Geotechnical Design of Large Diameter Impact Driven Pipe Pile Foundations: New East Span San Francisco-Oakland Bay Bridge

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Abstract: The San Francisco-Oakland Bay Bridge is one of the most heavily travelled 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-metre diameter, approximately 60- to 100-metre long pile foundations. Pile foundations will experience tension loads of up to approximately 90 MN and compression loads of approximately up to 140 MN during the design earthquake. This paper discusses the geotechnical design approach and techniques followed to evaluate the soil-pile-setup with time and the axial capacity of large diameter impact driven pipe piles for the project.

Keywords: Impact hammer, pile driving, bay mud, soil-pile setup, and acceptance criteria

1. Background

The San Francisco-Oakland Bay Bridge (SFOBB) is the primary link between the cities of San Francisco and Oakland and carries 10 lanes of traffic across San Francisco Bay in California, USA. Yerba Buena Island (YBI) bisects the bridge, 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 east span of the SFOBB, which is one of the most heavily travelled 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 5 segments: (1) an approximately 460-metre-long transition structure extending from the YBI Tunnel to the eastern tip of YBI, (2) main-span signature structure extending offshore from the tip of YBI with an approximately 625-metre-long, single-tower that rises 164-metre above the water and asymmetrical, self-anchored suspension (SAS) cable, (3) an approximately 2.1-kilometre-long, four-frame Skyway structure extending from the signature structure eastward to the Oakland Shore Approach, (4) an Oakland Shore Approach structure extending about 700 metres from the Skyway structure to the north side of the Oakland Mole, and (5) an earthen fill transition from the Oakland Shore Approach structure to the roadways leading to and from the existing bridge.

Figure 1. Site Vicinity Map

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2. Site-Specific Geology

The geologic formations that underlie the skyway include, in the following sequence: Young Bay Mud, Merritt Posey-San Antonio formations, Old Bay Mud, upper Alameda Marine sediment and the Lower Alameda Alluvial sediment. Table 1 and Fig 2 provide key engineering soil properties and an illustration of the subsurface conditions encountered along the new bridge alignment.

The Self-Anchored-Suspension (SAS) signature portion of the bridge is supported by three pier supports. The West Pier-W2 and main Tower foundation supports are situated in the area of shallow, steeply sloping bedrock on and adjacent to YBI. West Pier W2 consists of two supports, one for eastbound and the other for the westbound direction. The two W2 foundations are 19 m x 19 m mass gravity foundations with a thickness of 10 m and will be constructed in rock.

Table 1. Engineering Soil Properties

Geological Formation	Friction Angle, Degrees	Undrained Shear Strength, kPa	Submerged Unit Weight, kN/m³
YBM: very soft to firm Clay	-	10 - 65	4 - 7
MPSA Dense ID 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

Figure 2. Interpreted Stratigraphic Section Along New Bridge Route

I. Proposed Foundations

The proposed bridge foundation types are largely based on the Geotechnical and geological conditions at the site. Along the new bridge alignment, the bedrock surface is relatively shallow and dips steeply near the shoreline of YBI then steeping to the east down to elevation of -100 metres at about 350 metres offshore from YBI. The bedrock contact then slopes gently toward Oakland. At the Oakland end, the bedrock surfaces are buried by about 135 to 140 metres of sediments.

Excavation of the rock is currently underway. The westbound footing includes four 2.5 m diameter, 10 m long cast-in-drilled-hole (CIDH) shafts socketed in slightly weathered to fresh rock while the eastbound footing is placed directly on rock. The westbound footing is embedded in steep bedrock slopes and the CIDH shafts provide lateral shear capacity.

In contrast, the SAS-East Piers and the Skyway and Oakland Shore approach portions of the bridge piers are underlain by over 85 to 135 metres of marine and alluvial sediments. SAS-East Piers

2E and 2W foundations consist of (12) 2.5-m diameter steel pipe piles up to 100 m long. Skyway eastbound and westbound Piers 3 through 16 consist of (160) 2.5-m diameter piles up to 100 metre long. 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 of these piers will be supported by an octagonal or rectangular pilecap with 4 to 6 piles, at 1:8 batters.

The Oakland Shore approach eastbound and westbound Piers 17 through 22 consist of 12 piers with 104 steel pipe piles and 1.8 m in diameter. 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 of these piers will be supported by a rectangular 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 1700 kilo-joule Menck MHU-1700 hydraulic impact hammer.

4. Axial Pile Capacity

4.1 Skin Friction

Since, the selected pipe pile diameters were outside the normal range of onshore/offshore piles and to reflect the site-specific soil conditions, the modified American Petroleum Institute (API) method was used for calculating axial capacities. The API design procedure (1993) 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:

$$f = \alpha S_u$$

$$\alpha = \left(\frac{c}{p'}\right)^{1/2} \left(\frac{S_u}{\sigma'_1}\right)^{-1/2}$$

Where: α -adhesion factor, S_u -undrained shear strength at any depth, σ'_1 -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-Marinc formations, the applicability of this assumption to the conditions at the bay bridge site was researched in more detail.

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 soil 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:

$$\alpha = (0.31)^{1/2} \left(\frac{S_u}{\sigma'_1}\right)^{-1/2} \text{ for } \frac{S_u}{\sigma'_1} < 1.0$$

$$\alpha = (0.31)^{1/2} \left(\frac{S_u}{\sigma'_1}\right)^{-1/4} \text{ for } \frac{S_u}{\sigma'_1} > 1.0$$

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

4.2 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 thickness of the interbedded clay layers. The value of the unit end-bearing pressure (q_u) for each frame was calculated as:

$$q_u = \frac{3.24 X + 1197 Y}{100}$$

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

4.3 Soil-Pile-Setup and Axial Capacity

In order to evaluate the soil-pile-setup and the axial capacity of a large diameter impact driven pipe pile with time, a series of CAPWAP analysis were done and compared with upper- and lower-bound soil-pile-setup predictions based on Soderberg (1962) method and API predicted axial capacity. 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 [8]. 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 pile installation demonstration project (PIDP) to better understand the magnitude and distribution of soil resistance along the pile. Three restrikes were conducted on Pile-1, three re-strikes were conducted on Pile-2, and two re-strikes were conducted on Pile-3. Based on CAPWAP analyses, the trend of increase in total skin resistance with time is illustrated in Fig. 3. For comparison, total skin resistance profiles computed, based on API design method (1993), is shown in Fig. 3.

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 capacities of 78 to 88 MN, which are approximately 98 to higher than 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

Fig. 3. Total Skin Resistance, Combined CAPWAP Analysis

that the skin friction capacity will exceed the capacity of 80 MN predicted by the API design method.

5. Soil Resistance to Driving

Stevens et al. (1982) recommended that lower- and upper-bound values of soil resistance to driving (SRD) be computed for the coring pile condition especially for larger-diameter pipe piles. When a pile cores, relative movement between pile and soil occurs both on outside and inside of the 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 [3] 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.

5.1 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 API (1986). The SRD is then calculated by reducing the unit skin friction values over increments of depth by multiplying by a pile capacity factor, determined empirically.

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

Fig. 4. Estimates of PILE setup From PIDP

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

$$S_u/U_{unc} = (OCR)^{0.85}$$

$$U_{unc} = \sigma'_{vo} \times (0.11 - 0.0037 \times p_i)$$

where: U_{unc} - undrained shear strength-normally consolidated clay, σ'_{vo} - effective overburden pressure and p_i - plasticity index

The 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

5.2 Method-II

Based on the "Sensitivity Method"; the unit skin friction for static loading is first computed by using the method recommended by API (1983, 1993). The SRD 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 SRD were used to perform the wave equation analyses to predict the lower- and upper-bound acceptance criteria. Wave equation analyses were performed 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 centimetres for all soil types. A shaft damping value of 0.19 to 0.36 seconds per metre was assumed for clayey soil. The shaft damping value in clayey soil decreases with increasing shear strength [2]. The toe damping value of 0.49 seconds per metre was assumed for all soil types.

Fig. 5 presents results of predicted and PDA observed 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. 6 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 metres, 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 metres for Pile 2. The observed blow counts spikes at certain depths was as a result of soil-pile setup that occurred during driving delays such as splicing/welding of pile sections.

5.3 Acceptance Criteria

Piles driven to the required design tip elevations and if meeting the coring case lower bound acceptance criteria for the given range of hammer energy (hammer efficiency and field blow counts) 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 early pile refusal is met (generally 5 to 10 m above the specified design pile tip elevation, depending on pier location) but the design elevation for lateral capacity is reached, the pile can be accepted if it met the following conditions.

- The specified primary hammer is operating at full rated energy according to the manufacturer'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.

5. 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.

Fig. 5. PDA Measured and Predicted SRD

Fig. 6. Observed and Predicted Blow Counts

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 (by either method for clayey soils) based on the coring case-lower bound acceptance criteria for the given range of hammer energy.

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