MICROPILE SOLUTIONS FOR CONSTRUCTION OF SAS SPAN OF THE NEW SAN FRANCISCO-OAKLAND BAY BRIDGE

Thuraisamy Thavaraj¹, Alex Sy², and David Dowdell³

ABSTRACT

The new east span of the San Francisco-Oakland Bay Bridge features a single tower self-anchored suspension (SAS) bridge that became the largest of its kind in the world when completed in 2013. During construction of the SAS bridge, micropiles were effectively used to solve three significant construction issues: (1) Construction of the SAS bridge required temporary towers and trusses to support the box girders until erection of the suspension cables is complete. Three of these towers were located on steep slopes of Yerba Buena Island bedrock outcrop. For these towers, vertical and sub-horizontal micropiles were used as a viable alternative to conventional piles, considering constructability, cost-effectiveness, schedule and environmental impact. All the micropiles were tested and performed well during construction of the bridge. (2) The installation method used for the main cable of the suspension bridge required the 160 m high main tower to be pulled 500 mm to the west, using 200 m long pull-back cables anchored to bedrock at Yerba Buena Island prior to the erection of the main cable. 18 m long micropiles were used as foundation for the tie-back cables inclined at 40 degrees. A reinforced concrete pile cap varying in thickness and width was used for testing the micropiles and for locking them off before the tiebacks were attached. (3) The bridge decks from the west approach connecting to the west pier of the SAS bridge needed to be pulled down 75 mm in order to make the hinge closure pour during construction. A tie-down system consisting of two pairs of steel beams with ten pull-down cables on each beam was used. Two rows of 20 m long vertical micropiles were used as foundation for the tie-down cables. The micropiles were tested using the adjacent micropiles as reaction piles, without a pile cap, and using the same steel beams. This paper describes the design and construction of the micropiles and the connection details between micropiles and the structure, focusing on some of the challenges faced during the design, installation and testing of the micropiles.

1. INTRODUCTION

The new east span of the San Francisco-Oakland Bay Bridge features a single tower self-anchored suspension (SAS) bridge and is the largest of its kind in the world since completion in 2013. The new east span replaced a double-deck steel bridge built in the mid-thirties.

The new SAS bridge with its unique design was built to withstand a much larger earthquake than the original structure. A span of the old steel bridge had collapsed during the 1989 Loma Prieta earthquake, subsequently repaired and was in service until the new

¹ Senior Geotechnical Engineer, Klohn Crippen Berger Ltd., Vancouver, BC, Canada, tthavaraj@klohn.com
² Vice President, Technical, Klohn Crippen Berger Ltd., Vancouver, BC, Canada, asy@klohn.com
³ Senior Structural Engineer, Klohn Crippen Berger Ltd., Vancouver, BC, Canada, ddowdell@klohn.com
SAS bridge was completed in 2013. During construction of the SAS bridge, micropiles were effectively used to solve three significant construction issues. This paper presents details of the issues and how they were solved using micropiles.

2. SAS BRIDGE AND TEMPORARY TOWERS

Figure 1 shows a schematic of the new bridge with its 160 m high main tower T1 in the middle, and Piers W2 and E2 at the west and east ends, respectively. The west and east spans of the SAS bridge are 180 m and 385 m, respectively. Pier W2 is founded on the Yerba Buena Island (YBI) bedrock outcrop. Tower T1 and Pier E2 are founded in San Francisco Bay on piles installed through marine sediments into sedimentary bedrock.

![Figure 1. New east span of the San Francisco-Oakland Bay Bridge](image)

During construction of the new bridge, a temporary bridge comprising towers and trusses was used to support the box girders of the new bridge until the loads from the box girders were transferred to the new suspension cables and the new permanent towers. Figure 2 (bottom part) is a schematic of the temporary bridge with the six pairs of temporary supporting towers along the eastbound and westbound lanes. Temporary Towers A, B and C were located on YBI between the permanent bridge Pier W2 and Tower T1. Temporary Tower D was located on either side of the permanent Tower T1 in the Bay, and Temporary Towers F and G were located between the permanent Tower T1 and Pier E2. The design loads on the temporary structures included construction loads, ship impact, wind and earthquake loads.

Along the bridge alignment, the general foundation conditions vary from sedimentary bedrock with interbedded sandstone, siltstone and claystone (Franciscan Formation) at the west end on YBI, to deep marine sediments with clay and sand overlying sloping bedrock at the east end in San Francisco Bay (Fugro-EMI, 2002). See Figure 3 for foundation conditions between Towers A and D.
Temporary Towers A and B, located on bedrock outcrop at YBI, were supported on micropiles on sloping ground and on drilled shafts on level ground. This paper focuses on the micropile foundations at A and B. Temporary Tower C, located near the shoreline, was supported on drilled shafts. The marine foundations for Towers D, F and G were supported on driven steel pipe piles. The design and construction of the drilled shaft and driven pipe pile foundations are described in Sy et al (2011) and Thavaraj et al (2013).
3. MICROPILE FOUNDATIONS FOR TEMPORARY TOWERS A AND B

As shown in Figure 4, on the north side or westbound lane, one footing (A1) for Tower A and two footings for Tower B (B1 and B2) were located on the north steep side slopes of the bedrock outcrop at YBI, and the remaining five footings were located on fairly flat ground. Some shallow fill covers the sedimentary bedrock sequence. The characteristics of the rock and thickness of the weathered zone vary spatially. Transition from intensely weathered to slightly weathered or fresh rock, however, occurs at relatively shallow depth.

![Figure 4. Temporary tower foundations A, B and C on Yerba Buena Island](image)

Micropiles were selected as a viable alternative to drilled shafts at the steep rock slope for foundations at A1, B1 and B2. The micropile foundations could be constructed using lighter crane-supported equipment and adapted more easily to the actual site conditions. Utilising micropiles minimized the disturbance to the area surrounding an existing historical wall at foundation B2. At the completion of the bridge construction, foundations of the temporary works were removed and the original ground surface restored. Minimizing the foundation size reduced the effort required for site restoration.

Each micropile foundation consisted of 6 or 8 vertical and 2 subhorizontal (batter) piles designed to resist the vertical and lateral loads, respectively, from the tower columns. The batter piles were oriented at 10 degrees to the horizontal. The loads were transferred from the tower column to the piles through a cast-in-place 1.6 m thick concrete pile cap. 200 mm diameter micropiles were used with 65 mm diameter center bars (Williams’ 1035 MPa all-thread bar). Two metre long casings were installed at the top within the intensely to moderately weathered bedrock; however, they were not designed to carry lateral loads. The micropiles were assumed to carry axial loads only, either in tension or compression. The bonded length of the micropile in contact with sound bedrock
was computed using an ultimate bond strength of 480 kPa, both in tension and compression. Maximum pile design load (including seismic) was 1800 kN. The static and seismic global stability of the slopes due to excavation and additional loads induced by the tower foundations was checked with rock wedge slope analysis (Swedge, 2005).

During construction, excavation of the rock faces and shoring support were required prior to the installation and testing of micropiles and construction of pile cap. Rock bolts, wire mesh and shotcrete were used for the temporary stabilization of rock faces. The 200 mm diameter holes for micropiles were drilled using a portable rotary percussion drill rig. The subhorizontal 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. Figures 5 to 7 show photos taken during micropile installation.

Figure 5. Typical micropile details and micropile installation at Tower A1

Figure 6. Drilling adjacent to historic retaining wall at B2

Figure 7. Micropile foundation construction at A1, B1 and B2
Tension proof tests were conducted on all vertical and batter micropiles. Custom test setup was used to test the batter piles. 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 target loads due to test setup problems that resulted in differential settlement and cracking of concrete bearing area and safety concern. The micropiles tested to the target loads showed consistent load-displacement behaviour that generally fell between the minimum and maximum elastic limits. The ten-minute creep displacements were also well within the allowable limit of 1 mm. Figure 8 shows typical tension proof test results on six vertical piles at micropile foundation A1.

Table 1. Maximum loads for tension proof tests

<table>
<thead>
<tr>
<th>Micropile Foundation</th>
<th>Maximum Tension Load (kN)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vertical Pile</td>
<td>Horizontal Batter Pile</td>
</tr>
<tr>
<td>A1</td>
<td>1100</td>
<td>500</td>
</tr>
<tr>
<td>B1</td>
<td>1800</td>
<td>500</td>
</tr>
<tr>
<td>B2</td>
<td>1800</td>
<td>1700</td>
</tr>
</tbody>
</table>

Figure 8. Tension proof test results for A1 piles and photo of test setup

An old retaining wall of historical importance, shown in Figures 4 and 6, was protected and monitored during construction but showed no movement due to adjacent micropile foundation construction at A1 and drilled shaft construction at A2. Figure 9 shows a photo of the temporary trusses and towers supported on micropile foundations A, B1 and B2 on the north rock slope of YBI, during construction of the SAS bridge.
4. MICROPILE FOUNDATION FOR MAIN TOWER PULL-BACK CABLES

Due to the asymmetrical spans, installation of the main cable of the suspension bridge required the 160 m high main tower, T1, be pulled 500 mm to the west, using 200 m long pull-back cables inclined 40 degrees to the horizontal, prior to the erection of main cables. The pull-back cables were supported by a group of ten 18 m long micropiles inclined at the same angle of 40 degrees to the horizontal. Figure 10 shows the Tower T1 tie-back cable arrangement.

The micropiles were founded on bedrock at YBI. Nine of these micropiles were spaced at 1.5 m and one pile at the north end was spaced at 2.4 m. As shown in Figure 11, 203 mm diameter micropiles were used with 65 mm diameter center bars with yield stress of 724 MPa. The minimum bond length was 15.6 m and the remaining unbonded length in the upper part was sleeved using a PVC pipe. The micropiles were nominally pressure grouted using a neat mix of concrete with water-cement ratio of 0.45 to obtain minimum 28-day compressive strength of 35 MPa. The tensile capacity of the micropile was computed assuming a bond strength 480 kPa for the pile in contact with the sound Franciscan Formation bedrock. The group effect of the single row of ten piles was checked using the method proposed by Littlejohn and Bruce (1977). The unbonded length was selected considering the condition of bedrock in the upper part, group effect and the stability of adjacent slope.

Tension proof tests were conducted on all micropiles to a maximum load of 2050 kN, in general accordance with the FHWA (2005) guidelines. Piles were then locked off at a tension load of 1200 kN.
Figure 10. Plan (left) and elevation (right) views of Tower T1 tie-back cables

Figure 11. Testing/tensioning bearing pad and micropile foundation details
Prior to testing, there were concerns regarding the bearing capacity of overburden soils (fills) present above the bedrock due to lack of site-specific geotechnical information. Therefore, instead of testing all the micropiles to the full testing load, 1 m x 1 m square bearing pads were constructed at the locations of three selected micropiles, to assess the bearing capacity of the actual foundation material. The three micropiles chosen for testing were: Micropile 1 at the north end, Micropile 5 in the middle, and Micropile 10 at the south end. During application of tension test load on the micropile, both the elongation of micropile and the settlement of the 1 m x 1 m bearing pad were measured. The test results obtained for the foundation material indicated that the 1 m$^2$ pad at Micropile 10 was capable of sustaining the full test load of 2050 kN and the settlement of the pad was approximately 6 mm at full load. However, the 1 m$^2$ pad at Micropile 5 settled 25 mm at full test load and the 1 m$^2$ pad at Micropile 1 settled more than 25 mm at half of the full test load.

Based on these test results, it was decided to excavate the relatively weak fill material and use a continuous reinforced concrete pad (or pile cap) varying in thickness (from 0.45 m to 1.4 m) and width, as shown in Figure 11. The ground conditions at the south end were much stiffer than the conditions at the north end. Also, the northern most pile was located close to the slope. The thickness, width and stiffness of the pad were adjusted to suit the ground conditions, so that the pile cap would not undergo excessive displacements or failure during testing and in service.

Testing was subsequently conducted using the reinforced concrete pad as reaction base. During testing, both the elongation of the center bar and the settlement or deflection of the bearing pad were measured. The test results indicated that each of the micropiles sustained the test load with maximum displacement in the range of 19 mm to 28 mm and the corresponding settlement of bearing pad varied between 0.4 mm to 2.4 mm. All micropiles passed the ten-minute creep test (i.e. less than 1 mm) at the full test load of 2050 kN and the ten-minute creep varied between 0.3 mm and 0.6 mm.

Figure 12 shows photos of the micropile installation, the bearing pad used for testing and tensioning the micropiles, and a typical test arrangement. Figure 13 shows photos of the tie-back cables for tower T1 and the tie back cables supported on micropile foundation.
5. MICROPILES FOR WEST APPROACH DECK PULL-DOWN CABLES

The bridge decks of the west approach span on YBI needed to be pulled down approximately 75 mm during construction in order to make the hinge closure pour to complete the connection between the approach span and the west pier W2 of the SAS bridge. It was opted to use a tie-down system rather than adding weight to the approach span; the latter was initially attempted but subsequently abandoned. The weight required to deflect the bridge girder would have been quite large and would have introduced a seismic design issue; whereas a tie-down system would not.

The tie-down system consisted of two pairs of steel beams with ten tie-down cables on each beam. This arrangement was used for both the east line and west line bridge deck girders. Two rows of 20 m long vertical micropiles, unevenly spaced between 1.0 m and 1.8 m, were used as foundation for the tie-down cables. Hydraulic jacks were used to lower the bridge deck girder by pulling on the tie-down cable against the steel beam. One jack with capacity of 1780 kN was used at each cable. The cables were tensioned simultaneously until the deck was lowered by the required amount of 75 mm. Figure 14 shows the tie-down cable arrangement and the micropile foundation used for the east line bridge girders. The system used for west line girders was similar.

The micropiles were founded on bedrock at YBI. As shown in Figure 15, 165 mm diameter micropiles were used with 65 mm diameter center bars with yield stress of 1035 MPa. The minimum bond length in contact with the sound bedrock was 12 m, and the remaining 8 m unbonded length in the upper part was sleeved using a 76 mm Schedule 80 PVC pipe. The micropiles were nominally pressure grouted using a neat mix of concrete with water-cement ratio of 0.45 to obtain minimum 28-day compressive strength of 35 MPa. The tensile capacity of the micropile was computed using a bond strength of 480 kPa for the pile in contact with sound rock.

The group effect of the two rows of twenty piles (each row with ten piles at variable spacing and the rows separated by 1.22 m) was checked using the method proposed by Littlejohn and Bruce (1977). A factor of safety of two was used to calculate the required depth of the cone having friction angle of 55 degrees. To allow for group effect, the required unbonded length was taken as 8 m. The upper 3.5 m of the unbonded length of the micropile was cased using a 219 mm steel casing with 12.7 mm wall thickness and yield stress of 240 MPa. The cased part of the micropiles was predominantly in contact with surficial fill or intensely to moderately weathered bedrock.
Figure 14. Tie-down cables and tie-down beam arrangement

Figure 15. Typical micropile details and load test arrangements
Tension proof tests were conducted on all micropiles to a maximum load of 1650 kN in general accordance with the FHWA (2005) guidelines. The same steel beam and jack system was used to test the micropiles. Due to concern regarding the bearing capacity of the upper soils, each micropile was tested by using the adjacent micropiles as reaction piles, and the test arrangements used for testing inner and outer piles are shown in Figure 15. Utilizing the steel beam for testing eliminated the need to construct and later demolish a reinforced concrete beam for each set of tie-downs.

Each pile was loaded to maximum tension load of 1650 kN and followed by a ten-minute creep test. The maximum displacements ranged between 18 mm and 30 mm, and the corresponding residual displacements after unloading to alignment load ranged between 1 mm and 7 mm. The ten-minute creep varied between 0 and 0.97 mm. Figure 16 shows photos of the installed micropiles, and the test beam and typical load test arrangement.

Following testing, the same steel beams were then moved to the top of the approach span deck and the tie-down cables were tensioned until the required movement of the bridge deck was obtained. Figure 17 shows photos of the tie-down cables tensioned using the same test beams as used for testing the micropile foundations.
6. SUMMARY

During construction of the SAS bridge, micropiles were effectively used to solve three significant construction issues. Firstly, three of the temporary towers were located on the steep side slopes of Yerba Buena Island bedrock outcrop. The foundations had to carry both vertical and horizontal loads. Vertical and subhorizontal micropiles were selected as viable alternative to conventional piles, considering constructability, cost-effectiveness, schedule and environmental impact. All the piles were tested and performed well during construction of the bridge.

Secondly, the installation method used for the main suspension cable required the 160 m high main tower to be pulled 500 mm to the west using 200 m long pull-back cables anchored to bedrock at Yerba Buena Island prior to the erection of the main cable. 18 m long micropiles were chosen as the foundation for the tie-back cables inclined at 40 degrees. A reinforced concrete pile cap varying in thickness and width was used for testing the micropiles to maximum of 2 MN load and for locking them off at 1.2 MN before the tie-backs were attached. The micropiles performed well until erection of the bridge cables was complete.

Thirdly, the bridge decks from the west approach connecting to the west pier of the SAS bridge needed to be pulled down 75 mm, in order to make the hinge closure pour during construction. A tie-down system consisting of two pairs of steel beams with ten tie-down cables on each beam was used. Two rows of 20 m long vertical micropiles were used as a viable foundation for the tie-down cables. All micropiles were tested to maximum load of 1.7 MN, using the adjacent micropiles as reaction piles, without a pile cap, and using the same steel beams that were subsequently used for tying down the cables.

7. REFERENCES


