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ABSTRACT

The San Francisco-Oakland Bay Bridge is one of the most heavily traveled bridges in the world. The east span of the bridge will be replaced due to seismic safety concerns. The new bridge will be founded mostly on large 1.8 to 2.5-meter diameter, approximately 60- to 100-meter long pile foundations. American Petroleum Institute (API) method was used for pile design, since the selected foundation type is similar to those used for offshore oil platforms and the subsurface conditions are in many ways similar to those in the Gulf of Mexico. Piles foundations will experience tension loads of up to approximately 90 MN and compression loads of up to approximately 140 MN during the design earthquake. Prior to Production pile driving, a cofferdam will be constructed at each pier location, the pilecap footing box will be placed inside and the piles will be driven through the footing box with the use of a 500 to 1700 kilojoule Menck MHU-500 and MHU-1700 hydraulic impact hammer. Based on the Pile Installation Demonstration Project that was conducted in the fall of 2000, a soil-pile setup with time and acceptance criteria was established to accept the production piles.

INTRODUCTION

The San Francisco-Oakland Bay Bridge (SFOBB) carries 10 lanes of traffic across San Francisco Bay in California, USA. Yerba Buena Island (YBI) bisects the bridge, which is the primary link between the cities of San Francisco and Oakland, longitudinally, with the west span extending from San Francisco to YBI and the east span extending from YBI to Oakland, Figure 1. The existing east span bridge is a 3.5-km-long, double-decked structure that was constructed in the 1930s. The bridge is used by an average of over 280,000 vehicles every day.

NEW EAST SPAN BAY BRIDGE

The east span of the SFOBB, which is one of the most heavily traveled bridges in the world, will be replaced due to seismic safety concerns. The proposed east span bridge will be constructed along a parallel alignment to the north of the existing bridge. The new bridge will consist of an approximately 460-meter-long transition structure extending from the YBI Tunnel to the eastern tip of YBI, an approximately 625-meter-long, single-tower that rises 164-meter above the water and asymmetrical, self-anchored suspension cable, main-span signature structure extending offshore from the tip of YBI, an approximately 2.1-kilometer-long, four-frame Skyway structure extending from the signature structure eastward to the Oakland Shore Approach, an Oakland Shore Approach structure extending about 700 meters from the Skyway structure to the north side of the Oakland Mole and an earthen fill transition from the Oakland Shore Approach structure to the roadways leading to and from the existing bridge.

SITE SPECIFIC GEOLOGY

The geologic formations that underlie the skyway area include in the following sequence: Young Bay Mud (YBM), Merritt-Posey-San Antonio (MPSA) formations, Old Bay Mud (OBM), Upper Alameda Marine (UAM) sediment and the Lower Alameda Alluvial (LAA) sediment. Fig. 2 provides an illustration of the subsurface conditions encountered along the new bridge alignment.

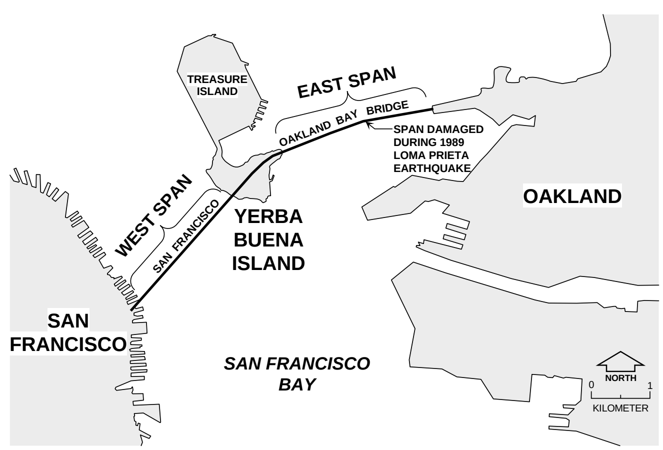

Fig.1. Site Vicinity Map

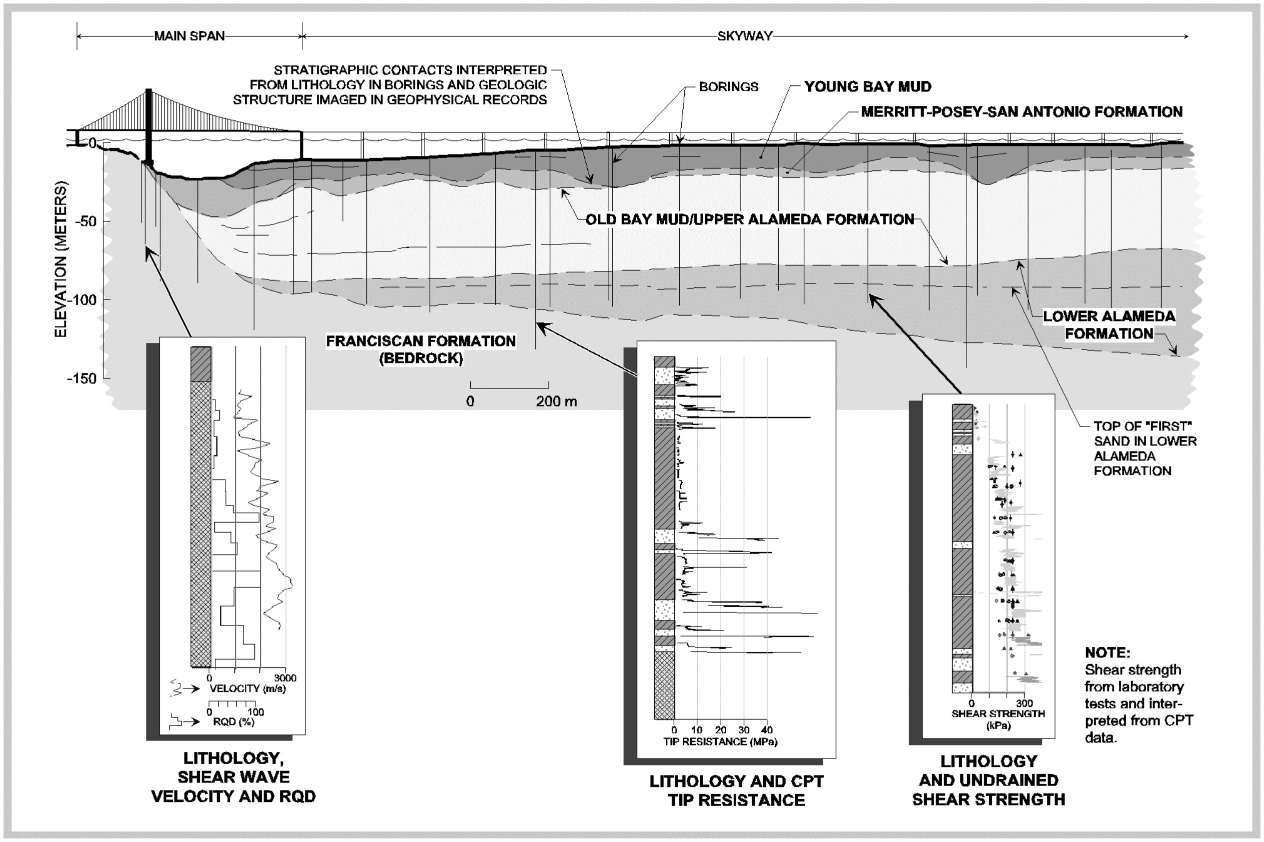

Figure 2. Interpreted Stratigraphic Section Along New Bridge Route

Table 1. Engineering Soil Properties

Geological Formation	Ø' Degree	Su kPa	γ' kN/m^3
YBM: very soft to firm Clay	-	10 - 65	4 - 7
MPSA: Dense to Very Dense Sand with Stiff to Very Stiff Clay Layers	35 - 42	60 -175	6 - 11
OBM/UAM Sediments: Very Stiff to Hard Clay	-	100 - 250	6 - 9
LAA Sediments: Dense to Very Dense Sand and Hard Clay	40 +	225 - 400	9 - 12

YBM, the youngest geologic unit in the bay, is marine clay that has been deposited since the end of the last sea level low stand, which was about 11,000 years ago (Atwater et al., 1977). YBM sediments are generally very soft to firm, normally to slightly over-consolidated, high plasticity clay. The YBM includes sand layers and or seams within several depth intervals that may correlate to different depositional periods. The undrained shear strength of YBM generally increases with depths from 2 to 4 kilopascals (kPa) at the surface to 20 to 65 kPa at the base of the sequence. At the primary and alternate test location, the YBM is approximately 4 to 6 and 20 meters thick, respectively.

MPSA formations, which is beneath the YBM, a layered sequence of dense to very dense sand with layers of stiff to very stiff sandy clay and clay is present over portions of eastern Bay. The sequence is generally considered to be composed of late Pleistocene, non-marine sediments deposited during the late Wisconsin glacial stage (90,000 to 11,000 years ago). The Merritt formation reportedly is primarily coarse-grained, aeolian sediments that were blown in from the west during sea level low stands (Rogers and Figuers, 1991).

The Posey member, typically considered the basal member of the San Antonio Formation, is reportedly of primarily alluvial origin and was likely deposited within channels that were active during sea level low stands. Although primarily non-marine deposits, the late Wisconsin glacial stage also included periods of sea level fluctuations that may have produced estuarine environments. The fine-grained layers within the sequence are likely associated with those periods. At the primary and alternate test location, the MPSA is approximately 7 to 8 and less than 5 meters thick, respectively.

OBM sediments, which underlies the San Antonio Formation and overlies the Alameda Formation is considered to be an 80,000 to 130,000 year old marine deposit (Sloan, 1982). The surface of the OBM is extensively channeled. The OBM typically is a very stiff to hard, over-consolidated, plastic clay that includes several, often discontinuous, crusts. The undrained shear strength of OBM increases with depth and typically range from 90 to 175 kPa at the top of the sequence to 150 to 250 kPa at the base of the sequence. The sequence also includes numerous crust layers with shear strengths 25 to 50 kPa higher than adjacent layers. Those crust layers are interpreted to be old soil horizons that were exposed to air during sea level changes. The OBM is typically about 20 to 25 meters thick.

Alameda Formation generally lies directly above the Franciscan Formation (FF) bedrock in the marine portion of the project area. The Alameda Formation is considered to be of late Pleistocene age (Sloan, 1982) and has been informally divided into an Upper Alameda Marine (UAM), primarily fine-grained marine member, and Lower Alameda Alluvial (LAA), primarily alluvial member (Rogers and Figuers, 1992a and 1992b).

UAM is composed primarily of very stiff to hard, over-consolidated, plastic clay with occasional silt, clayey silt and sand layers. OBM/UAM sediments, which consist primarily of very stiff to hard fat clay. Except for the increased occurrence of coarser-grained interbeds, the UAM clays are similar to the marine clays overlying OBM. The combined thickness of these marine clays is typically about 60 meters. At the primary test location the OBM/UAM is 60 to 65 m thick with an intermediate 6 m thick very dense sand layer known as the Upper Alameda Marine Paleochannel (UAMPC) sand. At the Alternate test location the OBM/UAM is 50 to 55 meters thick.

Lower Alameda Alluvial (LAA) sediments includes a 3 to 10 meters thick cap layer of very stiff to hard lean clay underlain by a sequence of primarily very dense granular alluvial sediments. The PIDP specified pile tip elevations were anticipated to be approximately 10 meters into the LAA-sand.

PROPOSED FOUNDATIONS

The proposed bridge foundation types are largely based on the geotechnical/geological conditions at the site. Along the new bridge alignment, the bedrock surface is relatively shallower near the shoreline of YBI and relatively steeper slope to the east to approximately down to elevation of –100 meters at about 350 meters offshore from the YBI. Farther to the east, the slope of the bedrock decreases. At the Oakland end, the bedrock surfaces buried by more than 135 meters of sediments.

Self-Anchored-Suspension (SAS) portion of the bridge is supported by three pier supports. The west piers-W2 and main tower foundation supports are situated in the area of shallow, steeply sloping bedrock on and adjacent to YBI. West Pier W2 of self-anchored suspension bridge consists two supports, one for eastbound and the other for the westbound. East and westbound W2 foundations (square 19 m x 19 m with a thickness of 10 m mass gravity foundation) will be constructed in rock after excavation of the rock. The westbound footing is supported on four 2.5 m diameter, 10 m long CIDH piles socketed in fresh rock while the eastbound footing is placed directly on rock. The reason for supporting the westbound footing with piles is attributed to difference in rock characteristics from the eastbound footing. Also, the westbound footing overlies a relatively steep slope.

In contrast, the SAS-East Piers and the Skyway and Oakland Shore approach portion of the bridge piers are underlain by over 85 to 135 meters of marine and alluvial sediments. Skyway eastbound and westbound Piers 3 through 16 (total of 28 piers and 2.5-meter diameter, up to 100 meter long, 160 piles) and SAS-East Piers (Pier-2E and Pier-2W, total of 2 piers and 2.5-meter diameter, up to 100 meter long, up to 60 meter long, 12 piles), each pile will experience tension loads of up to approximately 80 to 90 MN and compression loads of up to approximately 120 to 140 MN during the design earthquake. Each peir (Piers 2 through 16) will be supported by an octagonal or retangular pilecap with 4 to 6 piles, at 1:8 batters. Oakland Shore approach eastbound and westbound Piers 17 through 22 (total of 12 piers and 1.8 meter diameter, 104 piles), each pile will experience tension loads of up to approximately 9 to 30 MN and compression loads of up to approximately 18 to 42 MN during the design earthquake. Each peir (Piers 17 through 22) will be supported by an retangular or square pilecap with 8 to 9 piles.

Prior to pile driving, a cofferdam will be constructed at each pier location (except Piers 2 through 6), the pilecap footing box will be placed inside and the piles will be driven through the footing box with the use of a 500 kilo-joule Menck MHU500 or 1700 kilo-joule Menck MHU-1700 hydraulic impact hammer.

AXIAL PILE CAPACITY: SKIN FRICTION

The API design procedure (1993a,b) for calculating axial pile capacity in clay soils was adapted from an Offshore Technology Conference paper by Randolph and Murphy (1985) in which the unit skin friction transfer is calculated using the relationship:

$$f = \alpha S_u$$

$$\alpha = \left(\frac{c}{p'}\right)^{1/2}\left(\frac{S_u}{\sigma'}\right)^{-1/2}$$

Where: α-adhesion factor, S_u-undrained shear strength at any depth, σ'-effective overburden stress at any depth, c-undrained shear strength and p'-vertical effective overburden pressure

The API adopted a lower-bound value of 0.25 for the c/p' ratio for Gulf of Mexico clays, resulting in a default value of 0.5 for the $(c/p')^{1/2}$ term Since, the large diameter overwater pile foundations derive much of the skin friction capacity from the clay layers of the Old Bay Mud and the Upper Alameda-Marine formations detailed consideration was given to the applicability of this assumption to the conditions at the bay bridge site

The results of static pile load tests performed on piles supported in Young Bay Mud were evaluated. In those tests the ultimate side shear transfer in the pile load tests were estimated to be equal to the undrained shear strength. Hindcast analysis to match the observed static load-settlement behavior of the pile resulted in an estimate of the undrained strength ratio (c/p') of 0.31. A number of, K_o Consolidated-Undrained Triaxial Tests and Direct

Simple Shear tests were performed on samples as a part of the marine site characterization for the project. The results of those tests also suggested a c/p' value on the order of 0.31.

Based on available site-specific data, the design methods presented in API (1993a,b) were modified by increasing the value of the implicit c/p' ratio in API from 0.25 to 0.31. The ultimate unit shear transfer in the clay strata was then calculated as:

$$\alpha = (0.31)^{1/2}\left(\frac{S_u}{\sigma'}\right)^{-1/2} \quad \text{for} \quad \frac{S_u}{\sigma'} \leq 1.0$$

$$\alpha = (0.31)^{1/2}\left(\frac{S_u}{\sigma'}\right)^{-1/4} \quad \text{for} \quad \frac{S_u}{\sigma'} > 1.0$$

These formulations result in axial skin friction capacities in clay that are approximately 11 percent greater than those given by the standard API formulation.

END BEARING

In order to develop estimates of end-bearing capacity, a statistical evaluation was performed to evaluate the probable presence of clay layers at the pile tip. To account for the potential presence of clay layers, values for the end-bearing capacity were developed using weighted averages based on the relative amounts and thicknesses of the interbedded clay layers expected to be encountered. The value of the unit end-bearing pressure (q_u) for each frame was calculated as:

$$q_u = \frac{3.24\,X + 11.97\,Y}{100}$$

Where: X-percentage of clay layers and Y-percentage of sand layers

PILE INSTALLATION DEMONSTRATION PROJECT

The pile installation demonstration project (PIDP) was conducted as a part of the SFOBB East Span Seismic Safety Project. Three full-scale large diameter steel pipe piles, 2.438-meter diameter, up to 100-meter long, wall thickness 40-70 millimeter, one vertical and two on a 1:6 batter, were driven near the planned bridge alignment in San Francisco Bay. The PIDP piles are the deepest large diameter piles ever driven in the world. The expected ultimate axial compression capacity was estimated based on American Petroleum Institute method (1986, 1993) to be on the order of 120 MN. Two hydraulic hammers MHU500 (550 kJ) and MHU1700 (1870 kJ) were selected for the installation. These hammers may not have sufficient energy to drive these piles at their ultimate capacity following long-term set-up. However, the repeatedly sheared and remolded clayey soils near the pile tip and wall, the soil plug behavior and the excess pore pressure generation in the Young & Old Bay Mud and Upper Alameda formations during driving were expected to reduce the shaft and end bearing resistance, and therefore reduce the energy required to drive these piles to design penetration.

COMBINED CASE PILE WAVE ANALYSIS (CAPWAP)

Satisfactory CAPWAP evaluations and estimates of the ultimate pile capacity require that the hammer energy be sufficient to move the pile enough to fully mobilize the available soil resistance. If the energy is not sufficient to mobilize the full soil resistance then the CAPWAP analyses may significantly underestimate the available static pile capacity. However, based on the data collected during continuous installation driving and after a series of restrikes at different set-up times, the combined CAPWAP analysis (Stevens 2000) can be used to better estimate the available static pile capacity even when the hammer is not able to mobilize the entire soil resistance during a series of restrike(s).

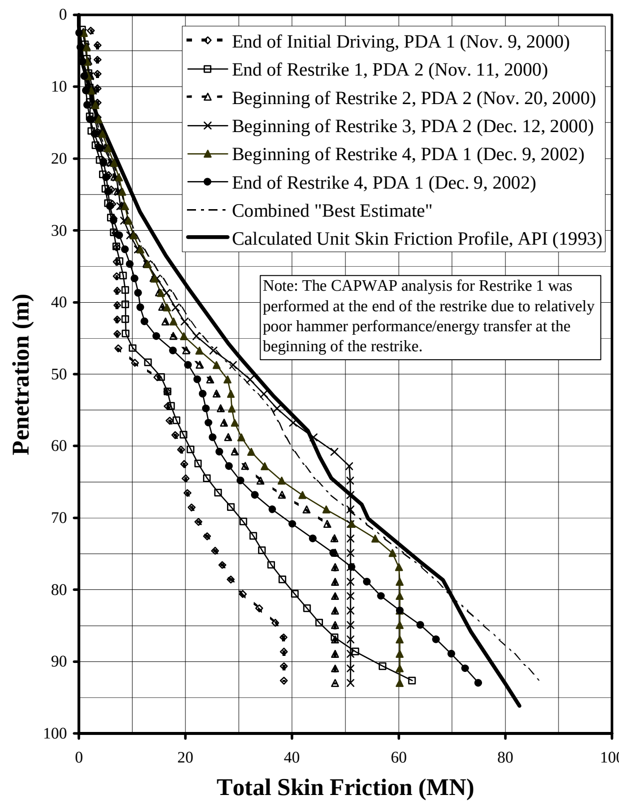

Fig. 3. Total Skin Resistance - Combined CAPWAP Analysis

A combined “best estimate” skin friction distribution was obtained by summing the largest mobilized skin friction increment values along the length of the pile during initial driving or subsequent re-strike(s). It is important to recognize that the largest mobilized skin resistance increment may come from the CAPWAP analysis performed at the end of re-strike. This is due to the fact that subsequent hammer blows will breakdown the setup in the upper portion of the pile and mobilize skin resistance in the lower portion of the pile that was not mobilized at the beginning of the restrike.

A series of restrikes were conducted on the PIDP piles to better understand the magnitude and distribution of soil resistance along the pile. Four restrikes were conducted on Pile-1, three re-strikes were conducted on Pile-2, and two re-strike was conducted on Pile-3. Based on CAPWAP analyses, the trend of increase in total skin resistance with time is illustrated on Fig. 3. For comparison, total skin resistance profiles computed based on API design method (1993a, b) is shown on Fig. 3.

PILE CAPACITY/OBSERVED SETUP

Fig. 4 presents the total skin resistance at each re-strike versus time. It appears that after 33-days of set up, Pile-1 has the static skin resistance capacity of 70 MN, which is approximately 88 percent of design skin resistance capacity. For Pile 2, the total skin resistance capacity was approximately 67 MN after 22 days of set-up. It appears that after 23 to 24 months of set up, all three piles have the static skin resistance capacity of 78 to 88 MN, which are approximately 98 to over 100 percent of design skin resistance capacity. However, due to the presence of predominantly clayey soils at the site, it is anticipated that soil pile setup will continue for several months. Also, it is anticipated that the skin friction capacity will exceed the predicted capacity of 80 MN by the API design method.

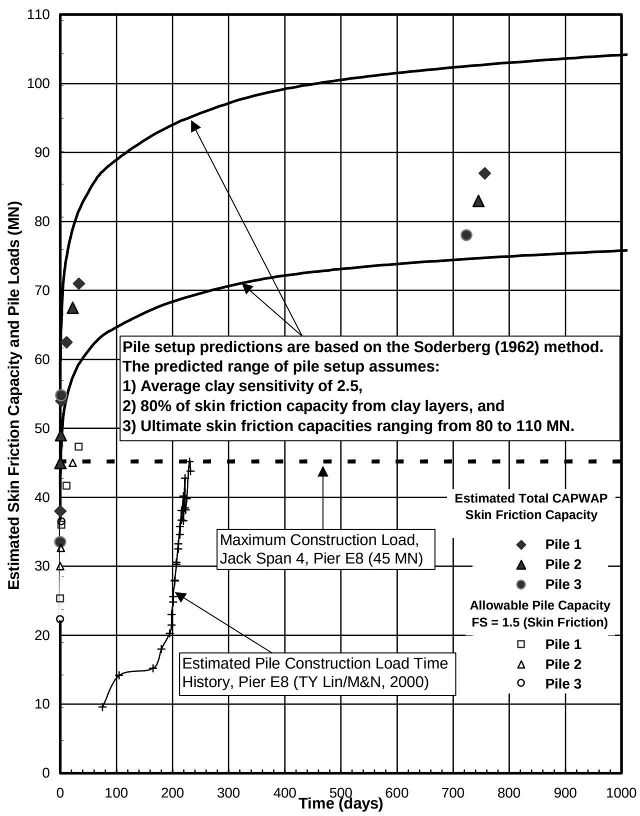

Fig. 4. Estimates of PILE setup From PIDP

PREDICTED SETUP IN CLAY

The predicted set-up in clayey soils was based on a method developed by Soderberg (1962). This method was based on site-specific radial consolidation coefficient (c_h). The coefficient of radial consolidation was defined by:

$$T = (c_h \times t)/r_p^2$$

In Soderberg's hypothesis, the time factor (T) was set equal to unity at time t equal to t_{50}. Therefore, the equation may be rearranged as

$$c_h = T\left(r_p^2 / t\right) = r_p^2 / t_{50}$$

The linear relationships describing the range of predicted increase of axial geotechnical capacity with time are identical to that recommended in Bogard and Matlock (1990c).

Lower Bound:

$$Q_t = Q_u(0.20 + 0.80U)$$

Upper Bound:

$$Q_t = Q_u(0.30 + 0.67U)$$

Where percent consolidation (U) given by:

$$U = (t/t_{50})/(1 + t/t_{50})$$

The predicted ranges of set-up for the method is based on an assumed range of set-up factors of 3 to 5. The set-up factor is the ratio of final ultimate capacity to the capacity at the end of initial driving. The horizontal coefficient of consolidation (c_h) was chosen for the Soderberg method based on available consolidation test data in the Young and Old Bay Mud and a series of multiple orientation consolidation tests that were performed as a part of site characterization. The vertical coefficient of consolidation (c_v) was selected from the lower one-third of the available test data range as presented in Fig. 5. Which was approximately 8 square meters per year (m^2/year). Also, based on the multi-oriented consolidation test data in the over consolidated range of stresses; the c_v value was multiplied by the ratio of c_h to c_v (approximately 1.5) to obtain a c_h value of approximately 12 m^2/year.

Fig. 4 presents the predicted set-up that based on Soderberg's method and the calculated setup based on CAPWAP analysis for the three piles. The combined CAPWAP analyses may be conservative since the skin friction was not mobilized in the lower portion of the pile during re-strikes, and the skin resistance mobilized on a particular pile segment is assumed to be larger of the actual resistance mobilized during continuous driving (Stevens 2000). However, the skin resistance is likely to continue to increase with time and likely to exceed the calculated (API 1993a, b) ultimate skin friction of pile capacity.

SOIL RESISTANCE TO DRIVING (SRD)

Stevens et al. (1982) recommended that lower and upper bound values of SRD be computed for the coring pile condition especially for larger diameter pipe piles. The data presented in Fig. 6 confirms the assumption of large diameter piles coring through the soil during continuous driving. When a pile cores, relative movement between pile and soil occurs both on outside and inside of pile wall. The lower bound was computed assuming that the skin friction developed on the inside of the pile is negligible. An upper bound is computed assuming the internal skin friction is equal to 50 percent of the external skin friction. For a plugged pile, a lower bound is computed using unadjusted values of unit friction and unit end bearing. An upper bound plugged case for granular soils is computed by increasing the unit skin friction by 30 percent and the unit end bearing by 50 percent. For cohesive soils the unit skin friction is not increased and the unit end bearing is computed using a bearing capacity factor of 15, which is an increase of 67 percent. For sandy soils, the unit skin friction and unit end bearing values that were used to predict the SRD were the same as those used to compute static pile capacity. For clayey soils, the SRD was computed using two methods.

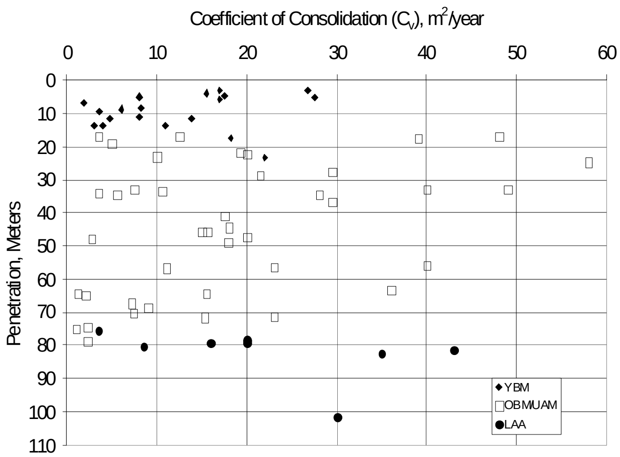

Fig. 5 Coefficient of Consolidation Profile

Method–I; The unit skin friction was computed from the stress history approach presented by Semple and Gemeinhardt (1981). The unit skin friction and unit end bearing for static loading is first computed by using the method recommended by the American Petroleum Institute (1986). The SRD is then calculated by incrementally reducing the unit skin friction values by multiplying by a pile capacity factor, determined empirically.

$$F_p = 0.5 \times (OCR)^{0.3}$$

The over-consolidation ratio (OCR) was calculated with the following equations.

$$Su/Uunc = (OCR)^{0.85}$$

$$Uunc = \sigma'_{vo} \times (0.11 - 0.0037 \times PI)$$

Where: U_{unc}-undrained shear strength-normally consolidated clay, σ'_{vo}-effective overburden pressure and PI-plasticity index OCR can also be estimated from CPT tip resistances with the use of following equation.

$$OCR = \sigma'_p / \sigma'_{vo}$$

$$\sigma'_p = 0.33 \times (q_c - \sigma'_{vo})$$

Where: q_c = cone tip resistance and σ'_p= preconsolidation stress

Method-II; Based on “Sensitivity Method”; the unit skin friction for static loading is first computed by using the method recommended by American Petroleum Institute (1983, 1993). The SDR is then calculated by incrementally reducing the unit skin friction values by measured clay sensitivities, which is the ratio of the undisturbed to remolded clay shear strengths. The computed lower and upper bound SDR were used to perform the wave equation analyses to predict the lower and upper bound acceptance criteria.

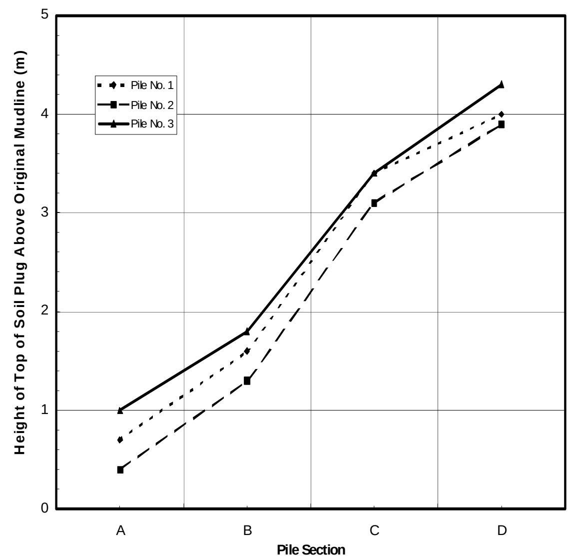

Fig. 6. Summary of Soil Plug Measurements

WAVE EQUATION ANALYSES

Wave equation analyses were performed to predict the blow counts with the penetration depth using GRLWEAP (1997-2) program for the continuous driving case. The soil quake and damping parameters recommended by Roussel (1979) were used. The shaft and toe quakes were assumed to be 0.25 centimeters for all soil types. A shaft damping value of 0.19 to 0.36 seconds per meter was assumed for clayey soil. The shaft damping value in clayey soil decreases with increasing shear strength (Coyle and Gibson 1970). The toe damping value of 0.49 seconds per meter was assumed for all soil types.

OBSERVED VS PREDICTED SRD

Fig. 7 presents results of predicted and PDA observed soil resistance to driving (SRD) profiles for Pile 2. Interestingly, the predicted and observed blow count profiles, as expected, correlate well with the predicted and PDA observed SRD. PDA observed SRD was computed based on maximum Case Method and damping coefficient (J) of 0.5. The observed blow counts and SRD spikes at penetration depths of about 45 and 70m were as a result of soil pile set-up that occurred during driving delays such as splicing and welding of pile sections. Therefore, it demonstrates very well that wave equation analyses can be used to reasonably predict the blow counts with the penetration depth provided that similar hammer energies are applied in the model.

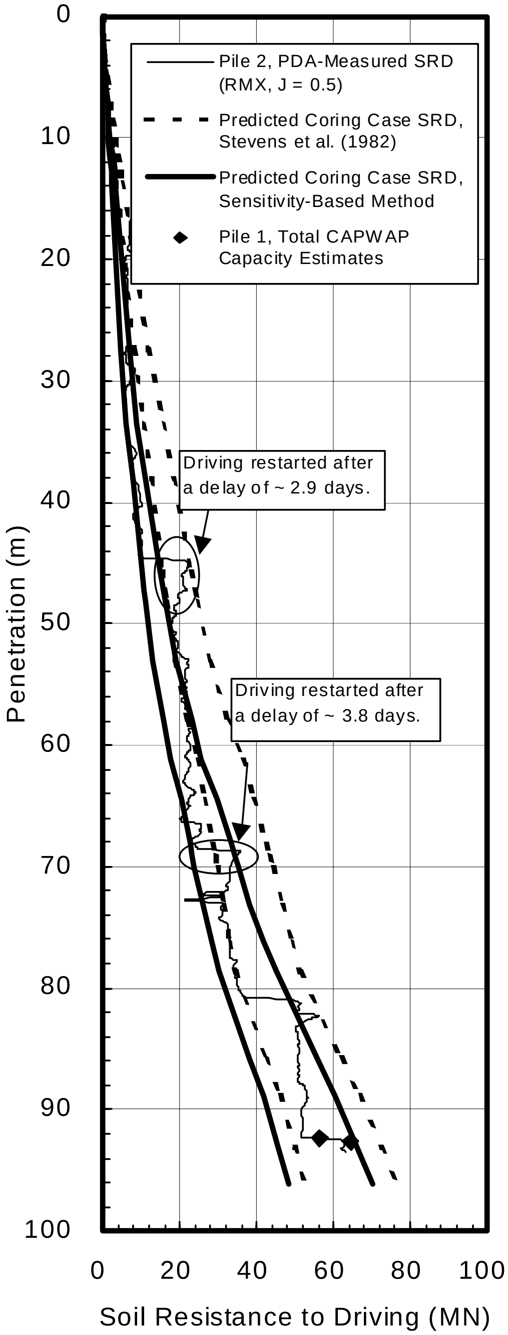

Fig. 7. PDA Measured and Predicted SRD Pile-2

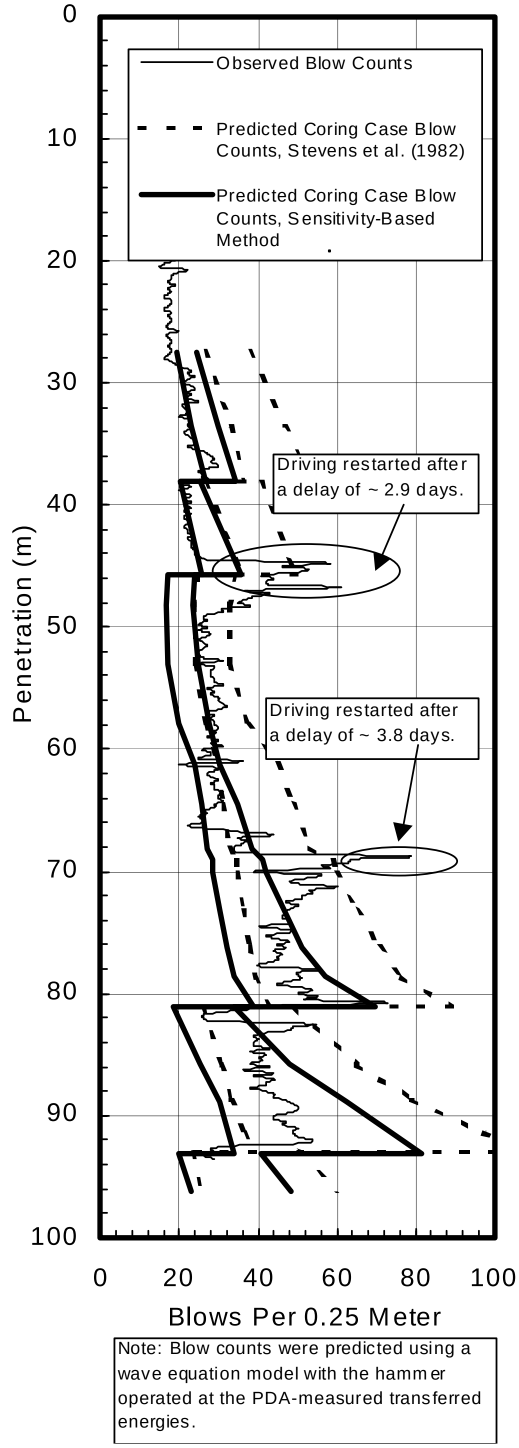

Fig. 8. Observed and Predicted Blow Counts Pile-2

OBSERVED VS PREDICTED BLOW COUNTS

Fig. 8 presents the results predicted by Case-Goble formulation and wave equation analyses performed based on the PDA measured driving system performance data and observed field blow counts for Pile 2. During continuous driving observed blow counts below a penetration of 35 to 40 meters, tend to follow the lower bound of predicted blow counts based on Method-I and are generally bound by upper and lower bounds based on the "Sensitivity Method" (Method-II). The sensitivity method seems to better predict the observed blow counts in the soft Young Bay Mud sediments even in the upper 35 to 40 meters for Pile 2. The observed blow counts spikes at certain depths was as a result of soil pile set-up that occurred during driving delays such as splicing/welding of pile sections.

The results also indicate that after 3 to 5m of driving, the setup was broken down and observed blow counts seems to converge with the predicted blow counts. Fig. 8 also demonstrates that if piles are driven to the required design tip elevations, piles can be accepted based on the coring case lower bound acceptance criteria for the given range of hammer energy or efficiencies by either method for clayey soils.

ACCEPTANCE CRITERIA

Piles are driven to the required design tip elevations and if it meets the coring case lower bound acceptance criteria for the given range of hammer energy (hammer efficiency and field blow counts) then the piles will be accepted by either method. However, if the piles do not meet the lower bound acceptance criteria for the PDA measured range of hammer energy then additional restrikes and CAPWAP analyses will be performed to evaluate the capacities prior to accepting the piles.

If piles are refusing (generally 5 to 10 meters, depending on the pier locations) above the required design tip elevations, and meets the lateral capacity requirements then the pile can be accepted if it met the following conditions.

- The specified primary hammer is operating at full rated energy according to the manufacture's specifications.
- Pile driving resistance exceeds either 250 blows per 250 mm over a penetration of 1500 mm or 670 blows for 250 mm of penetration.
- If pile-driving operation is interrupted for more than one hour, the above definition of refusal shall not apply until the pile has been driven at least 250 mm following the resumption of pile driving.
- At any time, 670 blows in 125 mm shall be taken as pile driving refusal.

CONCLUSIONS

Load tests in San Francisco Bay soils have shown that the unit side shear resistance on a pile can equal the undrained shear strength of the supporting soil, whereas the average skin friction values used in the static estimates generated using the modified API (1993) methodology can be on the order of 70 percent of the undrained shear strength of the surrounding soils. These load tests therefore indicate that the available side shear resistance may be as much as 40 percent higher than that, which would be used for design. The data from the PIDP also suggests that skin friction capacity may exceed those predicted using the API procedures.

The combined CAPWAP analysis can be used to estimate the capacity of driven piles with time and to proof-test the piles even if the hammer does not have sufficient energy to drive or mobilize the pile at their ultimate capacity. The combined CAPWAP analysis can also be used to establish the soil-pile setup with time for the clayey soils. It will be valuable data during staged construction in order to establish waiting periods prior to loading the piles.

Wave equation analyses can be used to predict the blow counts with the penetration depth using GRLWEAP program for the continuous driving case. Provided that piles are driven to the required design tip elevations, piles can be accepted based on the coring case, lower bound acceptance criteria for the given range of hammer energy or efficiencies by either method for clayey soils.

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