GEOTECHNICAL INVESTIGATION AND GEOLOGIC HAZARDS EVALUATION

FORT BRAGG FIRE DEPARTMENT STATION #830 141 MAIN STREET FORT BRAGG, MENDOCINO COUNTY, CALIFORNIA

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prepared for

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GEOTECHNICAL INVESTIGATION AND GEOLOGIC HAZARDS EVALUATION

Fort Bragg Fire Department Station #830
141 Main Street, Mendocino County, California

This report presents results of the geotechnical and geologic hazards investigation performed for the Fort Bragg Fire Department Station #830, Mendocino County, California. The geotechnical investigation and geologic hazards evaluation were performed and prepared by BACE Geotechnical (BACE), a division of Brunsing Associates, Inc., in accordance with our Service Agreement dated December 12, 2008 and authorized by I. L. Welty & Associates on December 17, 2008.

Based upon the results of our investigation and review of available geologic data, we conclude that the site is suitable for the building improvements or future reconstruction of the site. Our subsurface exploration has determined that the existing buildings are underlain by Pleistocene terrace deposits consisting of very loose to medium dense silty sands or sands with few fines. These terrace deposits overlay bedrock and range from 9.5 to 13.5 feet thick. Ground water was at 8 to 9 feet below the ground surface in January 2009, but can be very close to the ground surface during or shortly after periods of rainfall. The terrace deposits in a saturated condition could undergo liquefaction during or just after a strong seismic event on the nearby San Andreas Fault or the more distant Maacama Fault. The result of our analysis has determined the "theoretical" liquefaction induced settlement of the soils is approximately one and 2.5 inches at Borings B-3 and B-5, respectively. Foundations directly on top of liquefiable material can experience comparable or even larger differential settlements.

Due to the potential for liquefaction at the site, the existing structures should be supported on either a drilled cast-in-place concrete piers and grade beam foundation system or the liquefiable soils need to be reinforced by either compaction/pressure/chemical grouting to lower the liquefaction potential. Earthquake induced forces and earthquake-related ground settlement (induced liquefaction) are the only potential geologic hazards identified at this site. Design recommendations for drilled piers are given in this report.

The rough estimated cost to perform only the drilling operations for the drilled piers are $1,000 to $2,000 per hole if drilled holes need to be cased due to caving sands, and $600 to $700 per hole if drilled holes do not cave during drilling operations. BACE did not experience caving soils during our exploration, but with loose and saturated sands the contractor should be prepared to case the pier holes.

The rough estimated costs for compaction grouting and permeation (chemical) grouting have been obtained from grouting contractors. The estimated cost for compaction grouting ranged from $150,000 to $250,000 from one company and $450,000 to $475,000 from another company. The cost for permeation grouting is estimated to be about $900,000 to $950,000.

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation and geologic hazards evaluation performed by BACE Geotechnical (BACE), a division of Brunsing Associates, Inc., for Fort Bragg Fire Department, Station #830, located at 141 North Main Street, Fort Bragg, Mendocino County, California. The site (Assessor Parcel Number APN 008-161-04) is situated on the west side of North Main Street (Highway 1) between W. Alder Street and W. Oak Street, approximately eight-tenths of a mile south of Pudding Creek, and one mile north of Noyo River, as shown on the Vicinity Map, Plate 1.

The purpose of our geotechnical investigation and geologic hazard evaluation was to evaluate the soil, rock and groundwater conditions at the site in order to provide conclusions regarding feasibility or the site improvements and recommendations for upgraded foundation design criteria. Correspondence with Lee Welty, I.L. Welty & Associates, Inc. (Welty), indicates that the existing three buildings attached together, will be structurally evaluated and where applicable, structures will be upgraded (seismically retrofitted). We understand that Welty will also be evaluating specific foundation elements within the existing buildings. The existing buildings and surrounding property are shown on a 2008 plot plan overlain on a 1997 topographic survey map and floor plan of the property, presented herein as our Site Plan, Plate 2. BACE also understands that if the cost of retrofitting the existing buildings is too costly, there is a potential that the existing buildings could be removed and replaced with a new fire station.

Our approach to providing the geotechnical and geologic information necessary to perform this evaluation utilized our knowledge of the geologic conditions in the site vicinity and our experience with similar projects. Field exploration and laboratory testing for this investigation were directed towards confirming anticipated geotechnical and geologic conditions in order to provide the basis for our conclusions and recommendations. Our report provides conclusions and recommendations for two different conditions: Condition 1 – existing structures are to be seismically retrofitted; Condition 2 – existing structures are removed and new structures are to be constructed.

The scope of our services, as outlined in our Professional Services Agreement dated December 12, 2008, consisted of site reconnaissance, subsurface exploration, laboratory testing, geologic and engineering analyses, report preparation, and presentation of report and conclusions to Welty, the City of Fort Bragg and the Fort Bragg Fire Protection Authority.

2.0 INVESTIGATION AND LABORATORY TESTING

2.1 Research and Analysis

As part of our investigation, we reviewed published geotechnical literature, and geologic and fault maps for the site and vicinity. A list of the published references reviewed for this investigation is presented in Appendix A.

We performed computer analyses for site seismic response and liquefaction potential using the FRISKSP and LIQUEFY2 programs, respectively. The results of our analyses are presented in the Discussion and Conclusions section of this report.

2.2 Field Reconnaissance

BACE's Principal Engineering Geologist and Senior Civil Engineer performed an initial site reconnaissance on December 12, 2008, to assess site conditions, observe the existing structure, including foundation stem walls, slabs, and pavements, check for evidence of settlement distress, and mark test boring locations. Our Senior Civil Engineer also returned on January 6, 2009, to observe site conditions and subsurface drilling operations. Site Photograph A on Plate 3 shows the site and existing buildings from the southeast.

2.3 Aerial Photograph Studies

To augment our field reconnaissance, we obtained oblique-angle aerial photographs from the California Coastal Records Project (www.californiacoastline.org). The 2005 photo is presented herein as our Coastal Oblique Aerial Photograph on Plate 4. We also studied a stereo pair of vertical aerial photographs, dated June 24, 1981. We studied these photographs to identify lineaments, creek offsets, etc., that may be due to faulting. The results of our aerial photograph studies are discussed below.

2.4 Field Exploration

Our subsurface drilling exploration was conducted on January 6, 2009, and consisted of drilling, logging, and sampling five exploratory test borings, Borings B-1 through B-5. The approximate test boring locations are shown on Plate 2. Each of the test borings was advanced to practical drilling refusal with the exception of B-4, which was terminated due to suspected petroleum hydrocarbon contamination. In each of the other borings, practical drilling refusal was encountered within bedrock between approximately 11 to 20 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. The relatively undisturbed samples were obtained from the test borings using a 2.5-inch outside diameter (O.D.) Modified California split-barrel sampler, driven by a 140-pound drop hammer falling 30 inches per blow. The inside of the 2.5-inch sampler barrel contained 1.9-inch inside diameter liners for retaining the soil/rock materials. Hammer blows required to drive the 2.5-inch sampler were converted to Standard Penetration Test (SPT) blow counts for correlation with empirical test data, using a conversion factor of 0.79. Sampler penetration resistance (blow counts) provides a relative measure of soil consistency and strength, and is utilized in our engineering analyses. Selected samples were also obtained using a 2-inch outside diameter, SPT sampler containing 1.4-inch inside diameter liners.

Graphic logs of the borings, showing the various soil types encountered and the depths of the samples taken, are presented on Plates 5 through 9. The soils are classified in accordance with the Unified Soil Classification System ASTM D 2487 outlined on Plate 10. The various descriptive properties used to describe the soils are listed on Plate 11, and rock descriptive properties are listed on Plate 12.

2.5 Laboratory Testing

Selected samples obtained during our subsurface exploration were tested in BACE's and subcontractors' laboratories to assess their pertinent geotechnical engineering characteristics. Laboratory testing consisted of moisture content-dry density, grain-size distribution, unconsolidated-undrained triaxial compression, and direct shear tests.

The test results are presented opposite the samples tested on the test boring logs. A Key to Test Data is provided on Plate 10. In addition, grain size distribution test results are presented on Plates 13 and 14, triaxial compression test data on Plate 15, and direct shear test results on Plate 16. Near-surface soil samples were sent to JDH Corrosion Consultants, Inc. for corrosivity potential testing consisting of resistivity, pH, chloride, sulfate and redox. Corrosivity test results are presented in Appendix B.

3.0 SITE CONDITIONS

As shown on Plate 1, the site is on the west side of North Main Street (Highway 1), just northwest of the intersection with West Oak Street in Fort Bragg, California. According to the USGS Fort Bragg 7-1/2 Minute Quadrangle Topographic Map (Plate 1), the approximate site elevation is 60 feet above Mean Sea Level. The site coordinates are 39.44240 degrees north (latitude) and –123.80621 degrees west (longitude).

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. Existing buildings on the property consist of three attached buildings built in three different phases with the long axis of the group of buildings oriented north south. The North Wing (North Apparatus Room) was constructed in 1947, the South Wing (South Apparatus Room) was built in 1977, and the centrally-located Office and Crew Rooms were constructed in 1977 and 1997. The ground surface within the property is nearly level and is mostly paved. Paved alleys form the westerly and northerly boundaries of the property, and North Main Street forms the easterly boundary of the property. Paved parking areas are located south of the main building.

A buried creek channel extends easterly from Soldier Bay through the Fort Bragg community. Based upon present USGS topographic contours, it appears that the channel was once present near the north end of the fire station property. No evidence of creek channel soils or backfill materials were encountered in our borings. The backfill soils encountered in Boring B-2, as described in Section 4.2 of this report, are not likely to be channel backfill soils (too clean and well compacted).

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 in this part of the Mendocino coast is comprised of well-consolidated sedimentary rocks, such as sandstone and shale, of the Cretaceous-Tertiary Period coastal belt Franciscan Complex. The bluff top 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 geologic conditions in the vicinity of the site are shown on the Regional Geologic Map, Plate 17.

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 6.7 miles (10.8 kilometers [km]) west-southwest of the site. The active Maacama Fault (north segment) is located approximately 21.2 miles (34.1 km) northeast of the site, as shown on the Regional Active Fault Map, Plate 18. Other, possibly active faults, such as the Pacific Star Winery Fault, are located several kilometers 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

Our test borings indicate that one to two feet of fill, consisting of asphalt and aggregate base materials, covers most of the site. Thicker fill soils are present within localized portions of the property. Approximately 11 feet of aggregate (sand with angular rock fragments) fill soils were encountered in Boring B-2. This fill appears to have been compacted based upon the sampler blow counts. The fill appears to be backfill for utilities or a previous area of weak soils removed to maintain passage of fire equipment; its areal extent is uncertain.

Poorly consolidated Pleistocene Epoch marine terrace deposits mantle the bedrock in the site vicinity. 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 are mostly comprised of very loose to medium dense, damp to saturated sands. Silty sands were observed in some of the borings, but most of the terrace deposits we observed are sands with few fines (silt- and clay-sized particles). The terrace deposits generally extend between 9.5 and 13.5 feet below the ground surface (bgs) and appear to generally thicken toward the west.

Franciscan Complex bedrock was encountered in our test borings at approximately 9.5 to 13.5 feet bgs. The bedrock generally consists of sandstone, siltstone and shale. The sandstone is gray brown, blue gray and green gray with orange brown mottling. The sandstone is generally intensely to little fractured, friable to hard and deeply to little weathered. The siltstone is reddish brown to gray brown and green brown. The siltstone is crushed to little fractured, soft to hard and deeply to little weathered. The dark gray shale is intensely to little fractured, moderately hard to hard and moderately to little weathered. The upper few feet of bedrock is generally deeply weathered and reddish-brown to gray-brown. Weathering generally decreases with depth, and hardness increases. Since the ground surface at the site is entirely overlain with terrace deposits, and no bedrock surface outcrops were observed, a site-specific geologic map was not prepared. Geologic Cross sections A-A' and B-B' on Plates 19 and 20, respectively, depict our interpretation of the subsurface conditions and show the approximate existing building locations.

Practical drilling refusal in hard bedrock was encountered at 20, 11, 13.5, and 17.5 feet in borings B-1, B-2, B-3, and B-5, respectively. Boring B-4 was terminated in bedrock at 9.5 feet when a strong petroleum odor was encountered.

Ground water was encountered in our borings at depths ranging from 8 to 9 feet below the ground surface in borings B-1, B-2 and B-5. These unstabilized ground water level measurements were taken shortly after completion of the borings. No water was encountered in borings B-3 and B-4. BACE has reviewed its own previous report in the site vicinity along with review of the State Water Resources Control Board GeoTracker. Ground water levels may be anticipated to rise to within a few feet of the ground surface in response to increased recharge during and shortly after periods of prolonged rainfall, and/or following rainy seasons.

4.3 Landslides and Slope Stability

The topography of the site is of a sufficiently shallow gradient that we do not consider landsliding to pose a potential hazard at the site. No evidence of active landsliding, slumps or debris slides was observed in the site vicinity during our site reconnaissance or exploration. No landslides are shown within the building areas on the published geologic maps we reviewed for this investigation and evaluation. The nearest banks of the former lumber mill pond are over 400 feet west of the property. The nearest ocean bluffs are 1500 feet from the property.

4.4 Faulting and Seismicity

The site is within the Coast Ranges geomorphic province, a zone of high seismic activity associated with the active San Andreas Fault system, which passes off the Mendocino coast about 6.7 miles (10.8 kilometers) southwest of the site, as shown on the Regional Active Fault Map, Plate 18. According to publish maps, the site is not located within an Alquist-Priolo Earthquake Fault zone.

The San Andreas Fault system is the boundary between the northward moving Pacific Plate (west of the fault) and the southward moving North American Plate (east of the fault). In the Fort Bragg area of coastal Mendocino County this movement is distributed across a complex system of strike-slip, right-lateral, parallel and subparallel major regional active faults, the closest of which include the San Andreas and Maacama (north) faults, and other more distant faults located more than 50 kilometers (km) from the site.

Fort Bragg was badly damaged during the 1906 earthquake on the San Andreas Fault. According to eyewitness accounts quoted in the Lawson Report (Reference 17) "Several brick buildings were completely demolished; others had parts of their walls broken off. Even a number of wooden buildings collapsed or were partly wrecked."

No evidence of active or incipient faulting was observed at the property during our site exploration, and no faults are shown on, or trending toward the property on the published geologic maps we reviewed. No visual linements indicative of faulting were observed in the aerial photographs that we studied. An inactive fault is located approximately 0.72 miles (1.16 kilometers) southwest of the site as shown on Plate 17. Inactive faults within the Franciscan Complex are common. Without geomorphic evidence of Holocene activity, such as creek offsets, linear depressions, scarps, etc., such faults can be considered "inactive".

The northwest trend of Newman Gulch lines up directly with the northwest bend of Pudding Creek. An ancient (inactive) fault may be responsible for these features. This possible fault feature is approximately 3500 feet northeast of the fire station property.

In general, 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. A non-quantitative Earthquake Shaking Potential Map reproduced from the California Geological Survey (CGS) website is presented on Plate 21. Due to its proximity to the San Andreas Fault, the Fort Bragg area is within a region expected to experience strong ground shaking during future earthquakes.

5.0 DISCUSSIONS AND CONCLUSIONS

5.1 General

Based upon the results of our investigation and review of available geologic data, we conclude that the site is suitable for the building improvements or future reconstruction at the site. The main geotechnical constraints that need to be considered in the design and construction of site improvements are strong seismic shaking from future earthquakes, potential soil liquefaction and densification, potential settlement, and tsunami/storm wave hazard. These considerations, their suggested mitigation measures, and other potential hazards are discussed below. Other specific aspects of this project are discussed in the following subsections.

After submitting a draft report to I. L. Welty & Associates and discussion with Lee Welty, BACE understands that the current approach is to use compaction grouting to reinforce the terrace deposit sands that have a potential of liquefaction. BACE has provided recommendations for compaction grouting along with other recommendations.

5.2 High Ground Water

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. 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.

5.3 Weak Soils

The majority of sandy deposits overlying the bedrock at the site are loose to medium dense (as observed in our borings). Fill soils are likely to have variable degrees of compaction. Therefore, these soils are not suitable for support of the existing shallow foundations or future foundations. Recommendations for deepening of foundations below the weak soil zones or reinforcing the soils are presented in the Recommendations Section of this report.

5.4 Settlement

The terrace deposits encountered during our investigation are compressible for anticipated (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 under static conditions, settlement due to seismic conditions could be greater. See Section 5.6, Soil Liquefaction and Densification, for approximate settlement due to seismic loads, and for additional settlement considerations. If a mat foundation, compacted aggregate piers or compacted stone column system is to be considered, BACE should be retained to provide estimated settlement behavior.

5.5 Potential Seismic Hazards

5.5.1 Faulting

The August 1998 CDMG Active Fault Near-Source Zones map (page B-11 of Reference 12), Plate 18, covering the site vicinity indicates that the site is approximately 10.8 kilometers from the nearest known seismic source. Since the nearest active fault (San Andreas Fault) is located about 10.8 kilometers (6.7 miles) away, and no evidence of faulting was observed at the site, nor shown on the geologic maps and literature we reviewed, we do not consider surface fault rupture to pose a hazard to the site.

5.5.2 Seismic Parameters for Nearby Faults

As previously stated, the site coordinates are 39.44240 degrees North (Latitude) and – 123.80621 degrees West (Longitude). Nearby active faults (within 50 kilometers) are summarized in Table 1.

Table 1
Seismic Parameters for Nearby Active Faults [7]

Fault	Geometry	Closest Distance from Fault to Site[1]		Maximum Moment Magnitude	Average Slip Rate (mm/yr)	Fault Class (CGS, 2003)
		Miles	Kilometers			
San Andreas (North Coast North)	Right lateral-Strike Slip	6.7	10.8	7.3	24.0	A
Maacama (North)	Right lateral-Strike Slip	21.2	34.1	7.1	9.0	B
Maacama (Central)	Right lateral-Strike Slip	28.2	45.3	7.1	9.0	B

5.5.3 Ground Shaking

As is typical of the Mendocino County coastal area, the property will be subject to strong ground shaking during future earthquakes on the San Andreas Fault. According to the

USGS the 1906 earthquake, which occurred on the San Andreas central segment, has been redetermined to have a moment magnitude of 7.7 to 7.9. The USGS deaggregation tool, http://eqint.cr.usgs.gov/deaggint/2002/index.php, the most probable model magnitude is considered to be M7.88, which BACE is using for our analysis. As shown above in Table 1, CGS has given the San Andreas, north coast north segment, a maximum moment magnitude of 7.3. Typically, structures founded in firm soil and rock materials, and designed in accordance with current building codes, are well suited to resist the effects of ground shaking. 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.

Probabilistic seismic hazard analysis (PSHA) was performed, using USGS seismic Hazard Curves and Uniform Hazard Spectra software version 5.0.9 and FRISKSP version 4.00. The input and output data files for the analyses are presented in Appendix C.

Horizontal peak ground acceleration values were calculated for ground motions having a 10-percent chance of exceedance in 50 years. Horizontal peak ground acceleration value has been calculated using Ss/2.5 (in accordance with ASCE 7-5 11.8.3) and a site-specific evaluation. The calculated horizontal peak ground acceleration using Ss/2.5 is 0.60g and the site-specific evaluation is 0.67g. For magnitude-scaling in the liquefaction analyses discussed in Section 5.5.5 of this report, this probabilistic acceleration of 0.67g was judgmentally associated ("deaggregated") with an earthquake having a maximum moment magnitude of 7.9, occurring on the San Andreas which has the closest distance from the site of approximately 10.8 km. The effects of ground shaking, including ground acceleration and horizontal ground motion, and seismically induced ground failures, are discussed in the following subsections and in Section 5.5.5.

5.5.4 Seismic Design Criteria

BACE understands that the structures do not have irregular shapes or other structural constraints that could require special seismic design provisions (beyond current code provisions). Therefore, the structures may be designed in accordance with California Building Code criteria for an essential facility, with review by California Geological Survey (CGS), in accordance with CGS's most current guidelines. Structural design using normalized response spectral accelerations or scaled time histories of earthquake ground motions appear un-warranted for these structures. Recommended geological seismic design parameter valves, for use in structural seismic design of the buildings are provided in Section 6.3 of this report.

5.5.5 Liquefaction

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

As observed in our test borings, the soil layers overlying the bedrock contain very loose to medium dense, saturated sands that could liquefy during a strong earthquake on the nearby San Andreas Fault. Due to the poor grain-size distribution of the terrace sands present at the site, low density, low strength and the presence of shallow groundwater, we consider the site to have a potential for liquefaction during seismically induced ground shaking, resulting in differential settlement. To confirm this conclusion we performed laboratory testing of the soils and a computer seismologic analysis using LIQUEFY2 Version 1.50. (see Appendix D for LIQUEFY2 data). The laboratory testing and analysis confirmed that the terrace deposits above the bedrock, in a saturated condition are liquefiable.

The approach used to perform the analyses included using a probabilistically determined horizontal peak ground acceleration value of 0.67g at the site, in accordance with ASCE 7-05. BACE also used recommendations presented in Recent Advances in Soil Liquefaction Engineering: a unified and consistent framework by Seed and others for evaluating the liquefaction potential. The analyses indicate that the silty sand or sand with few fines encountered in our borings would be potentially susceptible to liquefaction as shown in Appendix D.

Where the probability of liquefaction factor of safety (FS) was 1.3 or less, we performed an analysis to quantify induced vertical settlement due to liquefaction. This analysis was based on Tokimatsu and Seed procedures (1987) and "Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework" by Seed and others, April 30, 2003. The results of our analysis determined that the "theoretical" liquefaction induced soil settlement is approximately 0.9 inches and 2.6 inches at Boring B-3 and B-5, respectively, see Appendix D for liquefaction-induced settlement calculations.

If liquefaction were to occur at this site the possible consequences include settlement of the ground surface associated with the post-earthquake dissipation of excess pore-water pressures from the liquefied zones. In addition, existing structures could experience relatively large differential settlement.

Existing or future buildings on shallow spread footings could be severely damaged by liquefaction within the upper soil zones. If the soils directly under a building foundation should liquefy, the foundation elements could shift dramatically, especially if the foundations are not adequately reinforced.

To mitigate the concern of liquefaction, existing or future new structures should be supported on drilled piers penetrating the underlying supporting bedrock. Recommendations for drilled piers are given in Section 6.0 of this report. As an alternative, for the existing structures the terrace deposits can be reinforced by compaction/pressure/chemical grouting.

Lateral spreading or lurching is generally caused by liquefaction of marginally stable soils underlying gently to steeply-inclined slopes. In these cases, the saturated soils move toward an unsupported face, such as an incised river channel or body of water. We conclude that conditions for lateral spreading do not exist at the site, and that there is not a potential for lateral spreading.

5.6 Tsunami/Storm Waves

As typical of the Mendocino 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. In 1964, a magnitude 9.2 earthquake in Alaska caused 21-foot waves in Crescent City and significant effects and damage as far south at San Francisco Bay. More recently, the 1992 Cape Mendocino tsunami spurred additional research within the scientific community of historical tsunamis and risk analysis of future events along the California coast. Local earthquakes, or those occurring within the Pacific Basin, may trigger tsunami waves that have the potential to reach (and potentially damage properties on) the California Coast. The region from Cape Mendocino (Humboldt County) to north of Monterey is considered at moderate risk of tsunami events generated by local earthquakes.

Within the City of Fort Bragg, designated "low-lying areas" include these areas below the 60-foot elevation level. Since the property elevation is approximately 60 feet or more above MSL, the property is not within a designated low-lying area. Wave inundation is thus unlikely during an earthquake or large storm event.

5.7 Flooding

Review of the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM), Community-Panel Number 060184 0005 C, dated June 16, 1992, indicates that the Site is in Zone X. Zone X is an area FEMA has determined to be outside the 500-year flood plain.

5.8 Soil Corrosivity

In order to assess the potential of the near-surface soil chemistry to corrode subsurface utility conduits and other subsurface structures, we submitted samples of the near-surface soils to our subconsultant, JDH Corrosion Consultants, Inc. for basic soil corrosivity testing. The laboratory test results, interpretation of the raw data and associated recommendations for corrosion control are presented in Appendix B.

6.0 RECOMMENDATIONS

6.1 Site Grading (for new structures)

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 or stockpiled for later use in landscape areas, as appropriate.

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 Structural Areas, existing weak soils should be removed to a depth of at least 5 feet below soil subgrade (SSG) 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 SSG. In some areas, deeper excavations may be necessary to remove weak soils, if encountered.

After the recommended excavations, a BACE representative should observe the exposed soils to confirm suitable materials are exposed. These soils should then be scarified to about six inches deep, moisture conditioned at or near optimum moisture content (OMC) and compacted to at least 90 percent relative compaction (RC) as determined by the ASTM D 1557 test procedure, latest edition. These moisture conditioning and compaction procedures should be observed by BACE to check 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 BACE 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 OMC, and compacted to at least 90 percent RC, to achieve planned grades.

6.2 Foundation Support (for existing or new structures)

6.2.1 General

As encountered in our test borings, the site is underlain by approximately 9.5 to 13.5 feet of weak, relatively loose soils that are highly permeable, and seasonally saturated by the high ground water conditions and high liquefaction potential. These soils are unsuitable for existing foundation support in their current state or for new foundations. Structure foundations and concrete slabs placed directly upon weak, porous or liquefiable soils could undergo damaging differential settlement due to porous soil collapse or loss of shear strength due to liquefaction when loaded in a saturated condition.

Foundations for the existing structures must penetrate through these upper, weak-liquifiable soils using cast-in-place drilled piers, or the liquefiable soil needs to be reinforced by either compaction/pressure/chemical grouting. Foundations for new structures can be cast-in-place drilled piers, mat foundation on top of reinforced soils, or shallow foundation and slabs gaining support on aggregate piers, compacted stone columns, or dynamic compaction. If mat foundation, aggregate pier, or compacted stone column is selected BACE should be retaining to provide recommendations. Our recommendations for drilled piers and mat foundation elements are presented below.

6.2.2 Cast-In-Place Drilled Piers (for existing or new structures)

Support for the existing structures or new structures can be obtained using a drilled, cast-in-place concrete pier and grade beam foundation system. Piers should be a minimum of 18 inches in diameter and spaced no closer than three pier diameters, center to center. The piers should penetrate a minimum of five feet into bedrock or drilling refusal using a heavy-duty drill rig, as identified by BACE personnel. The weak and/or liquefiable terrace deposits should be neglected for support (9.5 to 13.5 feet as observed in our test borings). The minimum average pier depth is anticipated to range from about 14.5 to 18.5 feet below the existing ground surface or deeper based on structural loads. Pier depth should be verified in the field by BACE personnel.

For static conditions skin friction can be obtained from the terrace deposits and the underlying bedrock. An allowable skin friction of 160 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 700 psf 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.

Piers also need to be designed to resist downdrag due to liquefaction. Downdrag load (or negative skin friction) can be assigned 600 psf of shaft area within the terrace deposits.

For static conditions resistance to lateral loads using passive earth pressure of 280 psf per foot of depth (triangular distribution) in the terrace deposits. Resistance to lateral loads should be neglected within the weak/liquifiable materials for dynamic conditions. Resistance to lateral loads can be obtained in the bedrock using passive earth pressure of 1,000 psf plus 250 psf per foot of depth (trapezoidal distribution). 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 BACE 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 BACE. If necessary, pier holes should be dewatered prior to placement of reinforcing steel and concrete. Alternatively, concrete can be tremmied 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 placed or designed to be left in place in low over head areas.

6.2.3 Mat (for new structures)

Satisfactory foundation support for new structures 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.

6.2.4 Spread Footing (for existing or new structures)

If terrace deposits below the existing structures are upgraded by means of compaction/pressure/chemical grouting, footings can be assigned an allowable soil bearing pressure of 2,000 pounds per square foot (psf) for dead plus long-term-live loads. A 25 percent increase in bearing pressure is allowable for dead plus all live loads, and 50 percent increase in bearing pressure is allowable for total load, including wind or seismic loads. New footings, if needed, should conform to current California Building Code (CBC) dimensions and depths.

Resistance to lateral loads can be obtained using passive earth pressure against the face of the foundations. An allowable passive pressure of 280 psf per foot of depth (triangular distribution) in the upgrade terrace deposits.

6.2.5 Compaction Grouting (for existing or new structures)

The liquefaction potential soils above the bedrock can be upgraded by compaction grouting. An experienced contractor using current state of the art methods should perform the compaction grouting. BACE should observe grouting, and assess if grouting has reinforced the liquefaction potential soils. To assess the grouting BACE will need to perform test borings during the grouting operations. The compaction grouting will need to reinforce the subsurface soils to a minimum Standard Penetration Test (SPT) blow count (N) of at least 20.

6.3 Seismic Design Criteria

The structures should be designed and/or constructed to resist the effects of strong ground shaking (on the order of Modified Mercali Intensity IX) in accordance with current building codes. The California Building Code (CBC) 2007 edition indicates that the site classification for the property is Site Class F, due to the liquefiable soils. BACE is anticipating that the fundamental period of vibration will be equal to or less than 0.5 seconds, for 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, BACE will need to re-evaluate the site and may need to perform a site response analysis. For design purposes BACE is using Site Class D and CBC indicates that the following seismic design parameters are appropriate for the site:

Site Class = D
Mapped Spectral Response Acceleration at 0.2 sec S_S = 1.500g
Mapped Spectral Response Acceleration at 1.0 sec S_1 = 0.675g
Modified Spectral Response Acceleration at 0.2 sec S_{MS} = 1.500g
Modified Spectral Response Acceleration at 1.0 sec S_{M1} = 1.012g
Design Spectral Response Acceleration at 0.2 sec S_{DS} = 1.000g
Design Spectral Response Acceleration at 1.0 sec S_{D1} = 0.675g
Site Coefficient F_a = 1.0
Site Coefficient F_v = 1.5
Occupancy Category = IV
Seismic Design Category = D

6.4 Retaining Walls

If retaining or subsurface walls are utilized for existing or new structures, walls should be provided with permanent back drainage to prevent buildup of hydrostatic pressure. Drainage and backfill details are presented on Plate 22. 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 23. These pressures do not consider additional loads resulting from adjacent foundations, vehicles, or other downward surcharge loads. BACE 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 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.

6.5 Concrete Slab Floor Support (for existing or new structures)

If a structural concrete slab is used (i.e., a slab 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 the upper 3 to 5 feet of subgrade needs to be compacted for resisting lateral loads on piers).

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 3 to 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 0.9 to 2.6 inches or more.

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 RC 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 24. Where migration of moisture through the floor slab would be detrimental to its intended use, the installation of a vapor retarder membrane should be considered. The membrane should be at least 10-mils thick and should be overlapped a minimum of 2 feet between adjoining sheets or taped with moisture resisting tape. Construction of vapor retarders does not guarantee the prevention of moisture moving through slabs.

6.6 Site Drainage

Because surface and/or subsurface water is often the cause of foundation stability problems, care should be taken to intercept and divert concentrated surface flows and subsurface seepage away from the building foundations. Roof runoff water should be directed away from the buildings and dispersed, as much as practical, across the site. Drainage across the site should be by sheet-flow. Surface grades should maintain a recommended five percent gradient away from building foundations.

7.0 ADDITIONAL SERVICES

Prior to construction, BACE should review the final grading and foundation plans, and soil related specifications for conformance with the intent of our recommendations. During construction, BACE should be retained to provide periodic observations, together with the appropriate field and laboratory testing during site grouting, preparation, subdrain installations, and/or foundation excavations (pier drilling). Foundation excavations should be reviewed by BACE 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

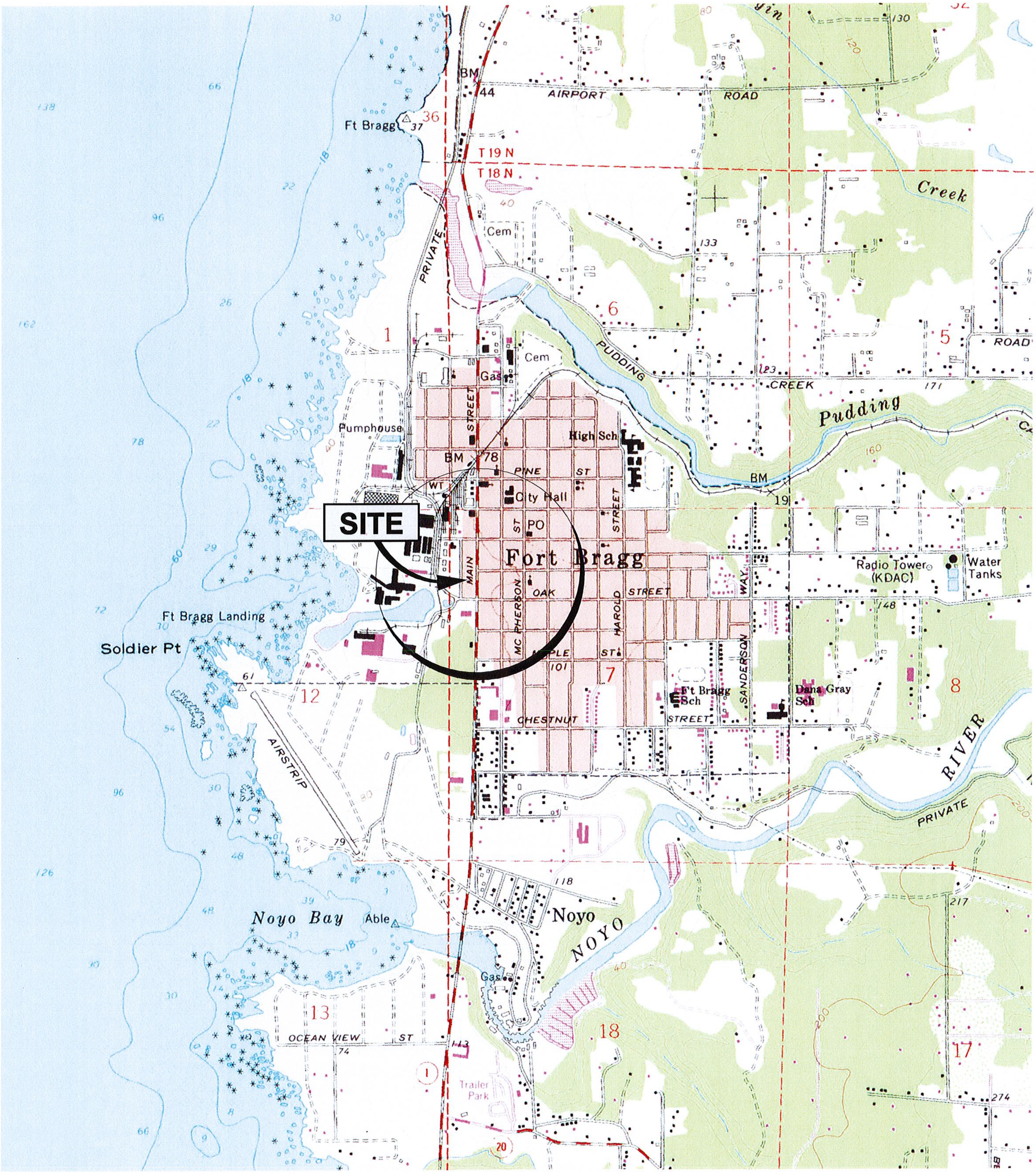
This geotechnical investigation and engineering geologic reconnaissance of the property were performed in accordance with the usual and current standards of the profession, as they relate to this and similar localities. No other warranty, expressed or implied, is provided as to the conclusions and professional 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 BACE, 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 BACE 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 current building locations, 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.



REFERENCE:

Fort Bragg, 1978,
7.5 Minute Quadrangle Topographic Map, USGS

NORTH

39.44240° North
-123.80621° West

APPROXIMATE SCALE (FEET)

0 1,000 2,000 4,000

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Appr.: KAC

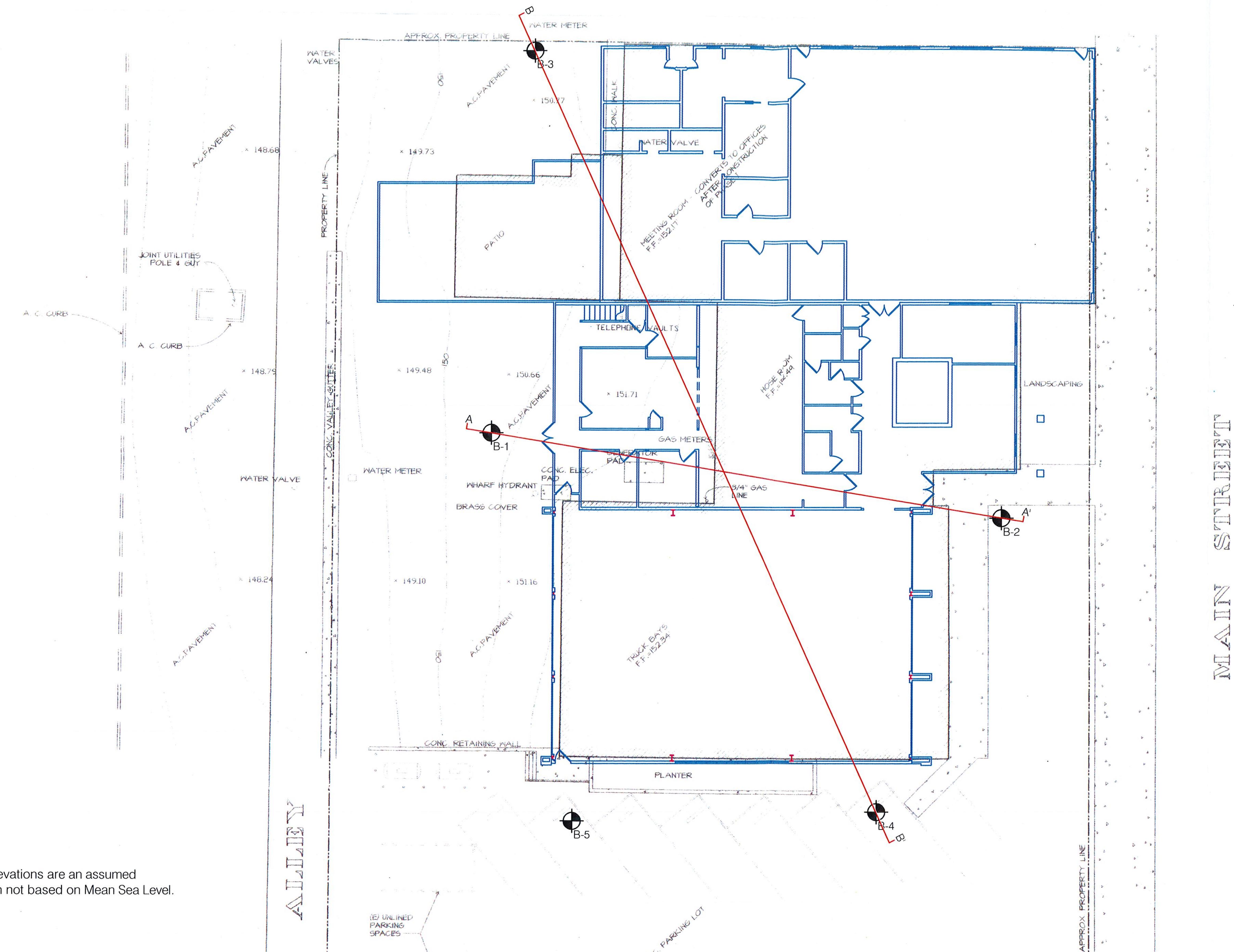
Date: 07/16/09

VICINITY MAP

FORT BRAGG FIRE DEPARTMENT
STATION #830
141 Main Street
Fort Bragg, Mendocino County, California

PLATE

1



LEGEND

Number and approximate location of test boring (B-5)

Geologic cross section locations (B – B')

Note: Elevations are an assumed elevation not based on Mean Sea Level.

REFERENCE:
Floor Plan by I.L. Welty and Associates, dated 12/2008. Overlaying Existing Site Plan by Edward Taubold Architect, dated 2-24-97.

NORTH

APPROXIMATE SCALE (FEET)

0 10 20 40

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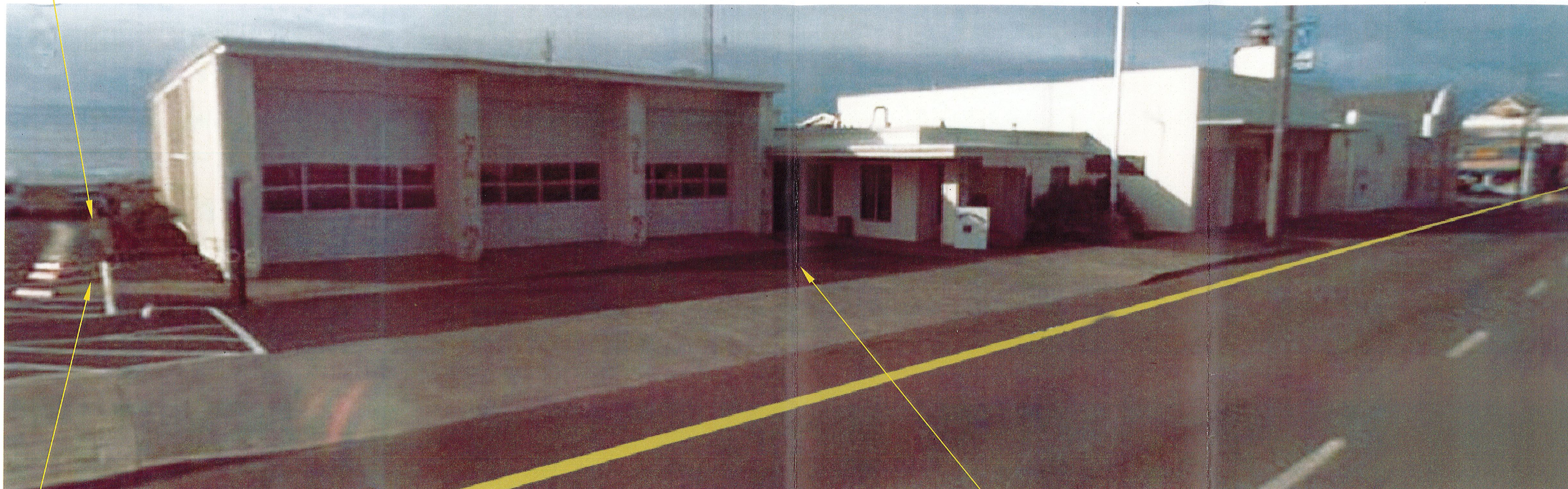
SITE PLAN
FORT BRAGG FIRE DEPARTMENT
STATION #830
141 Main Street
Fort Bragg, Mendocino County, California

PLATE
2

SITE PHOTOGRAPH A

Fort Bragg Fire Department Station #830
Looking north-northwest



REFERENCE:
Google Maps Image

BACE Geotechnical a division of Brunsing Associates, Inc. (707) 528-6108	Job No.: 10511.3 Appr.: KAC Date: 07/16/09	SITE PHOTOGRAPH A FORT BRAGG FIRE DEPARTMENT STATION #830 141 Main Street Fort Bragg, Mendocino County, California	PLATE 3

REFERENCE: California Coastal Records Project, www.californiacoastline.org, by permission.

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Date: 07/16/09

OBLIQUE AERIAL PHOTOGRAPH 2005
FORT BRAGG FIRE DEPARTMENT
STATION #830
141 Main Street
Fort Bragg, Mendocino County, California

PLATE
4

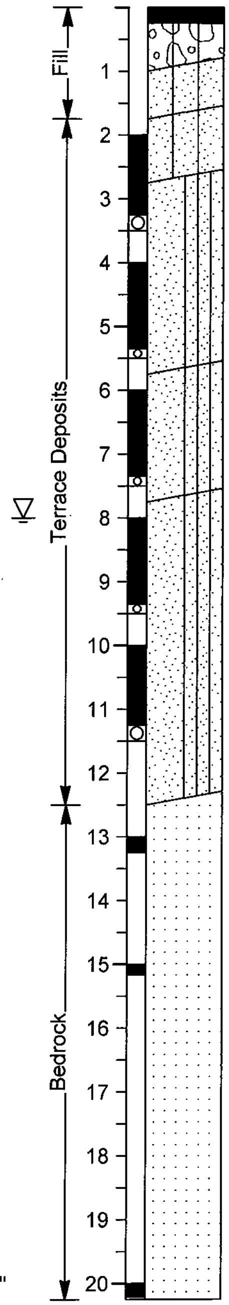
Log of Boring B- 1

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

Date: 1/6/09

Logged By: JEW **Elevation:** 150.5 feet ***

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)
15% Passing #200 100% Passing #4	CM	7.7	98	8**	3
	CM	7.0	107	17**	5
DS 1343 (1500)		13.2	116		
	CM			16**	7
6% Passing #200 100% Passing #4	CM	19.1	106	20**	9
DS 3183 (3000)		17.9	110		
	CM			12**	11
	SPT			50/3"	13
	SPT			50/2"	15
	SPT			50/2.5"	20



ASPHALT
LIGHT GRAY SILTY GRAVEL (GM) with sand
loose, damp, (Aggregate Base)
BROWN-GRAY SILTY SAND (SM)
loose, damp, medium to coarse grained sand
DARK BROWN SILTY SAND (SM)
loose, damp, fine to medium grained sand, organic odor

REDDISH-BROWN TO BUFF SAND (SP-SM) with silt
loose to medium dense, damp, medium to coarse grained, no odor, oxidized zones

BUFF TO LIGHT BROWN SAND (SP-SM) with silt
medium dense, wet, coarse grained sand

LIGHT BROWN SAND (SP-SM) with silt
medium dense, saturated, coarse grained with trace medium grained sand, mostly quartz, well rounded to subrounded

GRAY-BROWN TO BLUE-GRAY SANDSTONE
intense fracturing, friable to low hardness, deeply weathered, fine to medium grained

- becomes hard, little weathered

Notes:
1. No caving.
2. Free water encountered at about 8.0 feet.
3. Practical drilling refusal at 20.0 feet bgs.

*CA - California Modified Split Tube Sampler 3.0-inch O.D.
CM - California Modified Split Tube Sampler 2.5-inch O.D.
SPT - California Split Tube Sampler 2.0-inch O.D.
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Edward Taubold Architect, dated 2-24-97.

Scale: 1" = 3'

BACE GEOTECHNICAL a division of Brunsing Associates, Inc. (707) 528-6108	Job No.: 10511.3 Appr.: KAC Date: 07/16/09	**LOG OF BORING B- 1** FORT BRAGG FIRE DEPARTMENT STATION #830 141 Main Street Fort Bragg, Mendocino County, California	PLATE **5**

Log of Boring B- 2

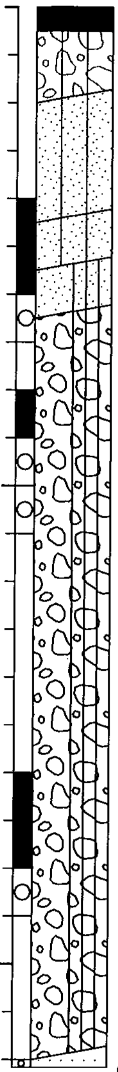
Equipment: Mobile B-53; 7-inch hollow-stem auger

Date: 1/6/09

Logged By: JEW **Elevation:** 152.5 feet ***

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)
		6.5	100		
	CM			27**	3
	CM			55/7.5"**	5
Tx 3773 (1152)		5.6	125		
	CM			49/9"** ▽	9
	SPT			50/1"	11

Fill (0 to 11 ft.)



ASPHALT

LIGHT BROWN TO DARK GRAY-BROWN SILTY SANDY GRAVEL (GM)
medium dense, damp

DARK BROWN SILTY SAND (SM)
loose to medium dense, damp, medium grained sand, trace pebbles, oxidized zones

REDDISH-BROWN SILTY SAND (SM)
medium dense, damp, fine to medium grained sand

LIGHT TO BUFF-BROWN SAND (SP-SM) with silt
medium dense, damp, medium to coarse grained sand

BROWN-GRAY TO BLUE-GRAY SANDY GRAVEL (GP-GM)
very dense, wet to saturated

GREEN-GRAY WITH ORANGE-BROWN MOTTLING SANDSTONE
intense to close fracturing, moderate hardness to hard, moderate weathering

Notes:
1. No caving
2. Free water encountered at about 9.0 feet bgs.
3. Practical drilling refusal at 11.0 feet bgs.

*CA - California Modified Split Tube Sampler 3.0-inch O.D.
CM - California Modified Split Tube Sampler 2.5-inch O.D.
SPT - California Split Tube Sampler 2.0-inch O.D.
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Edward Taubold Architect, dated 2-24-97.

Scale: 1" = 2'

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Date: 07/16/09

LOG OF BORING B- 2
FORT BRAGG FIRE DEPARTMENT
STATION #830
141 Main Street
Fort Bragg, Mendocino County, California

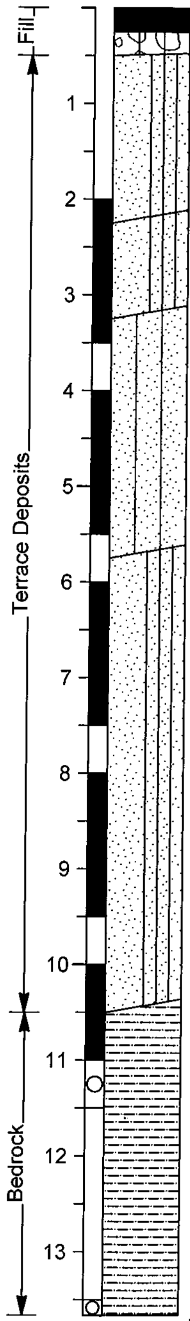
PLATE
6

Log of Boring B- 3

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

Date: 1/6/09

Logged By: JEW **Elevation:** 150.75 feet ***



Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)	Description
					0	ASPHALT
						LIGHT GRAY SILTY GRAVEL (GM) with sand medium dense, damp (Aggregate Base)
						DARK BROWN TO REDDISH-BROWN SILTY SAND (SP-SM) very loose to loose, damp, fine to medium grained sand
		13.3	91			REDDISH-BROWN TO BUFF-BROWN SAND (SP-SM) with silt very loose, damp, medium to coarse grained sand
	CM			4**	3	
						LIGHT BROWN TO BUFF-BROWN SILTY SAND (SM) very loose to medium dense, damp to wet, medium grained sand, primarily quartz, subrounded (rare), rounded (predominate)
19% Passing #200 100% Passing #4		10.1	112		4	
	SPT			21	5	
						BUFF-BROWN MOTTLED WITH ORANGE-BROWN SAND (SP-SM) with silt medium dense, damp to wet, medium grained sand, primarily quartz, subrounded (rare), rounded (predominate)
		9.2	112			
	SPT			22	7	
					8	- moisture increase at approximately 8.0 feet bgs.
8% Passing #200		18.8	109			
	SPT			16	9	
						REDDISH-BROWN TO GRAY-BROWN SILTSTONE intensely to little fractured, moderate hardness to hard, deep to moderate weathering with depth, calcite veins in fractures
	SPT			75/9"	11	
	SPT			50/2"		

Notes:
1. Caving occured after removing hollow-stem auger.
2. No free water encountered
3. Practical drilling refusal at 13.5 feet bgs.

*CA - California Modified Split Tube Sampler 3.0-inch O.D.
CM - California Modified Split Tube Sampler 2.5-inch O.D.
SPT - California Split Tube Sampler 2.0-inch O.D.
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Edward Taubold Architect, dated 2-24-97.

Scale: 1" = 2'

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Appr.: KAC
Date: 07/16/09

LOG OF BORING B- 3
FORT BRAGG FIRE DEPARTMENT
STATION #830
141 Main Street
Fort Bragg, Mendocino County, California

PLATE
7

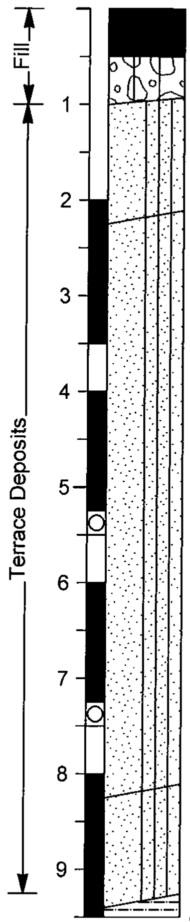
Log of Boring B- 4

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

Date: 1/6/09

Logged By: JEW **Elevation:** 152.5 feet ***

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)
	CM	6.6	89	10**	3
DS 348 (500)		7.7	111		
	CM			17**	5
11% Passing #200 100% Passing #4		7.7	112		
	SPT			17	7
	SPT			15	9



ASPHALT

LIGHT GRAY SILTY GRAVEL (GM) with sand
medium dense, damp

DARK BROWN-GRAY SILTY SAND (SP-SM)
loose, damp, fine to medium grained sand

LIGHT BROWN TO BUFF-BROWN SAND (SP-SM) with silt
loose to medium dense, damp to wet, medium to corase grained sand

GREEN-GRAY SAND (SP-SM) with silt
medium dense, wet, coarse-grained sand, strong petroleum odor

GREEN-GRAY SILTSTONE
crushed, soft, deeply weathered

Notes:
1. No free water encountered
2. No caving
3. Drilling terminated at 9.5 feet due to petroleum odor

*CA - California Modified Split Tube Sampler 3.0-inch O.D.
CM - California Modified Split Tube Sampler 2.5-inch O.D.
SPT - California Split Tube Sampler 2.0-inch O.D.
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Edward Taubold Architect, dated 2-24-97.

Scale: 1" = 2'

BACE GEOTECHNICAL a division of Brunsing Associates, Inc. (707) 528-6108	Job No.: 10511.3 Appr.: KAC Date: 07/16/09	**LOG OF BORING B- 4** FORT BRAGG FIRE DEPARTMENT STATION #830 141 Main Street Fort Bragg, Mendocino County, California	PLATE **8**

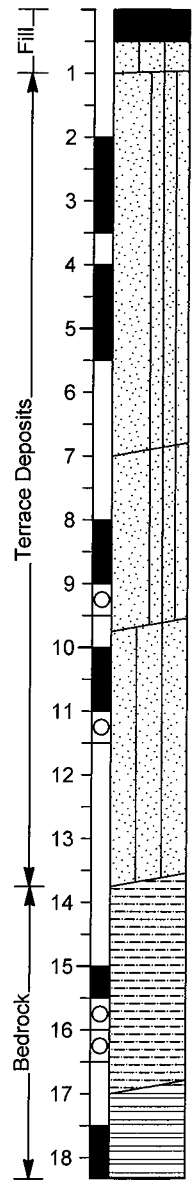
Log of Boring B- 5

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

Date: 1/6/09

Logged By: JEW **Elevation:** 149.0 feet ***

Laboratory Tests	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft.)	Description
					0	ASPHALT
						DARK BROWN TO LIGHT GRAY-BROWN SILTY SAND (SM) loose, damp, grains coarse with depth (fine to medium to medium/coarse
					1	REDDISH-BROWN TO ORANGE-BROWN SAND (SP-SM) with silt loose to medium dense, damp, medium to coarse grained sand, primarily quartz
	CM			19**	3	
		14.2	110			
	CM			17**	5	
					7	LIGHT BUFF-BROWN TO WHITE SAND (SP-SM) with silt loose to medium dense, saturated, medium grained, mostly quartz
		16.8	110			
	SPT			12	9	
19% Passing #200 87% Passing #4					10	GREEN-GRAY SILTY SAND (SM) loose, saturated, coarse sand and pebbles predominate
	SPT			9	11	
					14	GRAY-BROWN TO BLUE-GRAY SANDY SILTSTONE intense fracturing, moderately hard to hard, moderate weathering, calcite veining
	SPT			50/5"	16	
					17	DARK GRAY SHALE intense to little fracturing, moderately hard to hard, moderately to little weathering, calcite veining
	SPT			89/10"	18	



Notes:
1. No caving
2. Free water encountered at about 8.0 feet
3. Practical drilling refusal at 17.5 feet bgs. Sampling terminated at 18.3 feet bgs.

*CA - California Modified Split Tube Sampler 3.0-inch O.D.
CM - California Modified Split Tube Sampler 2.5-inch O.D.
SPT - California Split Tube Sampler 2.0-inch O.D.
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Edward Taubold Architect, dated 2-24-97.

Scale: 1" = 3'

BACE GEOTECHNICAL a division of Brunsing Associates, Inc. (707) 528-6108	Job No.: 10511.3 Appr.: KAC Date: 07/16/09	**LOG OF BORING B- 5** FORT BRAGG FIRE DEPARTMENT STATION #830 141 Main Street Fort Bragg, Mendocino County, California	PLATE **9**

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS) ASTM D 2487

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

Consol - Consolidation
LL - Liquid Limit
PI - Plasticity Index
EI - Expansion Index
SA - Sieve Analysis
■ Sample Retained
▨ Sample Recovered, Not Retained
☒ Bulk Sample
◘ Sample Not Recovered

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

▽ Ground Water Level During Exploration ▼ Second Ground Water Level Reading

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SOIL CLASSIFICATION CHART & KEY TO TEST DATA
FORT BRAGG FIRE DEPARTMENT
STATION #830
141 Main Street
Fort Bragg, Mendocino County, California

PLATE
10

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

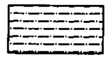
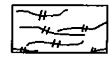
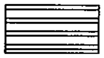
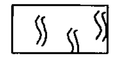
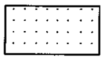
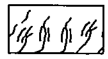
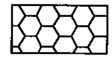
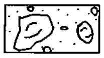
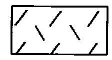
NATURAL MOISTURE CONTENT

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.

Generalized Graphic Rock Symbols

	Claystone		Siltstone		Tuff (Volcanic Ash)
	Shale		Chert		Andesite
	Sandstone		Serpentine		Basalt
	Conglomerate		Metamorphic Rock		Granite

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

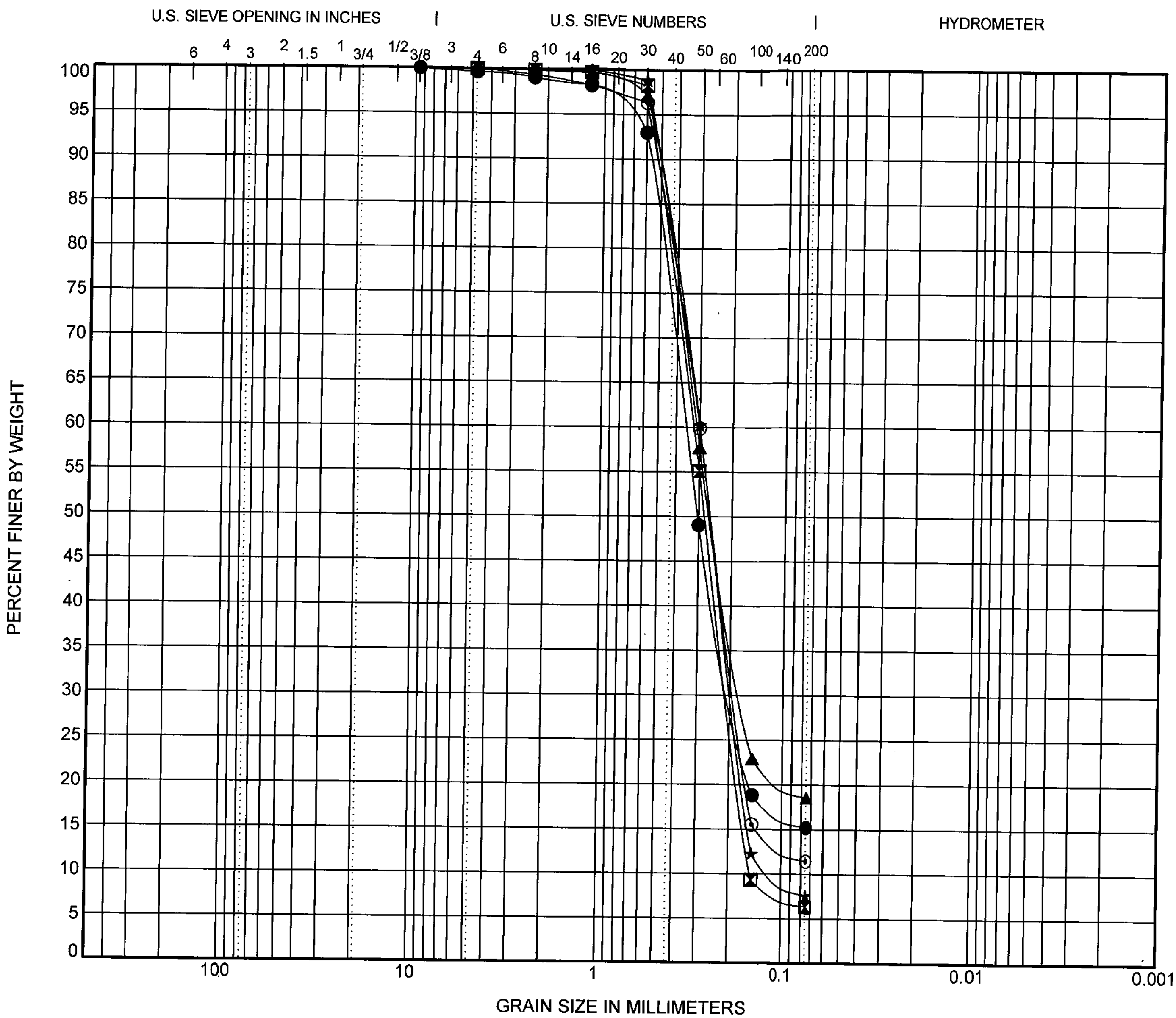
Fracturing Intensity	Thickness of Beds
Little	Greater than 4 feet
Occasional	1 foot to 4 feet
Moderate	6 inches to 1 foot
Close	1 inch to 6 inches
Intense	0.5 inches to 1 inch
Crushed	less than 0.5 inches

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.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● B-1	2.5 ft	**DARK BROWN SILTY SAND (SM)**					
⊠ B-1	8.5 ft	**LIGHT BROWN SAND (SP-SM) with silt**				0.86	2.14
▲ B-3	4.0 ft	**LIGHT BROWN TO BUFF-BROWN SILTY SAND (SM)**					
★ B-3	8.5 ft	**BUFF-BROWN MOTTLED WITH ORANGE-BROWN SAND (SP-SM) with silt**				1.17	2.79
⊙ B-4	6.5 ft	**LIGHT BROWN TO BUFF-BROWN SAND (SP-SM) with silt**				2.00	5.11

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● B-1	2.5 ft	9.5	0.357	0.194		0.3	84.4	15.2	
⊠ B-1	8.5 ft	4.75	0.325	0.205	0.152		93.7	6.3	
▲ B-3	4.0 ft	4.75	0.313	0.173			81.5	18.5	
★ B-3	8.5 ft	2.36	0.299	0.194	0.107		92.4	7.6	
⊙ B-4	6.5 ft	4.75	0.301	0.188			88.6	11.4	

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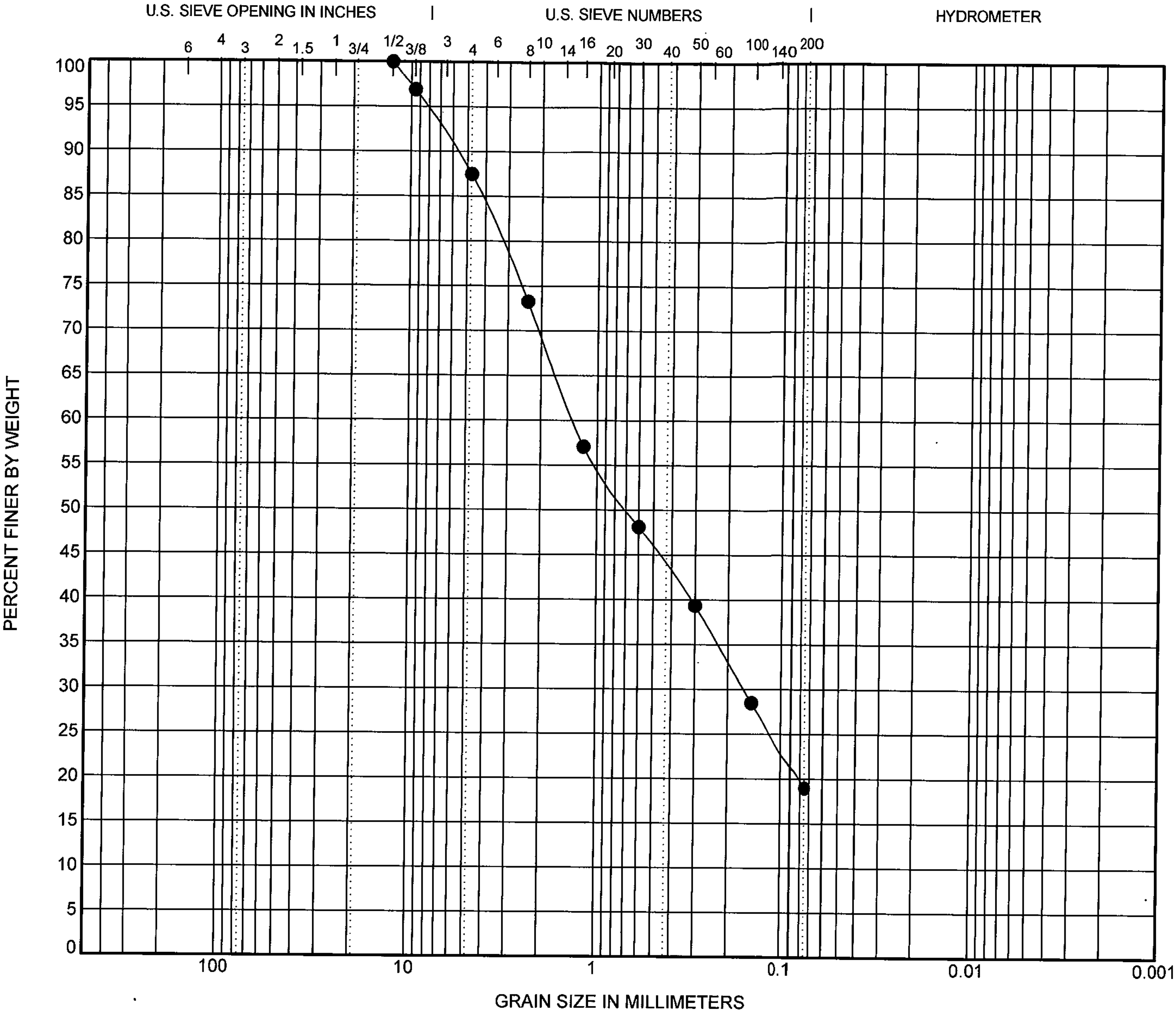
GRAIN SIZE DISTRIBUTION
FORT BRAGG FIRE DEPARTMENT
STATION #830
141 Main Street
Fort Bragg, Mendocino County, California

PLATE
13



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	B- 5	10.0 ft	GREEN-GRAY SILTY SAND (SM)					

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B- 5	10.0 ft	12.5	1.338	0.165		12.5	68.5	19.0	

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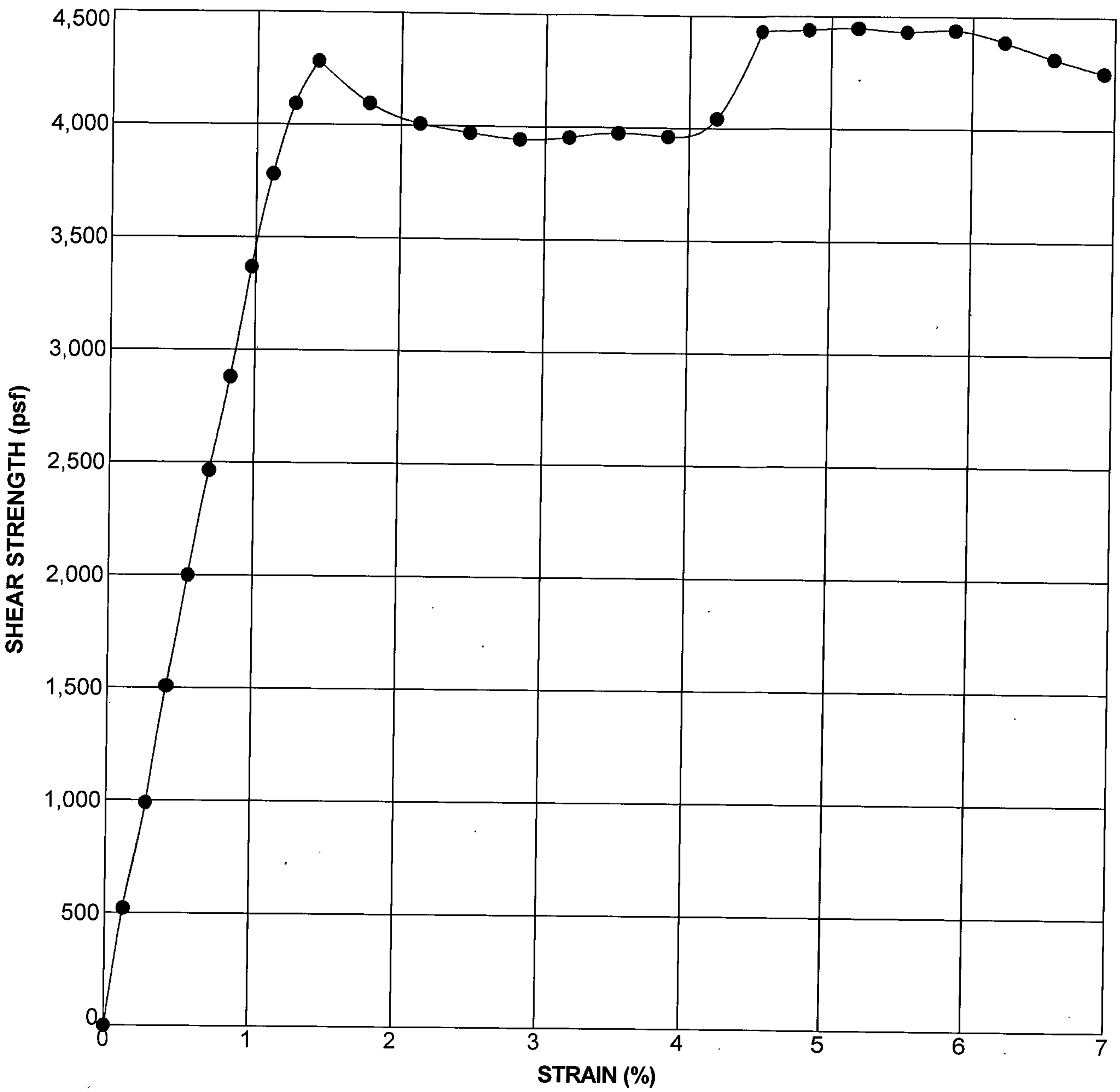
GRAIN SIZE DISTRIBUTION
FORT BRAGG FIRE DEPARTMENT
STATION #830
141 Main Street
Fort Bragg, Mendocino County, California

PLATE
14



Sample Source	Classification	Confining Pressure (psf)	Yield Strength (psf)	Strain (%)	Dry Density (pcf)	Moisture Content (%)
● B-2 @ 8.5 ft	BROWN-GRAY TO BLUE-GRAY SANDY GRAVEL (GP-GM)	1152	3773	1.1	125	5.6

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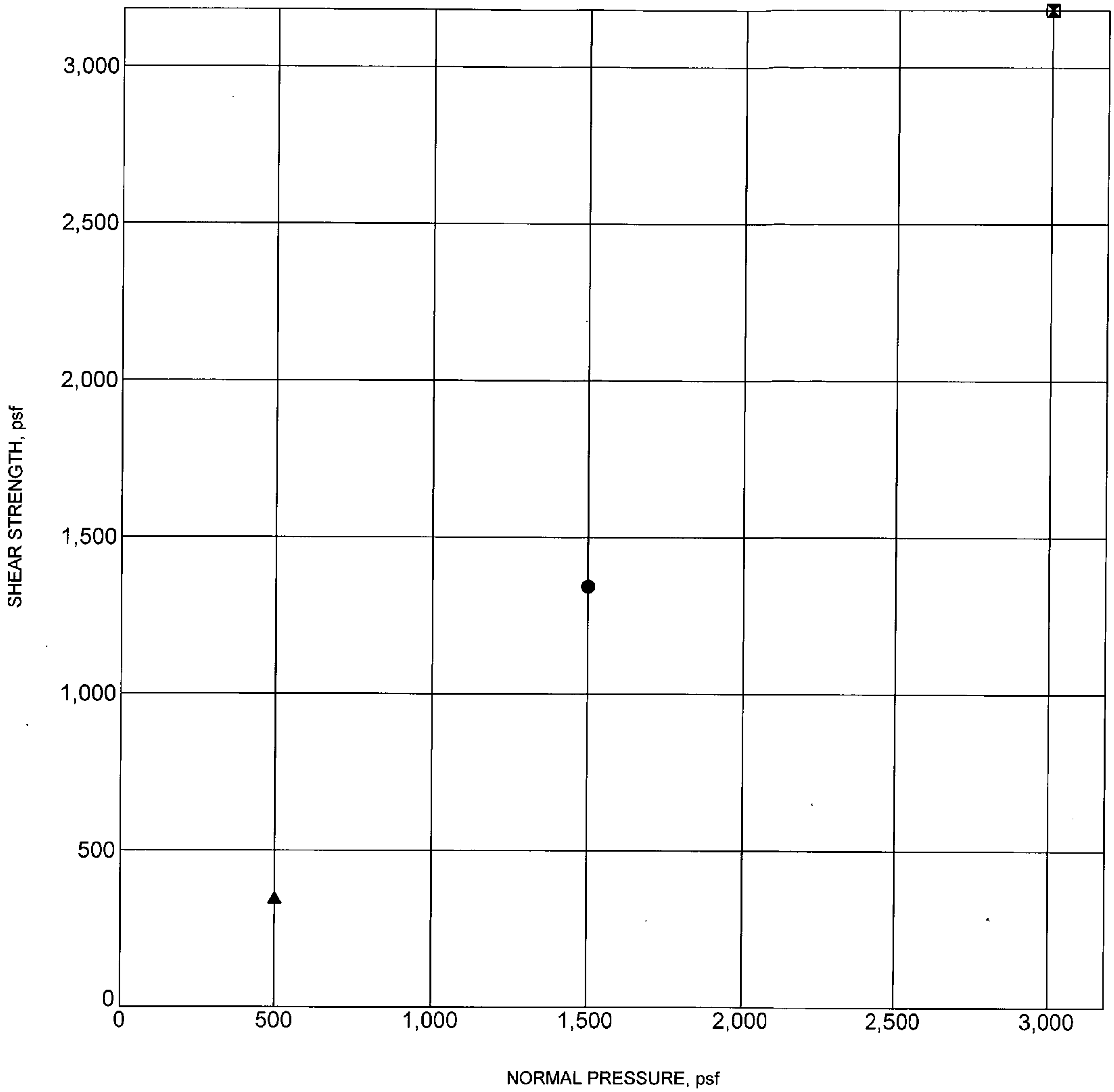
UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

FORT BRAGG FIRE DEPARTMENT
STATION #830
141 Main Street
Fort Bragg, Mendocino County, California

PLATE
15



	Specimen Identification			Classification	γ_d	MC%	c	ϕ (degree)
●	B- 1	at	6.5 ft	BUFF TO LIGHT BROWN SAND (SP) with silt	116	13.2	0	42
⊠	B- 1	at	10.5 ft	LIGHT BROWN SAND (SP-SM) with silt	110	17.9	0	47
▲	B- 4	at	4.5 ft	LIGHT BROWN TO BUFF-BROWN SAND (SP-SM) with silt	111	7.7	0	34
					Average		0	41

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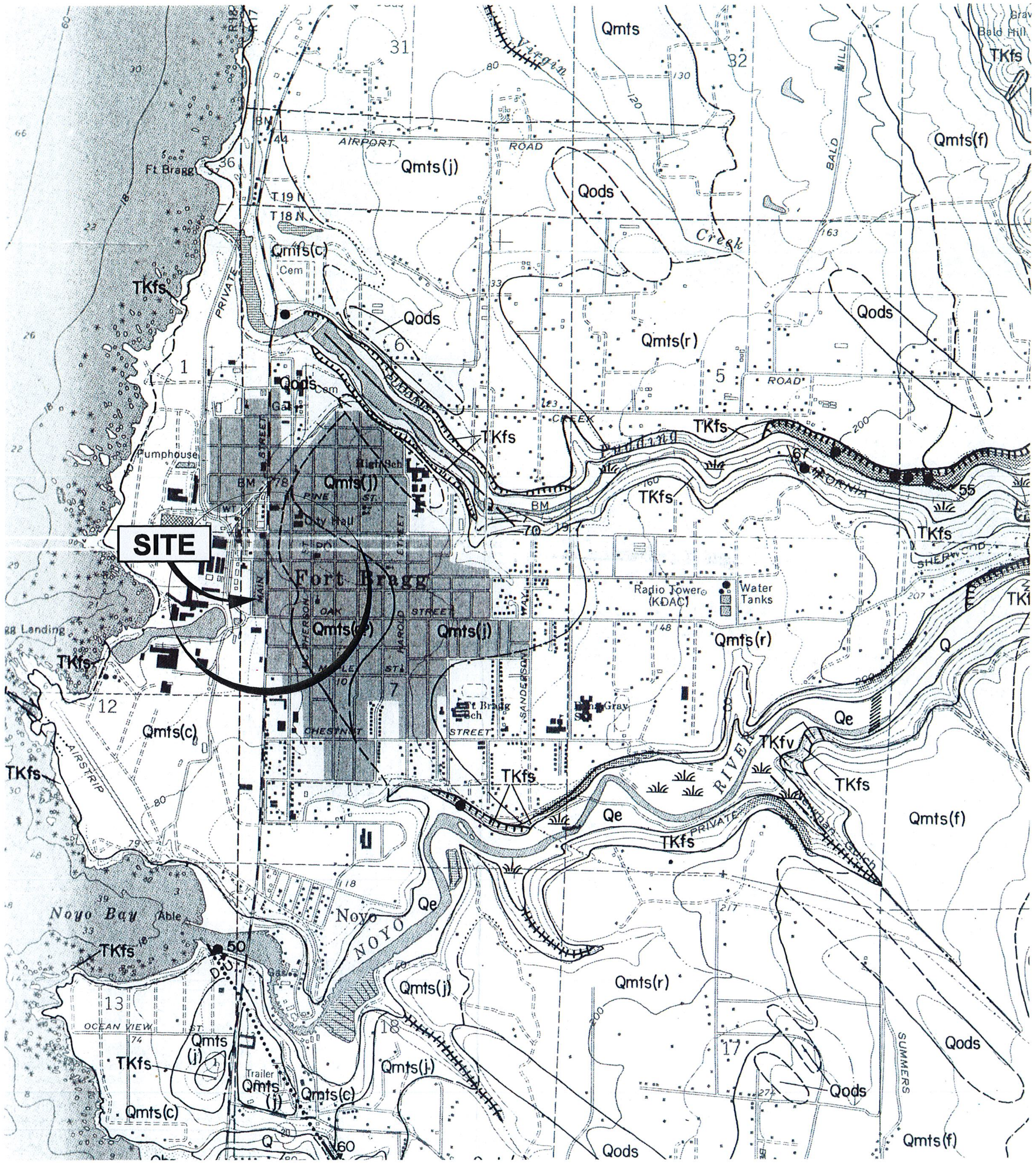
DIRECT SHEAR TEST

FORT BRAGG FIRE DEPARTMENT
STATION #830
141 Main Street
Fort Bragg, Mendocino County, California

PLATE
16



DEBRIS SLIDE: includes scarp and slide deposits; solid where active, dashed where dormant.

DEBRIS FLOW/TORRENT TRACK: solid where active, dashed where dormant.

DEBRIS SLIDE AMPHITHEATER/SLOPE

INNER GORGE: +++ where too narrow to delineate at this scale.

• ACTIVE SLIDE: too small to delineate at this scale.

Q ALLUVIUM (Holocene): unconsolidated, fine-grained sand and silt along modern river flood plains; minor amounts of gravel in channel areas.

Qbs BEACH SAND (Holocene): unconsolidated medium- to coarse-grained quartz sand with lesser amounts of shell fragments and Coastal Belt Franciscan (TKfs) cobbles.

Qe ESTUARINE DEPOSITS (Holocene): unconsolidated dark grey silt and fine sand along intertidal salt marsh estuaries; generally gradational contact with alluvium (Q).

Qods OLDER DUNES (Pleistocene): well-sorted, semi-consolidated fine- to medium-grained quartz sand overlying various terrace deposits (Qmts); recognized by subdued elongate dune profile, generally trending NW; dune deposits tend to be better drained than underlying units.

Qmts MARINE TERRACE DEPOSITS undifferentiated, progressively older with increased elevation (Pleistocene): deposits generally consist of well-sorted quartz sand with minor gravel and have coarser textures near major drainages; dune sands may be present. Elevations of marine terrace deposits listed below are approximate and, due to minor regional deformation, apply only to map area.

Qmts(c) CASPAR POINT marine terrace sediments: name is from stratigraphically equivalent deposits exposed at Caspar Point (W1/2 of Section 1, T17N, R18W) on the Mendocino 7.5' quadrangle; thickness 0 to 30 feet, mostly unconsolidated fine sand, Indian midden deposits common, native arboreal vegetation absent; found from modern sea cliff to an elevation of generally 100(±10) feet.

Qmts(j) JUG HANDLE FARM marine terrace sediments: name is from stratigraphically equivalent deposits exposed at Jug Handle Farm (SE1/4 of Section 36, T18N, R18W) on the Mendocino 7.5' quadrangle; thickness 0 to 10 feet, with frequent relict stacks of TKfs, sporadically forested; elevation generally 100 to 160(±10) feet.

Qmts(r) RAILROAD marine terrace sediments: name is from stratigraphically equivalent deposits exposed along old Caspar railroad right-of-way (Section 31, T18N, R17W) on the Fort Bragg 7.5' quadrangle; elevation generally 160 to 220(±20) feet.

EXPLANATION

Qmts(f) FERN CREEK marine terrace sediments: name is from stratigraphically equivalent deposits exposed along Fern Creek Road (Section 6, T17N, R17W) on the Mendocino 7.5' quadrangle; hardpan sporadically developed in map area; elevation generally 220 to 320(±20) feet.

Qmts(h) HANS JENNY PIT terrace and marine terrace sediments: name is from stratigraphically equivalent deposits exposed in soil test pits along Gibney Lane (NE1/4 of Section 5, T17N, R17W) on the Mendocino 7.5' quadrangle; hardpan well developed in map area; elevation 320 to 415(±25) feet.

Qmts(l) LOWER CASPAR ORCHARD marine terrace sediments: name is from stratigraphically equivalent deposits exposed at Caspar Orchard (SW1/4 of NW1/4 of Section 10, T17N, R17W) on the Mendocino and Glenblair SW 7.5' quadrangles; hardpan usually broken in map area; elevation 415 to 515(±25) feet.

Qmts(u) UPPER CASPAR ORCHARD marine terrace sediments: name is from stratigraphically equivalent deposits exposed at Caspar Orchard (NE1/4 of NW1/4 of Section 10, T17N, R17W) on the Glenblair SW 7.5' quadrangle; elevation generally 515 to 680 (±30) feet.

TKfs COASTAL BELT FRANCISCAN (Tertiary- Cretaceous): well-consolidated clastic sedimentary rocks including sandstone, shale with minor limestone, and conglomerate; NW trending streams tend to lie in more sheared shale.

LITHOLOGIC CONTACT: dashed where approximately located, dotted where projected or inferred.

LINEAMENT: linear feature of unknown origin observed on aerial photographs.

50 STRIKE AND DIP OF BEDDING: when appearing in Quaternary units the symbol represents the underlying bedrock.

BORROW PIT

SPRING

MARSH

SLOPES > 70 PERCENT: compiled from map contours, aerial photo interpretation, and field reconnaissance.

APPROXIMATE SCALE (FEET)

0 1,000 2,000 4,000

NORTH

REFERENCE:
Geology and Geomorphic Features Related to Landsliding, Fort Bragg 7.5 - Minute Quadrangle, CDMG Open File Report 83 - 5, 1983

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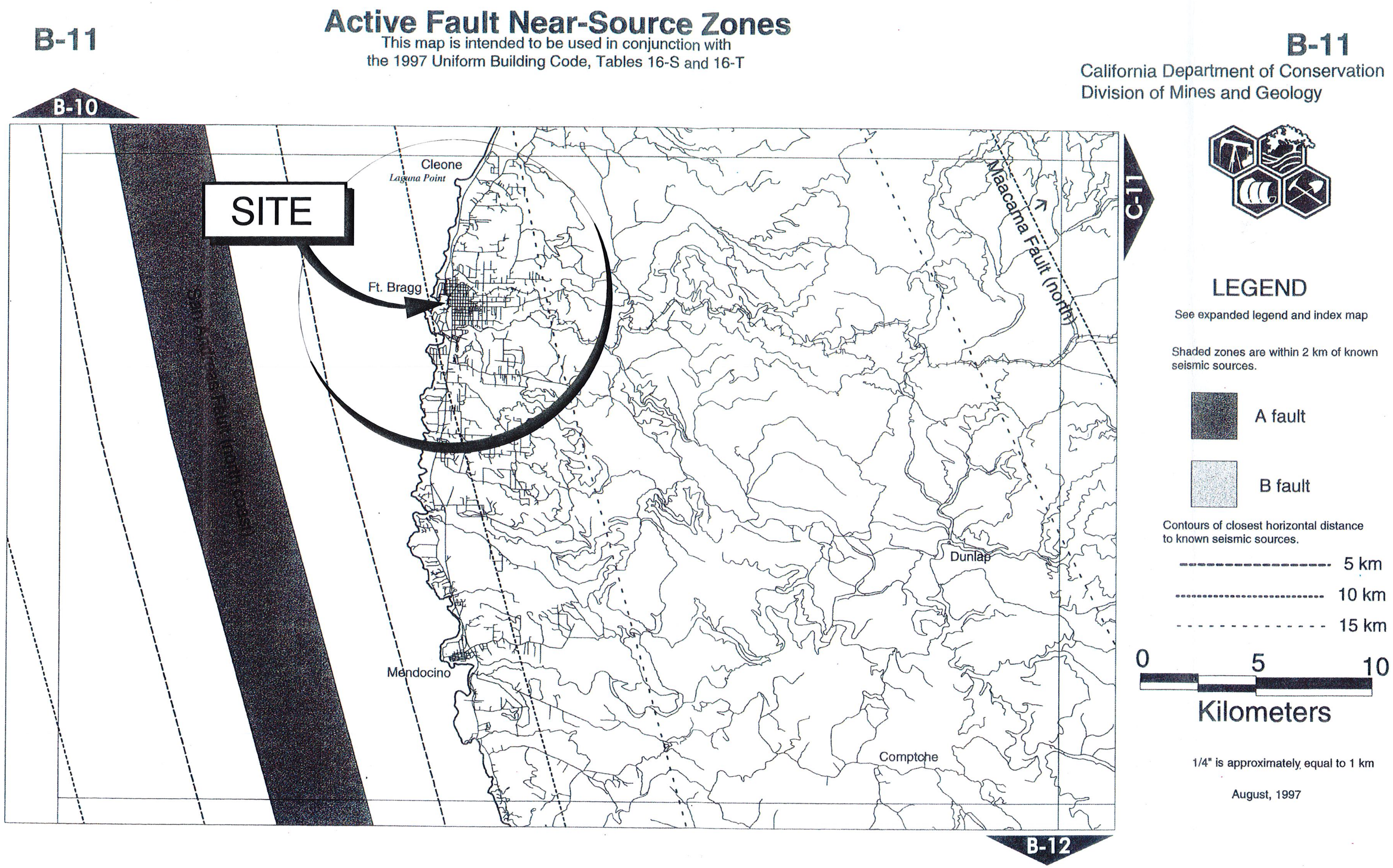
REGIONAL GEOLOGIC MAP
FORT BRAGG FIRE DEPARTMENT
STATION #830
141 Main Street
Fort Bragg, Mendocino County, California

PLATE
17



REFERENCE:

Sheet B-11 of Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, 1998, CDMG and International Conference of Building Officials.

NORTH

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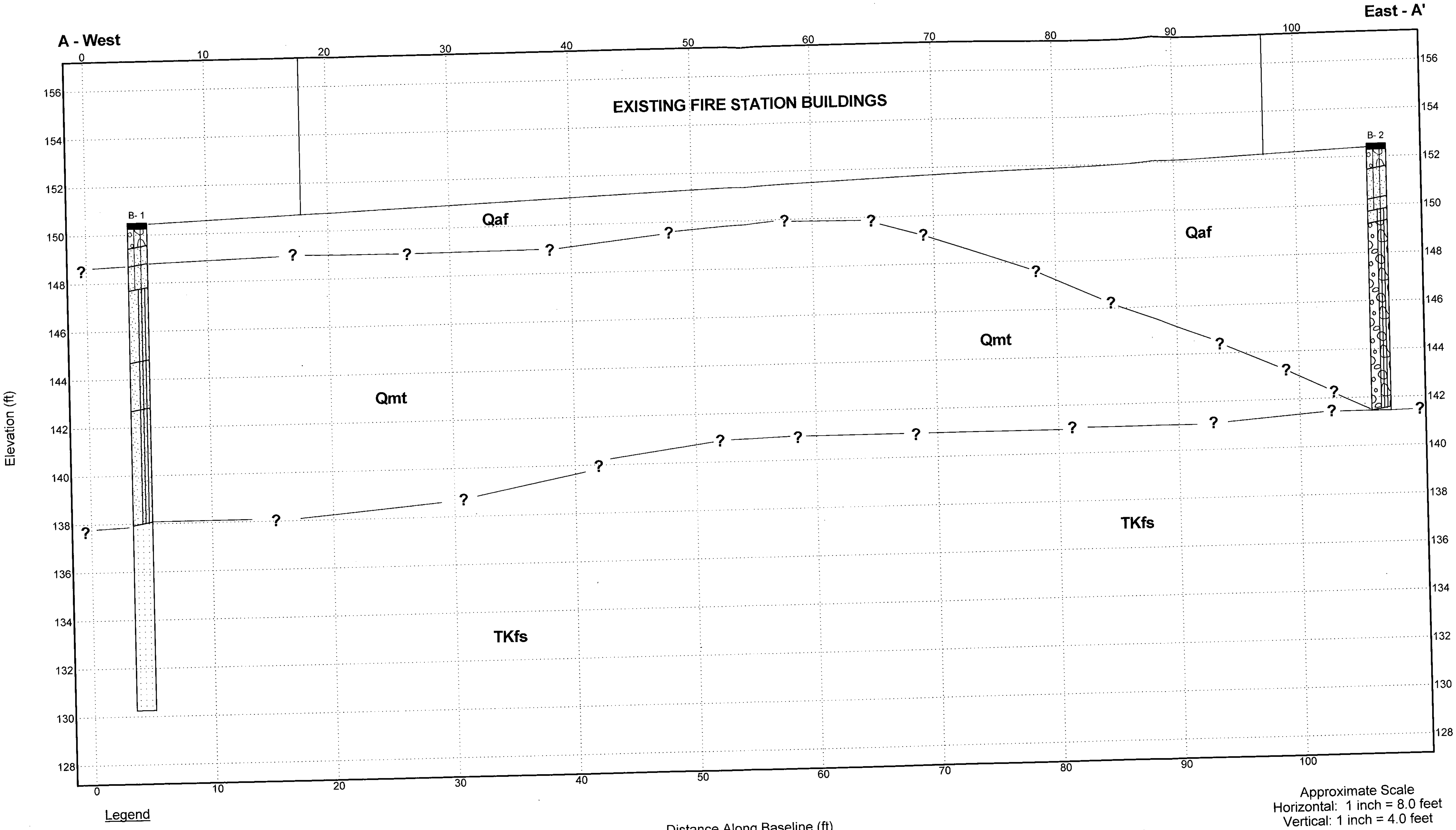

Approximate Scale
Horizontal: 1 inch = 8.0 feet
Vertical: 1 inch = 4.0 feet

Legend

— — ? Geologic contact, dashed and queried where certain

Qaf - Historic Artificial Fill Deposits

Qmt - Quarternary Marine Terrace Deposits

TKfs - Tertiary-Cretaceous Coastal Belt Franciscan Complex

Note:
Boring widths are exaggerated for clarity
See Plate 2 for elevation reference

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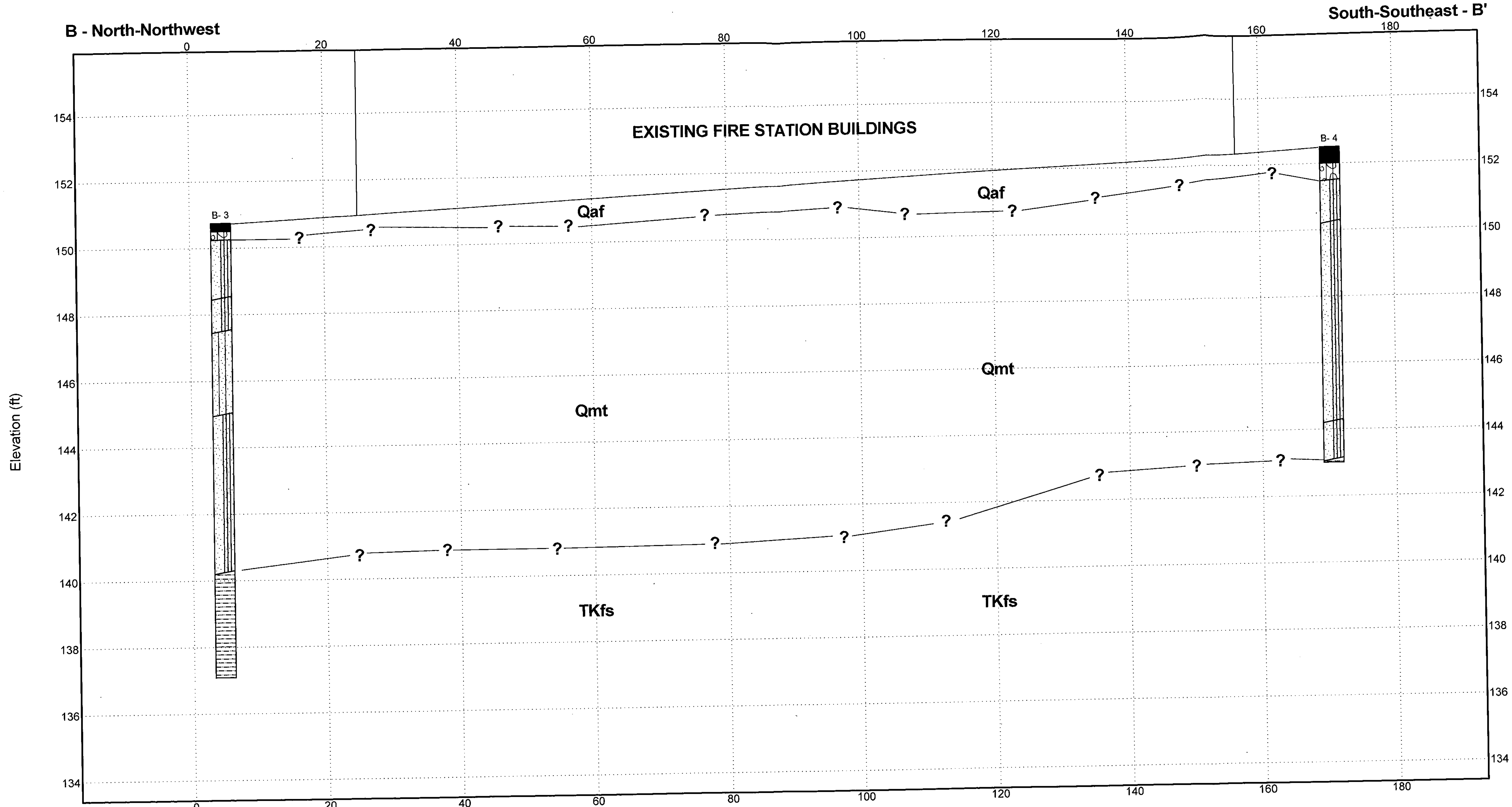
SECTION A - A'
FORT BRAGG FIRE DEPARTMENT
STATION #830
141 Main Street
Fort Bragg, Mendocino County, California

PLATE
19



Approximate Scale
Horizontal: 1 inch = 15.0 feet
Vertical: 1 inch = 3.0 feet

Legend

— —? Geologic Contact, dashed and queried where uncertain

Qaf - Historic Artificail Fill Deposits

Qmt - Quaternery Marine Terrace Deposits

TKfs - Tertiary-Cretaceous Coastal Belt Franciscan Complex

Note:
Boring widths are exaggerated for clarity
See Plate 2 for elevation reference

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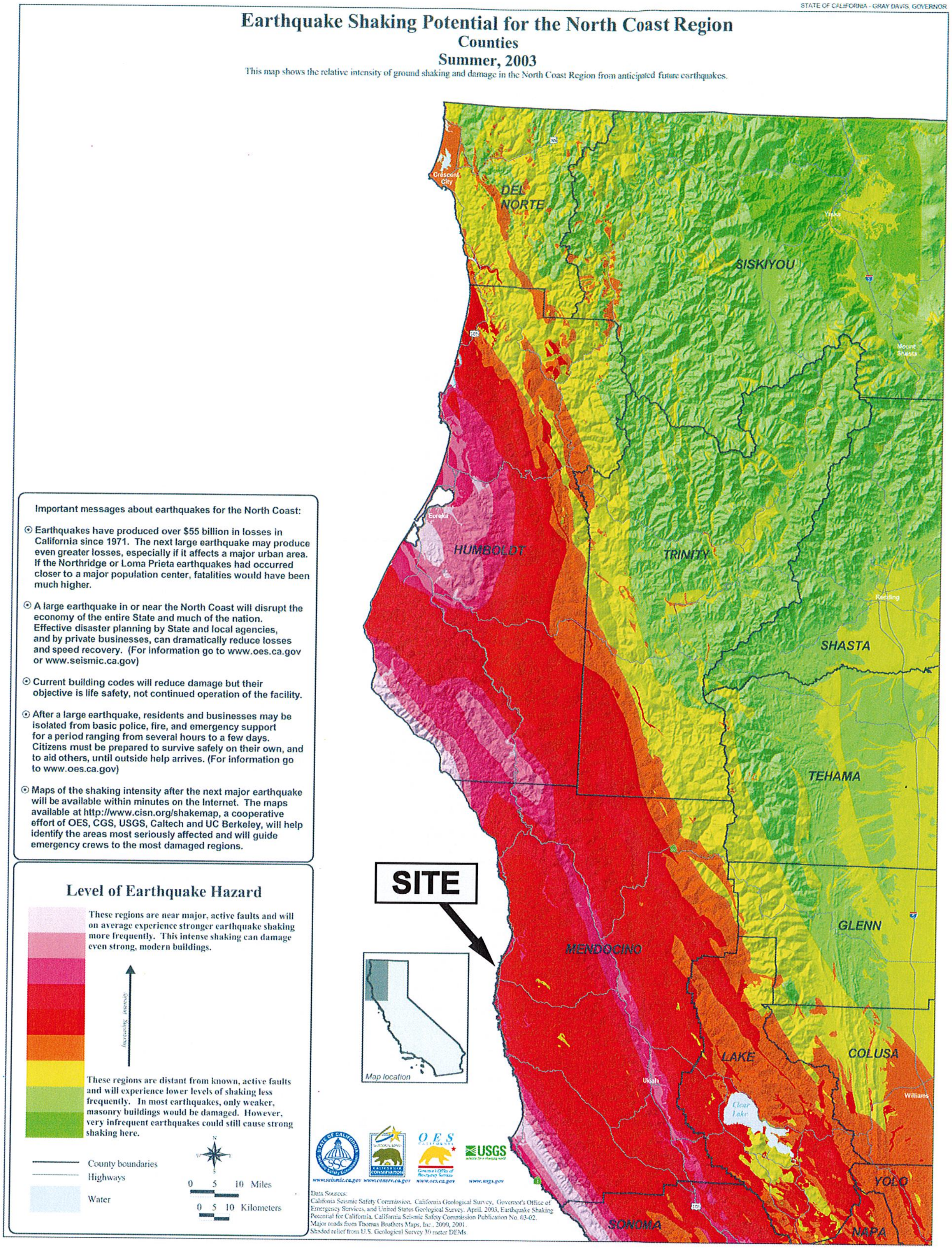
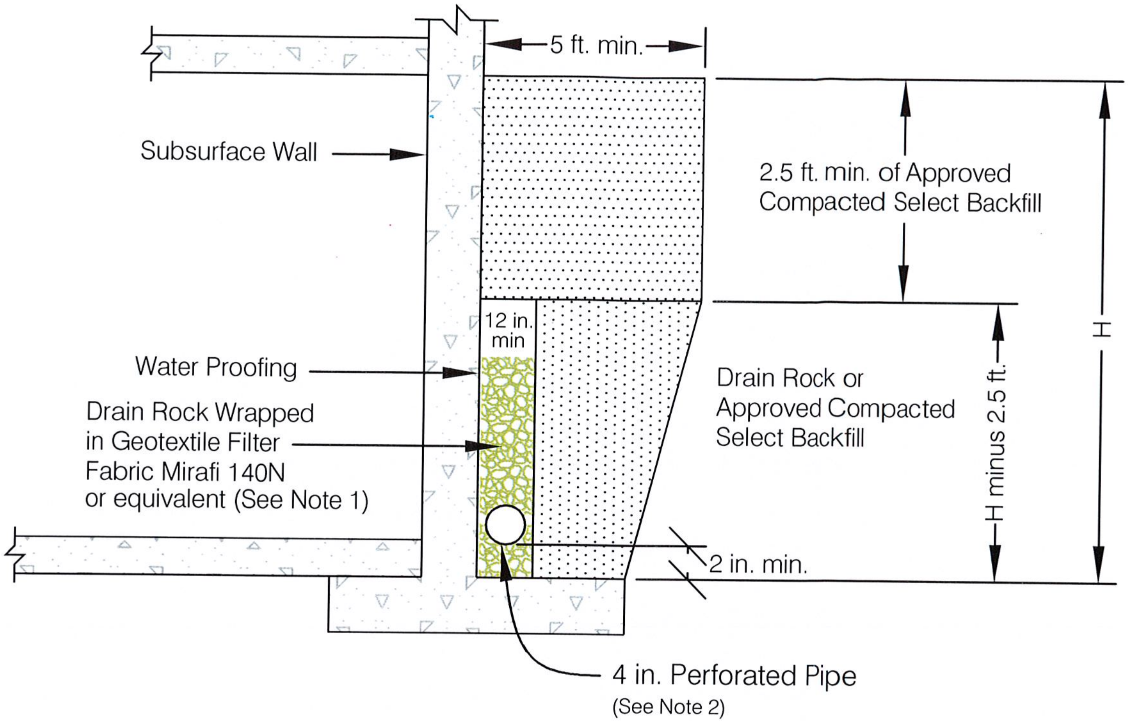


REFERENCE:

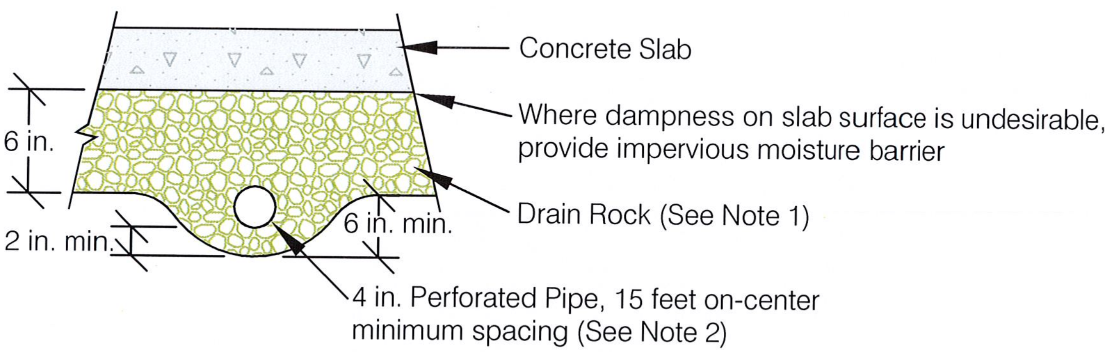
California Geological Survey, Probabilistic Seismic Hazard Maps. www.seismic.ca.gov/pub/intensitymaps/ncoast_county_print.pdf

NORTH

BACE Geotechnical a division of Brunsing Associates, Inc. (707) 528-6108	Job No.: 10511.3 Appr.: EEO Date: 07/16/09	EARTHQUAKE SHAKING POTENTIAL MAP FORT BRAGG FIRE DEPARTMENT STATION #830 141 Main Street Fort Bragg, Mendocino County, California	PLATE 21



SUBSURFACE WALL DRAINAGE DETAIL
(Not to Scale)



PIT UNDER SLAB DRAINAGE DETAIL (see Note 3)
(Not to Scale)

NOTES:

(1) Drain rock should be clean, free-draining material graded in size between the No. 4 and $\frac{3}{4}$ or 1-$\frac{1}{2}$ inch sieves.

(2) Pipe should be SDR 35, or equivalent, perforations should be placed down, sloped at least 1% to drain to gravity outlet or sump with automatic pump. A clean-out pipe with cap should be installed at the upslope end of the the pipe, pipe elbows should be 45 degrees or less (for "snake" access).

(3) Recommended only where pits are bottomed lower than about three feet below existing ground surface.

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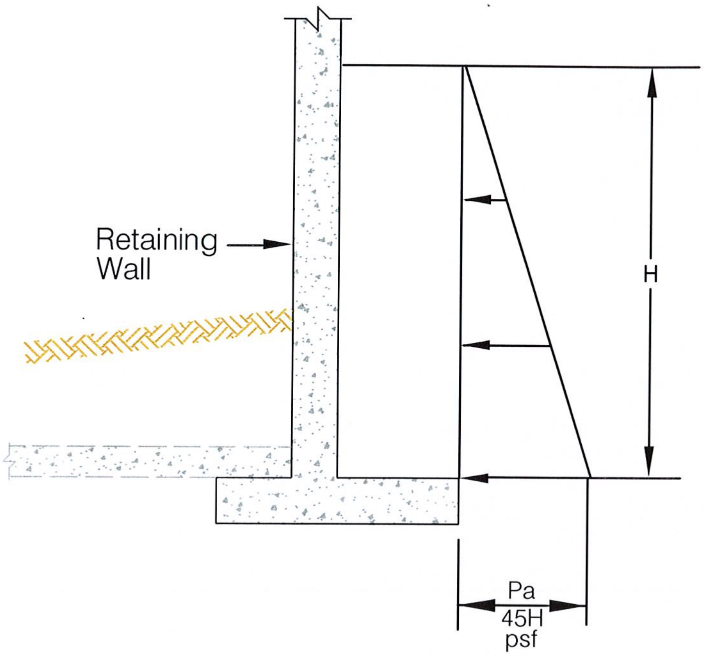
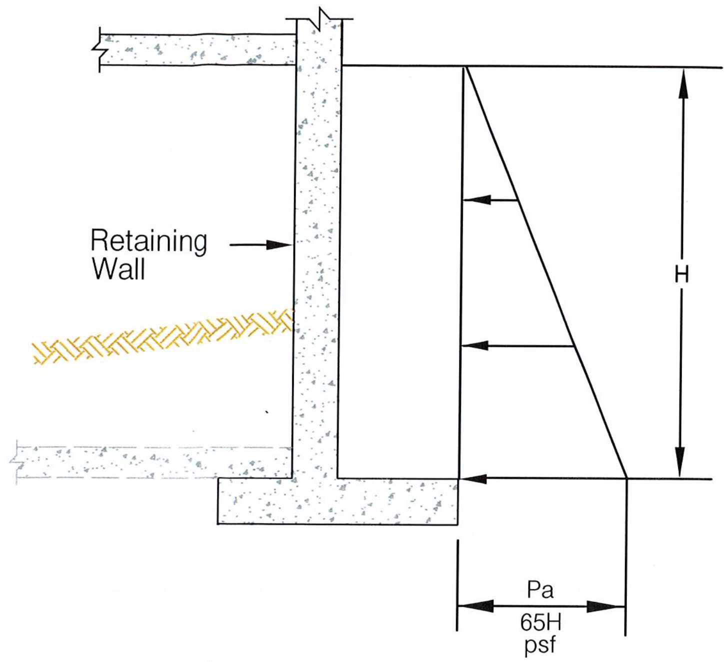


ACTIVE SOIL PRESSURE DIAGRAM

For walls that are free to yield slightly (See Note 2)



AT-REST SOIL PRESSURE DIAGRAM

For braced walls of substantial rigidity (See Note 2)

NOTES:

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

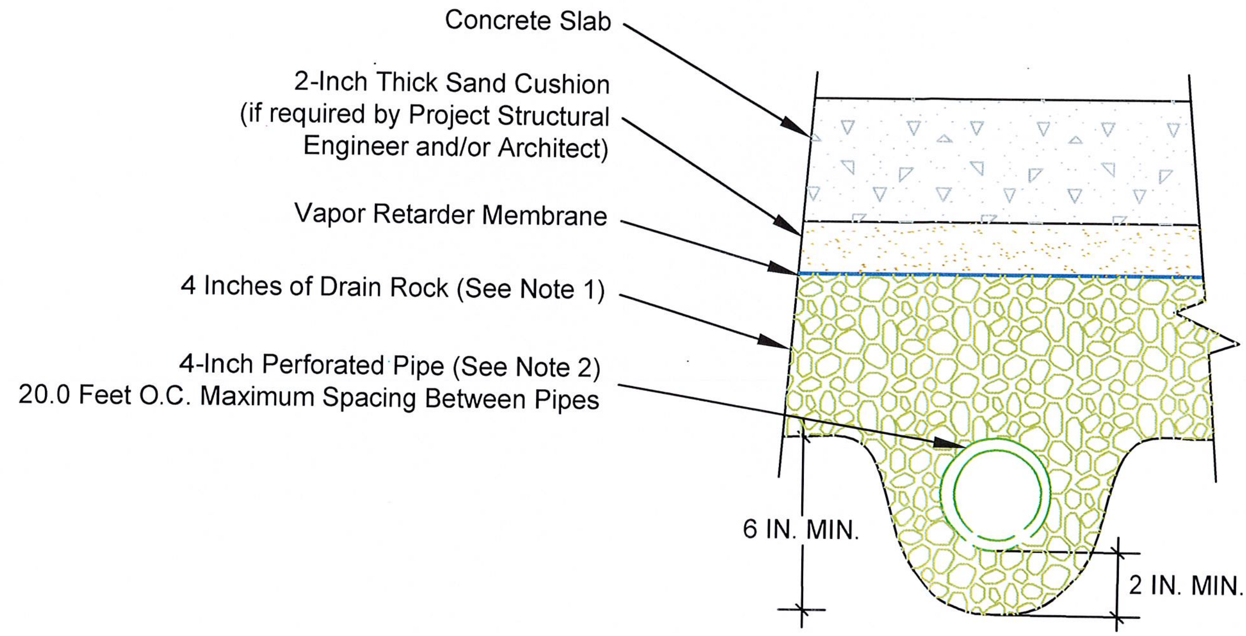
(2) If the wall, at surface of the backfill, cannot yield about 0.1% of its' height, at-rest soil pressures should be used.

(3) The above pressures assume a drained condition. See Plate 22 for drainage and backfill details.

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

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

BACE Geotechnical a division of Brunsing Associates, Inc. (707) 528-6108	Job No.: 10511.3 Appr.: KAC Date: 07/16/09	RETAINING WALL LATERAL EARTH PRESSURES FORT BRAGG FIRE DEPARTMENT STATION #830 141 Main Street Fort Bragg, Mendocino County, California	PLATE 23



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.

2. Pipe should be SDR 35 or equivalent, perforations placed down, sloped at least 1% 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.

BACE Geotechnical a division of Brunsing Associates, Inc. (707) 528-6108	Job No.: 10511.3 Appr.: KAC Date: 07/16/09	UNDERSLAB DRAINAGE DETAILS FORT BRAGG FIRE DEPARTMENT STATION #830 141 Main Street Fort Bragg, Mendocino County, California	PLATE 24

DISTRIBUTION

Twenty Six Copies	I.L. Welty & Associates, Inc. C/o Lee Welty 703 North Main Street Fort Bragg, California 95437

APPENDIX A

Selected References

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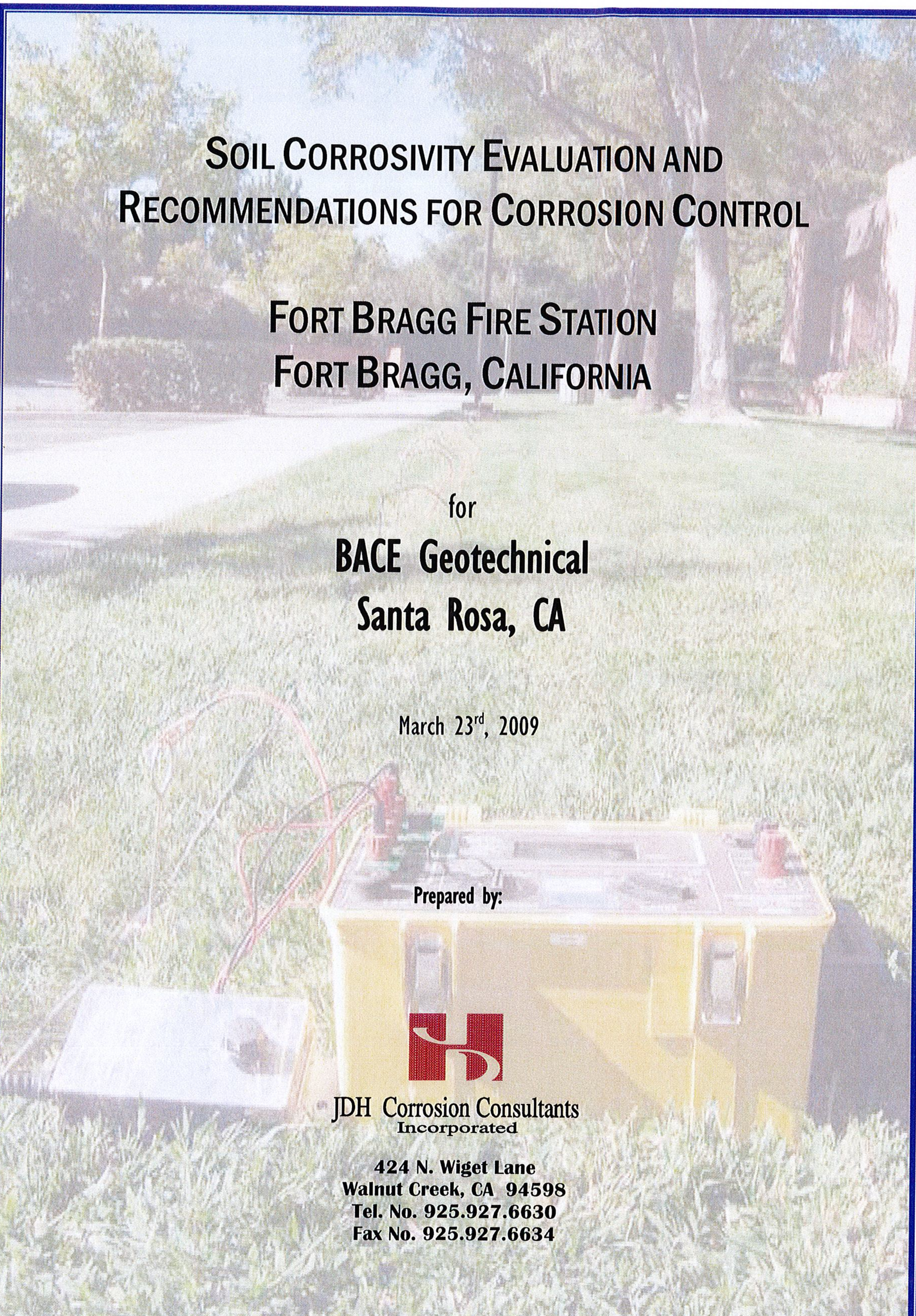
APPENDIX B

JDH Corrosion Consultants, Inc.

Corrosivity Testing Results and Recommendations for Corrosion Control



Soil Corrosivity Evaluation and Recommendations for Corrosion Control

Fort Bragg Fire Station
Fort Bragg, California

for

BACE Geotechnical
Santa Rosa, CA

March 23rd, 2009

Prepared by:

JDH Corrosion Consultants
Incorporated

424 N. Wiget Lane
Walnut Creek, CA 94598
Tel. No. 925.927.6630
Fax No. 925.927.6634

March 23, 2009

BACE Geotechnical
5468 Skylane Blvd., Ste 201
Santa Rosa, CA 95403

Attention: **Mr. Kieth Colorado**
Civil Engineer

Subject: **Soil Corrosivity Evaluation & Recommendations for Corrosion Control**
Fort Bragg Fire Station
Fort Bragg, CA

Dear Mr. Colorado,

Pursuant to your request, ***JDH Corrosion Consultants, Inc***., is providing corrosion analysis for the above referenced project site and we have provided herein recommendations for long-term corrosion control for the concrete foundations and underground utilities at this site based on Cerco Analytical, Inc.'s report dated, March 5th, 2009.

PURPOSE

The purpose for the initial evaluation is to determine the corrosion potential resulting from the soils at the subject site, and to provide recommendations for long-term corrosion control for the concrete foundations and the buried metallic utilities.

SOIL TESTING AND ANALYSIS

One (1) soil sample was collected from the project site by ***BACE Geotechnical*** field personnel and transported to a testing laboratory, **CERCO Analytical, Inc.** (DOHS certificate no. 2153) located in Concord, CA, for chemical analysis. Each sample was analyzed for pH, chlorides, resistivity, sulfates and Redox potential using ASTM test methods as detailed in the table below. The preparation of the soil samples for chemical analysis was in accordance with the applicable specifications.

The results of the chemical analysis data reported are as follows:

CERCO Analytical, Inc.
Soil Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	N.D. mg/kg	Non-corrosive
pH	7.1	Non-corrosive
Resistivity (@100% saturation)	27,000 ohm-cm	Mildly Corrosive
Redox	420 mV	Non-corrosive
Sulfate	N.D. mg/kg	Non-corrosive**

* With Respect to bare steel or ductile iron
** With respect to mortar coated steel

Chemical Testing Analysis

The chemical analysis provided by ***BACE Geotechnical*** indicates that the soils are "mildly corrosive" with respect to steel and ductile iron based upon the resistivity measurements. The chloride levels indicate "non-corrosive" conditions to steel and ductile iron, and the sulfate levels indicate "non-corrosive" conditions for concrete structures placed into these soils with regard to sulfate attack. The pH of the soils is slightly acidic which classifies them as "non-corrosive" to buried steel and concrete structures and the Redox potential indicates aerobic soils which are classified as "non-corrosive" to buried steel structures.

DISCUSSION

Reinforced Concrete Slab Foundations

Due to the low level of water-soluble sulfates and chlorides in these soils, special requirements for sulfate resistant cement are not required. However, the minimum depth of cover for the reinforcing steel should be as specified in the California Building Code (CBC) and the America Concrete Institute (ACI 318).

Underground Metallic Pipelines

The soils at the project site are considered to be "mildly corrosive" to ductile/cast iron, steel and dielectric coated steel. Therefore, we recommend the use of coatings for steel pipes and fittings, and polyethylene encasement for ductile iron pipe and fittings. All underground metallic pipelines should also be electrically isolated from above grade structures and reinforced concrete structures in order to minimize potential galvanic corrosion problems.

RECOMMENDATIONS

Reinforced Concrete Slab Foundations

We recommend using a Type II concrete mix with a water-to-cement ratio as specified in the California Building Code (CBC) for soils containing less that 0.10% water soluble sulfate by weight. Adhering to the minimum depth of cover for the reinforcing steel in the foundations as specified in the CBC is recommended for the subject structures as well.

Ductile Iron Pipe (Pressure Piping such as Domestic Water and Fire)

1. Direct buried ductile iron pipe should be encased in 8-mil polyethylene as specified in AWWA specification C-105. Epoxy coatings are also an acceptable alternative type of coating system for the pipe and/or fittings such as valves.

2. All rubber gasket joints, fusion epoxy coated flanges and flexible couplings on ductile iron pipelines should be bonded with insulated copper cable to insure electrical continuity of the pipeline and fittings.

3. Insulating flanges and/or couplings should be installed to electrically isolate the buried portion of pipeline from other metallic pipelines, reinforced concrete structures and above grade buildings or structures.

4. As an alternate, non-metallic piping may be used in lieu of ductile iron piping as allowed by State and local codes. Non-metallic piping does not require the implementation of any special type of corrosion prevention

measures. However, all metallic valves, fittings and appurtenances on non-metallic piping will require protection as specified below.

Ductile Iron Fittings & Metallic Valves (On Plastic Piping)

1. All direct buried ductile iron fittings installed on non-metallic piping shall be provided with a bituminous coating from the factory and encased in an 8-mil polyethylene bag in the field in accordance with AWWA Specification C-105. All bolts, restraining rods, etc. shall be coated with bitumastic prior to encasement in the polyethylene bag.

2. All metallic valves shall be coated from the factory (i.e. using powered epoxy or equivalent type of coating system) and all bolts shall be coated with bitumastic in the field and the entire valve shall be encased in an 8-mil polyethylene bag in accordance with AWWA Specification C-105.

Steel Pipelines (Natural Gas Pipelines & Risers)

1. A fusion-bonded epoxy coating system or a suitable tape coating should be applied to all buried steel pipelines in accordance with ANSI/AWWA C214-95, "AWWA Standard for Tape Coating Systems for the Exterior of Steel Water Pipelines." Also, a tape coating per AWWA Standard C209-95 is recommended for special sections, connections and fittings.

2. Insulating flanges and/or couplings should be installed to electrically isolate the buried portions of steel pipelines from other metallic pipelines, reinforced concrete structures and above grade structures.

3. All non-insulating rubber gasket joints, fusion epoxy coated flanges and flexible couplings should be bonded with insulated copper cable to insure electrical continuity of the pipeline and fittings.

4. As an alternate, non-metallic piping may be used in lieu of steel piping as allowed by State and local codes. Non-metallic piping does not require the implementation of any special type of corrosion prevention measures.

Sewer and Storm Drain Lines

No special corrosion considerations are required for the gravity sewer and storm drain lines.

Copper Water Pipelines (Service Lines)

1. Direct buried copper water services should be encased in 6-mil minimum polyethylene as specified in AWWA specification C-105.

2. All copper water laterals shall be electrically isolated from metallic water mains via the use of insulating type corporation stops installed at the water main.

Fort Bragg Fire Station #10511.3

Conterminous 48 States
2006 International Building Code
Latitude = 39.4424
Longitude = -123.80621
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.01 deg grid spacing

Period (sec)	Sa (g)
0.2	1.500 (Ss, Site Class B)
1.0	0.675 (S1, Site Class B)

Conterminous 48 States
2006 International Building Code
Latitude = 39.4424
Longitude = -123.80621
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)
0.2	1.500 (SMs, Site Class D)
1.0	1.012 (SM1, Site Class D)

Conterminous 48 States
2006 International Building Code
Latitude = 39.4424
Longitude = -123.80621
Design Spectral Response Accelerations SDs and SD1
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)
0.2	1.000 (SDs, Site Class D)
1.0	0.675 (SD1, Site Class D)

Conterminous 48 States
2006 International Building Code
Latitude = 39.4424
Longitude = -123.80621
MCE Response Spectrum for Site Class B
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0

Period (sec)	Sa (g)	Sd (inches)
0.000	0.600	0.000
0.090	1.500	0.119
0.200	1.500	0.586
0.450	1.500	2.963
0.500	1.349	3.295
0.600	1.124	3.954
0.700	0.964	4.613
0.800	0.843	5.272
0.900	0.749	5.931
1.000	0.675	6.590
1.100	0.613	7.249
1.200	0.562	7.908
1.300	0.519	8.567
1.400	0.482	9.226
1.500	0.450	9.885
1.600	0.422	10.544
1.700	0.397	11.203
1.800	0.375	11.862
1.900	0.355	12.521
2.000	0.337	13.180

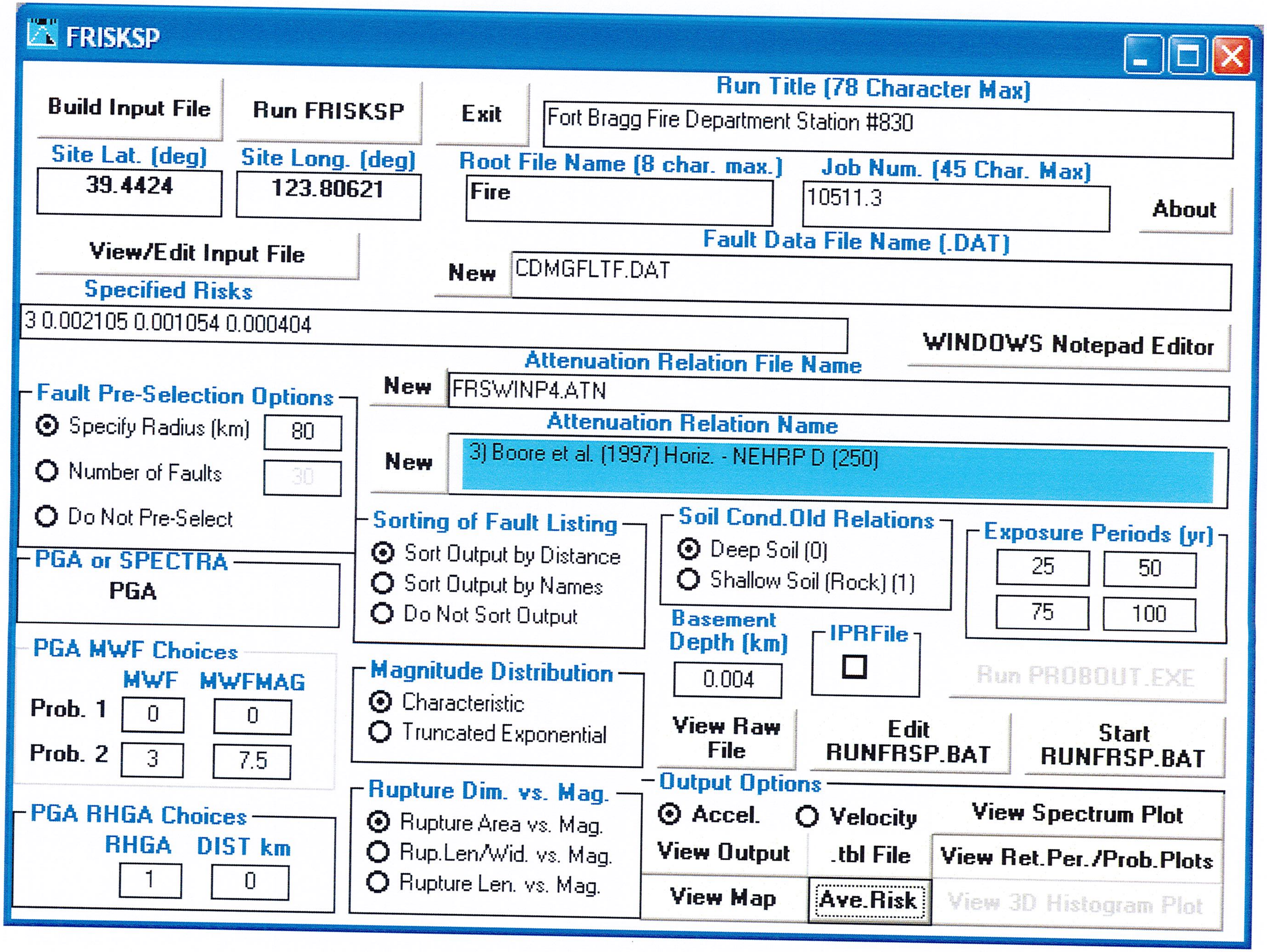
Conterminous 48 States
2006 International Building Code
Latitude = 39.4424
Longitude = -123.80621
Site Modified Response Spectrum for Site Class D
SMs = FaSs and SM1 = FvS1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)	Sd (inches)
0.000	0.600	0.000
0.135	1.500	0.267
0.200	1.500	0.586
0.675	1.500	6.668

0.700	1.445	6.919
0.800	1.265	7.908
0.900	1.124	8.896
1.000	1.012	9.885
1.100	0.920	10.873
1.200	0.843	11.862
1.300	0.778	12.850
1.400	0.723	13.839
1.500	0.675	14.827
1.600	0.632	15.816
1.700	0.595	16.804
1.800	0.562	17.793
1.900	0.533	18.781
2.000	0.506	19.770

Conterminous 48 States
2006 International Building Code
Latitude = 39.4424
Longitude = -123.80621
Design Response Spectrum for Site Class D
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)	Sd (inches)
0.000	0.400	0.000
0.135	1.000	0.178
0.200	1.000	0.391
0.675	1.000	4.445
0.700	0.964	4.613
0.800	0.843	5.272
0.900	0.749	5.931
1.000	0.675	6.590
1.100	0.613	7.249
1.200	0.562	7.908
1.300	0.519	8.567
1.400	0.482	9.226
1.500	0.450	9.885
1.600	0.422	10.544
1.700	0.397	11.203
1.800	0.375	11.862
1.900	0.355	12.521
2.000	0.337	13.180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Build Input File
Run FRISKSP
Exit
Run Title (78 Character Max)
Fort Bragg Fire Department Station #830
Site Lat. (deg)
39.4424
Site Long. (deg)
123.80621
Root File Name (8 char. max.)
Fire
Job Num. (45 Char. Max)
10511.3
About
View/Edit Input File
Fault Data File Name (.DAT)
New
CDMGFLTF.DAT
Specified Risks
3 0.002105 0.001054 0.000404
WINDOWS Notepad Editor
Attenuation Relation File Name
New
FRSWINP4.ATN
Fault Pre-Selection Options
Specify Radius (km) 80
Number of Faults 30
Do Not Pre-Select
Attenuation Relation Name
New
3) Boore et al. (1997) Horiz. - NEHRP D (250)
Sorting of Fault Listing
Sort Output by Distance
Sort Output by Names
Do Not Sort Output
Soil Cond.Old Relations
Deep Soil (0)
Shallow Soil (Rock) (1)
Exposure Periods (yr)
25
50
75
100
PGA or SPECTRA
PGA
Basement Depth (km)
0.004
IPRFile
PGA MWF Choices
MWF
MWFMAG
Prob. 1 0 0
Prob. 2 3 7.5
Magnitude Distribution
Characteristic
Truncated Exponential
Run PROBOUT.EXE
View Raw File
Edit RUNFRSP.BAT
Start RUNFRSP.BAT
Output Options
Accel.
Velocity
View Spectrum Plot
PGA RHGA Choices
RHGA
DIST km
1
0
Rupture Dim. vs. Mag.
Rupture Area vs. Mag.
Rup.Len/Wid. vs. Mag.
Rupture Len. vs. Mag.
View Output
.tbl File
View Ret.Per./Prob.Plots
View Map
Ave.Risk
View 3D Histogram Plot

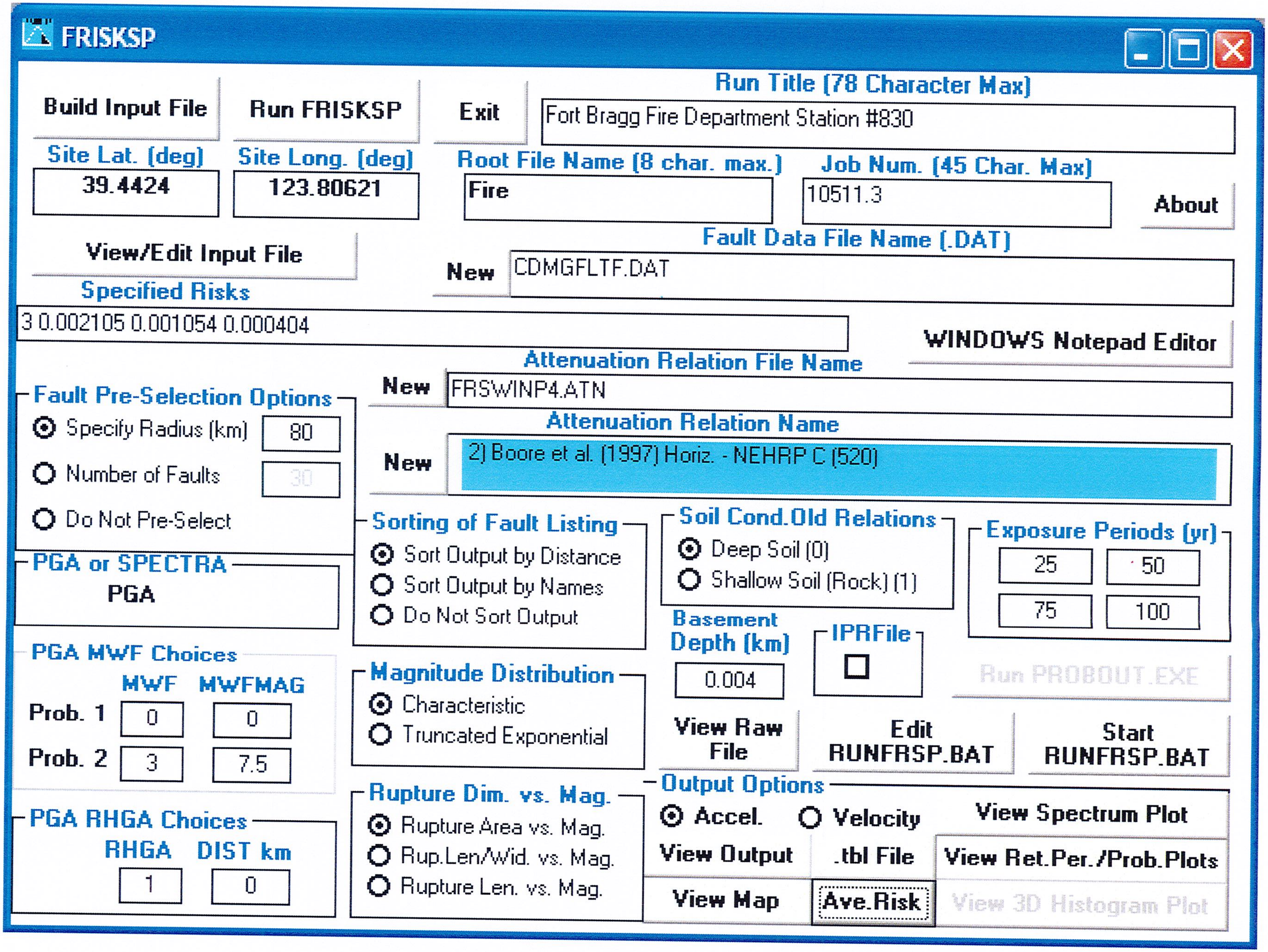
AVERAGE PGA SUMMARY

	Problem Name	.002105 475	.001054 949	.000404 2475	Risk 4	Risk 5
1	BOORE ET AL(1997) NEHRP D (250)1	0.67796	0.84623	1.09216	0.00000	0.00000
2	BOORE ET AL(1997) NEHRP D (250)2	0.66830	0.84444	1.10059	0.00000	0.00000
3		0.00000	0.00000	0.00000	0.00000	0.00000
4	AVERAGES:	0.67313	0.84533	1.09638	0.00000	0.00000
5		0.00000	0.00000	0.00000	0.00000	0.00000
6		0.00000	0.00000	0.00000	0.00000	0.00000
7		0.00000	0.00000	0.00000	0.00000	0.00000
8		0.00000	0.00000	0.00000	0.00000	0.00000
9		0.00000	0.00000	0.00000	0.00000	0.00000
10		0.00000	0.00000	0.00000	0.00000	0.00000
11		0.00000	0.00000	0.00000	0.00000	0.00000
12		0.00000	0.00000	0.00000	0.00000	0.00000
13		0.00000	0.00000	0.00000	0.00000	0.00000
14		0.00000	0.00000	0.00000	0.00000	0.00000
15		0.00000	0.00000	0.00000	0.00000	0.00000
16		0.00000	0.00000	0.00000	0.00000	0.00000
17		0.00000	0.00000	0.00000	0.00000	0.00000
18		0.00000	0.00000	0.00000	0.00000	0.00000

Back to Program

Print Data Table

REMEMBER: The above table contains the results for each of the problems analyzed. If each problem is an alternate method of computing each risk (as in the case of a multiple-pga run), then the averages shown are meaningul. However, if each problem is exclusive (as in the case of a spectra run), then the averages are not meaningful (and should be ignored).


FRISKSP
Build Input File
Run FRISKSP
Exit
Run Title (78 Character Max)
Fort Bragg Fire Department Station #830
Site Lat. (deg)
39.4424
Site Long. (deg)
123.80621
Root File Name (8 char. max.)
Fire
Job Num. (45 Char. Max)
10511.3
About
View/Edit Input File
Fault Data File Name (.DAT)
New
CDMGFLTF.DAT
Specified Risks
3 0.002105 0.001054 0.000404
WINDOWS Notepad Editor
Attenuation Relation File Name
New
FRSWINP4.ATN
Attenuation Relation Name
New
2) Boore et al. (1997) Horiz. - NEHRP C (520)
Fault Pre-Selection Options
Specify Radius (km)
80
Number of Faults
30
Do Not Pre-Select
Sorting of Fault Listing
Sort Output by Distance
Sort Output by Names
Do Not Sort Output
Soil Cond.Old Relations
Deep Soil (0)
Shallow Soil (Rock) (1)
Exposure Periods (yr)
25
50
75
100
PGA or SPECTRA
PGA
Basement Depth (km)
0.004
IPRFile
Run PROBOUT.EXE
PGA MWF Choices
MWF
MWFMAG
Prob. 1
0
0
Prob. 2
3
7.5
Magnitude Distribution
Characteristic
Truncated Exponential
View Raw File
Edit RUNFRSP.BAT
Start RUNFRSP.BAT
Output Options
Accel.
Velocity
View Spectrum Plot
PGA RHGA Choices
RHGA
DIST km
1
0
Rupture Dim. vs. Mag.
Rupture Area vs. Mag.
Rup.Len/Wid. vs. Mag.
Rupture Len. vs. Mag.
View Output
.tbl File
View Ret.Per./Prob.Plots
View Map
Ave.Risk
View 3D Histogram Plot

AVERAGE PGA SUMMARY

	Problem Name	.002105 475	.001054 949	.000404 2475	Risk 4	Risk 5
1	BOORE ET AL(1997) NEHRP C (520)1	0.51640	0.64426	0.83158	0.00000	0.00000
2	BOORE ET AL(1997) NEHRP C (520)2	0.50971	0.64288	0.83775	0.00000	0.00000
3		0.00000	0.00000	0.00000	0.00000	0.00000
4	AVERAGES:	0.51306	0.64357	0.83466	0.00000	0.00000
5		0.00000	0.00000	0.00000	0.00000	0.00000
6		0.00000	0.00000	0.00000	0.00000	0.00000
7		0.00000	0.00000	0.00000	0.00000	0.00000
8		0.00000	0.00000	0.00000	0.00000	0.00000
9		0.00000	0.00000	0.00000	0.00000	0.00000
10		0.00000	0.00000	0.00000	0.00000	0.00000
11		0.00000	0.00000	0.00000	0.00000	0.00000
12		0.00000	0.00000	0.00000	0.00000	0.00000
13		0.00000	0.00000	0.00000	0.00000	0.00000
14		0.00000	0.00000	0.00000	0.00000	0.00000
15		0.00000	0.00000	0.00000	0.00000	0.00000
16		0.00000	0.00000	0.00000	0.00000	0.00000
17		0.00000	0.00000	0.00000	0.00000	0.00000
18		0.00000	0.00000	0.00000	0.00000	0.00000

Back to Program

Print Data Table

REMEMBER: The above table contains the results for each of the problems analyzed. If each problem is an alternate method of computing each risk (as in the case of a multiple-pga run), then the averages shown are meaningul. However, if each problem is exclusive (as in the case of a spectra run), then the averages are not meaningful (and should be ignored).

APPENDIX D

LIQUEFY2 Program Data and
Liquefaction Induced Vertical Settlement Calculations

LIQUEFY2 - LIQUEFACTION (NCEER, 1997) Boring B-3

Begin Calculations | Exit

Job/Analysis Name (20 Characters Max): Fort Bragg Fire

Job Number (15 Characters Max): 10511.3

rd-Factor
- Seed (1985)
- NCEER (1997)
- (•) Idriss (1998) *

Rod Length Corr.?
- (•) Yes
- No
- Rod Stick Up (ft): 3.0

Magnitude Scaling Factor (MSF)
- Idriss (1997)
- Idriss (1998) *
- Andrus/Stokoe
- Y / N 20%
- Y / N 32%
- Y / N 50%
- (•) R.Seed (1998) *

Sampler Corr.	1.000	Help
Borehole Dia. Corr.	1.050	Help
Hammer Energy Corr.	1.080	Help

Calc. Water Depth (ft): 2.0

EQ. Mag. (Mw): 7.3

Peak Acc. (g): 0.670

--------- Surcharge Fill Loading ---------

Fill Unit Wt. (pcf): 0.0 | Fill Ht. (ft): 0.0

Output File: New | Firesta.OUT | WIN Notepad

Soil Profile Log Name (*.LDW): New | Firesta.LDW

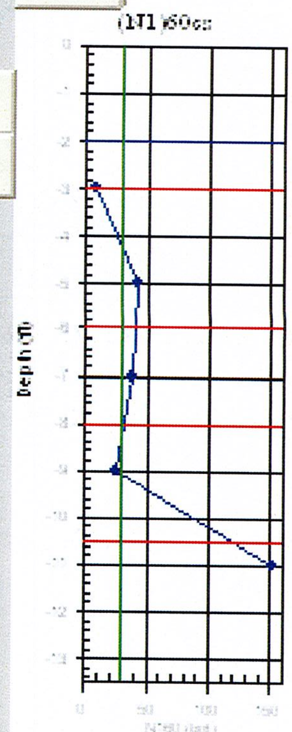
Liquefaction Results File (*.LAR): New | Firesta.LAR

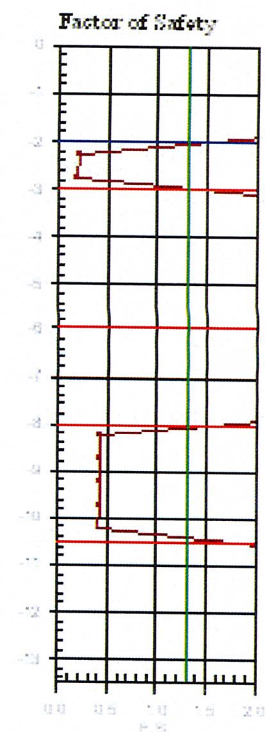
Update Page From File

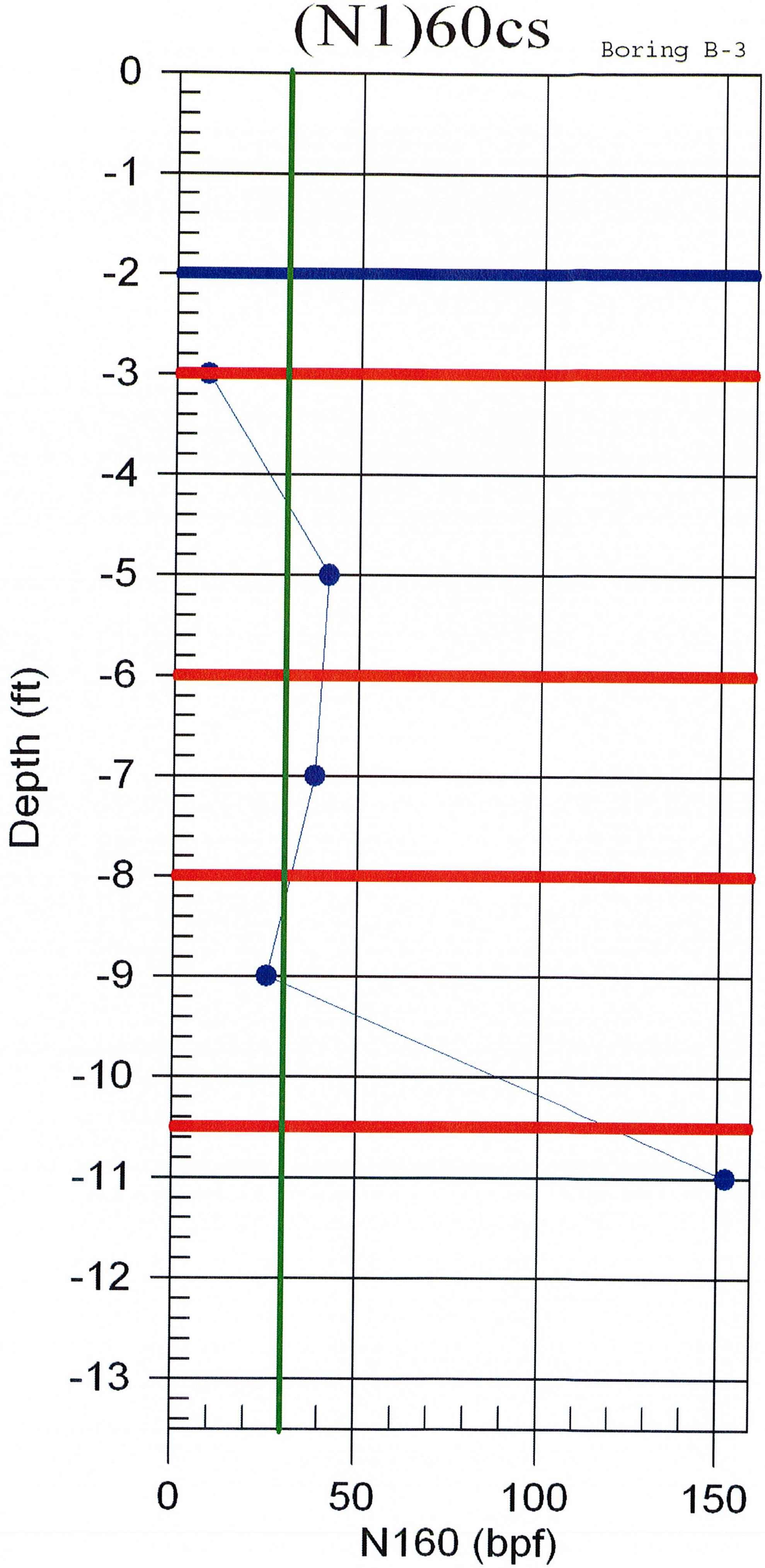
Write Table to LDW File

Boring Water (ft): 2.0

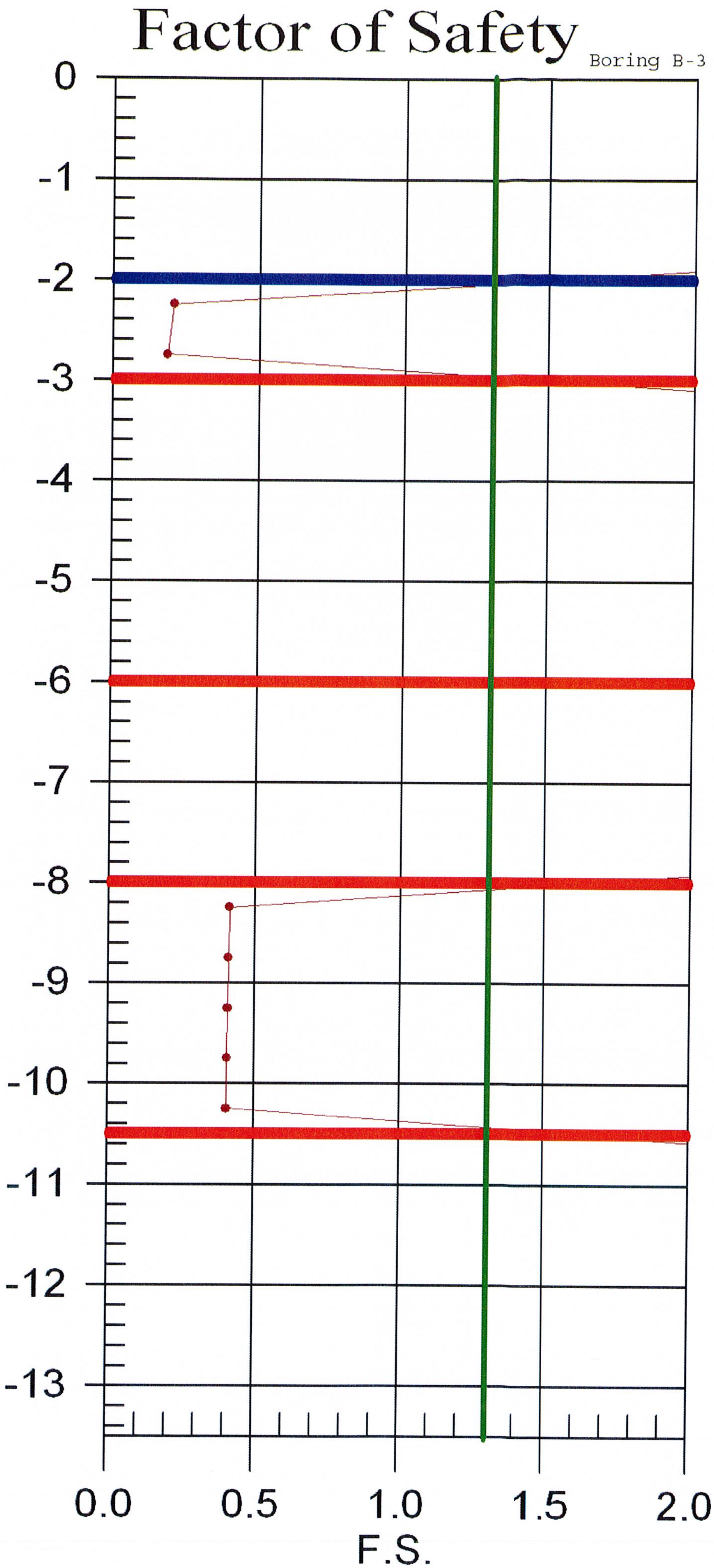
	Base	Field N	0/1	Unit Wt.	Fines	D50	SPT Depth
▶	3	4	1	103	12	0.3	3
	6	21	1	123	19	0.26	5
	8	22	1	122	8	0.26	7
	10.5	16	1	129	7	0.26	9
	13.5	100	1	132	10	0.7	11
*							







(N1)60cs
Boring B-3
Depth (ft)
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-11
-12
-13
0
50
100
150
N160 (bpf)


Factor of Safety
Boring B-3
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-11
-12
-13
0.0
0.5
1.0
1.5
2.0
F.S.

Job Fort Bragg Fire Department
Job # 10511.3
Date 3/19/2009

Induced Vertical Settlement due to Liquefaction

DBE	0.67	g		
acc. Gravity	32.2	ft/sec^2		
M_w	7.3			
$V_{s,40'}$	250	m/sec	820.21	ft/sec
DWF_M	1.1			

Layer	Depth Range (ft)		Thickness of layer (ft)	N	$N_{corrected}$	$(N_1)_{60}$	Soil Material	$(N_1)_{60,CS}$	CSR	CSRN	$\varepsilon_{v\,(\%)}$	ΔH (ft)	ΔH (inch)	ΔH (inch)
1	0	3	3	5	4	10.0	SP-SM	11.1	0.44	0.40	3.5	0.11	1.3	0.4
2	3	6	3	21	21	42.4	SP-SM	46.5	0.60	0.55	0.3	0.01	0.1	
3	6	8	2	22	22	39.8	SM	41.4	0.66	0.60	0.4	0.01	0.1	
4	8	10.5	2.5	16	16	25.7	SP-SM	26.7	0.70	0.63	1.7	0.04	0.5	0.5
5	10.5	13.5	3	100	100	142.5	Siltstone	148.7	0.72	0.66	0	0.00	0.0	

Based on boring B-3
As shown on Plate 7

Calculated settlement due to Liquefaction 0.9 Inches

Portion of layer is liquifiable
Liquifiable layer

Job	Fort Bragg Fire Department
Job #	10511.3
Date	3/19/2009

Boring B-3

Depth (ft)	N_m	σ'_{vo}	C_N	E_m/E_{ff}	$(N_1)_{60}$
3	4	309	2.5	1.000	10.0
6	21	490.8	2.0	1.000	42.4
8	22	610	1.8	1.000	39.8
10.5	16	776.5	1.6	1.000	25.7
13.5	100	985.3	1.4	1.000	142.5

C_{fines}	Fines (%)	$(N_1)_{60,CS}$
1.11	12	11.1
1.10	18.5	46.5
1.04	7.6	41.4
1.04	7	26.7
1.04	10	148.7

E_m	0.6
E_{ff}	0.6
E_m/E_{ff}	1

Boring	B-3	
γ_{water}	62.4	pcf
water depth	2	ft

Job Fort Bragg Fire Department
Job # 10511.3
Date 3/19/2009

Assume: P= Existing Over Burden Pressure (OB)

Depth (ft)	Δ Depth (ft)	Wet Density (pcf)	Wet Density-Water	Δ OB (psf)	OB (psf)
0	3	103	103	309	
3	3	123	60.6	181.8	309
6	2	122	59.6	119.2	490.8
8	2.5	129	66.6	166.5	610
10.5	3	132	69.6	208.8	776.5
13.5					985.3

Total Stress (psf)	pore water stress	Effective stress (psf)	rd	Effective stress (tsf)
309	0	309	0.9992	0.15
678	187.2	490.8	0.9982	0.25
922	312	610	0.9973	0.31
1244.5	468	776.5	0.9959	0.39
1640.5	655.2	985.3	0.9937	0.49

LIQUEFY2 - LIQUEFACTION (NCEER, 1997)

Boring B-5

Begin Calculations

Exit

Job/Analysis Name (20 Characters Max): Fort Bragg Fire

Job Number (15 Characters Max): 10511.3

rd-Factor
- Seed (1985)
- NCEER (1997)
- (●) Idriss (1998) *

Rod Length Corr.?
- (●) Yes
- No

Rod Stick Up (ft): 3.0

Magnitude Scaling Factor (MSF)
- Idriss (1997)
- Idriss (1998) *
- Andrus/Stokoe
- (●) R.Seed (1998) *
- Y / N 20%
- Y / N 32%
- Y / N 50%

Sampler Corr. 1.000 Help

Borehole Dia. Corr. 1.050 Help

Hammer Energy Corr. 1.080 Help

WIN Notepad

Calc. Water Depth (ft): 2.0

EQ. Mag. (Mw): 7.3

Peak Acc. (g): 0.670

--------- Surcharge Fill Loading ---------

Fill Unit Wt. (pcf): 0.0

Fill Ht. (ft): 0.0

Output File: New Firesta.OUT

Soil Profile Log Name (*.LDW): New Firesta.LDW

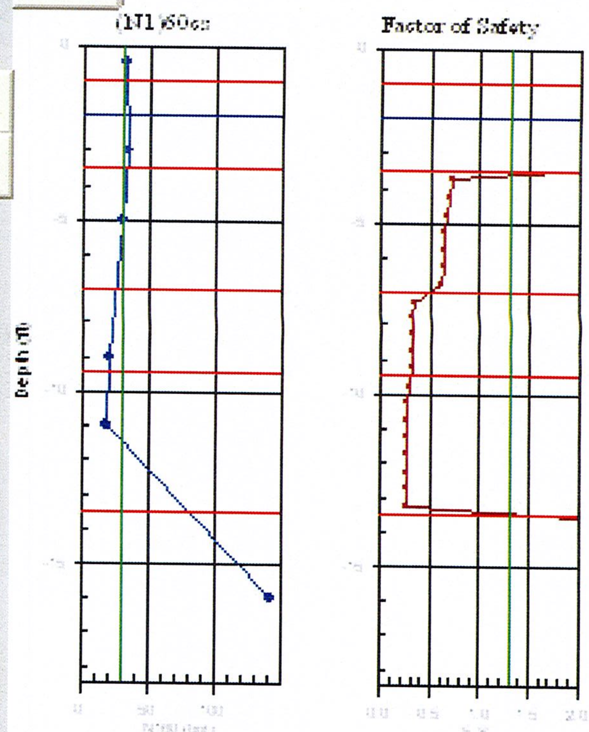
Update Page From File

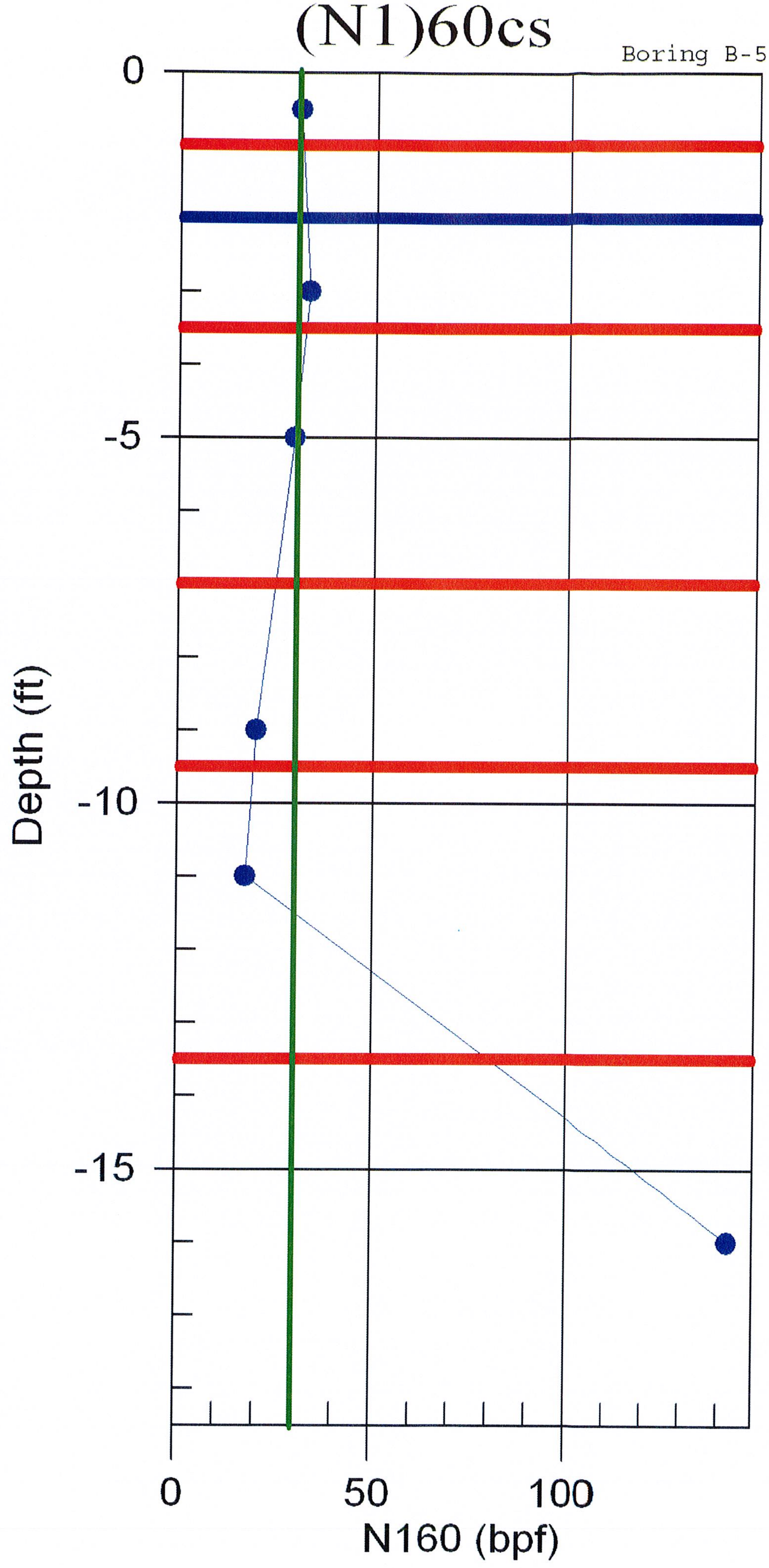
Write Table to LDW File

Liquefaction Results File (*.LAR): New Firesta.LAR

Boring Water (ft): 2.0

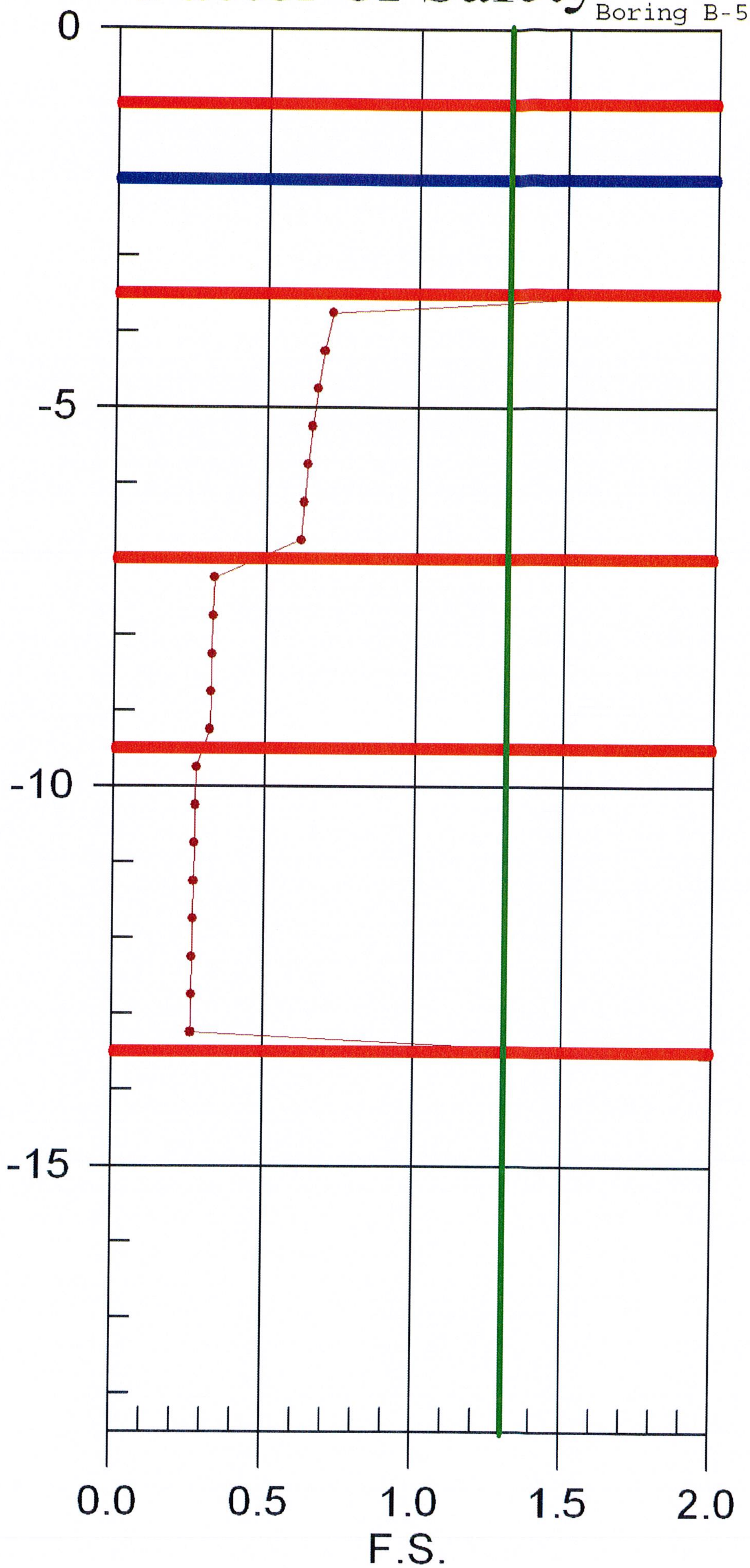
Base	Field N	0/1	Unit Wt.	Fines	D50	SPT Depth
1	15	1	105	18	0.7	0.5
3.5	19	1	126	8	0.3	3
7	17	1	126	7	0.27	5
9.5	12	1	128	12	0.3	9
13.5	9	1	115	19	0.7	11
18.5	100	1	125	10	0.4	16





(N1)60cs
Boring B-5
0
-5
-10
-15
Depth (ft)
0
50
100
N160 (bpf)

Factor of Safety



Job: Fort Bragg Fire Department
Job #: 10511.3
Date: 3/19/2009

Induced Vertical Settlement due to Liquefaction

DBE	0.67	g		
acc. Gravity	32.2	ft/sec^2		
M_w	7.3			
$V_{s,40'}$	250	m/sec	820.21	ft/sec
DWF_M	1.1			

Layer	Depth Range (ft)		Thickness of layer (ft)	N	$N_{corrected}$	$(N_1)_{60}$	Soil Material	$(N_1)_{60,CS}$	CSR	CSRN	ε_{v} (%)	ΔH (ft)	ΔH (inch)	ΔH (inch)
1	0	1	1	15	15	65.5	SM	71.1	0.44	0.40	0.1	0.00	0.0	
2	1	3.5	2.5	24	19	41.4	SP-SM	43.1	0.44	0.40	0.4	0.01	0.1	
3	3.5	7	3.5	22	17	30.7	SP-SM	31.9	0.58	0.53	1	0.04	0.4	0.4
4	7	9.5	2.5	12	12	18.9	SP-SM	20.4	0.64	0.58	2.2	0.06	0.7	0.7
5	9.5	13.5	4	9	9	12.6	SM	14.5	0.70	0.63	3.1	0.12	1.5	1.5
6	13.5	18.5	5	100	100	122.6	Siltstone	128.0	0.73	0.67	0	0.00	0.0	0.0

Based on boring B-5
As shown on Plate 9

Calculated settlement due to Liquefaction 2.6 Inches

Liquifible Layers

Job	Fort Bragg Fire Department
Job #	10511.3
Date	3/19/2009

Boring B-5

Depth (ft)	N_m	σ'_{vo}	C_N	E_m/E_{ff}	$(N_1)_{60}$
1	15	105	4.4	1	65.5
3.5	19	420	2.2	1	41.4
7	17	642.6	1.8	1	30.7
9.5	12	806.6	1.6	1	18.9
13.5	9	1017	1.4	1	12.6
18.5	100	1330	1.2	1	122.6

C_{fines}	Fines (%)	$(N_1)_{60,CS}$
1.09	18	71.1
1.04	8	43.1
1.04	7	31.9
1.08	12	20.4
1.15	19	14.5
1.04	10	128.0

E_m	0.6
E_{ff}	0.6
E_m/E_{ff}	1

Boring	B-5	
γ_{water}	62.4	pcf
water depth	2	ft

Job: Fort Bragg Fire Department
Job #: 10511.3
Date: 3/19/2009

Assume: P= Existing Over Burden Pressure (OB)

Depth (ft)	Δ Depth (ft)	Wet Density (pcf)	Wet Density-Water	Δ OB (psf)	OB (psf)
0	1	105	105	105	
1	2.5	126	126	315	105
3.5	3.5	126	63.6	222.6	420
7	2.5	128	65.6	164	642.6
9.5	4	115	52.6	210.4	806.6
13.5	5	125	62.6	313	1017
18.5					1330

Total Stress (psf)	pore water stress	Effective stress (psf)	rd	Effective stress (tsf)
105	0	105	0.9998	0.05
420	0	420	0.9991	0.21
861	218.4	642.6	0.9978	0.32
1181	374.4	806.6	0.9965	0.40
1641	624	1017	0.9937	0.51
2266	936	1330	0.9883	0.67