Based on the above, we recommend the following asphalt pavement thicknesses:

Table 4: Asphalt Concrete Design Thickness

		Thickness (inches)	
Area	T.I.	Asphalt Concrete Surfacing	Class 2 Aggregate Base
Driveway	5.0	2.5	6.0
Automobile Parking	3.5	2.5	6.0

These thicknesses are the recommended minimums. Increasing asphalt concrete thickness in place of Class 2 Aggregate Base would increase the life and durability of the pavement section, in particular if garbage (or heavy loaded) truck traffic is anticipated.

Class 2 Aggregate Base (AB) should have a minimum R-value of 78 and conform to the requirements contained in Section 26 of Caltrans (State of California) Standard Specifications, latest edition. AB should be placed in thin lifts and a manner to prevent segregation; moisture conditioned to near optimum moisture content, and compacted to at least 95 percent relative compaction to provide a smooth unyielding surface. The upper 6 inches of subgrade soils should be compacted to at least 95 percent relative compaction to provide a smooth, unyielding surfaces. Weak soils within pavement areas should be removed and compacted as described in Section 6.1.2 of this report.

6.6 Retaining Walls

If retaining walls are utilized, retaining or subsurface walls should be provided with permanent back drainage to prevent buildup of hydrostatic pressure. Drainage and backfill details are presented on Plate 26. Quality, placement and compaction requirements for backfill behind subsurface walls are the same as previously presented for select fill. Light compacting equipment should be used near the wall to avoid overstressing the walls.

Retaining walls should be designed to resist the lateral earth pressures presented on Plate 27. These pressures do not consider additional loads resulting from adjacent foundations, vehicles, or other downward surcharge loads. BAI can provide consultation regarding surcharge loads, if needed.

In addition to static loads, the retaining walls should also be designed to resist potential seismic loads, in accordance with new California Building Code requirements. For seismic loads, a pressure increment equivalent to an inverted triangular distribution is recommended, varying from 0 (zero) pounds per square foot (psf) at the bottom of the wall to 18H psf at the top of the embedded portion, where "H" is the height of the embedded portion (resultant dynamic thrust act at 0.6H above the base of the wall). The resultant distribution of both static and seismic pressures will thus be trapezoidal.

6.7 Swimming Pool Walls

The swimming pool excavations may encounter temporary seepage and even groundwater, especially during the winter-spring months. If dewatering is necessary, it can likely be accomplished by conventional pumping. However, installation of gravel drain blankets, geotextile filter fabrics, and sumps (at the contractor's option) will facilitate dewatering, and provide a reasonably dry working surface.

Pool walls should be designed using lateral earth pressures presented on Plate 27. The subsurface swimming pool walls and the pool bottom should be provided with permanent drainage to prevent buildup of hydrostatic pressure. Swimming pool drainage and backfill details are presented on Plate 28. Quality, placement and compaction requirements for backfill behind subsurface walls are the same as previously presented for select fill. Light compacting equipment should be used to avoid overstressing the walls.

Where the bottom of the pool is greater than about three feet below existing grade, we recommend that an under-slab drain as indicated on Plate 28 be provided. If shallow groundwater is present, the pool will be subject to "popping up" when empty. Therefore, drainage relief measures should be incorporated to reduce the buoyancy of the pool when empty. Drainage relief could consist of a six-inch thick blanket of drain rock (graded between the No. 4 and 1-inch sieve) wrapped in geotextile filter fabric placed on the pool excavation bottom, and two hydrostatic release "pop-up" valves placed at the deep end of the pool, as shown on Plate 28. Other drainage alternatives may be acceptable; BAI should review alternate drainage measures.

6.8 Site Drainage

Because surface and/or subsurface water is often the cause of foundation or slope stability problems, care should be taken to intercept and divert concentrated surface flows and subsurface seepage away from the building foundations and the neighboring bluff edge property. Roof runoff water should be directed away from the buildings and dispersed, as much as practical, across the lot. Drainage across the lot should be by sheet-flow. Surface grades should maintain a recommended five percent gradient away from building foundations. As mentioned in Section 3.0 of this report, water from the southerly adjacent property is currently being discharged near existing building foundations. BAI recommends drainage improvements as described herein be implemented during grading and construction, to mitigate the potentially problematic drainage situation.

If raised wood floors are used, the area under the floor should be graded to drain towards an under-building drain or sump with a conduit outlet(s) through or beneath the grade beams. Two-inch or four-inch PVC sleeves or equivalent should be placed within the forms, at or slightly below ground level, prior to concrete placement.

7.0 ADDITIONAL SERVICES

Prior to construction, BAI should review the final grading and foundation plans, and soil related specifications for conformance with our recommendations. During construction, BAI should be retained to provide periodic observations, together with the appropriate field and laboratory testing during site preparation, subdrain installations, and foundation excavations (pier drilling). Foundation excavations should be reviewed by BAI while the excavation operations are being performed. Our reviews and tests would allow us to check that the work is being performed in accordance with project guidelines, confirm that the soil and rock conditions are as anticipated, and to modify our recommendations, if necessary.

8.0 LIMITATIONS

This geotechnical investigation and engineering geologic reconnaissance of the ocean bluff property were performed in accordance with the usual and current standards of the profession, as they relate to this and similar localities. No other warranty, expressed or implied, is provided as to the conclusions and professional advice presented in this report. Our conclusions are based upon reasonable geological and engineering interpretation of available data.

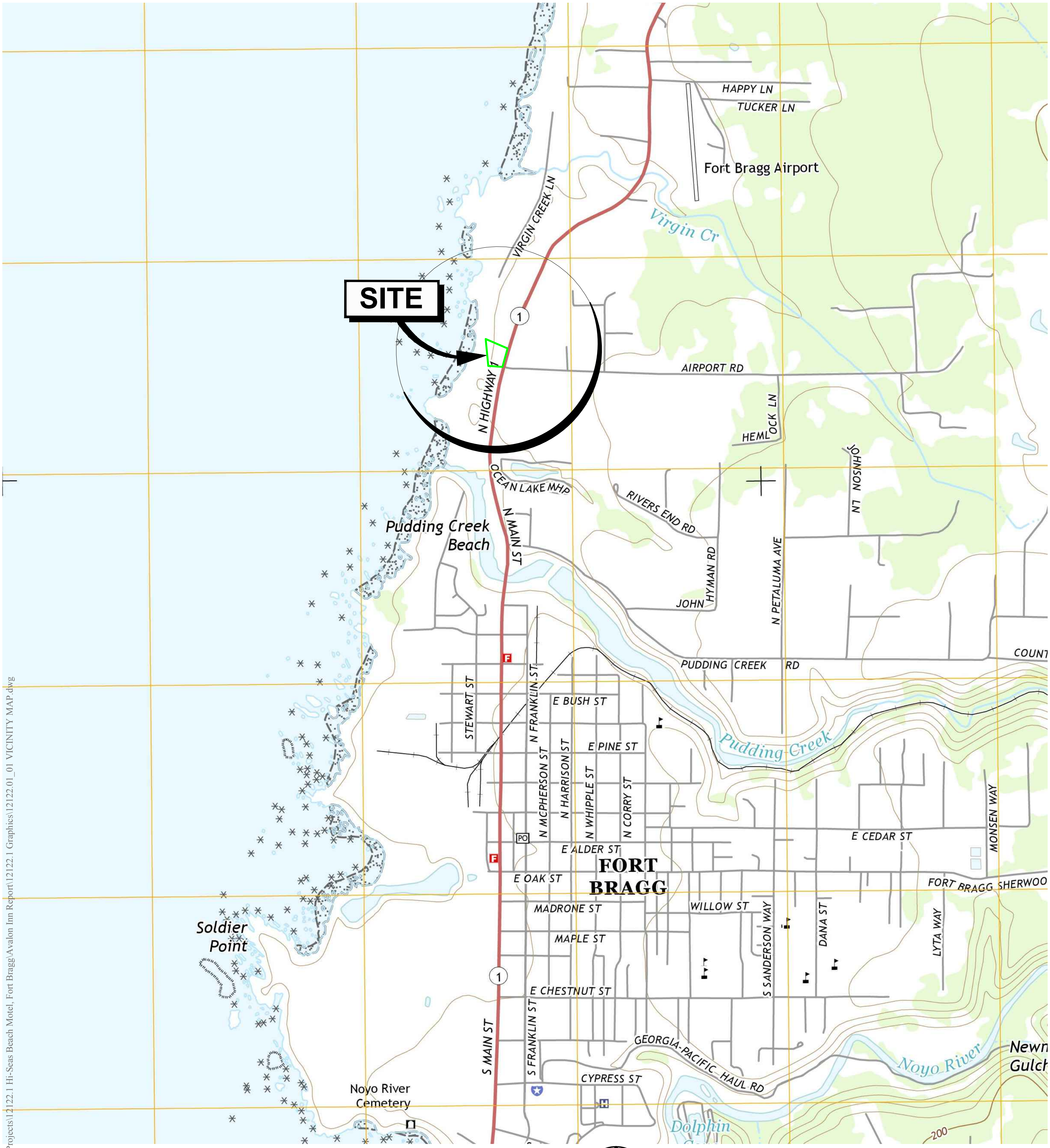
The samples taken and tested, and the observations made, are considered to be representative of the site; however, soil and geologic conditions may vary significantly between test borings and across the site. As in most projects, conditions revealed during construction excavation may be at variance with preliminary findings. If this occurs, the changed conditions must be evaluated by BAI, and revised recommendations be provided as required.

This report is issued with the understanding that it is the responsibility of the Owner, or his/her representative, to insure that the information and recommendations contained herein are brought to the attention of all other design professionals for the project, and incorporated into the plans, and that the Contractor and Subcontractors implement such recommendations in the field. The safety of others is the responsibility of the Contractor. The Contractor should notify the owner and BAI if he/she considers any of the recommended actions presented herein to be unsafe or otherwise impractical.

Changes in the condition of a site can occur with the passage of time, whether they are due to natural events or to human activities on this, or adjacent sites. In addition, changes in applicable or appropriate codes and standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, this report may become invalidated wholly or partially by changes outside of our control. Therefore, this report is subject to review and revision as changed conditions are identified.

The recommendations contained in this report are based on certain specific project information regarding type of construction and building location, which have been made available to us. If conceptual changes are undertaken during final project design, we should be allowed to review them in light of this report to determine if our recommendations are still applicable.

ILLUSTRATIONS



REFERENCE:
Fort Bragg Quadrangle
7.5-Minute Series, USGS, 2015

Latitude: 39.463676
Longitude: -123.806122

NORTH

APPROXIMATE SCALE (FEET)
0 1000 2000 4000

L:\Geotech Projects\12122.1 Hi-Seas Beach Motel, Fort Bragg\Avalon Inn Report\12122.1 Graphics\12122.01_01 VICINITY MAP.dwg

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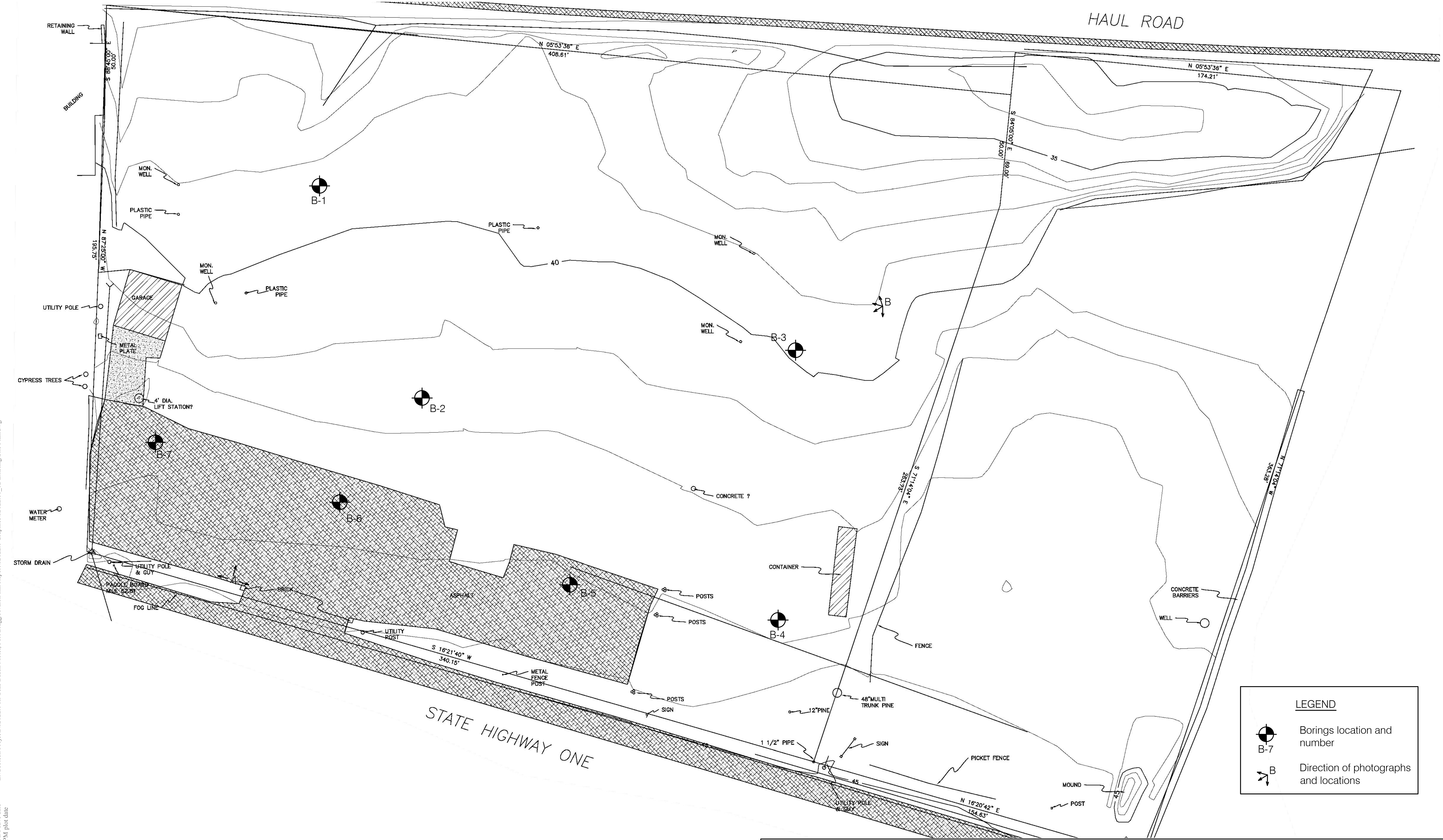
Santa Rosa, California 95403
Tel: (707) 528-6108

Job No.: 12122.01
Appr.: EEO
Date: 12/04/15

VICINITY MAP
THE AVALON INN
1201 North Main Street
Fort Bragg, California

PLATE
1



APPROXIMATE SCALE (FEET)

0 20 40 80

NORTH

REFERENCE:

Topographic Map, Sheet 1 of 1, prepared by Forrest Francis, Land Surveyor, dated 8/2014.

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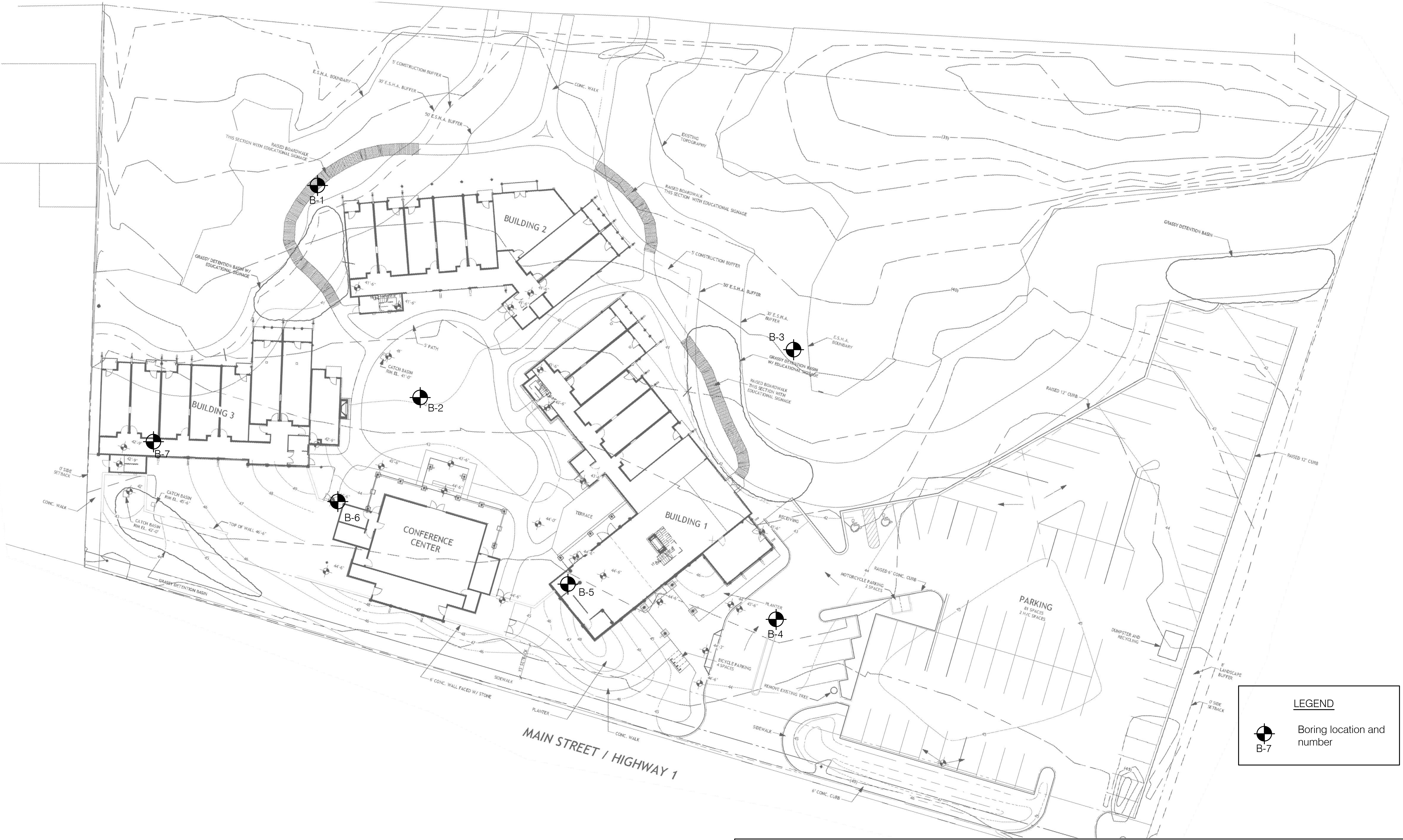
EXISTING SITE PLAN
THE AVALON INN
1201 North Main Street
Fort Bragg, California

PLATE
2



LEGEND

B-7 Boring location and number

APPROXIMATE SCALE (FEET)

0 20 40 80

NORTH

REFERENCE:

Schematic Design Set, Sheet A2-1, prepared by Epikos Land Planning + Architecture, dated 8/17/15.

Brunsing Associates, Inc. 5468 Skylane Blvd., Suite 201 Santa Rosa, California 95403 Tel: (707) 528-6108	Job No.: 12122.01 Appr.: EEO Date: 12/04/15	**PLANNED SITE PLAN** THE AVALON INN 1201 North Main Street Fort Bragg, California	**PLATE** **3**

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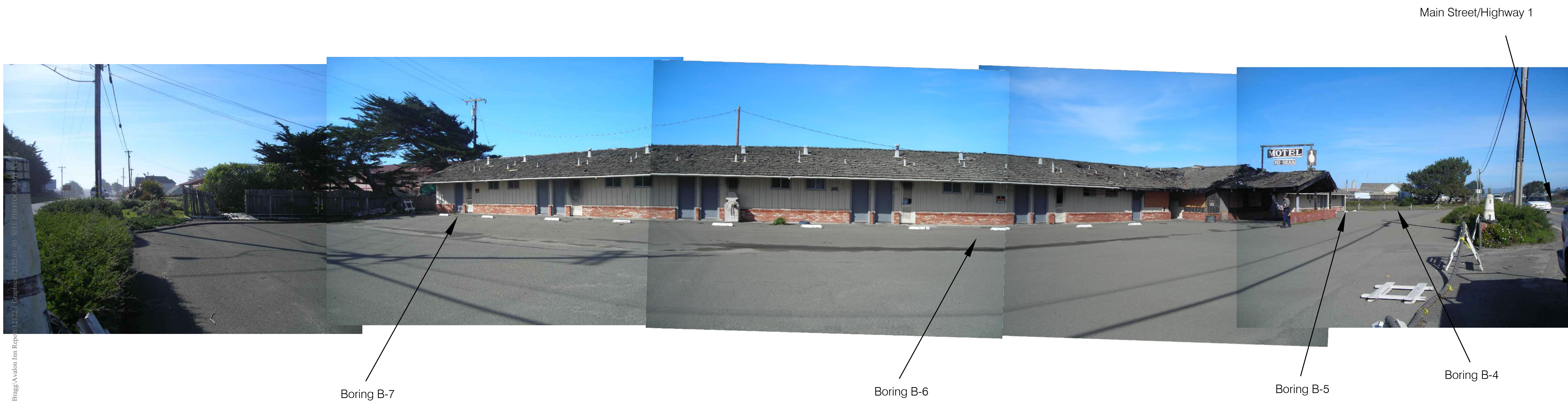
Date: 12/04/15

COASTLINE OBLIQUE AERIAL PHOTOGRAPH
2005 WITH GEOLOGIC MAPPING
THE AVALON INN
1201 North Main Street
Fort Bragg, California

PLATE
4

SITE PHOTOGRAPH A

Panoramic of hotel front, 2/8/08.



L:\Geotech Projects\12122.1 Hi-Seas Beach Motel, Fort Bragg\Avalon Inn Report\12122.1 Graphics\12122.01_05_SITE PHOTOGRAPH A

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Brunsing Associates, Inc. 5468 Skylane Blvd., Suite 201 Santa Rosa, California 95403 Tel: (707) 528-6108	Job No.: 12122.01 Appr.: EEO Date: 12/04/15	**SITE PHOTOGRAPH A** THE AVALON INN 1201 North Main Street Fort Bragg, California	**PLATE** **5**

SITE PHOTOGRAPH B

Looking east, south, and west; panoramic of rear property, 2/8/08.

12/4/2015 9:26:40 AM save date
12/8/2015 2:47:06 PM plot date

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SITE PHOTOGRAPH C

Bluffs and beach northwest of the site, 3-19-08.
See Plate 4 for photograph location.

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SITE PHOTOGRAPH C
THE AVALON INN
1201 North Main Street
Fort Bragg, California

PLATE
7

SITE PHOTOGRAPH D

Bluffs and beach west of the site, 3-19-08.
See Plate 4 for photograph location.

L:\Geotech Projects\12122.1 Hi-Seas Beach Motel, Fort Bragg\Avalon Inn Report\12122.1 Graphics\12122.01_08 SITE PHOTOGRAPH D.dwg

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SITE PHOTOGRAPH D
THE AVALON INN
1201 North Main Street
Fort Bragg, California

PLATE
8

SITE PHOTOGRAPH E

Bluffs and beach southwest of the site, 3-19-08.
See Plate 4 for photograph location.

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Job No.: 12122.01
Appr.: EEO
Date: 12/04/15

SITE PHOTOGRAPH E
THE AVALON INN
1201 North Main Street
Fort Bragg, California

PLATE
9

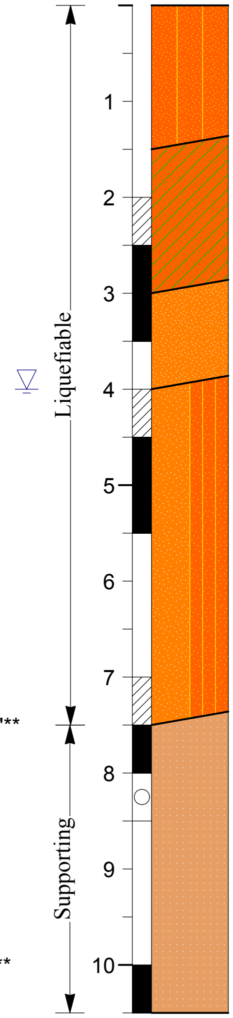
Log of Boring B- 1

Equipment: Mobile B-53; 7-inch hollow-stem auger

Date: 2/20/08 to 2/20/08 **Logged By:** SCL

Elevation: 40.5 feet *** **Latitude:** 39.463446 **Longitude:** -12.3806473

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)	Description
					0	DARK BROWN SILTY SAND WITH CLAY (SM) loose to medium dense, wet, with rottlets
					1.5	LIGHT GRAY AND ORANGE CLAYEY SAND (SC) medium dense, wet
	CA			15**	3	LIGHT GRAY AND ORANGE SAND (SP) medium dense, wet
					4	LIGHT GRAY SAND (SP-SM) medium dense, saturated
5% Passing #200		18.2	109		4.5	
	CA			15**	5	
	CA			45/9.5"**	7.5	BROWN-ORANGE SANDSTONE close to intense fracturing, modrate hardness to hard, moderate to deep weathering; red-stained fractures; white mica flakes
					9	becomes very hard at 9-9.5 feet
	CM			52/6"**	10	



NOTES:
1. Water encountered at 4.0 feet bgs during drilling
2. Caving to 3.0 feet after augers removed
3. Practical drilling refusal at 10 feet bgs
4. Backfilled with spoils at 11:30 am

* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Topographical Survey by Clifford Zimmerman, dated 11/1998.

Scale: 1" = 2'

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Job No.: 12122.01
Appr.: KAC
Date: 12/04/15

LOG OF BORING B- 1
THE AVALON INN
1201 North Main Street
Fort Bragg, California

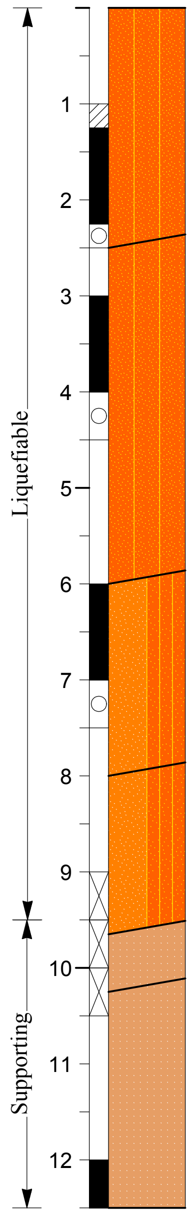
Log of Boring B- 2

Equipment: Mobile B-53; 7-inch hollow-stem auger

Date: 2/20/08 to 2/20/08 **Logged By:** SCL

Elevation: 42.5 feet *** **Latitude:** 39.463551 **Longitude:** -123.806174

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)
	CA			4**	2
	CA			15**	4
7% Passing #200		18.3	110		6
99% Passing #4					
DS 510(720)		19.3	109		
	CA			17**	7
	SPT			34	10
Tx 1449 (1440)	CM	9.3	81	64/6"***	12



DARK BROWN MOTTLED WITH ORANGE SILTY SAND (SM)
very loose, moist to wet, abundant rootlets; porous, less orange mottling below 1.5 feet bgs.

LIGHT GRAY TO ORANGE SILTY SAND WITH CLAY (SM)
medium dense, wet to saturated; orange color more intense below 3.5 feet

GRAY SAND (SP-SM)
occasional orange mottling, medium dense, saturated

very easy drilling

BROWN-MULTICOLORED SAND (SP-SM)
medium dense, saturated, medium to coarse grained

ORANGE-GRAY SANDSTONE
fracturing not visible, moderate hardness, deep weathering; white mica flakes visible

DARK GREENISH-GRAY SANDSTONE
close fracturing, moderate hardness, moderate to little weathering; white mica flakes visible

becomes very hard and little weathered to fresh at 12 feet

NOTES:
1. Water encountered at 3.5 feet bgs during drilling
2. Water level at 3.5 feet bgs 5.5 hours after drilling
3. No caving
4. Practical drilling refusal at 12.0 feet bgs
5. Backfilled with spoils at 5:30 p.m.

* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Topographical Survery by Clifford Zimmerman, dated 11/1998.

Scale: 1" = 2'

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Job No.: 12122.01
Appr.: KAC
Date: 12/04/15

LOG OF BORING B- 2
THE AVALON INN
1201 North Main Street
Fort Bragg, California

PLATE 11
SHEET 1 of 1

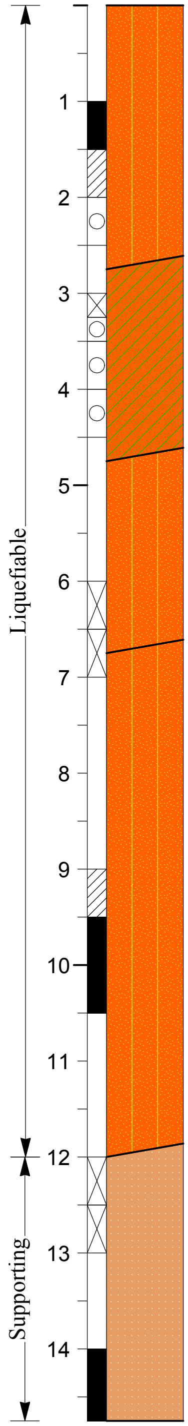
Log of Boring B- 3

Equipment: Mobile B-53; 7-inch hollow-stem auger

Date: 2/20/08 to 2/20/08 **Logged By:** SCL

Elevation: 40.5 feet *** **Latitude:** 39.464015 **Longitude:** -123.806369

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)
	CA			4**	2
	CA			5**	4
16% Passing #200 97% Passing #4	SPT			7	6.5
	SPT	19.7	108	7	10
	SPT			83	12.5
Tx 2178 (1728)		11.3	129		14
Tx 6357 (2016)	CM	11.4	125	38**	14.5



DARK BROWN SILTY SAND WITH CLAY (SM)
very loose, wet, porous, with abundant rootlets

MOTTLED DARK BROWN AND GRAY CLAYEY SAND (SC)
very loose to loose, wet, porous, with rootlets

BROWN-GRAY MULTICOLORED SILTY SAND (SM)
loose, saturated, medium grained sands

BRIGHT RED-ORANGE SILTY SAND WITH CLAY (SM)
loose, saturated; medium-grained sands

harder drilling at 12 feet

ORANGE-BROWN SANDSTONE
close fracturing, hard, deep weathering; white mica flakes visible, carbonate coating on fractures

NOTES:
1. Water encountered at apprximately 5 feet bgs. during drilling, rose to 2.5 feet bgs. 1/2 hour later
2. Caving/sloughing to approximately 4.5 feet bgs
3. Practical drilling refusal at 14.2 feet bgs
4. Backfilled with spoils at 1:40 pm.

* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Topographical Survery by Clifford Zimmerman, dated 11/1998.

Scale: 1" = 2'

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Tel: (707) 528-6108

Job No.: 12122.01
Appr.: KAC
Date: 12/04/15

LOG OF BORING B- 3
THE AVALON INN
1201 North Main Street
Fort Bragg, California

PLATE 12
SHEET 1 of 1

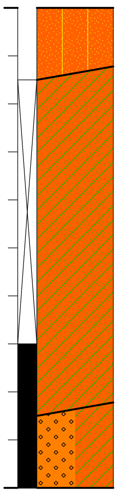
Log of Boring B- 4

Equipment: Mobile B-53; 7-inch hollow-stem auger

Date: 2/20/08 to 2/20/08 **Logged By:** SCL

Elevation: 43.5 feet *** **Latitude:** 39.464014 **Longitude:** -123.805801

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)	Sample
LL=25,PI=5 R-valve = 36					1	
					2	
					3	
					4	
6% Passing #200 94% Passing #4	CA	19.6	105	24**	5	



DARK BROWN SILTY SAND (SM)
loose, porous, moist, abundant rootlets

DARK BROWN CLAYEY SAND (SC)
loose to medium dense, wet

DARK BROWN-GRAY WELL GRADED SAND WITH CLAY (SW-SC)
medium dense, wet; gravels and sands are coated black, rounded

NOTES:
1. No free water encountered
2. No caving
3. Backfilled with spoils at 2 pm.

* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Topographical Survery by Clifford Zimmerman, dated 11/1998.

Scale: 1" = 2'

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Job No.: 12122.01
Appr.: KAC
Date: 12/04/15

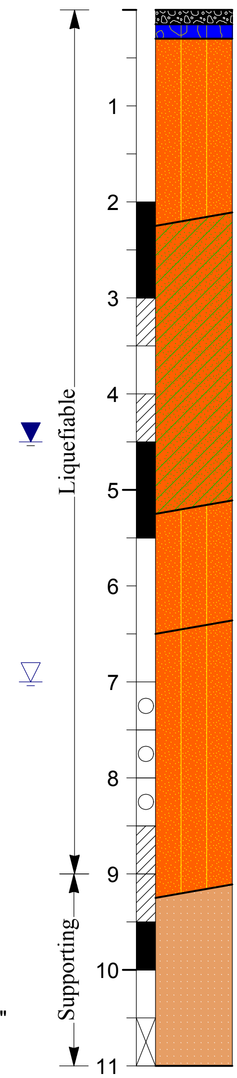
LOG OF BORING B- 4
THE AVALON INN
1201 North Main Street
Fort Bragg, California

PLATE 13
SHEET 1 of 1

Log of Boring B- 5

Equipment: Mobile B-53; 7-inch hollow-stem auger
Date: 2/20/08 to 2/20/08 **Logged By:** SCL
Elevation: 43.8 feet *** **Latitude:** 39.46373 **Longitude:** -123.805844

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot
	CA			6**
DS 483(576)		25.5	98	
	CA			8**
	CA			6**
	SPT	10.9	128	47
	SPT			50/6"



ASPHALT, 2 inches
BROWN-GRAY SILTY SANDY GRAVEL (GM)
medium dense, moist, 2.5 inches (baserock)
DARK GRAY WITH RUST PATCHES SILTY SAND (SM)
loose, moist to wet

LIGHT GRAY AND ORANGE CLAYEY SAND (SC)
loose, wet

GRAY SILTY SAND (SM)
loose, wet

BROWN-GRAY-MULTICOLORED SILTY SAND (SM)
loose, saturated

driller reports bit chatter at 9.5 feet
ORANGE-BROWN TO GRAY SANDSTONE
moderate to close fracturing, moderate hardness, moderate to deep weathering, some carbonate-filled fractures
very hard drilling at 10 feet

becomes intensely fractured, hard, and little weathered near 11 feet

NOTES:
1. Water encountered at approximate 7 feet bgs during drilling, rose to 4.5 feet half hour after drilling
2. No caving
3. Practical drilling refusal at 10.5 feet bgs
4. Backfilled with spoils, asphalt patched at 3 p.m.

BORING LOG 1 PER PAGE, 12122.01 GINT.GPJ, 12/8/15

* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Topographical Survery by Clifford Zimmerman, dated 11/1998.

Scale: 1" = 2'

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Date: 12/04/15

LOG OF BORING B- 5
THE AVALON INN
1201 North Main Street
Fort Bragg, California

PLATE 14

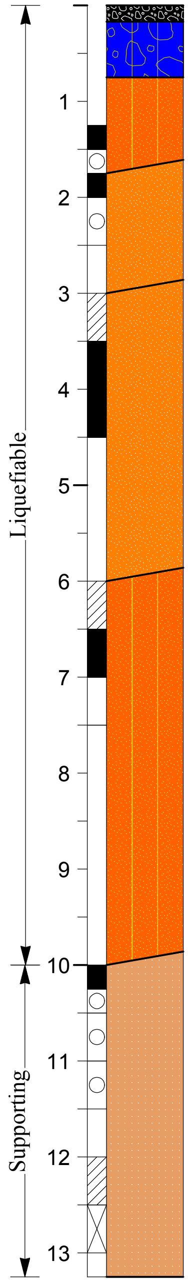
Log of Boring B- 6

Equipment: Mobile B-53; 7-inch hollow-stem auger

Date: 2/20/08 to 2/20/08 **Logged By:** SCL

Elevation: 43.5 feet *** **Latitude:** 39.463417 **Longitude:** -123.805968

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)
	CA			6**	2
	SPT	27.8	95	2	4
DS 650(864)	CA	17.3	110	8**	7
	CA			13**	11
	CM			55/8"***	12.5



ASPHALT, 2 inches

BROWN-GRAY SILTY SANDY GRAVEL (GM)
medium dense, moist, 7 inches (baserock)

DARK BROWN SILTY SAND (SM)
loose, moist, with rootlets

GRAY-BROWN SAND (SP)
loose, moist

GRAY-BROWN SAND (SP)
very loose, becomes wet and saturated at about 3.5-4 feet, some rootlets

LIGHT GRAY SILTY SAND (SM)
loose, wet to saturated

DARK GRAY SANDSTONE
close fracturing, moderate hardness, little to moderate weathering, mica flakes visible

becomes occasionally fractured, hard, and little weathered near 12 feet

NOTES:
1. Water ecountered at about 4.5 feet bgs during drilling
2. Practical drilling refusal at 12 feet bgs
4. Backfilled with spoils, asphalt patched at 3:45 pm.

* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Topographical Survey by Clifford Zimmerman, dated 11/1998.

Scale: 1" = 2'

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LOG OF BORING B- 6
THE AVALON INN
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PLATE 15

SHEET 1 of 1

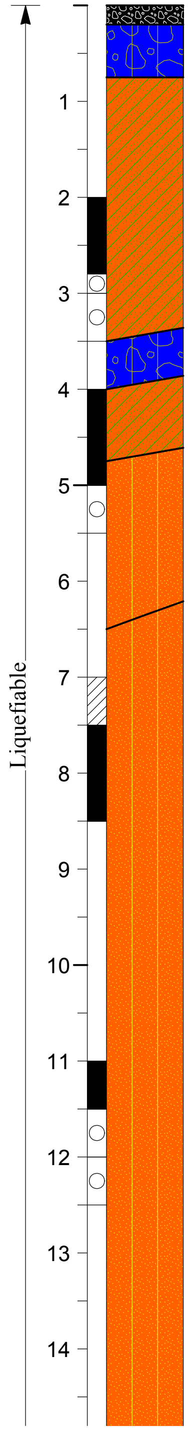
Log of Boring B- 7

Equipment: Mobile B-53; 7-inch hollow-stem auger

Date: 2/20/08 to 2/20/08 **Logged By:** SCL

Elevation: 43.5 feet *** **Latitude:** 39.463275 **Longitude:** -123.806113

Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)
CA			5**	3
CA			8**	5
CA			10**	8
	17.9	111		11
CM			9**	12



ASPHALT, 2.5 inches

BROWN SILTY SANDY GRAVEL (GM)
medium dense, moist, 6 inches (base rock)

MOTTLED ORANGE AND DARK BROWN CLAYEY SAND (SC)
loose, wet, slight chemical or foul odor; lateral streaks of orange and light gray

BROWN SILTY SANDY GRAVEL (GM)
loose, saturated; large rounded gravels

LIGHT GRAY AND ORANGE CLAYEY SAND (SC)
loose, saturated

LIGHT GRAY SILTY SAND (SM)
loose, saturated

ORANGE-TAN SILTY SAND (SM)
loose, saturated

same as above with lenses of coarse grained sands

* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Topographical Survery by Clifford Zimmerman, dated 11/1998.

Scale: 1" = 2'

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LOG OF BORING B- 7
THE AVALON INN
1201 North Main Street
Fort Bragg, California

PLATE 16
SHEET 1 of 2

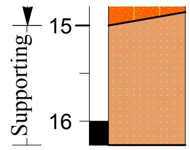
Log of Boring B- 7

Equipment: Mobile B-53; 7-inch hollow-stem auger

Date: 2/20/08 to 2/20/08 **Logged By:** SCL

Elevation: 43.5 feet *** **Latitude:** 39.463275 **Longitude:** -123.806113

Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)	Sample	Description
				15		ORANGE-BROWN SANDSTONE close fracturing, hard to very hard, deep weathering, mica flakes visible
CM			16**	16		



NOTES:
1. Water encountered at approximately 2.5 feet bgs during drilling
2. Water level at 2.5 feet bgs one hour after drilling
3. Some caving in sand layers
4. Practical drilling refusal at 16 feet
5. Backfilled with spoils, asphalt patched at 5:10 pm.

* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Topographical Survery by Clifford Zimmerman, dated 11/1998.

Scale: 1" = 2'

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LOG OF BORING B- 7
THE AVALON INN
1201 North Main Street
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PLATE 16

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE-GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (Less than 5% fines)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (Greater than 12% fines)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS 50% OR MORE OF COARSE FRACTION PASSING THROUGH NO. 4 SIEVE	CLEAN SANDS (Less than 5% fines)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES (Greater than 12% fines)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE-GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILT, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMOUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

KEY TO TEST DATA

LL - Liquid Limit
PI - Plasticity Index
Sample Retained
Sample Recovered, Not Retained
Bulk Sample
Sample Not Recovered
CA - California Modified Split Barrel Sampler 3.0-inch O.D.
CM - California Modified Split Barrel Sampler 2.5-inch O.D.
SPT - California Split Barrel Sampler 2.0-inch O.D.
SH - Shelby Tube
RC - Rock Coring
Recovery - Percent Core Recovered
RQD - Rock Quality Designation (length of core pieces >= 4-inches / core length)

Consol - Consolidation
EI - Expansion Index
SA - Sieve Analysis

Shear Strength, psf **Confining Pressure, psf**

Tx	320 (2600)	- Unconsolidated Undrained Triaxial
TxCU	320 (2600)	- Consolidated Undrained Triaxial
DS	2750 (2600)	- Consolidated Drained Direct Shear
FVS	470	- Field Vane Shear
UC	2000	- Unconfined Compression
PP	2000	- Field Pocket Penetrometer
Sat		- Sample saturated prior to test

Groundwater Level Reading
Second Groundwater Level Reading

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SOIL CLASSIFICATION CHART & KEY TO TEST DATA
THE AVALON INN
1201 North Main Street
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PLATE 17

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	Standard Penetration Test Blow Count (blows per foot)
Very loose	4 or less
Loose	5 to 10
Medium dense	11 to 30
Dense	31 to 50
Very dense	More than 50

CONSISTENCY OF FINE-GRAINED SOILS

Consistency	Identification Procedure	Approximate Shear Strength (psf)
Very soft	Easily penetrated several inches with fist	Less than 250
Soft	Easily penetrated several inches with thumb	250 to 500
Medium stiff	Penetrated several inches by thumb with moderate effort	500 to 1000
Stiff	Readily indented by thumb, but penetrated only with great effort	1000 to 2000
Very stiff	Readily indented by thumb nail	2000 to 4000
Hard	indented with difficulty by thumb nail	More than 4000

NATURAL MOISTURE CONTENT

Dry	No noticeable moisture content. Requires considerable moisture to obtain optimum moisture content* for compaction.
Damp	Contains some moisture, but is on the dry side of optimum.
Moist	Near optimum moisture content for compaction.
Wet	Requires drying to obtain optimum moisture content for compaction.
Saturated	Near or below the water table, from capillarity, or from perched or ponded water. All void spaces filled with water.

* Optimum moisture content as determined in accordance with ASTM Test Method D1557, latest edition.

Where laboratory test data are not available, the above field classifications provide a general indication of material properties; the classifications may require modification based upon laboratory tests.

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SOIL DESCRIPTIVE PROPERTIES
THE AVALON INN
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PLATE 18

Generalized Graphic Rock Symbols

Claystone	Siltstone	Tuff (Volcanic Ash)
Shale	Chert	Andesite
Sandstone	Serpentine	Basalt
Conglomerate	Greenstone	Schist

Stratification

Bedding of Sedimentary Rocks	Thickness of Beds
Massive	No apparent bedding
Very thick bedded	Greater than 4 feet
Thick bedded	2 feet to 4 feet
Thin bedded	2 inches to 2 feet
Very thin bedded	0.5 inches to 2 inches
Laminated	0.125 inches to 0.5 inches
Thinly laminated	less than 0.125 inches

Fracturing

Fracturing Intensity	Fracture Spacing
Little	Greater than 4 feet
Occasional	1 foot to 4 feet
Moderate	6 inches to 1 foot
Close	1 inch to 6 inches
Intense	0.5 inches to 1 inch
Crushed	less than 0.5 inches

Strength

Soft	Plastic or very low strength.
Friable	Crumbles by hand.
Low hardness	Crumbles under light hammer blows.
Moderate hardness	Crumbles under a few heavy hammer blows.
Hard	Breaks into large pieces under heavy, ringing hammer blows.
Very hard	Resists heavy, ringing hammer blows and will yield with difficulty only dust and small flying fragments.

Weathering

Deep	Moderate to complete mineral decomposition, extensive disintegration, deep and thorough discoloration, many extensively coated fractures.
Moderate	Slight decomposition of minerals, little disintegration, moderate discoloration, moderately coated fractures.
Little	No megascopic decomposition of minerals, slight to no effect on cementation, slight and intermittent, or localized discoloration, few stains on fracture surfaces.
Fresh	Unaffected by weathering agents, no disintegration or discoloration, fractures usually less numerous than joints.

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ROCK DESCRIPTIVE PROPERTIES
THE AVALON INN
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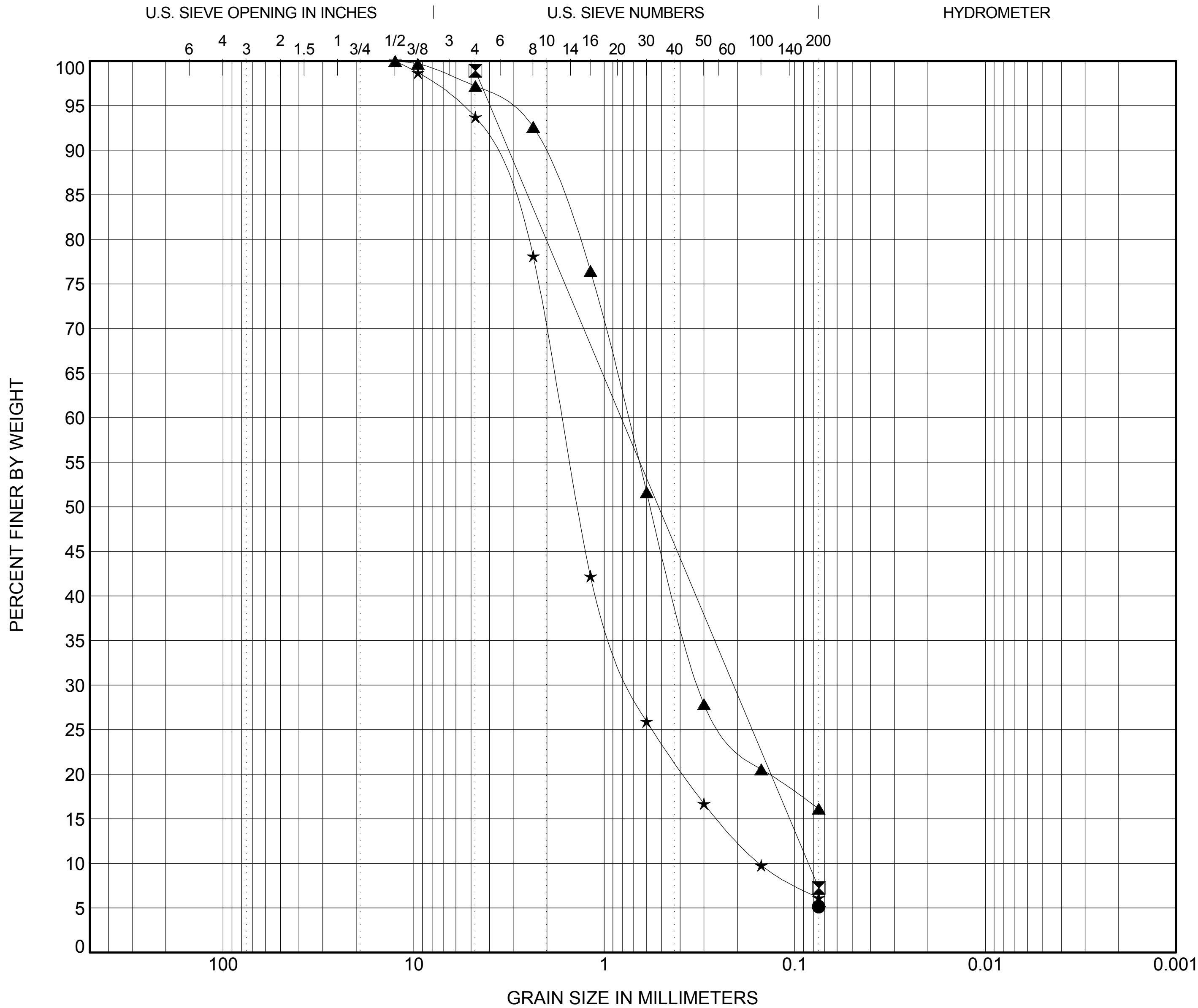

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● B- 1	4.5 ft	**LIGHT GRAY SAND (SP-SM)**					
⧗ B- 2	6.0 ft	**GRAY SAND (SP-SM)**				**0.64**	**9.61**
▲ B- 3	6.0 ft	**BROWN-GRAY MULTICOLORED SILTY SAND (SM)**					
★ B- 4	4.5 ft	**DARK BROWN-GRAY WELL GRADED SAND WITH CLAY (SW-SC)**				**1.98**	**10.85**

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● B- 1	4.5 ft	**0.075**						**5.1**	
⧗ B- 2	6.0 ft	**4.75**	**0.817**	**0.21**	**0.085**		**91.7**	**7.2**	
▲ B- 3	6.0 ft	**12.5**	**0.754**	**0.319**		**2.8**	**81.1**	**16.1**	
★ B- 4	4.5 ft	**12.5**	**1.664**	**0.711**	**0.153**	**6.3**	**87.6**	**6.1**	

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GRAIN SIZE DISTRIBUTION
THE AVALON INN
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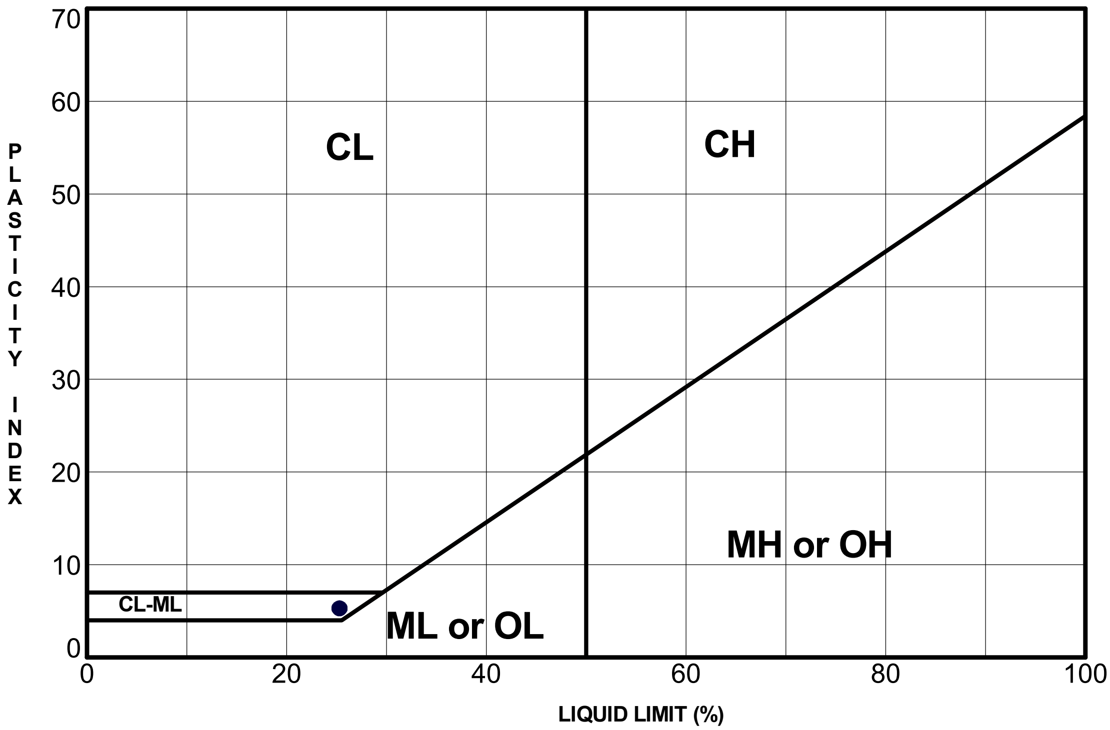
PLATE 20



SYMBOL	CLASSIFICATION AND SOURCE	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX	% PASSING No. 200 SIEVE
●	**DARK BROWN CLAYEY SAND (SC)** **B- 4 @ 1.0 feet**	**25**	**20**	**5**	

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ATTERBERG LIMITS TEST RESULTS
THE AVALON INN
1201 North Main Street
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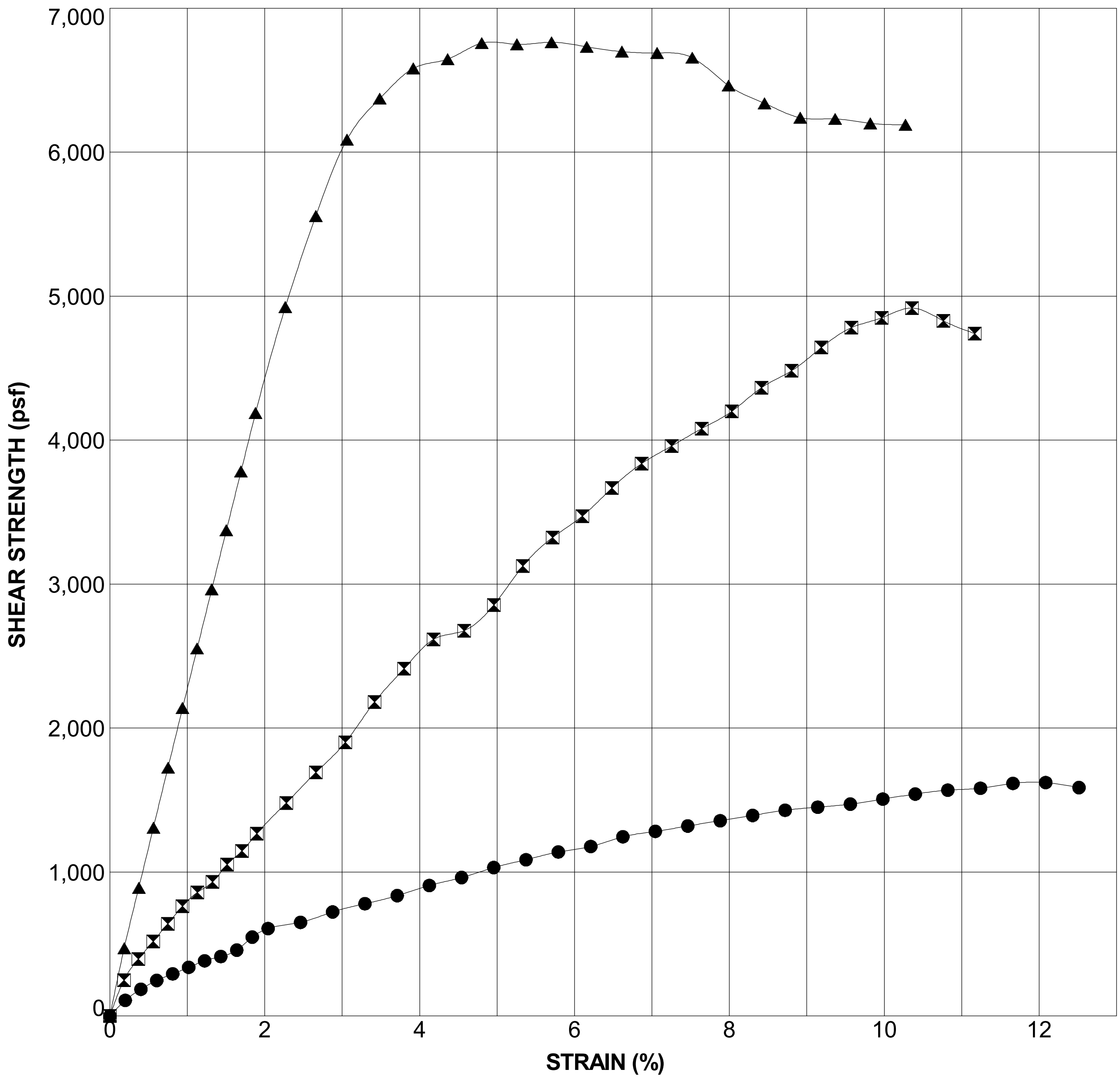
PLATE 21



Sample Source	Classification	Confining Pressure (psf)	Yield Strength (psf)	Strain (%)	Dry Density (pcf)	Moisture Content (%)
● B- 2 at 12 ft	DARK GREENISH-GRAY SANDSTONE	1440	1449	4.1	81	9.3
⧗ B- 3 at 14 ft	ORANGE-BROWN SANDSTONE	1728	2178	3.4	129	11.3
▲ B- 3 at 14.5 ft	ORANGE-BROWN SANDSTONE	2016	6357	3.5	125	11.4

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UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

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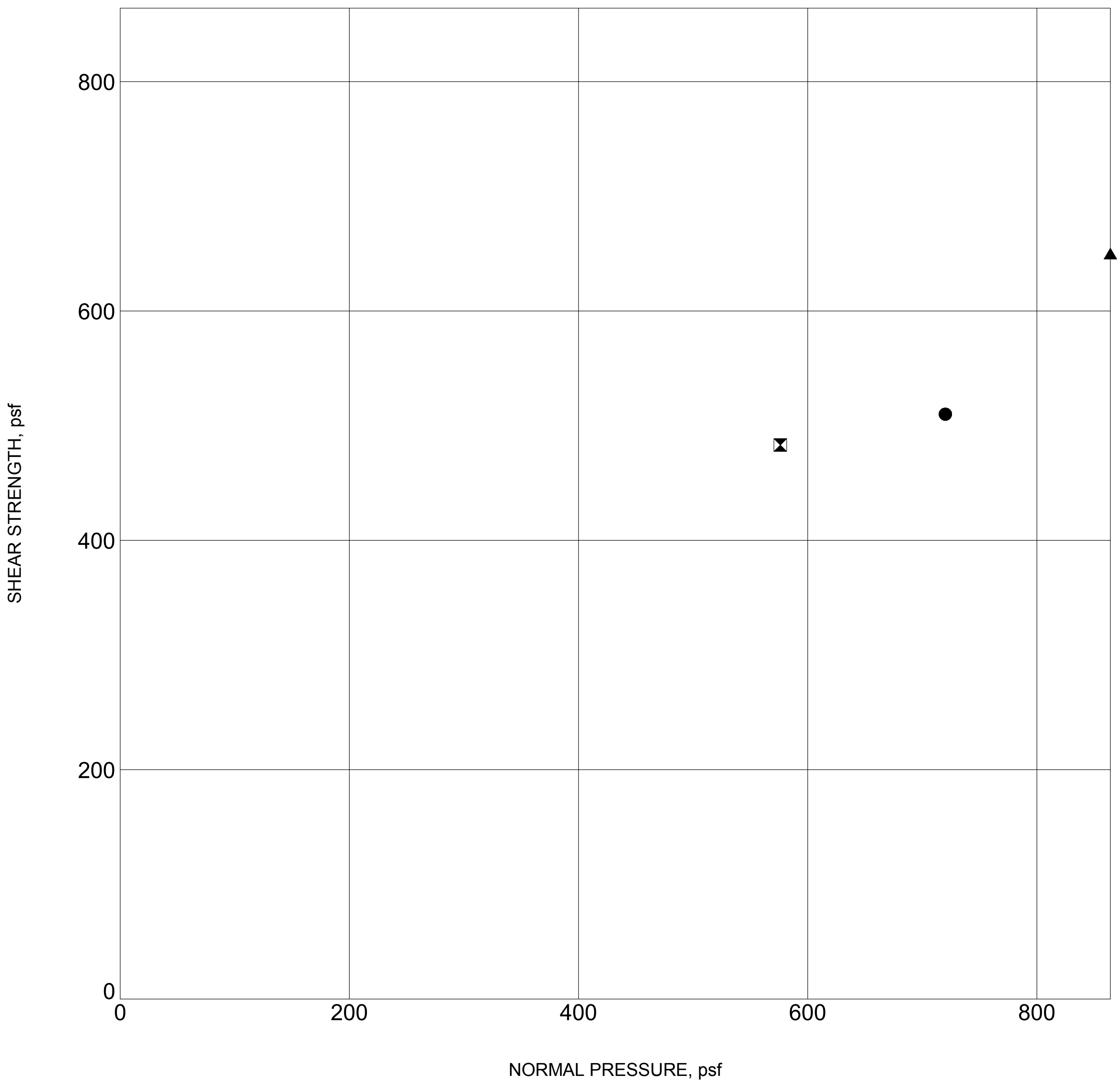
PLATE 22



Sample Source		Classification		γ_d	MC%	c	ϕ (degree)
●	**B- 2 at 6.5 ft**	**GRAY SAND (SP-SM)**	**Initial**	**109**	**19.3**	**0**	**35**
			After	**118**	**20.8**		
⧗	**B- 5 at 4.5 ft**	**LIGHT GRAY AND ORANGE CLAYEY SAND (SC)**	**Initial**	**95**	**25.5**	**0**	**40**
			After	**96**	**26.2**		
▲	**B- 6 at 7.0 ft**	**LIGHT GRAY SILTY SAND (SM)**	**Initial**	**110**	**17.3**	**0**	**37**
			After	**106**	**19.6**		

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DIRECT SHEAR TEST RESULTS
THE AVALON INN
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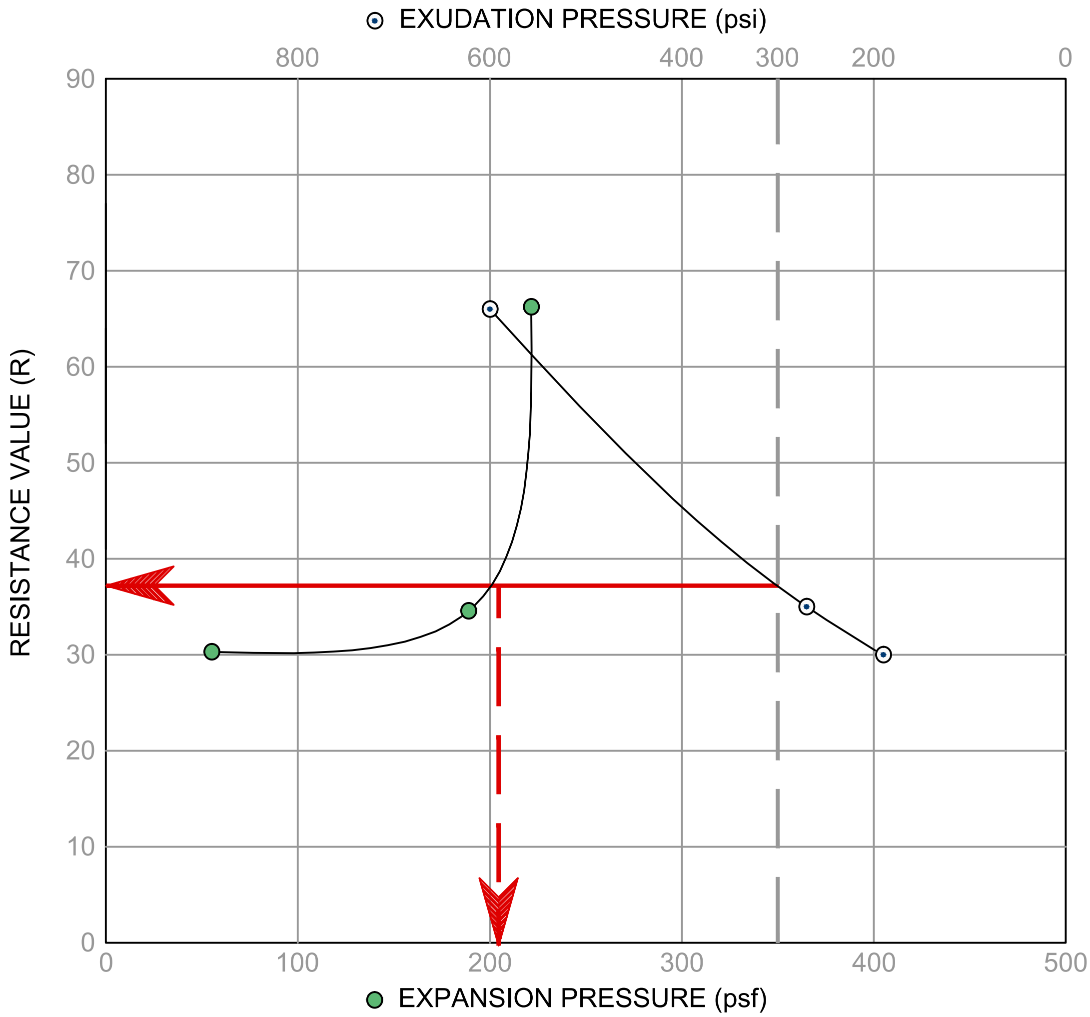
PLATE
23



Specimen Number	A	B	C
Exudation Pressure (psi)	191	606	275
Moisture Content (%)	16.0	13.7	15.0
Dry Density (pcf)	113	116	114
Expansion Pressure (psf)	65	237	194
Resistance Value (R)	30	66	35

			Values at 300 psi Exudation	
Sample Source	Classification	Sand Equivalent	Expansion Pressure (psf)	**R-Value**
B-4 @ 1 to 3.5 feet	DARK BROWN CLAYEY SAND (SC)	--	210	36

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RESISTANCE VALUE TEST DATA
THE AVALON INN
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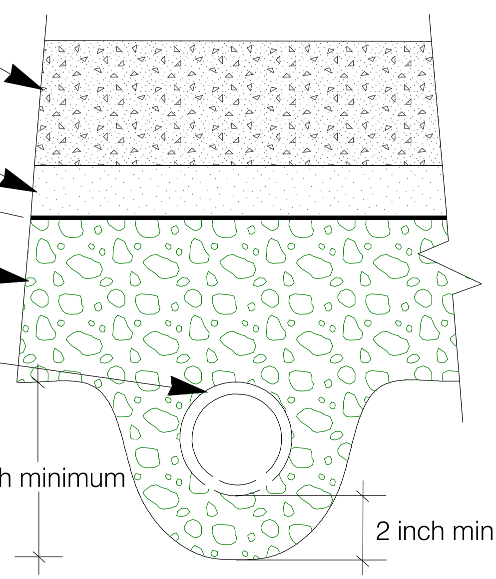
PLATE
24



NOT TO SCALE

NOTES:

1. Drain rock should be clean, free-draining material graded in size between the No.4 and 3/4 inch sieves.
2. Pipe should be SDR 35 or equivalent, perforations placed down, sloped at least 1 percent to gravity outlet, or sump with automatic pump.
3. A clean-out pipe with cap should be installed at the up-slope end of perforated pipe.
4. Vapor retarder should be at least 15-mils thick and installed in accordance with the manufacturer's specifications.

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UNDERSLAB DRAINAGE DETAILS

THE AVALON INN
1201 North Main Street
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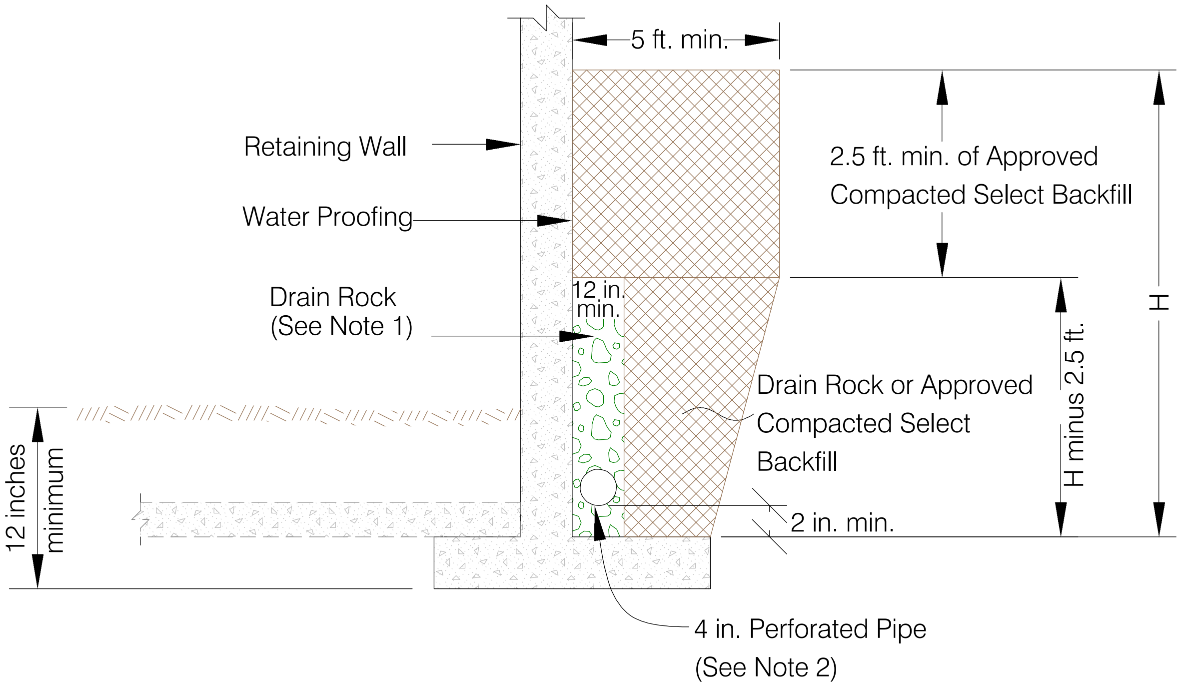
PLATE
25



RETAINING WALL DRAINAGE DETAIL
(Not to Scale)

NOTES:

(1) Drain rock should be clean, free-draining material graded in size between the No. 4 and 3/4 or 1-1/2 inch sieves and should be wrapped in a non-woven geotextile filter fabric (Mirafi 140N or equivalent), or Class 2 permeable material, without filter fabric, per Caltrans standard specifications, latest edition.

(2) Pipe should be SDR 35 or equivalent, placed with perforations down, and sloped at 1 percent to drain to gravity outlet or sump with automatic pump.

(3) A clean-out pipe with cap should be installed at the up-slope end of perforated pipe, and pipe elbows should be 45 degrees or less (for "snake" access).

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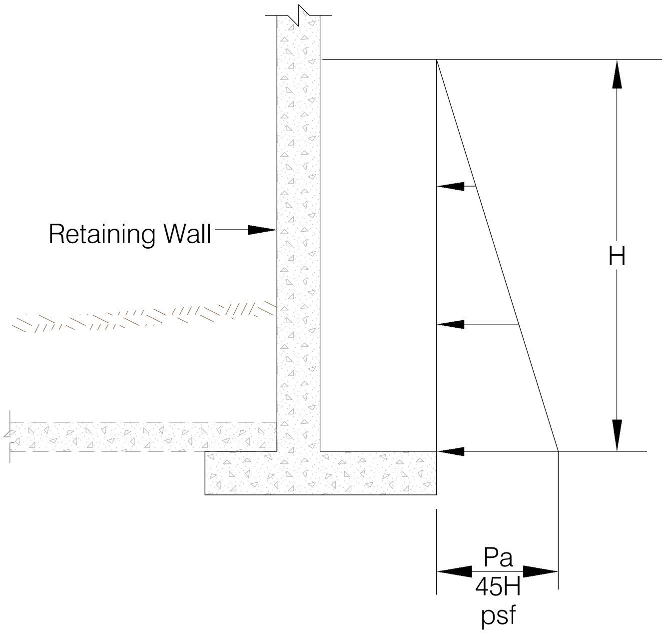
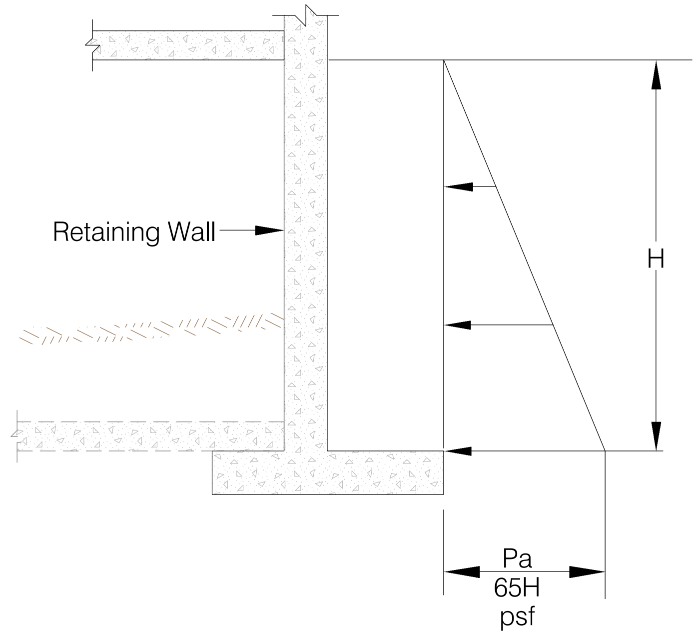
RETAINING WALL DRAINAGE DETAIL
THE AVALON INN
1201 North Main Street
Fort Bragg, California

PLATE 26



ACTIVE SOIL PRESSURES DIAGRAM
For walls that are free to rotate (See Note 2)



AT-REST SOIL PRESSURES DIAGRAM
For braced walls of substantial rigidity (See Note 2)

NOTES:

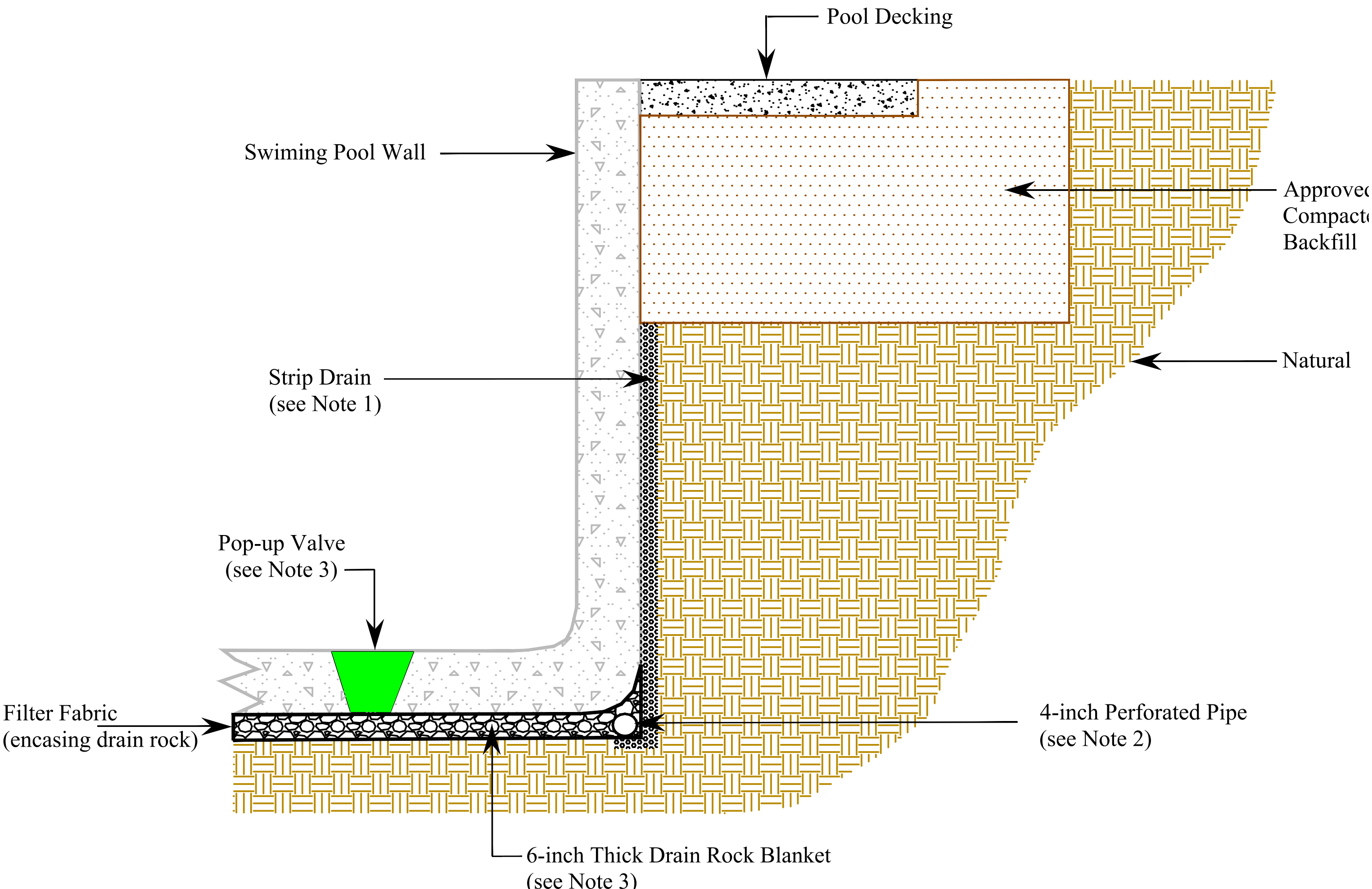
(1) The above are level backfill soil pressures only and do not include lateral loads resulting from other sources such as traffic, floor loads, adjacent foundations or other vertical loads.

(2) If the wall at the backfill surface cannot move more than about 0.1 percent of its height, at-rest soil pressures should be used.

(3) The above pressures assume a drained condition. See Plate 26 for drainage and backfill details.

(4) The above pressures should be used where backfill slope is flatter than 3 horizontal to 1 vertical (3:1). Where backfill slope is between 3:1 and 1.5:1, use active pressure of 55H psf and at-rest pressure of 87H psf, respectively.

(5) For design seismic pressures see the Retaining Walls section of this report.



TYPICAL SECTION
NO SCALE

NOTES:

(1) The strip drains should be AKWADRAIN by American Wick Drain Corporation or equivalent. The strip drains should be be spaced 8 feet apart. Drains should start a maximum of 1.5 feet below the ground surface behind the pool walls and slope downward beneath the pool.

(2) Pipe should be SDR 35, or equivalent, perforations placed down, sloped at least 1 percent to drain to gravity outlet or sump with automatic pump.

(3) Place a six-inch-thick blanket of drain rock (graded between No.4 and 1 inch sieve) wrapped in filter fabric placed on the pool excavation bottom and two hydrostatic release "pop-up" valves placed at the pool deep end.

(4) Other drainage alternatives may be acceptable; BAI should review alternate drainage plans.

L:\Geotech Projects\12122.1 Hi-Seas Beach Motel, Fort Bragg\Avalon Inn Report\12122.1 Graphics\12122.01_28 SWIMMING POOL AND BACKFILL DETAILS.dwg

12/4/2015 9:50:33 AM save date
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Brunsing Associates, Inc.
5468 Skylane Blvd., Suite 201
Santa Rosa, California 95403
Tel: (707) 838-3027

Job No.:	12122.011
Appr.:	KAC
Date:	12/04/15

SWIMMING POOL DRAINAGE AND BACKFILL DETAILS
THE AVALON INN
1201 North Main Street
Fort Bragg, California

PLATE
28

APPENDIX A

Liquefaction Induced Vertical Settlement and Lateral Spread Calculations

Project: The Avalon Inn
Project #: 12122.01
Date: 12/4/2015
Boring: B-1

Input Parameters:

Peak Ground Accel (g) =	0.400			
Earthquake Magnitude, M =	7.7			
Water Table Depth (m) =	0.6		(ft)	2.00
Average γ Above Water Table (kN/m^3) =		21.2	(lb/ft^3)	135
Average γ Below Water Table (kN/m^3) =		20.3	(lb/ft^3)	129
Borehole Diameter (mm) =	177.80		(in)	7
Requires Correction for Sample Liners (YES/NO):			no	
Rod Lengths Assumed Equal to the Depth Plus		1.5	m (for the above ground extension)	
Gravilty Acceleration (m/sec^2)		9.81		
Height of Exposed Face (m)		12	(ft)	40

SPT Sample Number	Depth (m)	Measured Depth (ft)	Measured N	Soil Type (USCS)	Flag "Clay"	Sat/Unsat	Fines Content (%)	Energy Ratio, ER (%)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	σ_{vc}' (kPa)	C_N	$(N_1)_{60}$	ΔN for Fines Content	$(N_1)_{60\text{-}cs}$	Stress Reduct, Coeff, r_d	CSR	MSF for Sand	K_σ for Sand	CRR for M=7.5 & σ_{vc}'=1atm	CRR	Factor of Safety	Depth (m)	Limiting Shear Strain γ_{lim}	Parameter F_α	Maximum Shear Strain γ_{max}	ΔH_i (m)	ΔLDI_i (m)	Vertical Reconsol. Strain ε_v	ΔS_i (m)	ΔS_i (in)
1	0.46	1.50	10	SM		0	15	75	1.25	1	0.75	1.00	10.8	10	10	1.70	18.33	3.3	21.59	1.00	0.261	0.94	1.10	0.227	N.A.	N.A.	0.46	0.133	0.431	0.000	0.46	0.000	0.000	0.000	0.00
2	0.91	3.00	15	SC		1	15	75	1.25	1	0.75	1.00	16.2	19	16	1.70	27.49	3.3	30.75	1.00	0.308	0.94	1.10	0.536	0.554	1.80	0.91	0.042	-0.140	0.004	0.46	0.002	0.001	0.000	0.00
3	1.22	4.00	15	SP		1	2	75	1.25	1	0.75	1.00	16.2	25	19	1.70	27.49	0.0	27.49	1.00	0.340	0.94	1.10	0.364	0.376	1.10	1.22	0.065	0.076	0.028	0.30	0.009	0.006	0.000	0.02
4	2.29	7.50	15	SP-SM		1	5	75	1.25	1	0.8	1.00	17.3	47	30	1.58	27.29	0.0	27.30	0.99	0.397	0.94	1.10	0.357	0.368	0.93	2.29	0.067	0.088	0.041	1.07	0.043	0.009	0.001	0.03
5	3.20	10.50	50	SS		1	20	75	1.25	1	0.85	1.00	61.1	65	40	1.28	77.94	4.5	82.42	0.98	0.418	0.94	1.10	2.000	2	2.00	3.20	0.000	-4.419	0.000	0.91	0.000	0.000	0.000	0.00

Liquefaction Potential

LDI= 0.05 (m) 2.124 (in)

S= 0.001 (m) 0.04 (in)

Project: The Avalon Inn
Project #: 12122.01
Date: 12/4/2015
Boring: B-2

Input Parameters:

Peak Ground Accel (g) =	0.400		
Earthquake Magnitude, M =	7.7		
Water Table Depth (m) =	0.6	(ft)	2.00
Average γ Above Water Table (kN/m^3) =	20.4	(lb/ft^3)	130
Average γ Below Water Table (kN/m^3) =	20.4	(lb/ft^3)	130
Borehole Diameter (mm) =	177.80	(in)	7
Requires Correction for Sample Liners (YES/NO):	no		
Rod Lengths Assumed Equal to the Depth Plus	1.5	m (for the above ground extension)	
Gravilty Acceleration (m/sec^2)	9.81		
Height of Exposed Face (m)	12	(ft)	40

SPT Sample Number	Depth (m)	Measured Depth (ft)	Measured N	Soil Type (USCS)	Flag "Clay"	Sat/Unsat	Fines Content (%)	Energy Ratio, ER (%)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	σ_{vc}' (kPa)	C_N	$(N_1)_{60}$	ΔN for Fines Content	$(N_1)_{60\text{-}cs}$	Stress Reduct, Coeff, r_d	CSR	MSF for Sand	K_σ for Sand	CRR for M=7.5 & σ_{vc}'=1atm	CRR	Factor of Safety	Depth (m)	Limiting Shear Strain γ_{lim}	Parameter F_α	Maximum Shear Strain γ_{max}	ΔH_i (m)	ΔLDI_i (m)	Vertical Reconsol. Strain ε_v	ΔS_i (m)	ΔS_i (in)
1	0.76	2.50	4	SM		1	15	75	1.25	1	0.75	1.00	4.3	16	14	1.70	7.33	3.3	10.59	1.00	0.288	0.94	1.10	0.122	0.126	0.44	0.76	0.443	0.901	0.443	0.76	0.338	0.036	0.033	1.28
2	1.83	6.00	15	SM		1	15	75	1.25	1	0.8	1.00	17.3	37	25	1.63	28.12	3.3	31.38	0.99	0.380	0.94	1.10	0.586	0.605	1.59	1.83	0.038	-0.182	0.010	1.07	0.010	0.002	0.000	-0.01
3	2.44	8.00	17	SP-SM		1	7	75	1.25	1	0.8	1.00	19.6	50	32	1.52	29.76	0.1	29.89	0.99	0.402	0.94	1.10	0.478	0.494	1.23	2.44	0.047	-0.082	0.022	0.61	0.014	0.004	0.000	-0.01
4	2.90	9.50	20	SP-SM		1	7	75	1.25	1	0.85	1.00	24.4	59	37	1.40	34.25	0.1	34.39	0.99	0.413	0.94	1.10	0.979	1.011	2.00	2.90	0.024	-0.392	0.000	0.46	0.000	0.000	0.000	0.00
5	3.81	12.50	100	SS		1	20	75	1.25	1	0.85	1.00	###	78	46	1.23	149.92	4.5	154.40	0.98	0.427	0.94	1.10	2.000	2	2.00	3.81	0.000	-11.466	0.000	0.91	0.000	0.000	0.000	0.00

Liquefaction Potential

LDI= 0.36 (m) 14.24 (in)

S= 0.032 (m) 1.25 (in)

Project: The Avalon Inn
Project #: 12122.01
Date: 12/4/2015
Boring: B-3

Input Parameters:

Peak Ground Accel (g) =	0.400		
Earthquake Magnitude, M =	7.7		
Water Table Depth (m) =	0.6	(ft)	2.00
Average γ Above Water Table (kN/m^3) =	20.4	(lb/ft^3)	130
Average γ Below Water Table (kN/m^3) =	20.4	(lb/ft^3)	130
Borehole Diameter (mm) =	177.80	(in)	7
Requires Correction for Sample Liners (YES/NO):	no		
Rod Lengths Assumed Equal to the Depth Plus	1.5	m (for the above ground extension)	
Gravilty Acceleration (m/sec^2)	9.81		
Height of Exposed Face (m)	12	(ft)	40

SPT Sample Number	Depth (m)	Measured Depth (ft)	Measured N	Soil Type (USCS)	Flag "Clay"	Sat/Unsat	Fines Content (%)	Energy Ratio, ER (%)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	$\sigma_{vc'}$ (kPa)	C_N	$(N_1)_{60}$
1	0.76	2.50	4	SM		1	15	75	1.25	1	0.75	1.00	4.3	16	14	1.70	7.33
2	1.45	4.75	5	SC		1	15	75	1.25	1	0.75	1.00	5.4	30	21	1.70	9.16
3	2.13	7.00	7	SM		1	16	75	1.25	1	0.8	1.00	8.1	44	29	1.70	13.69
4	3.66	12.00	7	SM		1	15	75	1.25	1	0.85	1.00	8.6	75	45	1.47	12.61
5	4.42	14.50	50	SS		1	20	75	1.25	1	0.85	1.00	61.1	90	53	1.19	72.43

SPT Sample Number	ΔN for Fines Content	$(N_1)_{60-cs}$	Stress Reduct, Coeff, r_d	CSR	MSF for Sand	K_σ for Sand	CRR for M=7.5 & σ_{vc}'=1atm	CRR	Factor of Safety	Depth (m)	Limiting Shear Strain γ_{lim}	Parameter F_α	Maximum Shear Strain γ_{max}	ΔH_i (m)	ΔLDI_i (m)	Vertical Reconsol. Strain ε_v	ΔS_i (m)	ΔS_i (in)
1	3.3	10.59	1.00	0.288	0.94	1.10	0.122	0.126	0.44	0.76	0.443	0.901	0.443	0.76	0.338	0.036	0.033	1.28
2	3.3	12.43	1.00	0.359	0.94	1.10	0.136	0.14	0.39	1.45	0.363	0.849	0.363	0.69	0.249	0.033	0.028	1.09
3	3.6	17.26	0.99	0.393	0.94	1.10	0.176	0.182	0.46	2.13	0.215	0.655	0.215	0.69	0.148	0.026	0.017	0.67
4	3.3	15.88	0.98	0.425	0.94	1.09	0.164	0.168	0.40	3.66	0.250	0.717	0.250	1.52	0.381	0.028	0.020	0.78
5	4.5	76.91	0.97	0.432	0.94	1.10	2.000	2	2.00	4.42	0.000	-3.915	0.000	0.76	0.000	0.000	0.000	0.00

Liquefaction Potential

LDI= 1.12 (m) 43.94 (in)

S= 0.097 (m) 3.82 (in)

Project:	The Avalon Inn
Project #:	12122.01
Date:	12/4/2015
Boring:	B-5

Input Parameters:

Parameter	Value	Unit	Value
Peak Ground Accel (g) =	0.400		
Earthquake Magnitude, M =	7.7		
Water Table Depth (m) =	0.6	(ft)	2.00
Average γ Above Water Table (kN/m^3) =	19.6	(lb/ft^3)	125
Average γ Below Water Table (kN/m^3) =	19.6	(lb/ft^3)	125
Borehole Diameter (mm) =	177.80	(in)	7
Requires Correction for Sample Liners (YES/NO):	no		
Rod Lengths Assumed Equal to the Depth Plus	1.5	m (for the above ground extension)	
Gravilty Acceleration (m/sec^2)	9.81		
Height of Exposed Face (m)	12	(ft)	40

SPT Sample Number	Depth (m)	Measured Depth (ft)	Measured N	Soil Type (USCS)	Flag "Clay"	Sat/Unsat	Fines Content (%)	Energy Ratio, ER (%)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	σ_{vc}' (kPa)	C_N	$(N_1)_{60}$
1	0.76	2.50	5	SM		1	15	75	1.25	1	0.75	1.00	5.4	15	13	1.70	9.16
2	1.60	5.25	7	SC		1	15	75	1.25	1	0.8	1.00	8.1	31	22	1.70	13.69
3	2.82	9.25	6	SM		1	15	75	1.25	1	0.85	1.00	7.3	55	34	1.69	12.42
4	3.35	11.00	50	SS		1	20	75	1.25	1	0.85	1.00	61.1	66	39	1.29	78.51

SPT Sample Number	ΔN for Fines Content	$(N_1)_{60-cs}$	Stress Reduct, Coeff, r_d	CSR	MSF for Sand	K_σ for Sand	CRR for M=7.5 & σ_{vc}'=1atm	CRR	Factor of Safety	Depth (m)	Limiting Shear Strain γ_{lim}	Parameter F_α	Maximum Shear Strain γ_{max}	ΔH_i (m)	ΔLDI_i (m)	Vertical Reconsol. Strain ε_v	ΔS_i (m)	ΔS_i (in)
1	3.3	12.43	1.00	0.289	0.94	1.10	0.136	0.14	0.48	0.76	0.363	0.849	0.363	0.76	0.277	0.033	0.028	1.09
2	3.3	16.95	1.00	0.375	0.94	1.10	0.173	0.179	0.48	1.60	0.223	0.669	0.223	0.84	0.187	0.026	0.018	0.69
3	3.3	15.68	0.99	0.422	0.94	1.10	0.162	0.167	0.40	2.82	0.256	0.726	0.256	1.22	0.312	0.028	0.020	0.80
4	4.5	82.99	0.98	0.432	0.94	1.10	2.000	2	2.00	3.35	0.000	-4.471	0.000	0.53	0.000	0.000	0.000	0.00

Liquefaction Potential

LDI= 0.78 (m) 30.52 (in)

S= 0.066 (m) 2.58 (in)

Project:	The Avalon Inn
Project #:	12122.01
Date:	12/4/2015
Boring:	B-6

Input Parameters:

Peak Ground Accel (g) =	0.400		
Earthquake Magnitude, M =	7.7		
Water Table Depth (m) =	0.6	(ft)	2.00
Average γ Above Water Table (kN/m^3) =	21.2	(lb/ft^3)	135
Average γ Below Water Table (kN/m^3) =	19.6	(lb/ft^3)	125
Borehole Diameter (mm) =	177.80	(in)	7
Requires Correction for Sample Liners (YES/NO):	no		
Rod Lengths Assumed Equal to the Depth Plus	1.5	m (for the above ground extension)	
Gravilty Acceleration (m/sec^2)	9.81		
Height of Exposed Face (m)	12	(ft)	40

SPT Sample Number	Depth (m)	Measured Depth (ft)	N	Soil Type (USCS)	Flag "Clay"	Sat/Unsat	Fines Content (%)	Energy Ratio, ER (%)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	σ_{vc}' (kPa)	C_N	$(N_1)_{60}$	ΔN for Fines Content	$(N_1)_{60-cs}$	Stress Reduct, Coeff, r_d	CSR	MSF for Sand	K_σ for Sand	CRR for M=7.5 & σ_{vc}'=1atm	CRR	Factor of Safety	Depth (m)	Limiting Shear Strain γ_{lim}	Parameter F_α	Maximum Shear Strain γ_{max}	ΔH_i (m)	ΔLDI_i (m)	Vertical Reconsol. Strain ε_v	ΔS_i (m)	ΔS_i (in)
1	0.23	0.75	12	GM		0	12	75	1.25	1	0.75	1.00	12.9	5	5	1.70	21.99	2.1	24.07	1.00	0.261	0.94	1.10	0.269	N.A.	N.A.	0.23	0.099	0.288	0.000	0.23	0.000	0.000	0.000	0.00
2	0.46	1.50	6	SM		0	16	75	1.25	1	0.75	1.00	6.5	10	10	1.70	11.00	3.6	14.57	1.00	0.261	0.94	1.10	0.153	N.A.	N.A.	0.46	0.288	0.772	0.000	0.23	0.000	0.000	0.000	0.00
3	0.91	3.00	6	SP		1	7	75	1.25	1	0.75	1.00	6.5	19	16	1.70	11.00	0.1	11.13	1.00	0.309	0.94	1.10	0.126	0.13	0.42	0.91	0.418	0.887	0.418	0.46	0.191	0.035	0.031	1.22
4	1.83	6.00	2	SP		1	6	75	1.25	1	0.8	1.00	2.3	37	25	1.70	3.91	0.0	3.94	0.99	0.383	0.94	1.10	0.080	0.083	0.22	1.83	0.500	0.948	0.500	0.91	0.457	0.058	0.055	2.15
5	3.05	10.00	8	SM		1	16	75	1.25	1	0.85	1.00	9.8	61	37	1.57	15.37	3.6	18.95	0.99	0.422	0.94	1.10	0.194	0.2	0.47	3.05	0.179	0.572	0.179	1.22	0.218	0.024	0.014	0.54
6	3.51	11.50	13	SS	RELIAI	1	20	75	1.25	1	0.85	1.00	15.9	70	41	1.41	22.41	4.5	26.89	0.98	0.430	0.94	1.10	0.343	?	?	3.51	0.070	0.114	0.000	0.46	0.000	0.000	0.000	0.00
7	4.11	13.50	83	SS		1	20	75	1.25	1	0.85	1.00	###	82	47	1.22	123.77	4.5	128.25	0.98	0.438	0.94	1.10	2.000	2	2.00	4.11	0.000	-8.826	0.000	0.61	0.000	0.000	0.000	0.00

Liquefaction Potential

LDI=	0.87 (m) 34.1 (in)	S=	0.100 (m)	3.92 (in)

Project: The Avalon Inn
Project #: 12122.01
Date: 12/4/2015
Boring: B-7

Input Parameters:

Peak Ground Accel (g) =	0.400		
Earthquake Magnitude, M =	7.7		
Water Table Depth (m) =	0.6	(ft)	2.00
Average γ Above Water Table (kN/m^3) =	21.2	(lb/ft^3)	135
Average γ Below Water Table (kN/m^3) =	19.6	(lb/ft^3)	125
Borehole Diameter (mm) =	177.80	(in)	7
Requires Correction for Sample Liners (YES/NO):	no		
Rod Lengths Assumed Equal to the Depth Plus	1.5	m (for the above ground extension)	
Gravilty Acceleration (m/sec^2)	9.81		
Height of Exposed Face (m)	12	(ft)	40

SPT Sample Number	Depth (m)	Measured Depth (ft)	Measured N	Soil Type (USCS)	Flag "Clay"	Sat/Unsat	Fines Content (%)	Energy Ratio, ER (%)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	σ_{vc}' (kPa)	C_N	$(N_1)_{60}$	ΔN for Fines Content	$(N_1)_{60-cs}$	Stress Reduct, Coeff, r_d	CSR	MSF for Sand	K_σ for Sand	CRR for M=7.5 & σ_{vc}'=1atm	CRR	Factor of Safety	Depth (m)	Limiting Shear Strain γ_{lim}	Parameter F_α	Maximum Shear Strain γ_{max}	ΔH_i (m)	ΔLDI_i (m)	Vertical Reconsol. Strain ε_v	ΔS_i (m)	ΔS_i (in)
1	0.23	0.75	12	GM		0	12	75	1.25	1	0.75	1.00	12.9	5	5	1.70	21.99	2.1	24.07	1.00	0.261	0.95	1.10	0.269	N.A.	N.A.	0.23	0.099	0.288	0.000	0.23	0.000	0.000	0.000	0.00
2	1.07	3.50	5	SM		1	16	75	1.25	1	0.75	1.00	5.4	22	17	1.70	9.16	3.6	12.74	1.00	0.327	0.95	1.10	0.138	0.144	0.44	1.07	0.351	0.839	0.351	0.84	0.294	0.032	0.027	1.06
3	1.98	6.50	8	SM		1	12	75	1.25	1	0.8	1.00	9.2	40	26	1.70	15.64	2.1	17.71	0.99	0.390	0.95	1.10	0.181	0.189	0.48	1.98	0.205	0.633	0.205	0.91	0.187	0.025	0.016	0.63
4	3.05	10.00	10	SM		1	12	75	1.25	1	0.85	1.00	12.2	61	37	1.55	18.89	2.1	20.96	0.98	0.422	0.95	1.10	0.218	0.228	0.54	3.05	0.143	0.466	0.143	1.07	0.152	0.022	0.010	0.41
5	4.57	15.00	9	SM		1	16	75	1.25	1	0.95	1.00	12.3	91	52	1.34	16.48	3.6	20.06	0.97	0.442	0.95	1.09	0.207	0.213	0.48	4.57	0.158	0.515	0.158	1.52	0.241	0.023	0.012	0.47
6	5.03	16.50	79	SS		1	20	75	1.25	1	0.95	1.00	###	100	56	1.17	125.78	4.5	130.26	0.97	0.445	0.95	1.10	2.000	2	2.00	5.03	0.000	-9.027	0.000	0.46	0.000	0.000	0.000	0.00

Liquefaction Potential

LDI= 0.87 (m) 34.44 (in)

S= 0.065 (m) 2.57 (in)

DISTRIBUTION

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