Temporary tower foundations for the New San Francisco-Oakland Bay Bridge self anchored suspension span

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ABSTRACT
The new east span of the San Francisco-Oakland Bay Bridge will feature a single tower self-anchored suspension (SAS) bridge that will become the largest of its kind in the world when it is completed in 2013. During construction of the SAS bridge, temporary towers and trusses are required to support the box girders until erection of the suspension cables is complete. The temporary towers are supported on various foundation types consisting of micro-piles, rock socketed drilled shafts, and large diameter steel pipe piles driven into marine sediments and bedrock. This paper presents details of the challenges faced during the design and construction of the tower foundations. Construction of the temporary towers and foundations began in 2008, the towers were in place by 2010, and placement of box girders and construction of the permanent main tower are currently underway.

RESUMEN
El nuevo tramo al este del Puente de la bahía entre San Francisco y Oakland (Bay Bridge) cuenta con una sola torre anclada auto-suspendida (SAS) que se convertirá en la mas grande de su clase en el mundo cuando se complete en el año 2013. Durante la construcción del puente de SAS, se requieren torres temporales y armaduras para apoyar las vigas de caja hasta que se instalen los cables de suspensión. Las torres temporales se apoyan en diversos tipos de fundaciones que consisten en micro-pilotes, pilotes de hormigón armado moldeados en roca in situ, y pilotes de acero de gran diámetro terminados en sedimentos marinos y roca. Esta publicación técnica detalla algunos de los desafios enfrentados durante el diseño y construcción de los cimientos de la torre. La construcción de las torres temporales y las fundaciones se comenzaron en el año 2008 y se completaron en el año 2010, y la colocación de vigas de caja y la construcción de la torre principal está actualmente en curso.

1 INTRODUCTION

The new east span of the San Francisco-Oakland Bay Bridge will feature a single tower self-anchored suspension (SAS) bridge that will become the largest of its kind in the world when it is completed in 2013. Figure 1 shows an artist's rendering of the new bridge with its main tower T1 in the middle, and Piers W2 and E2 at the west and east ends, respectively. The east and west spans of the self-anchored portion of the bridge will be 385 m and 180 m, respectively.

Figure 1. Artist's rendering of new Bay Bridge

During construction of the SAS bridge, temporary towers and trusses are used to support the box girders of the permanent bridge until the loads from the box girders are transferred through suspension cable to the permanent bridge Tower T1 and Piers W2 and E2. Six pairs of temporary towers are required along the eastbound and westbound lanes. Temporary Towers A, B and C are located on Yerba Buena Island between the permanent bridge pier W2 and tower T1. Tower D is located on either side of the main Tower T1 and Towers F and G are located between the main bridge Towers T1 and E2, as shown in Figure 2. The temporary structures need to be supported on competent foundations until the loads from the box girders are transferred to the permanent bridge towers T1, W2 and E2. The foundation soil condition varies from sedimentary bedrock at the west end with steep slopes to deep marine sediments at the east end. Consideration to cost-effectiveness, constructability, schedule and protection of environmentally sensitive areas was critical to the project when selecting the appropriate foundation types. The temporary towers are consequently supported on various foundations consisting of micro-piles, rock socketed drilled shafts and large diameter steel pipe piles driven into marine sediments and bedrock. This paper presents details of the challenges faced during the design and construction of the tower foundations.

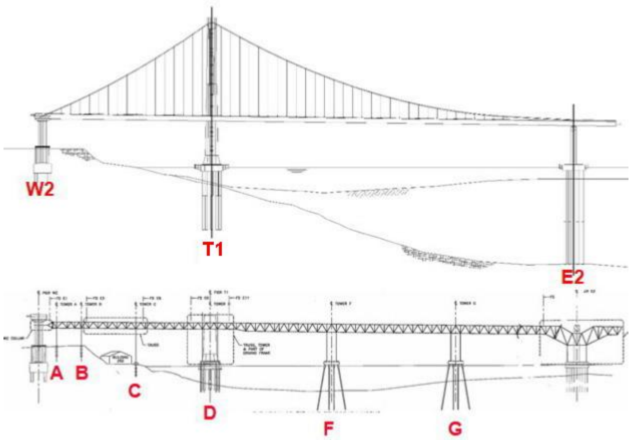

Figure 2. Location of Permanent Tower and Piers and Temporary Towers

2 SUBSOIL CONDITIONS AND FOUNDATIONS

Figure 3 shows the geological cross-section along the alignment of the new SAS bridge and the locations of the temporary Towers A through G. It also shows locations of the permanent Piers W2 and E2 and Tower T1, which will support the self-anchored bridge structure. The foundation conditions vary from sedimentary rock with interbedded sandstone, siltstone and claystone at the west end to very deep marine sediments with clay and sand overlying bedrock at the east end.

The primary geologic units at the site are:

- Young Bay Mud (YBM) - Very soft to soft or soft to firm clay;
- Merritt-Posey-San Antonio Formations (MPSA) - Very stiff clay layers;
- Old Bay Mud (OBM) - Very stiff to hard clay;
- Upper Alameda Marine Sediments (UAM) - Very stiff to hard clay;
- Upper Alameda Marine Paleochannel Sand (UAMPC) - Very dense sand with hard clay layers;
- Lower Alameda Alluvial Sediments (LAA) - Dense and very dense sand and hard clay layers; and
- Franciscan Formation Bedrock (FF) - Sandstone interbedded with siltstone and claystone

As shown in Figure 3, Towers A and B are located on bedrock outcrop. Tower C is located near the shoreline and Towers D, F and G are located in water. The depth to bedrock increases from a few metres at Tower C to about 90 m at Tower G. The thickness of the marine sediments also increases from Tower D to Tower G while sea floor (mudline) dips from EL -15 m at Tower C to about EL -25 m at Tower F and then rises to about EL -20 m at Tower G.

The following foundation types were selected for Towers A to G:

- Rock socketed drilled shafts for Towers A and B located on fairly flat ground with bedrock at shallow depth;
- Micropiles for Towers A and B located on steep slopes of bedrock outcrop;
- Rock socketed drilled shafts for Tower C located near the shoreline on relatively thin layer of overburden soils overlying bedrock at shallow depth;
- Driven steel pipe piles for Tower D located offshore with limited thickness of marine sediments and bedrock at relatively shallow depth; and
- Driven steel pipe piles for Towers F and G located off shore with deep marine sediments overlying bedrock.

3 TOWERS A AND B

As shown in Figure 4, on the north side or westbound lane, one footing for Tower A and two footings for Tower B are located on steep slopes of the bedrock outcrop at Yerba Buena Island, and the remaining five footings are located on fairly flat ground or bedrock

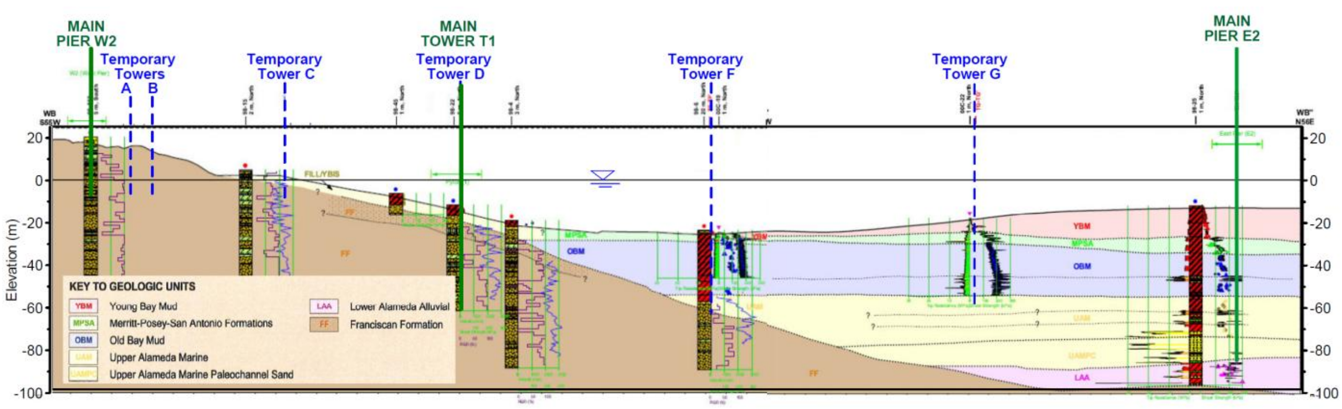

Figure 3. Geological cross section along the temporary Towers A to G

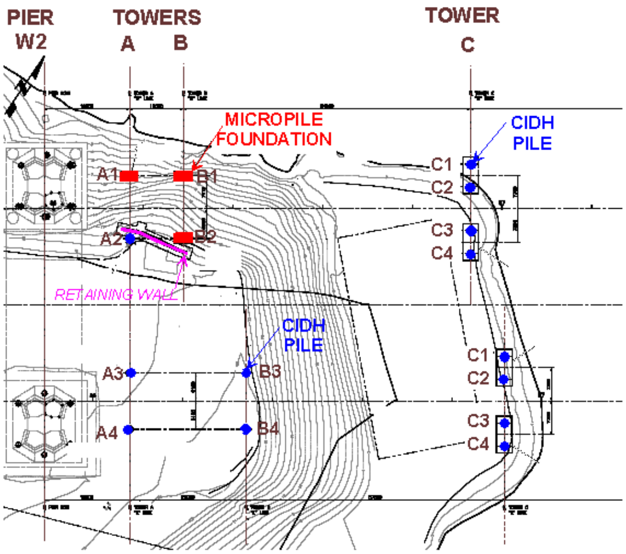

Figure 4. Temporary tower foundations A, B and C

The area is generally characterized by surficial Franciscan bedrock but the characteristics of the rock and thickness of the weathered zone vary spatially. However, the transition from intensely weathered to slightly weathered or fresh rock occurs at relatively shallow depth in this area.

3.1 CIDH Pile Foundations for Towers A and B

1.5 m diameter cast-in-drilled hole (CIDH) piles with rock sockets were used for the foundations on fairly flat part of the bedrock outcrop for Towers A and B. The CIDH piles were designed conservatively based on skin friction within the Franciscan bedrock alone, without consideration of skin friction within soil overburden, or end bearing in rock. The skin friction capacity under both compression and tension was estimated based on ultimate bond strength of 480 kPa. The pile load tests conducted for the Benicia-Martinez Bridge (FugroEMI, 2002) using Osterberg cells on 1.7 m diameter drilled shaft with sockets in bedrock consisting of interbedded shale, siltstone and sandstone yielded bond strengths of about 400 kPa to 720 kPa. Based on this information, 480 kPa of bond strength was selected in the absence of any site specific pile load test data. The socket lengths of the CIDH piles varied between 4 m and 7 m in slightly to moderately weathered rock and the pile toes were between 7 m and 8 m below the ground surface.

During construction, 1.5 m holes were pre-drilled using a rotary drill rig and casings were installed at the top part within the overburden soils and weathered rock. Rebar cages were then inserted and the holes were filled with concrete to create the CIDH piles.

3.2 Micropile Foundations for Towers A and B

Micropiles were selected as an alternative but viable option to CIDH piles at the steep rock slope at Towers A and B. The pile arrangements consisting of both vertical and near horizontal batter piles were designed to resist the vertical and lateral loads, respectively, from the tower columns. The loads are transferred from the tower column to the pile through a cast-in-place 1.6 m thick concrete pile cap. The batter piles were oriented at 10 degrees to the horizontal. 165 mm diameter micropiles were used with 65 mm diameter center bars (Williams 1035 MPa all thread bar). 2 m long casings were installed at the top within the intensely to moderately weathered bedrock. The micropiles were assumed to carry axial loads only, either in tension or compression. The bonded length of the micropile, which is in contact with the sound Franciscan Formation bedrock, was computed using a bond strength of 480 kPa, both in tension and compression. Design loads were up to 1800 kN per pile.

The static and seismic global stability of the slopes due to the excavation and the additional loads induced by the tower foundations in the vicinity of micropile and CIDH foundations was checked using the computer program Swedge (Rocscience, 2005).

During construction, excavation of rock faces and support were required prior to the installation and testing of micropiles and construction of pile cap. Rock bolts and wire mesh were used for the temporary stabilization of rock faces. The 200 mm diameter holes for micropiles were drilled using a rotary percussion drill rig. The sub-horizontal holes were drilled from a suspended platform held by a crane at the crest of the slope. The 65 mm diameter center bars were then inserted and grouted. Figure 5 shows the micropile construction at A1.

Figure 5. Micropile construction at A1

Tension proof tests were conducted on all vertical and batter micropiles. Custom test setup was used to test batter piles. Figure 6 shows the tension proof test on a vertical pile at B2. Twenty four piles were tested to the loads listed in Table 1 and the remaining four piles were tested to loads ranging between 56-75% of the loads in Table 1. The micropiles tested to the specified full load showed generally consistent load-displacement behaviour and they generally fell between the minimum and maximum elastic limits. The creep displacements were also well within the allowable limit. Figure 7 shows typical tension proof test results on six vertical piles at micropile foundation A1.

An old retaining wall of historical importance shown in Figure 4 was protected and monitored during construction but showed no movement due to adjacent micropile foundation construction A1 and CIDH pile construction at A2.

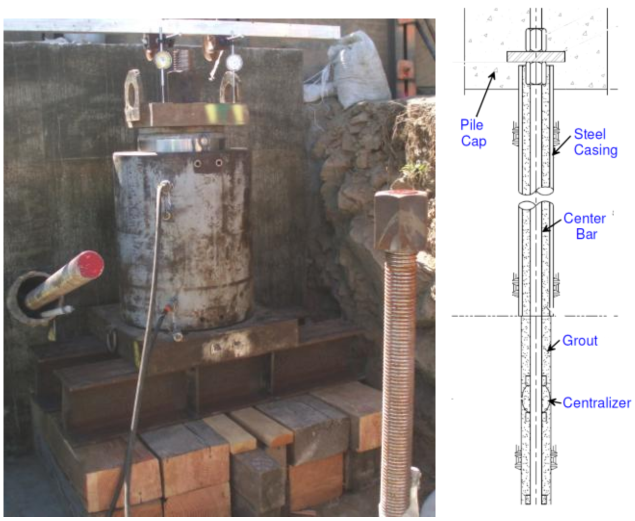

Figure 6. Tension proof test of a vertical pile at B2 and detail of a typical micropile

Table 1. Test Loads for Tension Proof Test

Foundation	Maximum Tension Load (kN)	
	Vertical Pile	Batter Pile
A1	1100	500
B1	1800	500
B2	1800	1500

4 TOWER C

The foundations for Tower C consist of four pile caps, each supported on two vertical 1.5 m diameter rock socketed concrete filled steel pipe piles. The piles were designed to carry vertical and lateral loads up to 17 MN and 1.7 MN, respectively. Figure 4 shows the layout of the piles and the pile cap. As shown in Figure 4, the pile foundations for Tower C are located on a filled platform at the bottom of the east rock slope at Yerba Buena Island, at the edge of the Bay and in close proximity to an existing historic structure, the Torpedo building, which needed to be protected during construction. Existing riprap protection, tide, dipping bedrock, varying thicknesses of fills and weathered rock and lack of sufficient drill hole information provided challenges during design and construction. Similar to Towers A and B, the piles were socketed in the slightly weathered bedrock and the top part of the piles were cased. The socket lengths ranged between 5.5 m and 8.5 m and the pile toe elevations varied from EL -7 m to EL -15 m.

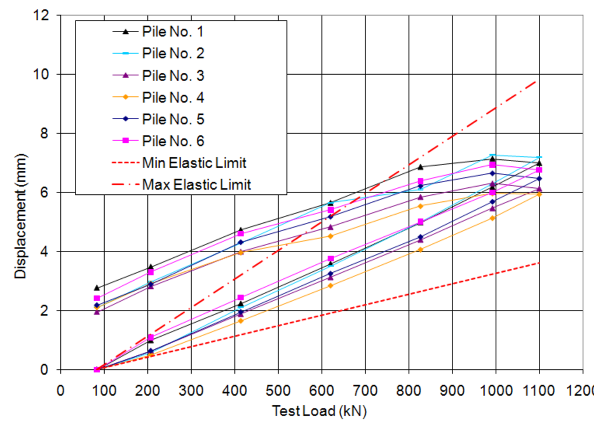

Figure 7. Tension proof test results for A1 vertical piles

Lateral pile load analyses were conducted using p-y curves to capture the pile-soil interaction. As established p-y curves are currently not available for piles socketed in Franciscan rock, p-y curves were generated using various methods and their sensitivity was checked. Figure 8 shows the p-y curves which included: Cases 1 and 2: linear p-y curves based on modulus of reaction for fresh rock and moderately weathered rock, respectively; Case 3: bi-linear p-y curves recommended by Reese et al. (2004) for strong rock; and Case 4: piecewise linear p-y curves based on 3D finite element analyses of larger diameter piles socketed in Franciscan rock (Fugro-EMI, 2002). The mobilized soil reaction was mostly in the linear range and the difference in the initial stiffness resulted in about 1.5 m difference in the depth to fixity which range between 6 m and 11 m for the piles.

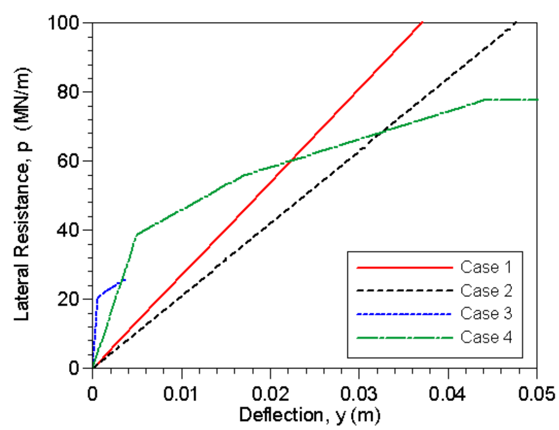

Figure 8 P-y Curves used for Franciscan Rock in sensitivity Analyses

During construction, the riprap protection was temporarily removed and a cofferdam and working pads were built using a combination of bulk aggregate bags (BABs) partially filled with gravel, fabric liners and crushed rock. The holes for the CIDH piles were drilled using a rotary drill rig. Prior to drilling into the sound rock, a 2.1 m diameter corrugated metal pipe was installed down to the sound rock, grouted and filled with lean mix concrete. The 1.5 m diameter holes were then drilled through the concrete rather than the crushed rock. The rebar cages were installed and filled with concrete to create the CIDH piles. Figure 9 shows drilling and rebar cage installation for CIDH pile at Tower C from a temporary working platform.

Figure 9. Drilling and rebar cage installation at Tower C CIDH pile

5 TOWER D

At Tower D, the site is characterized by Young Bay Mud underlain by rock fragments and Franciscan rock. The mudline and the bedrock surface both dip to the north-east. The elevation of the mudline at the pile locations ranges between about EL -10 m and EL -20 m and the elevation of the bedrock surface ranges between about EL -17 m and EL -22 m. The soil cover above bedrock includes soft fine grained clay, sand, gravel and rock fragments. The upper section of bedrock is intensely fractured and weathered, but its thickness is limited to about 7 to 9 m. Below EL -25 m to EL -31 m, the bedrock becomes slightly weathered to fresh. The interlayered bedrock sequence consists primarily of sandstone and siltstone. Figure 10 shows the pile layout for Tower D.

CIDH piles would have been an obvious choice for Tower D foundations. However, earlier CIDH pile installation for the main T1 tower foundation had encountered hole caving, necessitating drilling and backfilling with concrete in incremental stages until sound rock was reached. Driven steel pipe piles were chosen as an alternative for Tower D to avoid hole caving and environmental issues related to drilling and constructing under water. It also helped to save cost and expedited the construction. As there was not much precedence in driving steel pipe piles in Franciscan rock, the design and construction of pile foundations at Tower D provided somewhat unique challenges.

The steel pipe piles at Tower D are designed to carry vertical loads only and the lateral loads originating from the superstructure at this location are transferred to the pile cap of the permanent Tower T1, which was already constructed. The maximum geotechnical axial capacity of the piles is 13 MN. Each temporary Tower D is supported by four pile groups, each consisting of four 1.1 m diameter vertical steel pipe piles with 38 mm wall thickness. The steel pipe piles were pre-fitted with a reinforced shoe. The surface of the bedrock dips approximately 10 to 20 degrees toward the east at Tower D and the reinforced pile toe was expected to minimize the risk of pile toe damage upon encountering the dipping rock surface.

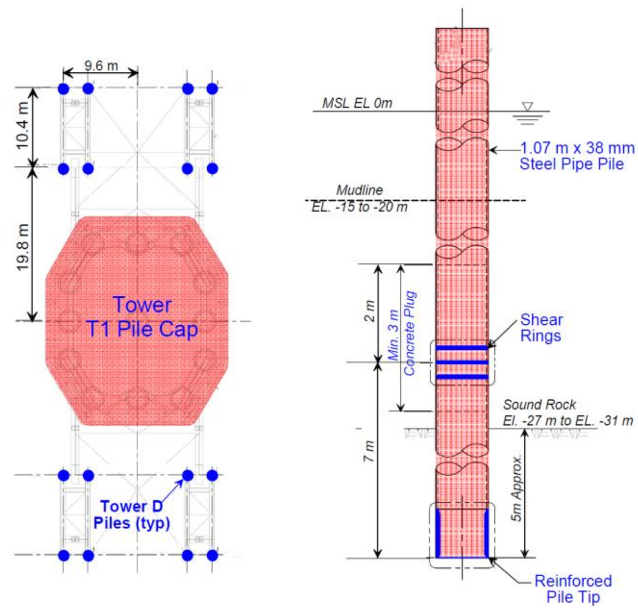

Figure 10. Tower D pile layout and details

Pile drivability analyses were conducted to evaluate the drivability of the open-ended steel pipe piles and to recommend a pile driving hammer system. The pile driving hammer system should be able to drive the piles to adequate depth or to refusal to achieve the required capacity without overstressing the piles. Several pile driving hammers were considered which included a Delmag D100-13 diesel hammer with a rated energy 360 kJ and Menck MHU-500T hammer with rated energy of 550 kJ.

The ratio of end bearing to total resistance can play a significant role in the induced driving stresses in the pile and, generally, increase in end bearing increases driving stresses. However, soil resistance to driving is difficult to assess in rock due to large variations in the degree and thickness of weathering and the nature of fracturing in the rock. Thus, drivability analyses were conducted for a range of end bearing values between 30% and 90% of the total resistance, which was kept slightly above the required capacity of 13 MN. In addition, two sets of quake and damping values were used for the rock to bound the uncertainty of these parameters for the rock. As expected, increase in shaft friction resulted in reduced blow counts for the same ultimate capacity and reduction in induced compressive stresses in the steel pipe pile. However, the effects were insignificant except in the case where the end bearing ratio was taken as 90% with a toe quake of 1 mm. In the latter case, the compressive stresses increased by about 20%. The analysis also showed that the Delmag D100-13 hammer cannot be used, if the end bearing component is 60% or greater. Thus, the Menck MHU-500T hammer was recommended for driving.

A rock plug may form during driving open ended piles and it could potentially induce large compressive stresses on the steel pipe pile just above the plug during driving. To address this issue, sensitivity analyses were conducted with and without a 5 m rock plug. Figure 11 shows the model used in the analyses. Note, during the initial design stage, several sensitivity analyses were conducted to assess the effect of shaft friction, toe and quake damping, height of rock plug which may form in the intensely weathered rock at the top by varying these parameters over possible ranges. Initial shaft friction distribution was estimated based on the test data collected during the driving of temporary casings for the main Tower T1 foundations.

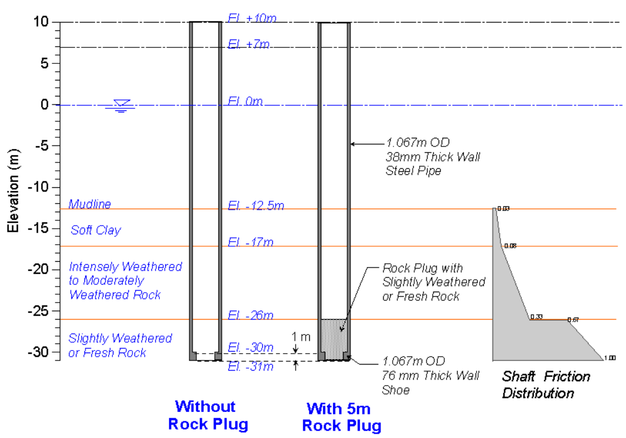

Figure 11. Pile configurations with and without rock plug

The drivability analyses were conducted using the computer program GRLWEAP (Grlweap, 2005). In the analyses, the piles were assumed to be driven to the full depth at full stroke for the Menck MHU-500T hammer. Figure 12 shows results from the analyses suggesting that the pile conforming to ASTM A572 Grade 50 will not be overstressed and the pile can be driven into the rock open-ended to achieve the necessary capacity. The initial refusal criteria set for the MHU-500T hammer was 8 blows for the last 25 mm of driving. Piles were monitored with PDA during initial driving to refusal to check pile integrity and capacity. Re-striking of piles had been called for after a period of 15 days to ensure that there is no loss of capacity due to potential degradation or relaxation of the rock.

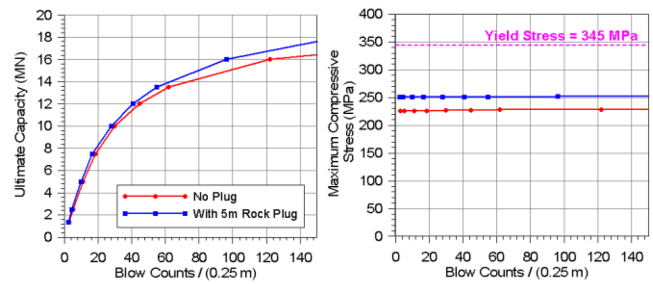

Figure 12. Wave equation analyses results with and without rock plug

As there was a concern that the piles, while driven open-ended, might not reach refusal nor achieve the intended capacity, a backup plan was considered during the design stage to increase the toe resistance and to achieve the design capacity. In such case, the soil and any weathered rock fragments above the hard rock plug inside the pile would be cleaned out and a concrete plug with a minimum thickness of 3 m would be formed to increase the toe resistance. To enhance axial load transfer between the pile and concrete, each pile was pre-fitted with three shear rings inside the pile. The concrete plug with shear rings was structurally designed to sustain more than 14 MN without slippage or failure. The estimated toe resistance was about 15 MPa for the rock with an unconfined compressive strength (UCS) in the range of 20 MPa to 130 MPa which was sufficient to achieve the required capacity. However, during construction, there was no need to increase toe resistance using concrete plug.

During construction, piles were initially vibrated through the upper soils using an APE600 vibratory hammer and then driven using the Menck MHU-500T hydraulic hammer to practical refusal. Figure 13 shows pile driving at Tower D using the Menck MHU500T hammer. The piles were re-struck twice using the same hammer and dynamic monitoring was conducted on eight piles (50% of total piles) using a Pile Driving Analyzer (PDA) during initial driving and re-strikes. CAPWAP analyses were also conducted to estimate the pile capacities. The final pile toe elevations varied between EL -26 m to EL -33 m and the lengths below the driving frame varied between 33 m and 40 m.

During initial drive, the pile capacity at the end of initial drive (EOID) estimated using the CAPWAP method was greater than 20 MN and well in excess of the required capacity of 13.2 MN. The pile capacities estimated using the CAPWAP method during the beginning of 1st re-strike (BOR) ranged between 18 MN and 23 MN and they were also well in excess of the required capacity. However, both the capacities and the blow counts generally showed decrease on re-strike, apparently due to the effect of pile end bearing relaxation. The reduction in total pile capacity generally ranged between 10% to 20% except for a pile with a 50% reduction which had a very high capacity (37 MN) during the initial drive. Review of blow counts during initial driving and re-striking indicated that the embedment into the sound rock appeared to have significant effect on the magnitude of relaxation. The piles that had greater embedment into the sound rock showed less relaxation. Thus, the piles were struck with more stringent criteria during re-strikes. The capacity of piles estimated from CAPWAP analyses at the beginning of the 2nd re-strike showed less relaxation effect and were well in excess of the required capacity of 13 MN. They ranged between 19 MN and 27 MN. The driving stresses were well below the limit (less than 250 MPa) during initial driving and re-strikes.

Figure 13. Pile driving at Tower D using Menck hammer

Tower D pile foundations have been monitored for settlement since its construction. The measured settlement in October 2010 varied between 3 mm and 9 mm and most of which is due to elastic shortening of the pile. By October 2010, full dead load on Tower D was in place.

6 TOWERS F AND G

The foundation soils at Towers F and G consist of very soft to firm clay underlain by very stiff clay to hard clay. The lower clay layers are interbedded with dense to very dense sand layers. The bedrock, which consists of sandstone, siltstone and claystone at these locations, dips steeply towards the northeast, with elevations ranging from about EL -60 m at Tower F to about EL -95 m at Tower G. The average mud line elevations at Towers F and G were about EL –26 and EL -18 m, respectively.

Figure 14 shows the typical pile layout of one tower foundation at Towers F and G. The foundations for Towers F and G consist of two pile groups, one for each of the eastbound lane and westbound lane towers. Each pile group has 14 perimeter batter and 4 interior vertical steel pipe piles. The diameters of the batter piles were 1.2 m and 1.1 m and the diameter of the vertical piles was 1.1 m. All the piles had 32 mm wall thickness. The batter angles were 7V:1H, 6V:1H and 5V:1H. Piles were designed to be driven open-ended to the specified toe elevation. However, there was a possibility that some of the piles at Tower F may encounter the dipping bedrock and thus, all the piles for the Tower F were pre-fitted with reinforced shoe and a practical refusal criterion into rock was also specified.

The ultimate axial pile capacities were estimated based on shaft resistance on the outside of the steel pipe piles and was taken to be the same both in compression and tension. The end bearing component was not included in the ultimate resistance calculation due to settlement concerns, except for the piles that are tipped in bedrock. The unit skin friction was estimated based on insitu, laboratory and piles load test data. The required ultimate axial compression resistances of the 1.2 m and 1.1 m batter piles at both towers were 9.5 MN and 10.0 MN, and their specified toe elevations varied between EL -58 m and EL -65 m with depth of penetration ranging from 33 m to 43 m. The required tension resistances were approximately half of the compression resistances for these batter piles. For the shorter 1.1 m diameter plumb piles at both towers, the required axial resistance was 5 MN and the specified toe elevation was approximately EL -48 m.

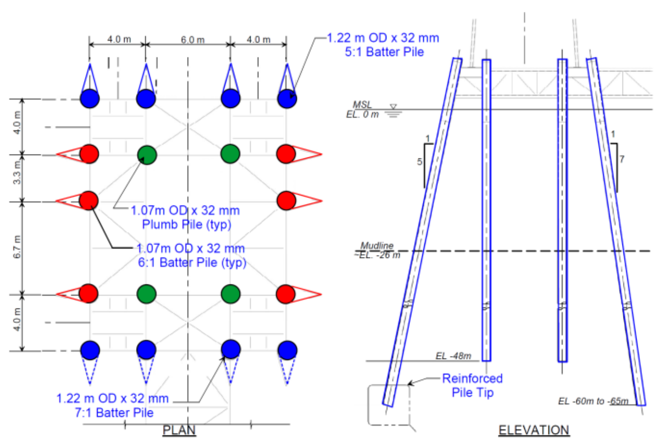

Figure 14. Pile layout at Tower F

Soil-pile interaction analyses using t-z springs were conducted to estimate pile settlement, which was limited to 25 mm at the mudline level. The estimated settlement at the mudline level ranged between 13 mm and 18 mm depending on the load, diameter and length. These settlements include elastic shortening of the piles.

Drivability analyses were conducted using the computer program GRLWEAP to confirm that Delmag D100-13 diesel hammer could be used to drive the piles to the required toe elevation or practical refusal. The setup effect in the clayey soils and potential for plugging during driving were considered in the drivability analyses. Based on the data from the Pile Installation Demonstration Project (Fugro-EMI, 2001), the skin friction during driving in the clayey soils was reduced by 60% (equivalent to a loss factor of 2.5). However, the skin friction in sandy soils and toe resistance were not reduced. During pile driving. whether the advancing open ended pipe pile is coring the soft soil or becoming partially or fully plugged depends on the soil conditions, pile diameter, pile roughness and pile acceleration during driving. However, large diameter piles driven continuously into the cohesive soils are unlikely to plug unless there is stoppage during driving that allows significant setup to occur. Owing to the uncertainty in the plugging behaviour of the pile, both coring and plugged pile conditions were considered in the drivability analyses.

Table 2 summarizes the three cases considered in the estimation of soil resistances to driving in the drivability analyses. These three cases were expected to cover potential range of soil resistances to driving at Towers F and G. In the drivability analyses, the piles were assumed to be driven to the full depth using the same hammer at maximum fuel setting. A 96 hour delay was also considered for pile splicing. Figure 15 shows the soil-pile configuration, unit skin friction distribution and the unit end bearing distribution used in drivability analyses for piles at Tower F. The drivability analyses indicated that a Delmag D100-13 hammer could be used to drive the piles at both Towers F and G to the specified toe elevations.

Table 2. Soil resistance to Driving Considered in Drivability Analyses

Case	Skin Friction		End Bearing
	Outer Perimeter	Inner Perimeter	
1-Coring Pile-Lower bound	Yes*	None	Yes (on Annulus)
2-Coring Pile-Upper bound	Yes*	Yes*	Yes (on Annulus)
3-Plugged Pile	Yes*	N/A	Yes (on Full area)

*Static values in clay reduced by 60%

Resistance to driving in rock is difficult to assess due to large variations in the degree and thickness of weathering and the nature of fracturing in the rock. However, the ratio of end bearing to total resistance can play a significant role in the induced driving stresses in the pile and, generally, the driving stresses near the toe increases as the end bearing component increases. Thus, in due consideration of softening of soils during driving, wave equation analyses were conducted for end bearing values of 15% and 50%, which would cover the potential range of end bearing component during driving. Analyses showed that an increase from 15% to 50% end bearing would increase the driving stresses just above the toe by about 50%. However, the maximum driving stress of about 170 MPa at the top of the pile is still much lower than the yield stress of 345 MPa.

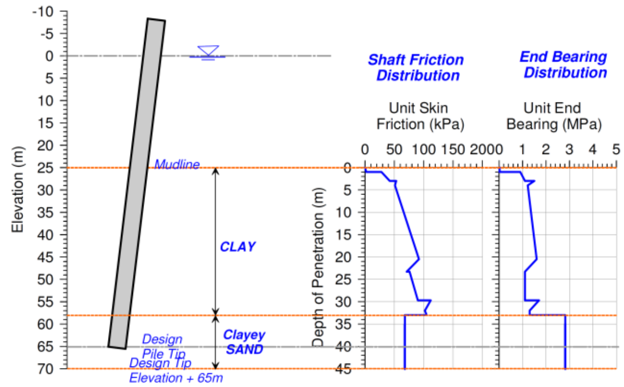

Figure 15. Tower F pile and unit skin and end bearing distribution

The pile run (i.e. the pile penetration due to self weight and weight of the hammer) was estimated to be in the range of 7 m to 12 m and was not expected to be an issue during pile driving.

During construction, the pre-fabricated driving frames were floated in place and the interior plumb piles were driven to the toe elevation using the Delmag D100-13 hammer. The batter piles were then driven using a vibratory hammer APE600 to expedite the pile driving. Only the last 5 m to 15 m was driven using the Delmag D100-13 impact hammer. At Tower F, six piles each at the north and south side were re-struck after approximately two to three weeks, to verify pile capacity.

At Tower F, six piles each on the north and south sides were monitored with PDA during initial driving and re-strike to check hammer performance, driving stresses and pile capacities. CAPWAP analyses were also performed to estimate pile capacities at the end of initial drive (EOID) and at the beginning of re-strike (BOR). Five out of the twelve piles at Tower F, which were monitored, reached refusal in bedrock. For piles that reached refusal, CAPWAP capacities ranged between 7 MN and 13 MN during initial drive and they increased to 11 MN to 17 MN at the time of re-strike.

Table 3 summarizes the capacity of four piles at the east bound lane at Tower F that were tipped in soil. The pile capacities at the end of initial drive and at the beginning of re-strike were estimated from CAPWAP analyses. The piles were re-struck approximately three weeks after the initial drive and the pile capacities increased by a factor of 1.9 to 3.3 in this period. The initial low capacities are partially due to the fact the piles were driven using the vibratory hammer. The pile capacities were expected to increase beyond the three-week period. The CAPWAP capacities of four piles at the west bound lane during the restrike ranged between 10.5 MN and

13.4 MN. These westbound lane piles were re-struck two weeks after initial drive.

Figure 16. Tower G batter pile installation

Table 3. Pile Capacities of Tower F Piles at Eastbound Lane

Pile	Pile Capacity (MN)		Capacity Ratio BOR/EOID
	EOID	BOR	
F105	4.5	13.0	2.9
F108	3.9	10.7	2.7
F115	4.0	13.1	3.3
F128	3.8	7.3	1.9

7 SUMMARY AND CONCLUSIONS

The temporary towers for the new SAS bridge are supported on foundations consisting of micro-piles, rock socketed drilled shafts, and large diameter steel pipe piles driven into marine sediments and bedrock. This paper presents details of the challenges faced during the design and construction of the tower foundations. The construction of the temporary towers and foundations was completed in 2010, and placement of box girders and construction of main tower are currently under way.

The foundation condition varies from sedimentary bedrock at the west end to deep marine sediments at the east end. Consideration to cost-effectiveness, constructability, schedule and protection of environmentally sensitive areas was critical to the project when selecting the appropriate types of foundations. The tower foundations on steep sloping bedrock on Yerba Buena Island were supported on 165 mm vertical and horizontal micro-piles while the towers on fairly flat bedrock outcrop and shallow bedrock were supported on 1.5 m diameter rock socketed drilled shafts. Tension proof tests were conducted on all the vertical and horizontal micro-piles. The offshore temporary tower foundations immediately adjacent to the main permanent bridge tower foundation consist of 1.1 m diameter steel pipe piles driven to refusal into the sound rock through shallow soft clay and weathered bedrock using a Menck MHU500T hydraulic hammer. As there was no precedence in driving large diameter open-ended pipe piles into the Franciscan bedrock, unique challenges were faced in predicting driving stresses, selection of appropriate hammers, pile embedment in bedrock, pile toe reinforcement, and the refusal criteria considering the potential pile relaxation effect. All piles were monitored with PDA equipment during initial driving and re-strike, and CAPWAP analyses were conducted to estimate pile capacities.

The two pairs of towers at the east end are supported on large vertical and battered steel pipe piles driven either to design embedment in the Bay Mud or to refusal into bedrock. The piles were 1.1 m and 1.2 m in diameter and up to 70 m in length. Drivability analyses with varying soil resistances to account for the setup effect in Bay Mud were conducted to predict pile run, driving stresses, selection of hammer and refusal criteria. Initial driving for these piles was carried out using an APE600 vibratory hammer to expedite pile driving and only the last 15 m or less were driven using a Delmag D100-13 diesel hammer. Selected piles were monitored with PDA equipment during initial driving and during re-strike and CAPWAP analyses were conducted to estimate pile capacities. Due to the set-up effect in the Bay Mud, the piles, which were tipped in soil, gained 1.9 to 3.3 times the capacity in the two weeks period prior to the re-strike.

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