Earth Mechanics

FUGRO

PRELIMINARY
MARINE GEOTECHNICAL SITE CHARACTERIZATION
SAN FRANCISCO-OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

VOLUME 1, SECTION 3.0
MARINE SITE INVESTIGATION

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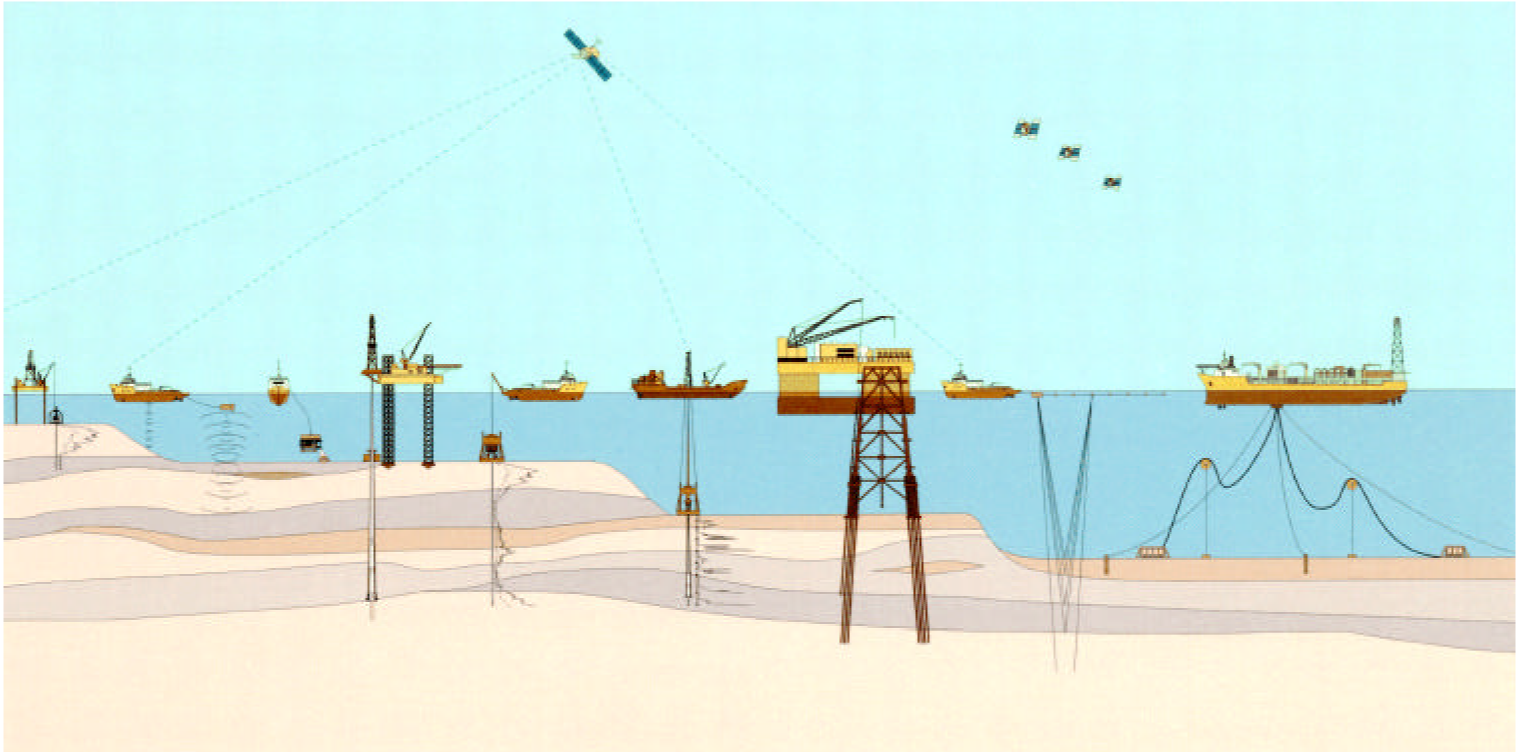


3.0 MARINE SITE INVESTIGATION

3.1 GENERAL OVERVIEW

The marine exploration program for the SFOBB East Span Replacement consisted of 14 wet-rotary borings drilled to depths of between 36 and 141 meters (119 and 462 feet) below the mudline. Drilling operations were conducted from two barge-mounted drill rigs. The barges used for the operation were supplied and operated by CS Marine Constructors, Inc., of Vallejo, California. The drilling equipment, in situ testing equipment, and laboratory testing equipment were supplied and operated by Fugro (Ventura, California, and Houston, Texas). Suspension logging equipment were provided by Agbabian Associates of Pasadena, California, and Welenco of Bakersfield, California.

The locations of the marine borings are shown on Plate 3.1. A summary of the dates of exploration for each boring location is provided on Table 3.1. The locations and surface elevations for each boring are also included on Table 3.1. The surveyed coordinates conform with California Coordinate System Zone 3, NAD '83 (meters). Diagrams of various sampling and in situ testing equipment are provided on Plates 3.2 through 3.6 and photos of the operations, barge, and equipment are included on Plates 3.7a through 3.7f. A Summary of Field Operations that provides a description of the drilling operations for each boring is included in each individual boring appendix.

3.2 VESSEL

Borings 98-1 through 98-6 were drilled from CS Marine's *Barge 38*, which is referred to in this report as the West Barge. Borings 98-7 through 98-12, 98-19, and 98-20 were drilled from CS Marine's *Barge 39*, which is referred to in this report as the East Barge. Due to the limited area available aboard the vessels, each of the drill barges was used in combination with a supply barge. The drill barges were approximately 9 meters by 33.5 meters (30 feet by 110 feet) long. The drill rig, pipe racks, field laboratories, suspension logging equipment, and part of the mud circulation system were located on the drill barge. The remainder of the mud circulation system (mud tanks, desander, water tank, etc.), drilling supplies (cement, saltwater gel, barite), a crane, and Halibut testing equipment were located on the supply barges. Skid-mounted drill rigs were welded to the drill barge's deck so that the borings could be advanced through a centerwell in the barge. Halibut remote vane tests were performed from the port side of the supply barge.

Fugro, GeoVision, and Welenco equipment were mobilized onboard the barges at CS Marine's dock in Vallejo. The drill barge and supply barge were fastened together, and pushed and positioned over the proposed drilling locations by the tug boats *Warrior* (West Barge) and *Betty L* (East Barge). The tug boats also served as anchor assist boats. The tug boats are owned and operated by Westar, Inc. The East Barge was held on position using two spuds and a four-point anchor spread. The West Barge was held on position using a four-point anchor spread only.

3.3 DRILLING EQUIPMENT AND PROCEDURES

3.3.1 Conventional Marine Drilling Procedures

The drilling equipment mobilized on each barge was tailored to the specific site conditions at the various drilling locations. A limited thickness of soil overlies bedrock at the locations explored from the West Barge. Since the majority of the explorations were to be in rock (foundations for the new bridge in this zone are expected to be founded in rock), the West Barge was set up as a primarily rock coring barge with soil sampling and minimal laboratory testing capabilities. A Failing 1500 DMX rotary drill rig was used for exploration performed from the West Barge.

Borings performed from the East Barge were anticipated to penetrate approximately 60 to 120 meters (200 to 400 feet) of soil before encountering bedrock. To characterize the soil sequences in these areas (where foundations are expected to bear in soil), the East Barge was set up for in situ testing using Fugro's Dolphin system in addition to soil sampling and rock coring. A Failing 2000 rotary drill rig was used for exploration performed from the East Barge.

Drilling and sampling within the soil sequences and upper weathered bedrock were accomplished by wet rotary drilling procedures using standard 127-millimeter [mm] (5-inch) API drill pipe and a drag bit. The drag bit used has a diameter of 250 mm (9-7/8 inches). For operations performed from the West Barge, the drag bit was directly connected to the drill pipe. For exploration performed from the East Barge, Fugro's thruster-drag bit assembly was connected to the drill pipe to facilitate the performance of in situ testing. Saltwater gel and weight material were used to suspend and remove drill cuttings, and to provide lateral pressure to support the sides of the borehole. Soil sampling and in situ testing were performed through the drill pipe.

Once bedrock was identified, the drill pipe was pulled to the deck, the drag bit or drag bit-thruster assembly removed, and the drill pipe lowered to the bottom of the hole to serve as casing. Rock core pipe was then lowered inside the drill pipe to the bottom of the hole and the hole was further advanced through rock using rotary drilling procedures and a Christensen 94-mm (3.7-inch) wireline core barrel system. Seawater was used to suspend and remove cuttings during rock coring operations.

Halibut vane shear testing was performed concurrently with rock coring. At the completion of rock coring suspension logging (P&S-wave logging, gamma/resistivity logging, caliper logging) was performed prior to abandoning the hole. Detailed descriptions of the soil sampling, in situ testing, rock coring operations, and suspension logging are presented below.

3.3.2 Project-Specific Modifications to Equipment and Procedures

Environmental regulations for drilling operations in the project area required that no drilling mud be expelled into the bay. To allow for the recirculation of drilling fluids, a 298-mm- (11-3/4-inch-) nominal-diameter casing was used to return the drill cuttings and drilling fluids to the deck. The project site is characterized by a tidal variation on the order of 2.1 meters (7 feet). To accommodate the resultant relative movement of the barge, a slip joint consisting of a 244-mm- (9-5/8-inch-) diameter casing that could slide inside a 298-mm- (11-3/4-inch-) diameter casing was used.

To facilitate the performance of downhole CPTs in shallow water depths while simultaneously providing for recirculation of drilling fluids, Fugro had to modify their standard equipment for use on the East Barge. This modification primarily involved the incorporation of a 0.86-meter- (34-inch-) diameter reaction mass and clamp system into the casing string used for mud recirculation.

To provide an adequate seal for the recirculation of drilling fluids, it was necessary to advance the casing to some depth below the mudline. For drilling operations from the West Barge, the slip joint was collapsed and the casing was driven below the mudline. Due to the relatively high end-bearing resistance offered by the reaction mass incorporated with the casing system onboard the East Barge, it was not always feasible to directly drive the casing an adequate distance below the mudline. Consequently, a 244-mm- (9-5/8-inch-) diameter stinger pipe was used inside the reaction mass to case the upper portion of the borehole.

At each boring location, the near-surface materials that would be disturbed by the installation of casing or by the stinger were initially sampled in an open hole. Subsequent drilling operations were performed in an offset cased hole. For drilling operations in an open hole, seawater was used to remove drill cuttings and to provide lateral support to the sides of the borehole. In addition to providing for the recovery of undisturbed samples and strength characterization of the near-surface materials, this procedure allowed for the evaluation of the most suitable depths to which the casing was to be driven aboard the West Barge, and for the most suitable length of stinger to be used on the East Barge. To seal off the upper portion of the hole and reduce the potential for loss of drilling fluids during operations in the cased hole, the bottom of the casing or the tip of the stinger was generally set in a relatively firm clay layer and below relatively loose near-surface sand layers. Since the reaction mass incorporated within the casing on the East Barge could only be driven a limited distance below the mudline, it was sometimes necessary to reconfigure the stinger for site-specific subsurface conditions.

Due to the presence of the reaction mass within the casing used on the East Barge, extreme care was required in selecting the length of casing, and in ensuring that the reaction mass was driven to the correct depth below the mudline such that the slip joint was capable of accommodating the tidal variations. If the reaction mass were driven too deep or the casing was too short, the slip joint would be fully extended during high tide and the buoyancy of the barge would tend to pull the casing and reaction mass out of the borehole. Conversely, if the reaction mass were not driven deep enough or if the casing were too long, the slip joint would fully collapse during low tide and there would be a potential for the casing to damage the rotary table of the drill rig.

These operations were especially critical at the locations of Borings 98-11, 98-12, and 98-20, which were characterized by relatively shallow water depths. At those locations, the length of the casing, and consequently the stroke in the slip joint, were limited by the water depth and the available lengths of casing pipe. Since the lowest water depths were on the order of 2.7 meter (9 feet), a slip joint that allowed for only approximately 2.7 meter (9 feet) of movement could be used at these locations. The depth to which the reaction mass needed to be driven was estimated on the basis of the water depth at the time the casing was set, and the predicted highest and lowest tides during the anticipated period of exploration. Since the tidal variation was approximately 2.1 meter (7 feet), the reaction mass was generally set such that there would be an approximately 0.3-meter (1-foot) margin for error at the anticipated highest and lowest tides. At the remaining boring locations where deeper water depths permitted the use of a longer slip joint, the reaction mass was generally driven below the mudline to allow a 0.6-meter (2-foot) margin for error at the anticipated lowest tide.

3.4 SURVEY AND WATER DEPTH MEASUREMENT

3.4.1 Datum

The horizontal and vertical datum specified by Caltrans were as follows:

- Horizontal Datum: California Coordinate System, Zone 3 in meters
- Vertical Datum: NGVD 29 Datum

Locations and elevations presented on the borings logs and used within this report are in reference to the above-specified project datum. The term depth or penetration, as used herein, is generally in reference to the mudline (or bay bottom) (i.e., 13-meter depth [below mudline]).

The NGVD 29 datum is generally equivalent to MSL datum, and is 0.942 meter (about 3.1 feet) above Mean Lower Low Water (MLLW) datum.

3.4.2 Position and Navigation

A Differential Global Positioning System (DGPS) and integrated navigation software, owned and operated by Fugro, were used to position vessels at the specified drilling locations and to determine final X-Y coordinates of the centerwell for those locations. The DGPS also was used to position the barge's anchors. Coordinates calculated from Fugro's DGPS system are considered accurate to within about 1 meter (3 feet).

Halibut remote vane tests were performed on the port side of the supply barge. The offsets of the Halibut frames were established by measuring from the centerwell of the drilling barges. Coordinates of the Halibut test locations were then calculated using the measured coordinates of the centerwell and the heading of the barges. As described subsequently, the Halibut basket was lowered from the crane to a slightly different location for each test. Consequently, the actual location of the Halibut vane shear tests fall within an approximately 3-meter (10-foot) radius of the reported coordinates.

3.4.3 Water Depth Measurement

Initial Water Depth. Prior to the initiation of drilling, the water depth was measured by two methods. An initial water depth was estimated using an echo sounder provided by Fugro Survey. The initial water depth was confirmed using an electronic bottom sensor. The bottom sensor is an electronic mudline sensor that is operated on an electric cable that latches into the drill bit. In this method, the drill pipe was lowered to the mudline until the drill bit was about 1.5 meters (5 feet) above the mudline, based on the initial water depth measured using the echo sounder. The bottom sensor was lowered through the drill pipe until it rested on a catch ring in the drill bit, as illustrated on Plate 3.2. The drill string was then slowly lowered to contact the mudline. When the bottom sensor touched the mudline, an electrical signal was received on the drill floor and the water depth was recorded as the length of the drill string below the water level.

Adjustment for Tidal Variations. Water depth variations at the site were constantly monitored using the echo sounder. During soil sampling operations, water depth measurements were generally taken at approximately 15-minute intervals. When the duration between samples or in situ tests was greater than 15 minutes, a water depth measurement was made prior to the recovery of each sample or the performance of each test. The driller would then adjust his reference marks so that the depth to which the drill pipe was advanced would provide for the recovery of samples or the performance of in situ tests at the specified penetrations below the mudline. During rock coring, water depth measurements were made at the commencement and completion of each core run. The change in water depth during the core run was used in combination with the depth by which the core pipe was advanced below the barge deck during the core run to calculate the length of the core run.

Variations in water depth were also monitored using predicted tide charts for the project area and the initial water depth. In general, the water depths measured using the echo sounder and calculated using the tide charts were within a few tenths of a foot of each other.

3.5 SAMPLING AND IN SITU TESTING INTERVALS

3.5.1 West Barge Borings

Soil samples were taken semi-continuously (1-meter [3-foot] intervals) in borings drilled from the West Barge, except in Boring 98-6 in which samples were taken semi-continuously to about 9-meter (30-foot) penetration and at about 1.5-meter (5-foot) intervals thereafter. At the top of bedrock, a standard penetration test (SPT) or driven wireline sample generally was collected to confirm top of bedrock. Once bedrock was identified, continuous rock coring was performed to the required total penetration. In Boring 98-5, rock coring also was performed to advance the hole through a gravel layer with cobbles and boulders between approximately 4- and 11-meter (13- and 36-foot) penetration.

3.5.2 East Barge Borings

For borings performed from the East Barge, samples were taken semi-continuously in the initial open hole, typically to a penetration of approximately 12 meters (40 feet). Semi-continuous sampling was extended to depths of 36 meters (119 feet), 34 meters (113 feet), and 17 meters (57 feet) in Borings 98-7, 98-8, and 98-10, respectively.

Below the depth of semi-continuous intervals, alternate sampling and in situ testing were performed. Wherever remote vane tests were feasible, piezocone penetration tests (CPTs) and remote vane tests were alternated. When the in situ test between samples was a remote vane test, the sample interval was approximately 2.4 meters (8 feet). When the in situ test between samples was a CPT, the sample interval depended on the stroke achieved during the performance of the CPT test. When a full 3-meter (9.8-foot) stroke was achieved, the sample interval was 3.6 meter (12 feet). When the CPT stroke was less than 3 meters (9.8 feet), the sample interval was reduced by the amount by which the CPT stroke was less than the full 3-meter (9.8-foot) stroke. Below approximately 90-meter (300-foot) penetration, the minimum sample interval was generally about 1.5 to 2 meters (5 to 7 feet).

At the top of bedrock, an SPT or driven wireline sample generally was collected to confirm top of bedrock. Once bedrock was identified, continuous rock coring was performed to the required total penetration.

Continuous in situ testing (without sampling) was generally performed in the offset borings between penetrations of approximately 6 meters (20 feet) and the depth to which semi-continuous sampling was performed in the open hole. This procedure was followed to facilitate the development of site-specific correlations between strengths estimated from in situ tests and laboratory strength tests. In selected borings, percussion samples were taken with a downhole wireline hammer at penetrations at which SPTs had been performed in the open hole boring.

3.5.3 Halibut Testing

At boring locations in which the near-surface soils were identified as being suitable for vane shear tests, Halibut remote vane tests were performed at 0.6-meter (2-foot) intervals. The Halibut tests were performed from 0.6-meter (2-foot) penetration to as much as 7-meter (24-foot) penetration. The depths to which Halibut testing was possible was constrained by the presence of shallow sand layers or stiff to very stiff clay layers. In borings drilled in relatively shallow water (Borings 98-10, 98-11, 98-12, and 98-20), the depth of Halibut vane testing was also limited to the maximum clear gap between the mudline and the bottom of the Halibut basket. For example, Halibut testing was only possible to a depth of 5.5 meters (18 feet) in Boring 98-11 since this was the maximum length of rod that could be added with the Halibut basket resting in the overside frame without penetrating the vane blade into soil.

3.6 SOIL SAMPLING EQUIPMENT AND PROCEDURES

3.6.1 Type of Samplers

Soil samples were obtained through the drill pipe at intervals specified above. A 63.5-mm-OD, 54.0-mm-ID (2.5-inch-OD, 2.125-inch-ID) liner sampler was used to obtain very soft to soft cohesive soil samples near the mudline. The remaining cohesive soil samples were generally taken using a 76-mm-OD, 72-mm-ID (3.0-inch-OD, 2.83-inch-ID) thin-walled Shelby tube sampler pushed into the soil with the weight of the drill string.

In granular soils, percussion (hammer) samples were obtained mainly with a 57-mm-OD, 54-mm-ID (2.25-inch-OD, 2.125-inch-ID) thin-walled Shelby tube, or a variable-OD, 54-mm-ID (2.125-inch-ID) thick-walled tube. Additional percussion samples were obtained, as per Caltrans requirements, in cohesive and granular soils using a 51-mm-OD, 35-mm-ID (2.00-inch-OD, 1.375-inch-ID) split spoon sampler, and a 76-mm-OD, 60-mm-ID (3.00-inch-OD, 2.375-inch-ID) modified California sampler.

3.6.2 Sampling Procedures

The liner sampler, which was used to aid sample recovery in very soft to soft cohesive sediments, was pushed 0.6 meter (2 feet) into the soil formation with the weight of a 79.4-kilogram (175-pound) wireline hammer and is noted as "WOH" on the boring logs.

Push samples, indicated by "PUSH" on the boring logs, were recovered by pushing a Dolphin push sampler into the soil formation using the weight of the drill string. The Dolphin push sampler is shown on Plate 3.3. In the push sampling technique, the drill bit was raised from the borehole bottom after the boring was drilled to the desired sampling depth. The push sampler was allowed to free fall through the drill pipe until it rested on a catch ring in the bottomhole assembly, with the mechanical pawls, sample tube, and tube adapter located outside the drill bit as shown on Plate 3.3. The drill string was then lowered to engage the mechanical pawls, and the weight of the drill string pushed the sample tube 0.6 meter (2 feet) into the soil. The drill string was then raised to pull the sample tube out of the soil. The push sampler was retrieved and raised to the drill floor using an overshot retrieving device attached to a wireline. After retrieving the sampler, the boring was advanced and the sampling procedure repeated.

Downhole wireline percussion tube samples were collected by lowering the sampler on a hammer and slide assembly through the drill pipe to the desired sampling depth. The hammer is a 79.4-kilogram (175-pound) sliding weight that was raised and dropped approximately 1.5 meters (5 feet) to achieve a maximum penetration of 0.6 meter (2 feet), or a maximum of 30 blows. In very dense granular soils, blow counts in excess of 30 blows are sometimes required to drive the sampler into the formation in order to obtain an adequate amount of sample for laboratory soil testing. The number of blows with their corresponding sampler penetrations are recorded on the boring logs.

The SPT split spoon sampler and modified California samplers were attached to N-rod drill pipe and lowered through the drill pipe to the bottom of the hole. The sampler was then driven with a 63.5-kilogram (140-pound) hammer attached to the top of the N-rod string. The hammer was raised approximately 0.75 meter (30 inches) using a typical two-wrap cathead hoist system on the drill rig and dropped for successive blows. SPTs were performed in general conformance with standard test method ASTM D1586. A few modified California samples also were obtained using the procedures described above for percussion tube samples.

3.7 IN SITU TESTING EQUIPMENT AND PROCEDURES

The Dolphin piezocone penetrometer, the Halibut remote vane, and the Dolphin remote vane (components of Fugro's Dolphin in situ testing system) were used to characterize and classify the subsurface soils. The piezocone penetrometer and in situ vane testing techniques are presented in the following paragraphs.

3.7.1 Piezocone Penetration Testing

CPTs were performed using Fugro's Dolphin piezocone penetrometer. The tests were performed to: (1) measure the in situ shear strengths of cohesive soils, and (2) estimate the relative density and frictional characteristics of granular soils. A schematic of the Dolphin piezocone penetrometer is presented on Plate 3.4. The CPTs were typically performed at about 3.7-meter (12-foot) intervals. The length of each push of the piezocone (which is indicated on the boring log) is dependent upon the soil resistance.

The first step in operating the Dolphin piezocone penetrometer was to lower a reaction mass-clamp assembly to the mudline prior to the start of the CPT testing operation. For the SFOBB East Span Replacement Project, the reaction mass-clamp assembly was incorporated within the casing used for recirculation of fluids. The Dolphin piezocone penetrometer was then allowed to free fall through the drill pipe to the Dolphin bottomhole thruster-drag bit assembly where it seated and latched inside the thruster under its own weight. The drill bit was then lowered to the borehole bottom and the clamping unit hydraulically activated from the drill floor to grip the drill string. The thrust for the piezocone was provided by the mud pressure developed in the drill string using an auxiliary high pressure pump. The piezocone was thrust into the soil formation at a controlled rate of penetration of about 2 centimeters (cm) per second with reaction being provided by the weight of the drill string and the reaction mass.

Calibrated strain gages in the cone electronically measured the cone tip resistance and sleeve resistance as the piezocone penetrated into the soil formation. The pore pressure is measured with an electronic pressure transducer. These measurements, as well as the penetration rate and flow data, were recorded at a rate of four samples per second and were stored temporarily in the downhole remote memory unit (RMU) of the Dolphin tool. The test was terminated after achieving a 3.0-meter (9.8-foot) stroke, or when refusal was encountered.

After the test was completed, the tool was retrieved with a high-speed wireline. Once the tool was back on deck, the data were transferred from the RMU to the computer. The computer generated a "quickplot" of the data, which provided generalized information on the cone tip resistance, sleeve resistance, pore pressure, rate of penetration, flow, and total tool stroke. The CPT data were processed and are shown on the logs presented for each boring.

CPTs were performed using a standard three-channel piezocone penetrometer, which measured cone tip resistance, sleeve resistance and pore water pressure. A standard 5-ton piezocone with an apex angle of 60 degrees, a base area of 1,000 square millimeters (mm^2), and a cylindrical sleeve area of 15,000 mm^2 was used. The pore water pressure measurements were taken immediately above the base of the piezocone (the filter is located 5 mm above the apex of the piezocone tip). The penetration rate of the piezocone penetrometer was approximately 2 cm per second. During the test, measurements of piezocone tip resistance (q_c), sleeve friction (f_s), and pore water pressure (u_2) were recorded simultaneously in the RMU of the Dolphin piezocone penetrometer. Detailed descriptions of the tool and testing procedures are presented in the previous section. The test equipment and procedures meet ASTM D-3441 specifications.

The presentation of results from CPTs consists of graphs showing q_c, f_s, u_2, and R_f versus depth below the mudline. The reference level of the test is the bottom of the borehole. The above terms are defined as follows:

q_c = piezocone resistance relative to the reference level of the test;

f_s = sleeve friction relative to the reference level of the test. A depth correction is applied so that sleeve friction is presented for the corresponding piezocone depth;

u_2 = pore pressure at the cylindrical extension above the base of the piezocone, relative to the reference level of the test; and

R_f = ratio of sleeve friction to piezocone resistance (f_s/q_c). This calculated ratio is for the piezocone depth.

3.7.2 Halibut Remote Vane Testing

Fugro's Halibut remote vane system was used to measure the in situ shear strengths of the near-mudline cohesive soils in the immediate vicinity of the soil boring location. In this system, the Dolphin remote vane was attached to the Halibut, which is a stabilizing ballast support frame and collar that rests on the mudline. Plate 3.5 presents a schematic drawing of the Halibut remote vane. The vane was set to achieve a fixed penetration below the square 1.5x1.5-meter (5x5-foot) base plate of the Halibut. The Halibut, attached to a wireline, was lowered to the mudline over the port side of the supply barges rather than through the drill pipe as in the downhole remote vane. The vane penetrates to the predetermined depth as the Halibut is lowered to the mudline. Eight to ten 445-N (100-pound) lead weights were added to the 4,448-N (1,000-pound) Halibut assembly to ensure full penetration of the vane.

Recording of test data began after the torque on the vane blade, driven by an electric motor, exceeded a preprogrammed torque threshold. The data were sampled once every second and stored temporarily in the RMU of the vane tool. Data collection was automatically terminated if the torque exceeded 33.9 N-meters (300 inch-pounds). The vane blade turns at a rate of about 18 degrees per minute. The test was allowed to run for about 3 minutes, after which the Halibut vane was raised above the mudline and repositioned to perform a second test at the same test depth. At the end of the second test, the Halibut was lifted back on the deck of the barge and the RMU was connected to a computer to begin data retrieval. The computer generated a "quickplot" of the torques developed. The undrained shear strengths were then computed from the maximum vane blade torques and transducer calibration factors.

After the data retrieval procedure was completed, the RMU was reinitiated and prepared to collect data for the next test depth. At the same time, 0.6-meter (2-foot) extension was added to the tool for the next test depth. The in situ shear strengths of cohesive soils measured by the Halibut remote vane are plotted on the boring logs.

Halibut vane shear tests were performed off the port side of the supply barge, concurrently with rock coring or while casing was being set. Periods of slack tide were selected for Halibut testing to reduce the potential for strong currents to tilt the Halibut basket. Tilting of the Halibut basket could result in inclined penetration of the vane blade and rods, and potential damage to the Halibut testing equipment.

3.7.3 Downhole Remote Vane Testing

The Dolphin remote vane was used to measure the in situ shear strengths of normally to lightly overconsolidated cohesive soils. Vane testing was generally not performed in hard clay strata where the in situ shear strength of the soils exceeds the measuring capacity of the vane. The vane testing was also not performed on highly stratified (interlayered clay and sand) soil profiles.

A schematic drawing showing the downhole Dolphin remote vane is presented on Plate 3.6. The downhole vane tests were performed immediately after the push sampling operation. The operational sequence for performing a downhole vane test involved suspending the drill string with the drill bit about 6.1 meters (20 feet) above the borehole bottom to prevent the blade of the vane from stabbing into the soil formation before testing. The tool was allowed to free fall through the drill pipe until it rested on a catch ring in the bottomhole assembly. The drill string was then lowered onto the mechanical pawls, and the four-bladed vane pushed to the desired test depth, which was generally 1.5 meters (5 feet) deeper than the previous sampling depth. The drill string was then raised about 0.9 meter (3 feet) to prevent contact between the pawls and the drill bit while the test was in progress.

The test was allowed to run for about 3 minutes. The test data were recorded as described earlier. At the end of the test, the drill string was lowered to push the tool an additional 0.6 meter (2 feet) to perform the second test. At the end of the second test, the tool was pulled out of the soil formation using the drill string and retrieved with an overshot retrieving device attached to a wireline. Once the tool was back on the drill floor, the RMU was connected to a computer for the data retrieval procedure. The computer generated a "quickplot" of the torques developed. The undrained shear strength was then computed from the maximum vane blade torques and transducer calibration factors. The maximum shear strength that can be measured by the Dolphin remote vane device is approximately 297 kilopascals (kPa) (5.5 kips per square foot [ksf]). After the data retrieval procedure was completed, the RMU was reinitialized and prepared for the next test.

3.7.4 Pressuremeter (Dilatometer) Testing

Seven pressuremeter tests were performed in Boring 98-2 from depths of 11.3 to 38.7 meters (37 to 127 feet) below mudline. The testing was performed by Hughes Insitu Engineering, Inc., of Vancouver, British Columbia.

The purpose of the pressuremeter testing was to measure the in situ maximum and average shear modulus of the Franciscan Formation bedrock for use in the foundation design. For this study, the testing was performed with a monocell pressuremeter with three independent electronic displacement sensors spaced at 120 degrees apart. The sensors are covered with a flexible pressuremeter membrane, clamped at each end, which is pressurized by use of hydraulic fluid to deform the adjacent rock. A protective sheet of stainless steel strips, pre-bent to the shape of the borehole wall, is placed over the membrane.

A pressuremeter test is performed by lowering the tool to the selected depth and manually inflating (expanding) the tool through the use of hydraulic fluid. As the tool is inflated, the average displacement of the rock mass is measured at the three sensor locations. The pressuremeter measures rock stress in a horizontal direction, not in the vertical direction as would occur under foundation loading. Additional information concerning the pressuremeter testing is summarized under separate cover in a report by Hughes Insitu Engineering (1998).

3.8 ROCK CORING PROCEDURE

Prior to coring, an inner casing consisting of 114-mm (4-1/2-inch)-diameter (ID) drill pipe was socketed about a meter (several feet) into the bedrock to help reduce the potential for caving. Once the inner casing was set, rock coring was performed using rotary drilling methods with saltwater as the drilling fluid. Continuous rock coring was performed for a depth (below the top of rock) of 17 to 61 meters (55 to 201 feet) using a Christensen 94-mm wireline coring system with a diamond-impregnated coring bit. H-size core (61-mm [2.4-inch] core diameter) was collected in every hole except Boring 98-2, where N-size core (52-mm [2.06-inch] core diameter) was collected to allow for pressuremeter (dilatometer) testing.

The Christensen core barrel is a hollow, double-tube core barrel (hollow tube equipped with a stationary inner tube) attached at the top of the drill pipe and fastened at its lower end with an annular diamond bit. For this study, a HC (61-mm- [24-inch-]) diameter core was specified. The outer tube of the HC core pipe is a 94-mm-OD (3.7-inch-OD) pipe. The inner tube assembly consists of a 65-mm (2.56-inch)-ID inner tube with a removable core lifter case and core lifter at one end, and a removable inner tube head, bearing, and overshot assembly on the opposite end. The inner tube latching device locks into a complementary recess in the wall of the outer tube such that the outer tube is allowed to rotate without causing rotation of the inner tube. When the core run is complete, the latch is activated and the inner tube assembly retrieved to the deck.

Coring was performed by rotating the bit and cutting an annular hole around the core, which then entered into the stationary inner tube of the core barrel. At the same time, drilling fluid comprised of saltwater was pumped via the swivel head into the drill pipe and flowed out around the annular space between the two tubes of the core barrel. It escaped through the openings between the cutting teeth and rose between the outer tube of the core barrel and the walls of the borehole on up the borehole outside the drill pipe. The drilling fluid prevented overheating of the bit and carried the drill cuttings to the surface. The drilling fluid also provided lateral pressure to support the sides of the borehole.

In this sampling technique, rock core samples were taken as drilling proceeded. The outer barrel (with cutting teeth rotating) cuts an annular hole around the core, which enters into the swivel-mounted stationary inner tube. The rock core sample with a 61-mm- (2.4-inch-) diameter was retained in the inner tube by a core lifter. At the end of each rock coring run, the inner tube was retrieved and raised to the deck using an overshot attached to a wireline. Then the rock core sample was removed from the inner tube prior to making another core run. Continuous rock core runs of about 1.5 meters (5 feet) were performed to reduce the potential for jamming the fractured rock in the 3-meter- (10-foot-) long core barrel. Actual rock core runs varied from a low of 0.3 meter (1 foot) to as long as 3 meters (10 feet), depending on the rock characteristics and coring conditions. Descriptions of the rock penetrated and the depths at which rock core samples were taken are presented on the boring logs.

3.9 DOWNHOLE GEOPHYSICAL LOGGING

Upon completion of the rock coring, a suite of electric wireline logs was performed. The suite of wireline logging performed included:

- Suspension log (primary and shear [P&S] wave velocity) in all borings except 98-19;
- Natural gamma and resistivity logs (soil lithology) in all borings except 98-19 and 98-20;
- Caliper log (borehole diameter) in all borings except 98-19 and 98-20; and
- Acoustic televiewer log (fracture identification) in Borings 98-1 through 98-5.

The suspension logging was performed by Agbabian Associates of Pasadena, California. The gamma, resistivity, caliper, and acoustic televiewer logging was performed by Welenco of Bakersfield, California.

In the suspension method, the system directly determines the average P&S wave velocity of a 1-meter- (3.3-foot-) high segment of the soil and rock column surrounding the borehole by measuring the elapsed time between arrivals of a wave propagating upward through the soil/rock column. The suspension probe includes both a shear-wave source and compressional-wave source, and two biaxial receivers that detect the source waves.

In general, suspension and e-logging was performed in multiple runs. Initially, the core pipe was withdrawn to the top of rock, and the lower portion of the hole was logged. Subsequently, core pipe and drill pipe were pulled to deck and suspension logging performed for the full depth of the hole. Depending on the stability of the hole, the core pipe and the drill pipe were withdrawn in one or more sections. Where potentially unstable layers were anticipated, the core pipe or drill pipe would be withdrawn to depths that were slightly below these layers so as to permit logging of the lower portions of the hole prior to the removal of support within the potentially unstable zones.

Continuous logs are available for borings where conditions permitted tool access. In general, continuous logs are available in the eastern borings (98-7 through 98-12) and Boring 98-3. In the western borings, the lower portion of the hole was logged prior to removing the inner casing string. With the exception of Boring 98-3, either the weathered bedrock or gravelly soil above the bedrock in the western borings caved when the inner casing was removed, precluding logging of the upper portion of the drill hole. Where conditions permitted, the upper soil portion of the western drill holes was logged to the depth where the drill hole had caved. Additional details of the suspension logging procedures and results are presented under separate cover in a report by Geovision (1998).

3.10 BOREHOLE ABANDONMENT

At the completion of each boring, the hole was grouted with a mixture of neat cement consisting of seven sacks of cement and one sack of gel per 100 gallons of water. The mixture was pumped down the hole using N-rod as a tremie pipe.

3.11 SAMPLE LOGGING AND FIELD LABORATORY TESTING

3.11.1 Soil Samples

During the field operation, all the soil samples were extruded from the samplers, examined and visually classified by Fugro's soil technicians, geologists, and engineers concurrent with the drilling, sampling, and in situ testing operations. The only exceptions were the specially preserved, 72-mm- (2.83-inch-) diameter, thin-walled Shelby tube samples obtained at selected depths in borings drilled from the East Barge. Those "save" tube samples were preserved by sealing each end of the sample with packers and wax.

Several types of strength tests were performed on the cohesive soil samples. Undisturbed and residual shear strength measurements were made on selected cohesive soil samples with a motorized miniature vane device at the bottom of the tube while the soil samples were still in the sample tubes. After sample extrusion, remolded miniature vane (MV) tests and undisturbed and remolded unconsolidated-undrained (UU) triaxial compression tests were performed on selected cohesive soil samples. Additional shear strength estimates were performed with a Torvane device, a Swedish fall cone device, and a hand-held pocket penetrometer.

Natural moisture content determinations were performed on all soil samples. Unit weights were measured on all cohesive soil samples and, where possible, on cohesionless soil samples. The results of the field classification and strength tests are included on the boring logs (provided in Appendices 98-1 through 98-12, 98-19, and 98-20).

Representative portions of each soil sample were sealed in airtight containers. Whenever possible, portions of undisturbed samples extruded from tubes were doubled wrapped with plastic wrap and aluminum foil, and stored in close-fitting, airtight, plastic quarts for additional laboratory testing. Disturbed samples and samples that had been tested were placed in plastic bags. The specially preserved "save" tube samples were sealed with end caps and wax, and were transported to Fugro's Houston laboratory for advanced laboratory testing.

Since the West Barge was set up primarily as a rock coring barge, minimal soil testing equipment was provided onboard this barge. Moisture content samples were weighed onboard the West Barge, and then transferred in containers with lids to the East Barge laboratory for drying. Additionally, samples for UU tests were transported in quarts to the East Barge where they were tested.

3.11.2 Rock Core Samples

Rock cores in the inner split tube of the core barrel were pried open so as to minimize core disturbance and carefully examined and classified by our field geologist or engineer. Particular attention was given to describing fractures and discontinuities within the sample. To assist our efforts in estimating the quality of the rock encountered in the borings, the core recovery and rock quality designation (RQD) were recorded. The cores were photographed in the core barrels prior to being extruded and stored in rock core boxes. Additional photographs were taken of the samples stored in each core box.

Whenever possible, Point Load Tests (PLTs) were performed on a sample from each core run. Care was taken during the performance of PLTs to note whether failures occurred along existing planes of weakness. The PLT is intended as an index test for the strength classification of rock materials. It can also be correlated with other rock strength parameters such as the unconfined compressive strength (q_u). The test measures the Point Load Strength Index ($I_{s(50)}$) and strength anisotropy index ($I_{a(50)}$) of rock specimens. Point load testing for this study was performed using a Rocktest Model PIL-5 testing device in general accordance with the suggested method for point load testing by the International Society for Rock Mechanics (ISRM, 1985). The PLT results for the borings are summarized on the individual boring logs.

3.12 DATA ENTRY

Data collected during the sampling, in situ testing, and laboratory testing programs performed onboard the barges were generally entered into a database. The electronic databases were then used to generate preliminary boring logs in the field using Fugro's proprietary software. In addition to providing preliminary information in a timely manner to Caltrans and the design team, the data processing also allowed for the controlled evaluation of data onboard. For example, bad UU tests were easily identifiable on the boring logs, which allowed for retesting of the relatively undisturbed samples almost immediately. Additionally, the preliminary boring logs were useful tools when making operational decisions such as estimating the most suitable length of stinger. At the completion of each boring, the data were electronically mailed (e-mailed) to our Ventura office. Boring logs and related test data are presented in the individual appendix for each boring. Additional field data have been provided in the Field Data Compilations (Fugro, 1998a) for each boring, dated April 20, 1998.

3.13 ONSHORE LABORATORY TESTING PROGRAM

The laboratory testing program for this study was performed in two phases. Visual classification, wet density determinations, water content determinations, and strength tests (including Torvane tests, pocket penetrometer tests, miniature vane tests, and UU triaxial compression tests) were performed concurrently with drilling and sampling operations. Additional laboratory soil testing was subsequently performed at Fugro's onshore soil mechanics laboratories in Ventura, California, and Houston, Texas. Dynamic testing was performed at Fugro's Houston laboratory and at the University of Texas at Austin. The onshore laboratory testing program included: classification tests (grain size, plasticity, organic content, specific gravity), consolidation tests, consolidated drained and undrained triaxial tests, resonant column tests, and cyclic direct simple shear tests. In addition, a rock testing program with unconfined compression and direct shear tests was conducted at the GeoTest Unlimited laboratory in San Leandro, California.

The test procedures generally conformed to the applicable Caltrans and ASTM standards. The following sections in this appendix provide descriptions of the laboratory tests performed in the static laboratory soil testing program. The total numbers of laboratory tests that have been conducted are listed on Table 3.2. The results of most laboratory tests conducted on soil and rock samples are included in the individual boring-specific appendices provided in Volumes 2A and 2B. Three types of special tests, however, are exceptions to that statement. Those test types are the: resonant column, cyclic simple shear, and torsional shear tests. The results of those tests are summarized in Section 5.3. Detailed test results are provided under separate cover in Fugro South (1998) and University of Texas at Austin (1998).

3.13.1 Classification Tests

The method of classifying soils according to their engineering properties used in this study was ASTM Standard Test Method D2487, which is based on the Unified Soil Classification System. The classification tests performed for this project include tests for moisture content, unit weight, grain size distribution, Atterberg limits, organic content, and specific gravity. All of those data are tabulated on the Summary of Test Results provided in the boring-specific appendices (Volumes 2A and 2B), and most of the results are also plotted on the boring log.

Tests for moisture content and unit weight of the soils were performed in general accordance with California Test 226. Plastic and liquid limits, collectively termed the Atterberg limits, were determined for selected cohesive soil samples and were performed in accordance with California Test 204. Wet densities of soil samples, where possible, were measured in the field by weighing soil samples of known volumes immediately after extrusion. Moisture content measurements for the above tests were from samples dried in an oven maintained at approximately 110 degrees Celsius. In the onshore laboratories, potentially organic samples were dried in an oven maintained at approximately 60 degrees Celsius.

Specific gravity of soil tests were performed in accordance with California Test 209. Organic content tests to estimate the weight percentage of organic matter in potentially organic soils were performed in accordance with ASTM D2974.

The gradation characteristics of selected samples were estimated in general accordance with the sieve and hydrometer analysis procedures of California Tests 202 and 203, respectively. In general, sieve analysis (for sizes greater than the No. 200 sieve) were performed on primarily granular samples (sand and gravel), and hydrometer analysis was performed on primarily fine-grained samples (silt and clay). Several granular samples were tested for only the percent passing the No. 200 sieve.

3.13.2 Shear Strength Tests

Undrained Shear Strength. Six procedures were used in the laboratory investigation to determine the undrained shear strengths of the cohesive soils under various conditions. Undisturbed and residual shear strengths of cohesive soil samples were determined in the field with a motorized miniature vane device while the samples were still in the sample tubes. Remolded MV tests, and undisturbed and remolded UU triaxial compression tests were performed in the field on selected samples after extrusion. Estimates of shear strength also were made in the field using a Torvane device, a fall cone device, and a hand-held pocket penetrometer. Additional UU triaxial compression tests were performed on selected cohesive soil samples in Fugro's Houston laboratory. K_o consolidated undrained triaxial compression tests were performed on six samples in Fugro's Houston laboratory. The test procedures are described in the following paragraphs.

Torvane Test. In the Torvane test, a small hand-operated device, consisting of a metal disc with thin, radial vanes projecting from one face, is pressed against the flat surface of the soil until the vanes are fully embedded. The device is then rotated through a torsion spring until the soil is sheared. The device is calibrated to indicate the undrained shear strength of the soil directly from the rotation of the torsion spring.

Pocket Penetrometer Test. This test is performed by slowly pressing a small flat-ended cylindrical metal rod (6.35-mm- [0.25-inch-] diameter) into the flat surface of the soil sample through a spring until it is embedded a predetermined distance. The resistance to penetration is recorded by the spring, which is calibrated to read the undrained shear strength of the soil based on spring compression.

Fall Cone Test. This test is performed by first lowering a conical tool of known dimensions and weight so that the tip just makes contact with the flat surface of the soil sample. The tool is then released so that it penetrates below the surface of the sample under its own weight. The device is calibrated to indicate the undrained shear strength of the soil directly from the measured penetration of the cone. The maximum undrained shear strength that could be measured with the apparatus onboard the barge is approximately 215 kPa (4.5 ksf).

Miniature Vane Test. Miniature vane tests are performed in accordance with ASTM D4648. In performing the miniature vane test, a small, 4-bladed vane is inserted into an undisturbed or remolded cohesive specimen. Torque is applied to the vane through a calibrated spring activated by a motorized pulley and belt system, which causes the vane to rotate slowly until soil shear failure occurs. The undisturbed or remolded shear strength of the soil is computed from the torque transmitted by the calibrated spring by multiplying the net rotation, in degrees, by the spring calibration factor. The maximum undrained shear strength that could be measured with Fugro's miniature vane device is approximately 215 kPa (4.5 ksf).

In selected undisturbed miniature vane tests, residual shear strengths of very soft to firm clay soils also are measured by allowing the vane to continue rotating after the initial soil shear failure has occurred. The tests are terminated when the torque applied to the vane through the calibrated spring has reached a constant value. The residual shear strength, which represents the soil shear strength at large strain, is computed by multiplying the net rotation (in degrees) by the spring calibration factor.

Unconsolidated-Undrained Triaxial Compression Test. In this type of strength test, either an undisturbed or remolded test specimen is enclosed in a thin rubber membrane and subjected to a confining pressure at least equal to the computed effective overburden pressure. A confining pressure of about 830 kPa [120 psi] (the pressure limitation of the triaxial cell in the field laboratory) was used in the field for UU triaxial compression tests performed on samples encountered at deep penetration. The test specimen is not allowed to consolidate under the influence of this confining pressure prior to testing. The test specimen is then loaded axially to failure at a constant rate of strain without permitting drainage from the specimen. The undrained shear strength of the cohesive soil is computed as one-half the maximum observed deviator stress.

Undisturbed and remolded shear strengths determined by this type of test are tabulated on the Summary of Test Results in the boring-specific appendices (Volumes 2A and 2B). Values of ε_{50} from undisturbed UU triaxial compression tests, confining pressure, percent strain at failure, and type of failure are tabulated with the stress-strain curves in the boring specific appendices.

K_o Consolidated Undrained Triaxial Compression Test. Triaxial Compression (CK_oUC) tests were performed to determine the stress-strain characteristics and the static shear strength of a soil specimen under applied axial stresses. This test uses procedures recommend by ASTM

D4767. Prior to setting up the test, the in situ effective vertical stress at the penetration of the sample was estimated on the basis of the interpreted unit weight profile at the boring location and the maximum past pressure that the sample had been subjected to was estimated from a consolidation test performed on a specimen from the same Shelby tube.

A cylindrical soil specimen was enclosed in a rubber membrane and placed inside a pressure chamber (triaxial cell). Tests were performed on specimens trimmed to 50.8-mm (2.0-inch) diameter (area = 20.27 cm^2 or 3.14 $in.^2$) and a height of about 114 mm (4.5 inches). The test specimens were saturated through back pressuring. Specimens were then K_o consolidated (diameter kept constant) in a drained state at a controlled rate of strain (about 0.3 percent per hour) to stresses greater than or equal to the estimated maximum past pressure thus inducing an OCR of 1.0. Unless the estimated in situ OCR was greater than 1.0, the specimen was cured (simulating aging) at constant vertical stress, and then sheared. When the estimated in situ OCR was greater than 1.0, the specimen was rebounded to the assigned OCR, cured, and then sheared. The curing period was generally about 24 hours and allowed for a minimum one log cycle of secondary consolidation at the test stress. Vertically or spirally oriented 6-mm- (1/4-inch-) wide, Whitman No. 54 filter strips placed at about 6-mm (1/4-inch) spacing provided radial drainage. The chamber pressure was kept constant and specimen drainage was not permitted during shear. A loading piston was advanced against the specimen cap at an applied rate of strain slow enough to produce approximate equalization of pore water pressure throughout the specimen at failure. The static stresses and pore water pressures, measured under undrained conditions, were used to express the measured strength parameters in terms of effective stress. Load, deformation, and pore water pressure measurements were recorded during loading using a data acquisition system.

The normalized undrained shear strength (S_u/σ'_{vc}) was estimated as the ratio of one-half the maximum observed deviator stress to the effective vertical stress prior to undrained loading. The in situ undrained shear strength was then calculated by multiplying the normalized undrained shear strength with the estimated in situ effective overburden pressure. A summary of the estimated in situ stresses, consolidation stresses, and undrained shear strengths for the CK_oUC triaxial compression tests is presented on Table 5.1. Additionally the following plots are provided for each test in the boring specific appendices:

- Axial strain versus effective vertical stress during consolidation
- Normalized shear stress (q/σ'_{vc}) versus normalized average effective stress, (p/σ'_{vc}) during undrained loading
- Normalized shear stress, normalized excess pore water pressure and obliquity versus axial strain during undrained loading

Drained Strength Tests. Multi-stage, consolidated-drained (CD) triaxial compression tests were performed to evaluate the frictional characteristics of the granular materials encountered in the boring. The tests were performed on samples recovered using either a 57-mm- (2-1/4-inch-) ID or a 74-mm- (2.8-inch-) ID, thin walled Shelby tube. The test specimen was trimmed down to a diameter of 51 to 71 mm (2.0 to 2.8 inches) and a height of 96 to 140 mm (3.8 to 5.5 inches) and placed in a rubber membrane with saturated filter paper strips around the perimeter of the specimen.

During the consolidation phase, the test specimen was allowed to consolidate to one of three test confining pressures. Young Bay Mud samples were tested at confining stresses ranging from the interpreted in situ vertical effective stress to approximately five to eight times the interpreted in situ vertical effective stress. Due to the relatively low confining stresses, it was considered relatively difficult to test these samples at stresses less than the in situ vertical stresses. Old Bay Mud/Upper Alameda Marine and Lower Alameda Alluvial samples were tested at stresses ranging from approximately half the interpreted in situ vertical effective stress to approximately four times the interpreted in situ vertical effective stress.

After the consolidation phase for the lowest confining pressure was completed and with the drainage line remaining open, the test specimen was sheared by increasing the axial load at a constant strain rate using a 5-ton Wykham-Farrance compression frame. The axial load was monitored by a load cell, deformation was measured by an LVDT, and volume change readings were taken with a calibrated burette. The load and deformation readings were continuously recorded by an IBM-compatible data logger.

The above procedures were repeated for two higher confining pressures. For the first two loading stages, the specimen was sheared until failure was indicated by a decrease in load, or 5 percent strain, whichever occurred first. After reconsolidation to a higher confining stress for the third stage, the sample was sheared to failure.

Consolidated drained triaxial test results, including friction angle, are summarized in Table 5.3. Additionally plots of stress versus strain, volumetric strain versus axial strain, and Mohr diagrams for the individual tests are presented in the boring-specific appendices (Volumes 2A and 2B).

3.13.3 Consolidation Tests

Incremental Consolidation Tests. Incremental consolidation tests were performed on selected high-quality cohesive samples to investigate the stress history of the soils at the boring location. The tests were performed in accordance with the recommendations of ASTM D2435. The consolidation test specimens were trimmed into 63.4-mm- (2.5-inch-) ID stainless steel rings. The trimmed test specimen and ring were then placed in a specially made cell where the base of the test specimen was sealed from the fluid (water) and the top of the test specimen was exposed to the fluid. A porous stone was placed on top of the test specimen and the loading ram was brought into contact with the porous stone. As the test specimen was compressed during loading, the pore fluid drained from the sample through the porous stone. The setup of the test specimen into the cell was performed with the entire cell under water so that there was no air trapped in the system that would affect the pore pressure response during loading.

After the cell was fully assembled, it was placed in a loading frame where the test specimen would be saturated before loading. Vertical loads were added in increments that usually doubled the previous load, yielding a load increment ratio of two. Each load increment was held for a period t_{100} (primary consolidation) determined by the logarithm of time method. The data readings were used to compute the vertical strain, vertical pressure, and coefficient of consolidation. Loading was continued until the effective stress applied was greater than the maximum past pressure applied to the test specimen, and the virgin compression portion of the consolidation curve was well defined. At that point, the test specimen was unloaded and then reloaded. The test specimen was rebounded again at the end of the test to produce a second rebound curve.

Controlled-Rate-of-Strain (CRS) Consolidation Test. CRS consolidation tests were performed in general accordance with ASTM D4186. CRS consolidation test specimens were trimmed into 63.5-mm- (2.5-inch-) ID stainless steel rings. The trimmed test specimen and ring were placed in a specially designed cell. The base of the test specimen was sealed from fluid (water) and the top of the specimen was exposed to the fluid. A porous stone was placed on top of the test specimen and the loading ram was brought into contact with the porous stone. As the test specimen was compressed during loading, the pore fluid drained from the top of the sample through the porous stone. A pressure transducer was connected to the bottom of the specimen, through the test cell, to measure the excess pore pressure during loading.

After the cell had been fully assembled, it was placed in a loading frame where the test specimen was saturated after a small setting load had been applied (to prevent swelling). The rate-of-strain was selected to produce a minimum excess pore pressure of 21 kPa (3 psi) and to limit the ratio of maximum excess pore pressure to applied vertical pressure to less than 15 percent (more commonly 10 percent). A limit of 10 to 15 percent was selected to obtain more reliable compressibility and rate of consolidation coefficients. The pore pressure ratio was continuously monitored during the first stage of loading and the rate of strain was automatically adjusted to keep the pore pressure ratio within the above limits. Loading was continued until the applied effective stress was greater than the estimated maximum past pressure applied to the test specimen and the virgin compression portion of the consolidation curve was well defined. The sample was loaded to between 20 and 30 percent strain, and then rebounded to produce a rebound curve.

A summary of the consolidation test results is presented in Table 5.2. Individual test results are presented in the boring-specific appendices (Volumes 2A and 2B) as curves of void ratio versus effective vertical pressure. Also plotted with these curves is the computed coefficient of consolidation at each effective vertical pressure.

Boring No.	Barge	Date Drilled		General Location	Coordinates, NAD83, CA Zone 3 (meters)		Approx. Locations re: 2-D Geophysical Survey
		Start	Complete		Easting	Northing	
98-1	West	10-Mar-98	12-Mar-98	North of E2	1,836,336	647,635	Line 99, Fix Mark 1644
98-2	West	15-Mar-98	20-Mar-98	North of E2	1,836,374	647,584	Line 18 X'g Line 107
98-3	West	1-Mar-98	5-Mar-98	Northeast of E2	1,836,347	647,682	Line 99 X'g Line 19
98-4	West	5-Mar-98	10-Mar-98	Northeast of E2	1,836,387	647,629	Line 107, Fix Mark 310.7, offset 15m north
98-5	West	12-Mar-98	15-Mar-98	Northeast of E2	1,836,418	347,608	Line 20, Fix mark 262.8
98-6	West	20-Mar-98	24-Mar-98	Northeast of E2	1,836,455	647,696	Line 21, Fix Mark 2380
98-7	East	11-Mar-98	16-Mar-98	Northwest of E3	1,836,618	647,712	Line 24, Fix Mark 199.5
98-8	East	4-Mar-98	11-Mar-98	North of E3	1,836,651	647,876	Line 26 X'g Line 98
98-9	East	23-Feb-98	4-Mar-98	North of E4	1,836,794	648,050	Line 30 X'g Line 97
98-10	East	16-Mar-98	22-Mar-98	North of E7	1,837,249	648,002	Line 38 X'g Line 99
98-11	East	22-Mar-98	27-Mar-98	North of E13	1,837,964	648,328	Line 54, Fix mark 2163.5
98-12	East	27-Mar-98	1-Apr-98	North of E17	1,838,314	648,251	Line 60 X'g Line 100
98-19	East	5-Apr-98	6-Apr-98	North of E3/E4	1,836,781	647,873	Line 28, Fix Mark 139
98-20	East	2-Apr-98	5-Apr-98	North of E11/E12	1,837,853	648,113	Line 50 X'g Line 100

Boring No.	Water Depth	Boring Depth	Elev. Bottom	Depth to Top of Rock	Elev. Top of Rock	Rock Thickness	Water Depth	Boring Depth	Elev. Bottom	Depth to Top of Rock	Top of Rock	Rock Length
	Meters (re: MSL)						Feet (re: MSL)					
98-1	-5.9	45.7	-51.7	10.4	-16.3	35.4	-19.5	150	-169.5	34	-53.5	116
98-2	-14.9	53.3	-68.3	1.8	-16.8	51.5	-49	175	-224	6	-55	169
98-3	-11.9	49.4	-61.3	18.9	-30.8	30.5	-39	162	-201	62	-101	100
98-4	-19.8	69.5	-89.3	8.2	-28.0	61.3	-65	228	-293	27	-92	201
98-5	-23.2	45.1	-68.3	14.3	-37.5	30.8	-76	148	-224	47	-123	101
98-6	-24.4	65.5	-89.9	35.1	-59.4	30.5	-80	215	-295	115	-195	100
98-7	-15.5	104.2	-119.8	79.9	-95.4	24.4	-51	342	-393	262	-313	80
98-8	-12.8	113.4	-126.2	84.1	-96.9	29.3	-42	372	-414	276	-318	96
98-9	-12.8	107.0	-119.8	90.2	-103.0	16.8	-42	351	-393	296	-338	55
98-10	-7.6	125.0	-132.6	106.7	-114.3	18.3	-25	410	-435	350	-375	60
98-11	-4.3	138.1	-142.3	120.7	-125.0	17.4	-14	453	-467	396	-410	57
98-12	-3.7	141.4	-145.1	124.4	-128.0	17.1	-12	464	-476	408	-420	56
98-19	-12.8	37.5	-50.3	---	---	---	-42	123	-165	---	---	---
98-20	-4.0	90.8	-94.8	---	---	---	-13	298	-311	---	---	---

NOTE: Borings 98-1 through 98-12 are numbered from West to East.

SUMMARY OF MARINE DRILLING PROGRAM

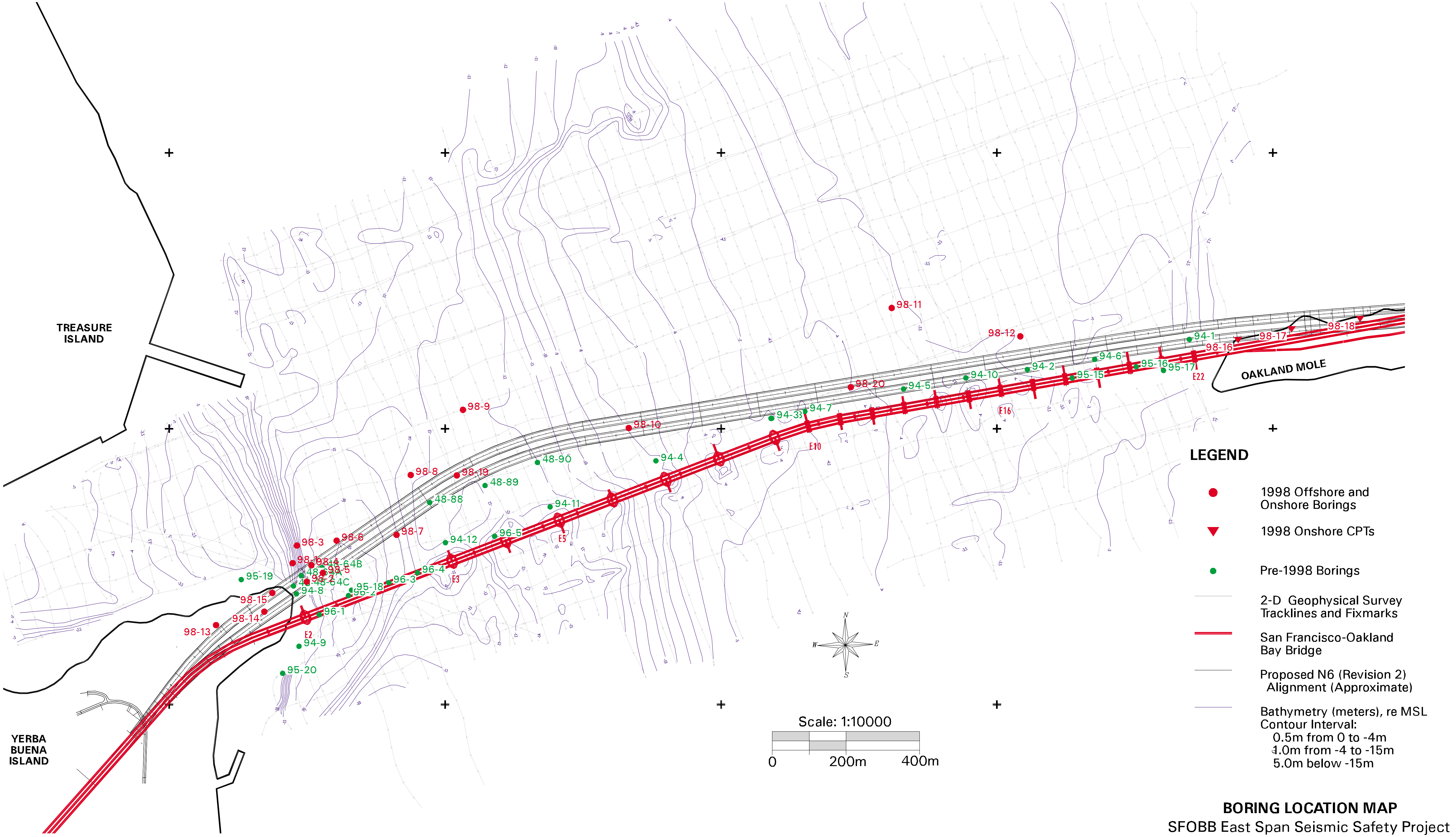
Task Order No. 3 Marine Exploration
SFOBB East Span Seismic Safety Project

TABLE 3.1

FUGRO

		Number of Tests														Totals
		98-1	98-2	98-3	98-4	98-5	98-6	98-7	98-8	98-9	98-10	98-11	98-12	98-19	98-20	
SOIL	**Index Tests**															
	Atterberg Limits	10	2	15	2	11	14	31	35	30	31	37	31	18	33	300
	Unit Weight	19	4	26	4	16	39	66	73	52	56	62	71	24	47	559
	Water Content	15	2	28	5	23	44	76	79	64	67	79	75	32	61	650
	Percent Passing #200	4	1	4	5	5	6	23	19	15	26	23	29	4	11	175
	Sieve Analysis	1	--	3	4	2	3	10	6	7	7	7	8	2	4	64
	Hydrometer	2	--	1	--	1	4	10	11	10	10	11	12	2	2	76
	Specific Gravity	1	--	2	--	--	4	8	6	5	7	6	9	--	3	51
	Organic Content	3	1	2	--	--	--	2	--	1	2	--	2	2	4	19
	Minimum-Maximum Density	--	--	--	--	--	--	--	--	--	--	--	2	--	--	2
	Consolidation															
	CRS Consolidation	--	--	1	--	--	2	2	4	1	3	3	3	1	2	22
	Incremental Consolidation	--	--	--	--	--	--	2	--	3	1	2	2	1	2	13
	Shear Strength Tests															
	Miniature Vane-Residual	5	1	4	--	1	1	7	9	7	3	3	8	9	16	74
	Miniature Vane-Remolded	--	--	--	--	--	--	5	4	8	3	5	3	4	10	42
	Miniature Vane-Undisturbed	14	4	17	--	9	21	46	51	28	25	27	24	22	33	321
	Torvane	1	--	10	--	10	19	38	19	25	19	21	21	14	30	248
	Pocket Penetrometer	2	--	10	3	10	14	46	28	16	28	19	25	3	18	222
	Swedish Fall Cone	--	--	--	--	--	--	15	12	14	7	10	--	19	28	105
	UU-Triaxial - Undisturbed	--	--	7	--	5	12	14	16	16	16	20	16	8	16	146
	UU-Triaxial - Remolded	--	--	5	--	2	5	9	15	16	10	15	17	2	11	107
	CK_oU-Triaxial	--	--	--	--	--	--	2	--	--	1	3	--	--	--	6
	CD-Triaxial	--	--	--	--	--	--	1	--	1	3	--	3	--	2	10
	Dynamic Tests															
	Resonant Column	--	--	--	--	--	--	1	1	2	--	--	1	--	--	5
	Cyclic Direct Simple Shear	--	--	--	--	--	--	2	2	1	--	1	1	--	--	7
	Other															
	Carbon Dating															
ROCK	Point Load Test	18	36	18	40	19	20	17	13	5	4	3	6	--	--	199
	Unconfined Compression	5	20	10	14	9	6	--	--	--	--	--	--	--	--	64
	Triaxial Compression	2	--	1	7	2	2	--	--	--	--	--	--	--	--	14
	Direct Shear	9	5	2	5	5	1	--	--	--	--	--	--	--	--	27
	Unit Weight	5	20	11	19	10	8	--	--	--	--	--	--	--	--	73
	Modulus/Poisson's Ratio	6	20	11	20	10	8	--	--	--	--	--	--	--	--	75

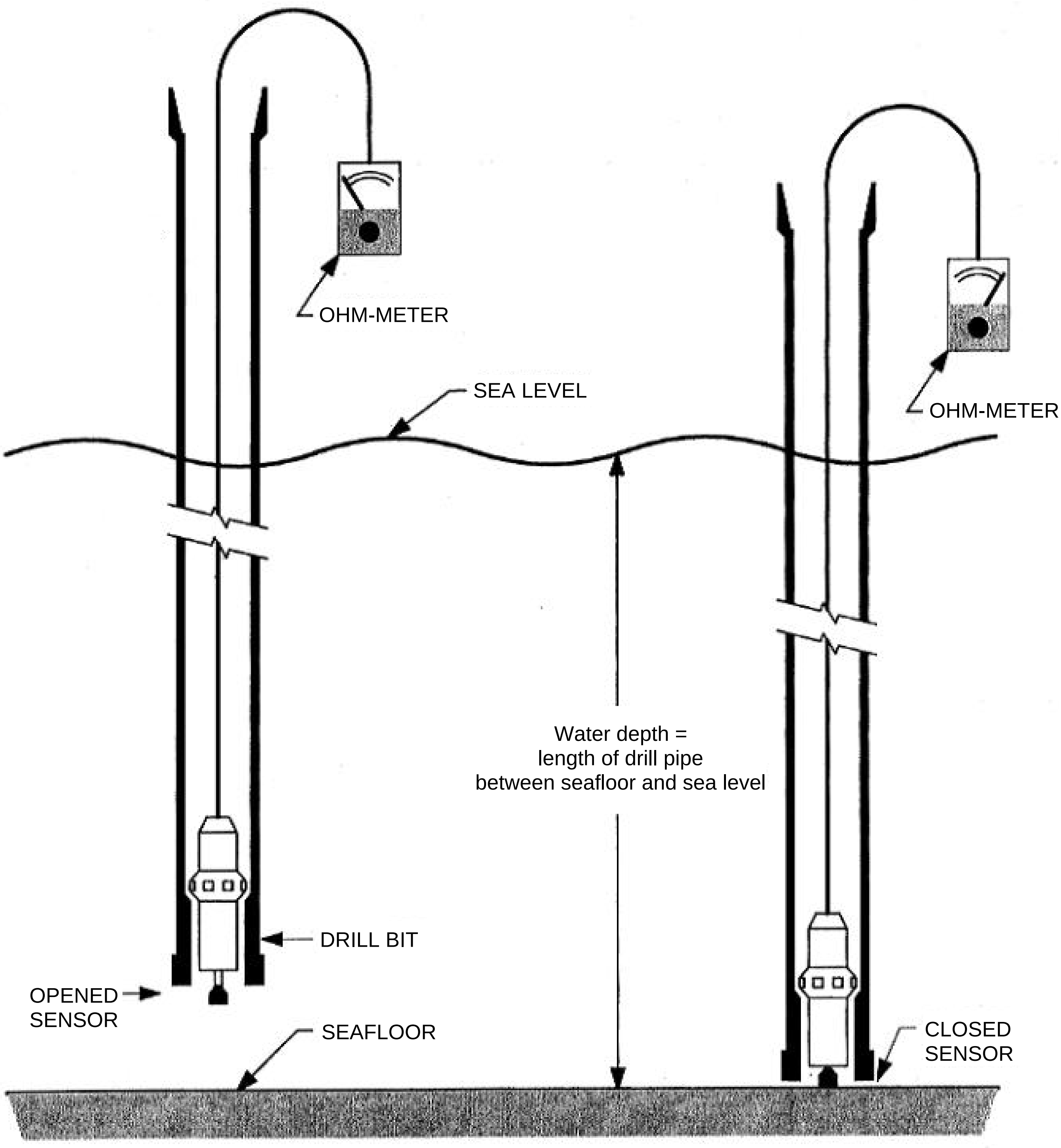
SUMMARY OF LABORATORY TESTING PROGRAM
Task Order No. 3 Marine Exploration
SFOBB East Span Seismic Safety Project



BORING LOCATION MAP

SFOBB East Span Seismic Safety Project

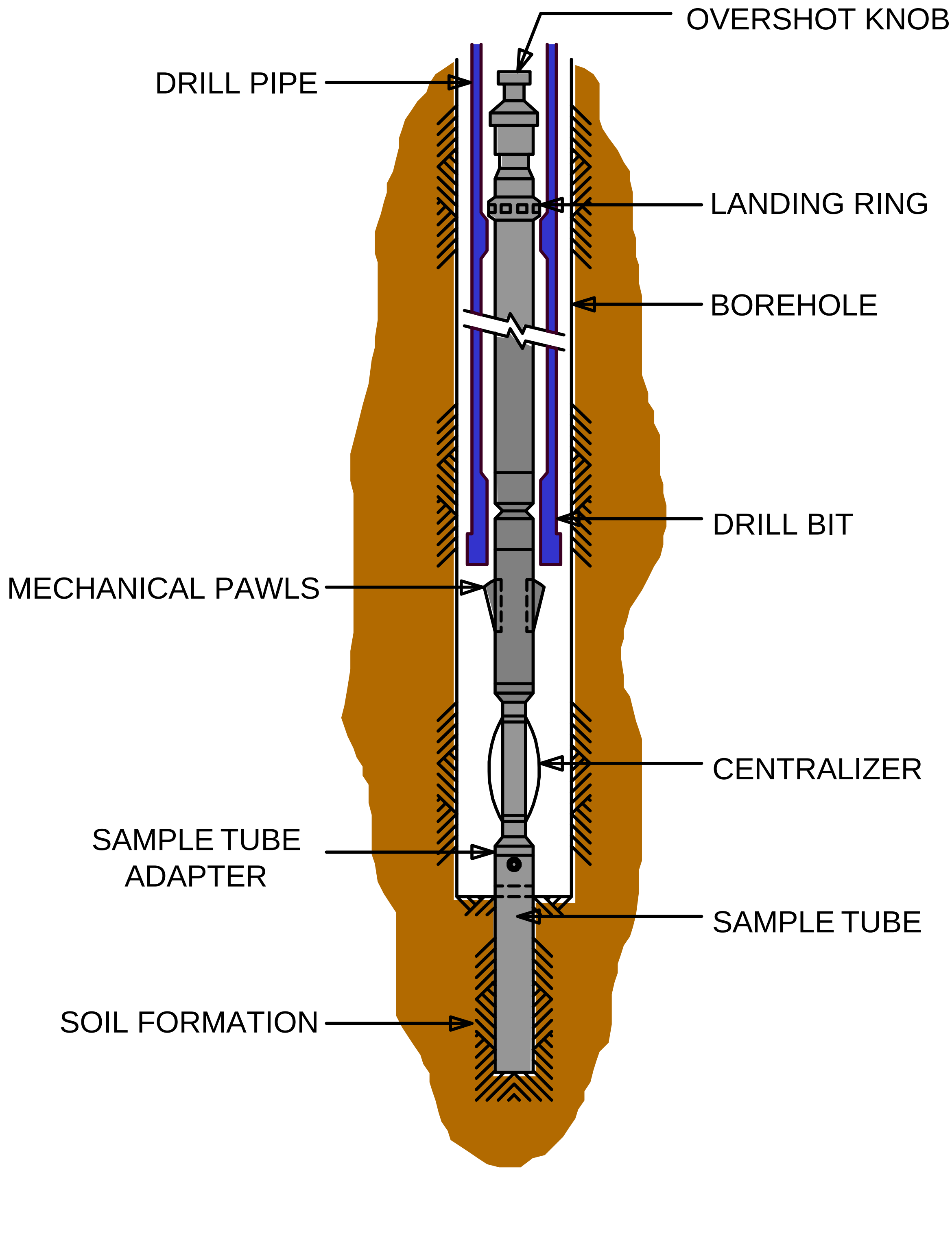
PLATE 3.1



WATER DEPTH MEASUREMENT USING BOTTOM SENSOR

SFOBB East Span Seismic Safety Project

SFOBB Task Order No. 5
Project No. 98-42-0054



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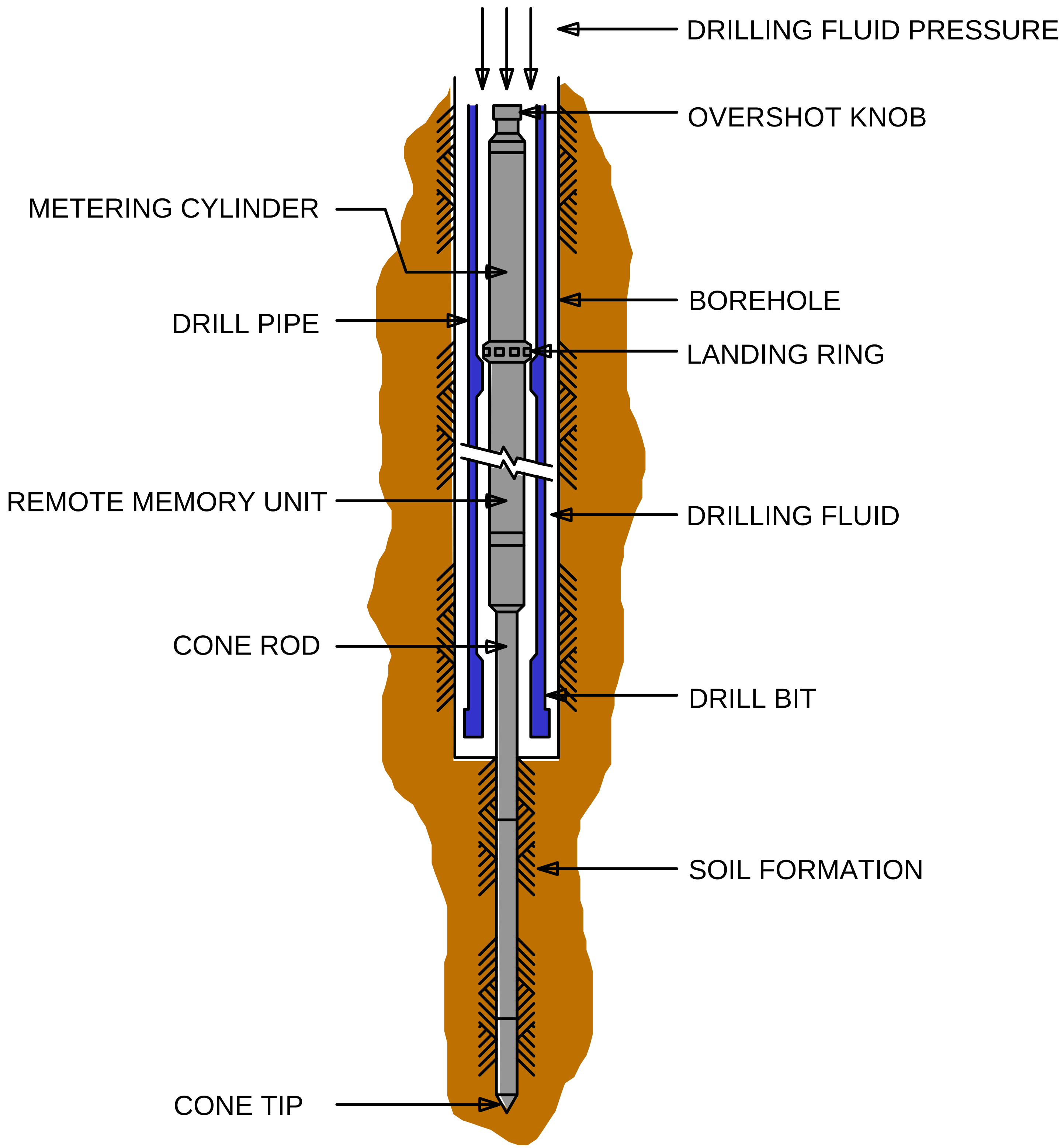
THE DOLPHIN PUSH SAMPLER
SFOBB East Span Seismic Safety Project

PLATE 3.3

SFOBB Task Order No. 5
Project No. 98-42-0054



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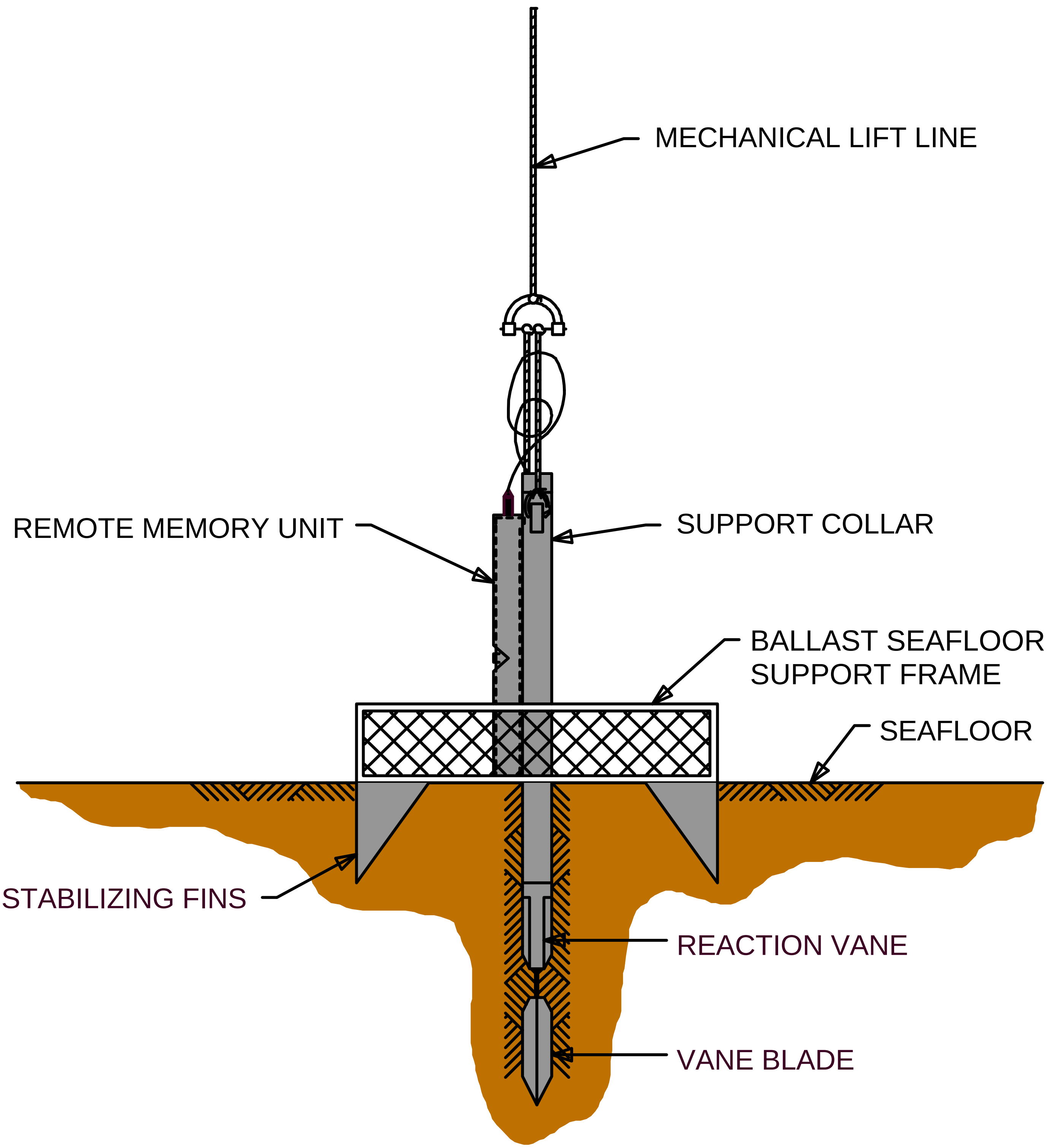
THE DOLPHIN CONE PENETROMETER
SFOBB East Span Seismic Safety Project

PLATE 3.4

SFOBB Task Order No. 5
Project No. 98-42-0054



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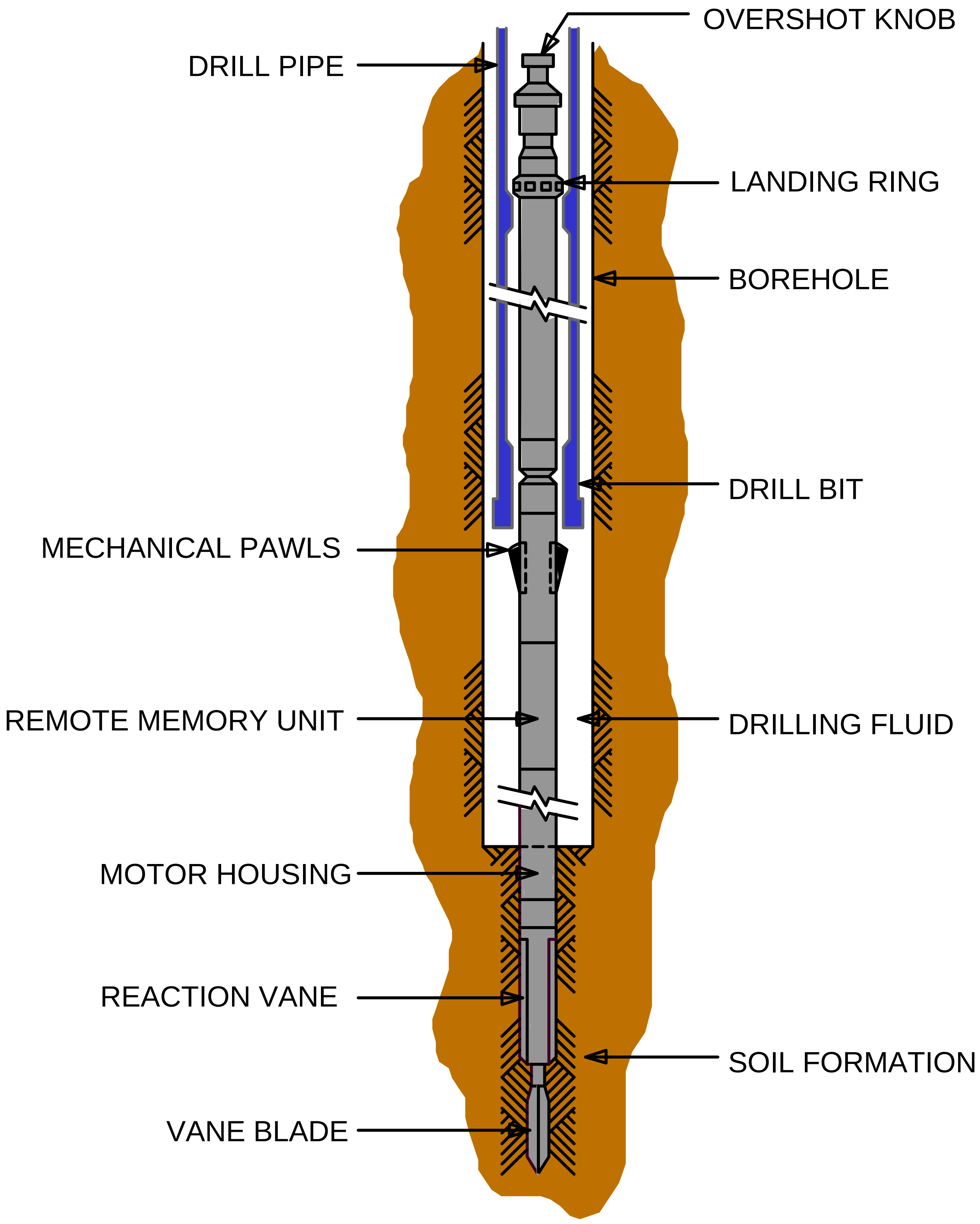
THE HALIBUT REMOTE VANE
SFOBB East Span Seismic Safety Project

PLATE 3.5



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THE DOLPHIN DOWNHOLE REMOTE VANE
SFOBB East Span Seismic Safety Project

Daytime operations: East- and West-Barge.

Nighttime operations: East- and West-Barge.

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FIELD OPERATIONS PHOTOGRAPHS,
MARINE SITE INVESTIGATIONS
SFOBB East Span Replacement Project

PLATE 3.7a

West-Barge layout showing drill barge, supply barge, tug-boat and crew boat.

Failing 2000 drill rig: lowering casing with slip joint fully extended.

FIELD OPERATIONS PHOTOGRAPHS, MARINE SITE INVESTIGATIONS

SFOBB East Span Replacement Project

Drill floor, East-Barge.

East-Barge mud circulation system.

FIELD OPERATIONS PHOTOGRAPHS, MARINE SITE INVESTIGATIONS

SFOBB East Span Replacement Project

PLATE 3.7c

Lowering Halibut remote vane equipment.

Assembling downhole remote vane.

FIELD OPERATIONS PHOTOGRAPHS,
MARINE SITE INVESTIGATIONS
SFOBB East Span Replacement Project

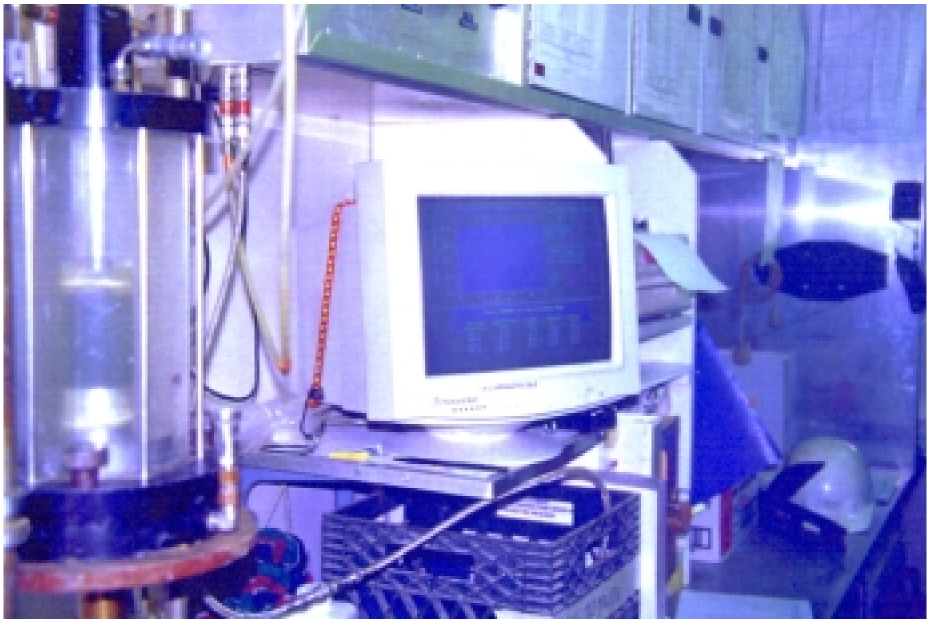
PLATE 3.7d

Retrieving thin-walled 3-inch tube sample.

Extruding soil sample from 2¼-inch tube sampler.

FIELD OPERATIONS PHOTOGRAPHS,
MARINE SITE INVESTIGATIONS
SFOBB East Span Replacement Project

PLATE 3.7e



Performing Undrained Unconsolidated Triaxial Compression test.

Performing Minivane test in 3-inch tube sampler.

FIELD OPERATIONS PHOTOGRAPHS,
MARINE SITE INVESTIGATIONS
SFOBB East Span Replacement Project