Quality Assurance Issues Related to the Installation of High Capacity Micropiles, Richmond-San Rafael Bridge Seismic Retrofit Project, California

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Abstract

Six Performance Piles and 458 long, high capacity production micropiles were installed in the existing foundations as part of the project to retrofit the Richmond-San Rafael Bridge. The Quality Control/Quality Assurance (QA/QC) measures associated with the installation of the micropiles included those required by the Contract as well as additional measures implemented by the Micropiling Contractor. The project was made especially challenging by the existence of only minimal pre-bid information on bedrock strength, poor quality and highly variable bedrock conditions along the bridge alignment, an unusual grout mix design, and discrepancies in load test data collected by load cell versus jacks. These challenges were addressed effectively through QA/QC procedures that included Working Drawings which tabulated information for each micropile, load test data reduction and analysis in the field, finite difference analysis of load test data, review of published literature relating to grout/rock shear strength, and supplemental test geotechnical test borings and laboratory analyses at select piers. These effectiveness of the QA/QC measures was optimized by rigorously implementing such measures at the beginning of the project.

Introduction

Quality Assurance and Quality Control (QA/QC) on the Richmond-San Rafael (RSR) Bridge Seismic Retrofit project was important during every step of the micropiling program, and probably more rigorous than typical, because of uncertainty in a number of key design and construction issues. These key issues included difficult and variable subsurface geologic conditions, grout mix design, performing load tests in a marine overwater environment, the amount of bond length required to achieve design load, and unknowns relating to the performance of unusually long micropiles when subjected to very high tension loads. The Micropiling Contractor added additional internal QA/QC procedures at the beginning of construction to supplement those required by the Contract. This entailed adding more administrative and technical field personnel and consultants to the project than typical to complete and review the micropiling QA/QC documentation, and performing supplemental geotechnical exploration and laboratory testing midway through the micropiling program to confirm the suitability of shear strength values applied to the bond zone. Ultimately, the extra effort expended in QA/QC documentation and analysis improved the quality of the construction process and the micropiles installed. Although this paper provides

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general "case history" information on the project, it is focused on the QA/QC and verification aspects which characterize this conference. Special attention is paid to these aspects in relation to site assessment, pile design, drilling and grouting processes, initial Performance Testing and subsequent routine Proof Testing.

Description of Bridge Structure

The Richmond-San Rafael (RSR) Bridge is one of seven State owned and operated toll bridges in Northern California undergoing a major seismic retrofit (Figure 1). The bridge is a 5.5-mile-long cantilever and truss structure that spans the northern end of San Francisco Bay between Contra Costa and Marin Counties on California Interstate I-580. It opened in September 1956 and currently is used by about 56,000 motorists a day. The bridge was selected for retrofitting because of its high volume of use and close proximity to the San Andreas and Hayward faults. Both faults are within 10 miles of the RSR Bridge alignment and are believed to be capable of generating earthquakes of magnitude 7.5 or greater. In 1999, the original budget for retrofitting all seven Bay Area toll bridges was $2.6 billion of which $484 million was assigned to the RSR Bridge. The budget for the RSR Bridge has almost doubled since that time to an estimated $914 million in August 2004.

Figure 1. Richmond-San Rafael Bridge looking west across the San Francisco Bay toward Marine County, California.

The bridge's west approach is a concrete trestle supported on 37 bents. Moving east along the bridge, the trestle transitions to a double decked steel truss which constitutes the majority of the bridge alignment which is supported on 78 piers. Each pier is comprised of either two or four columns that are connected by concrete spandrel beams and diaphragm walls (Figure 2). Each column is founded on a precast concrete bell that is supported by driven HP 14x89 steel piles. The number of

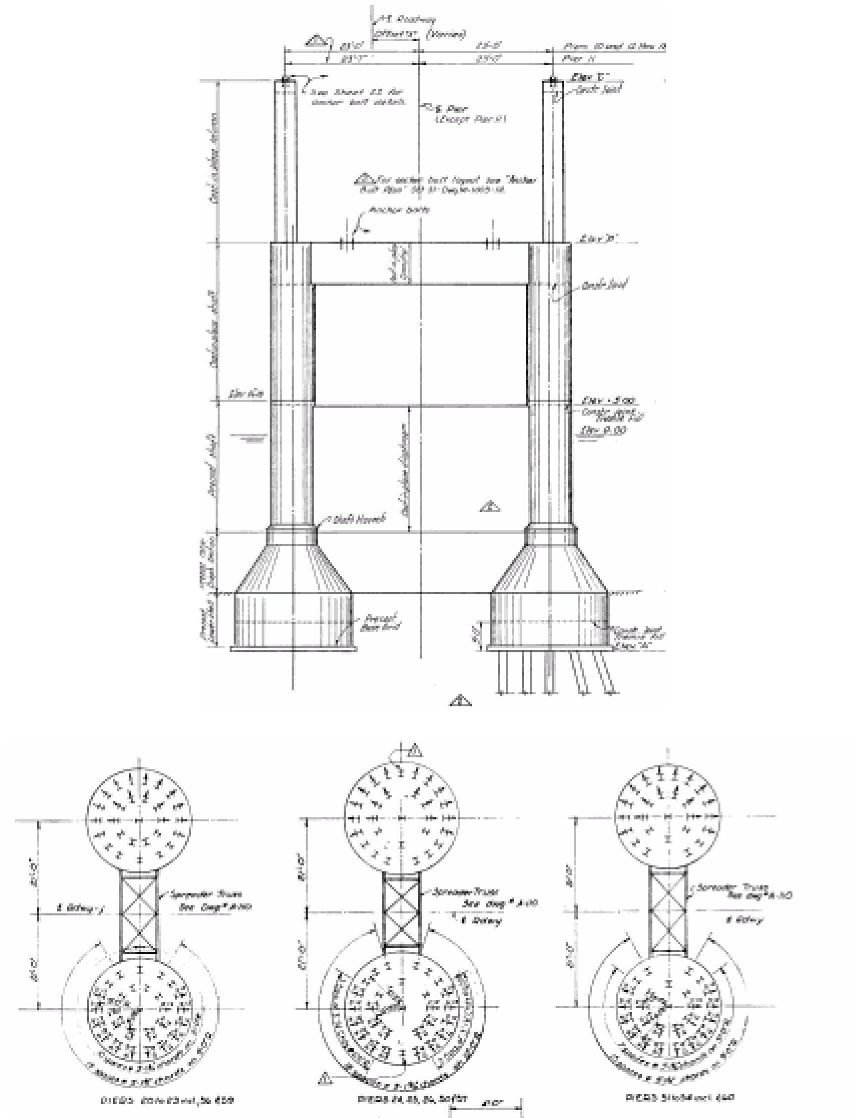

Figure 2. Typical profile view of two column pier with belled foundation and typical plan view of H-pile layout in driving template at base of bell (source: Caltrans, 1953 Richmond-San Rafael Bridge as-built drawings).

steel piles supporting each precast bell ranges from 9 to 33 (typically two rows of H-piles per bell, both battered and vertical) with lengths ranging from 13 feet to 160 feet. The tops of the H-piles extend about 3 to 6 feet up inside the precast bells and the piles tip-out in Franciscan Formation bedrock. The prescribed micropile design required that micropiles be installed between the existing H-piles and that no existing H-piles be encountered either above or below the concrete bell during concrete coring or micropile drilling activities in order to maintain the integrity the original bridge foundation support. Only piers of two column construction were retrofitted with micropiles. The four column piers were retrofitted with 66 inch diameter cast-in-drilled-hole (CIDH) and 150 inch diameter cast-in-steel-shell (CISS) piles.

Work Scope

The work scope included retrofitting 31 marine piers with high capacity marine micropiles. Micropiling work began in July 2001 and was completed in February 2004. During that time, a total of 6 Performance Piles and 458 production micropiles were installed in an over-water marine environment using floating work platforms secured to driven pipe piles and the existing structure (Figure 3). The work scope included coring a 14 inch diameter hole through the existing concrete bell at each

Figure 3. Pier 57 Production Pile work platform. The boat in foreground is docked in front of the metal frame with buoyancy tank that made up the majority of the work platform structure. The work platforms were pinned-off to 24 inch diameter driven pipe piles (shown in the background) at the highest tide to minimize the influence of tidal changes on platform motion. The pier's concrete columns are shown at the center of the picture.

micropile location and then advancing a 12 inch diameter permanent conductor casing through the corehole and into unconsolidated alluvium until bedrock was encountered. The coring, drilling and micropile installations where facilitated by steel templates at each bell which indicated the location of each micropile and provided conductor casings down to the top of the concrete bell. The templates consisted of two cylindrical steel tanks connected by 20 inch diameter conductor casings that served to guide the drill string from platform deck level near mean sea level to the top of the concrete bell. The lower tank was flooded to help keep it in place and the upper tank was left empty for buoyancy. Circular access portholes were created where the 20 inch diameter conductor casings intersected the metal cylinders (Figure 4). A total of four different templates were required to accommodate the different combinations of bell/column diameters, H-pile layouts, and number of micropiles required at each bell. Originally, it was intended that the templates would

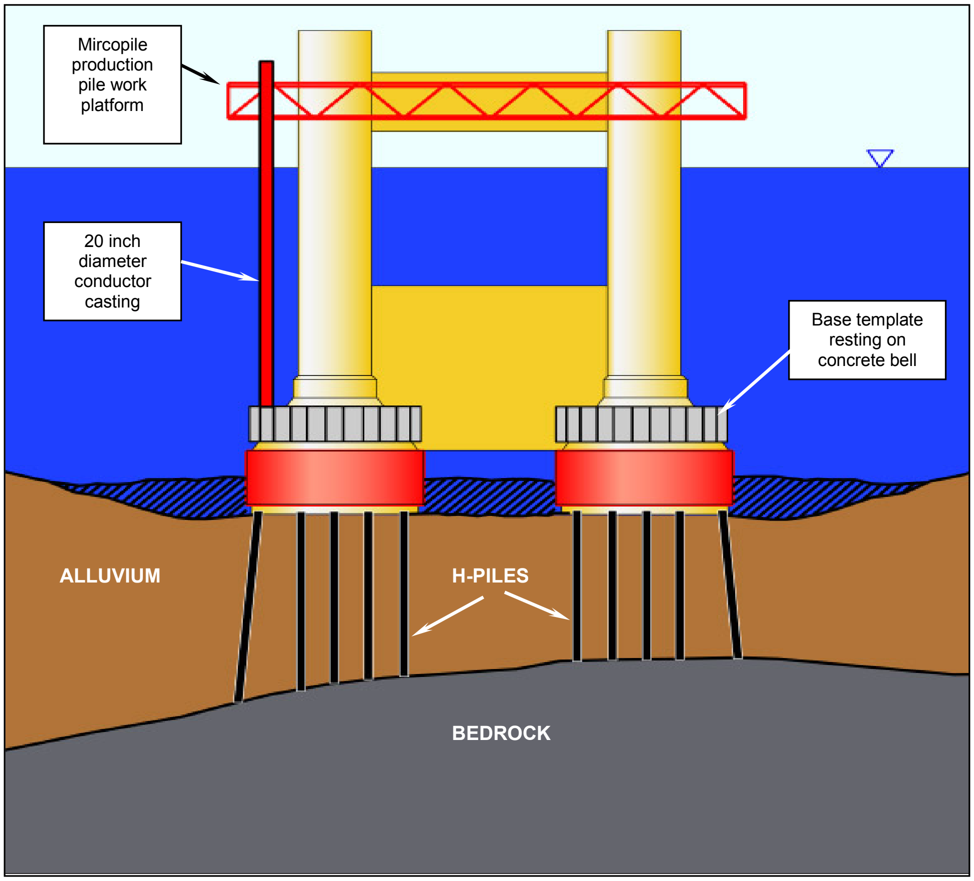

Figure 4. Schematic of micropile template resting on top of concrete bell with 20 inch diameter conductor casings extending up to deck level on the Production Pile work platform.

rest upon the upper inclined surface of the concrete bells but because of bells/column/diaphragm misalignment at most of the piers, it was more effective to hang the templates from the concrete columns at the work platform level. In either instance, the templates were positioned via an engineering survey that tied the template location to an underwater H-pile survey performed at the bottom of the bell. The theoretical horizontal clearance between the H-piles and coring equipment was about 3 inches horizontal distance as measured from the perimeter of the corehole. Roughly 40% of the micropiles were installed between existing vertical H-piles. Installations between vertical piles was of importance to the Owner in that undermining of existing H-pile tips (and foundation support) was to be avoided. Thus, the 12 inch diameter micropile casing tips between vertical H-piles were seated 10 feet below H-pile tips. These were referred to as “type B” micropiles in the Contract Documents. The remaining 60% of the piles were installed between battered piles. The Contract required shallower casing seatings for these piles, specifically a minimum of "zero feet and zero inch" embedment into the Franciscan Formation. These were referred to as “type A” micropiles in the Contract Documents.

The micropiles consisted of 8 inch diameter steel pipe reinforcement with wall thickness of either 1 inch or 0.875 inch, and about 12 feet of shear rings at the top and bottom of the reinforcement. Reinforcement was delivered to the work platforms full length whenever possible with welds completed on shore. In some instances where low overhead clearance was an issue, the reinforcing steel was delivered to the platform in sections and welded over the hole. At piers A through 10 on the west side of the bridge, the low overhead restrictions were quite severe and resulted in all micropiles needing about 8 welds. The Micropiling Contractor proposed to substitute welded 8 inch reinforcement piles with a structurally equivalent system of No. 20 All Thread Bars (ATB's) connected by nuts which the Owner agreed to. Micropiles were grouted in two stages. The top of first stage grout was placed at 10 feet below the bottom of the bell immediately after placing the 8 inch or ATB reinforcement in the drill hole. Upon completion of all load tests at a pier, the second stage grout was placed up to the top of the bell.

The design working loads (DWL's) for the production piles ranged from 290 to 1,140 kips (Table 1). Production piles were proof tested to 100% of the DWL while Performance Piles were tested to 120% of DWL. The Contract required that two production micropiles per pier be proof tested, resulting in a total of 62 proof tests for the project. Five Performance Piles were identified in the Contract, each corresponding to one of five geologic zones along the bridge alignment. Prior to beginning production micropiling work in any of the five geologic zones, the Contractor first had to install and test a Performance Pile that satisfied the pass/fail elongation criterion established by the Owner. This criterion for both Proof and Performance Piles was based on a "not to exceed" total vertical movement, e, calculated from:

$$e = \frac{PL_1}{AE} + 0.33\,\frac{PL_2}{AE} + 0.12\,D$$

where e = total vertical movement or elongation of Performance or Proof Pile

P = test load applied to Performance or Proof Pile

L1= free length of micropile reinforcement (top of 8 inch or ATB reinforcement to tip of 12 inch permanent casing)

L2= bonded length of micropile reinforcement (tip of 12 inch permanent casing to tip of 8 inch reinforcement or ATB)

A = steel reinforcement cross sectional area

E = elastic (Young's) modulus for steel reinforcement

D = micropile diameter

Table 1. Micropile Schedule from Bid Documents

Pier	Geologic Control Zone	Reinforcement Outside Diameter (in)	Reinforcement Wall Thickness (in)	No. Micropiles per Bell (ea)	Design Load Compression (kips)	Design Load Tension (kips)	Tension Test Loading (kips)	Max Allowable TVM/Performance Pile TVM (in)	Approx. Bedrock (ft)	Miniumn Length Bedrock Embedment (ft)	Top of Reinforcement Pipe (elev. ft)	Tension Test Loading	Tension Test Loading
Pier A-N	A	8.625	0.875	8	520	600	600	0.55	-41	43	-18.25	600	0.55
Pier A-S	A	8.625	0.875	8	520	600	600	0.53	-39	44	-18.25	600	0.53
Pier I-N	A	8.625	0.875	8	520	600	600	0.57	-43	43	-18.25	600	0.57
Pier 1-S	A	8.625	0.875	8	520	600	600	0.59	-45	42	-18.25	600	0.59
Pier 2-N	A	8.625	0.875	8	520	600	600	0.71	-56	39	-18.25	600	0.71
Pier 2-S	A	8.625	0.875	8	520	600	600	0.71/0.91	-56	39	-18.25	600	0.71
Pier 3-N	A	8.625	0.875	6	460	360	460	0.74	-69	29	-18.25	460	0.74
Pier 3-S	A	8.625	0.875	6	460	360	460	0.74	-69	29	-18.25	460	0.74
Pier 4-N	A	8.625	0.875	6	460	360	460	0.81	-87	26	-18.25	460	0.81
Pier 4-S	A	8.625	0.875	6	460	360	460	0.82	-88	26	-18.25	460	0.82
Pier 5-N	A	8.625	0.875	6	460	360	460	0.97	-105	23	-18.25	460	0.97
Pier 5-S	A	8.625	0.875	6	290	360	360	0.97	-105	23	-18.25	460	0.97
Pier 6-N	B	8.625	0.875	4	290	180	290	0.71	-120	12	-20.25	290	0.71
Pier 6-S	B	8.625	0.875	4	290	180	290	0.73	-123	12	-20.25	290	0.73
Pier 7-N	B	8.625	0.875	4	290	180	290	0.73	-124	12	-20.25	290	0.73
Pier 7-S	B	8.625	0.875	4	290	180	290	0.72	-122	12	-20.25	290	0.72
Pier 8-N	B	8.625	0.875	6	540	480	540	1.07	-102	22	-20.25	540	1.07
Pier 8-S	B	8.625	0.875	6	540	480	540	0.99	-82	24	-20.25	540	0.99
Pier 9-N	B	8.625	0.875	6	540	480	540	1.04	-99	22	-20.25	540	1.04
Pier 9-S	B	8.625	0.875	6	540	480	540	0.99	-92	23	-20.25	540	0.99
Pier 10-N	B	8.625	0.875	6	560	480	560	1.37	-128	19	-22.25	560	1.37
Pier 10-S	B	8.625	0.875	6	560	480	560	1.15	-97	23	-22.25	560	1.15
Pier 11-N	B	8.625	0.875	6	550	460	550	1.29	-121	20	-22.25	550	1.29
Pier 11-S	B	8.625	0.875	6	550	460	550	1.29	-104	21	-22.25	550	1.29
Pier 12-N	B	8.625	0.875	6	540	480	540	1.1	-106	21	-22.25	540	1.10
Pier 12-S	B	8.625	0.875	6	540	480	540	0.99	-96	23	-22.25	540	0.99
Pier 13-N	B	8.625	0.875	6	560	480	560	1.15	-106	22	-24.25	560	1.15
Pier 13-S	B	8.625	0.875	6	560	480	560	1.15	-100	22	-24.25	560	1.15
Pier 14-N	B	8.625	0.875	6	560	480	560	1.18	-112	21	-24.25	560	1.18
Pier 14-S	B	8.625	0.875	6	560	480	560	1.15	-107	22	-24.25	560	1.15
Pier 15-N	B	8.625	0.875	6	560	480	560	1.24	-118	20	-24.25	560	1.24
Pier 15-S	B	8.625	0.875	6	560	480	560	1.18	-112	21	-24.25	560	1.18
Pier 16-N	B	8.625	0.875	6	550	460	550	1.31	-128	19	-26.25	550	1.31

(continues)

Table 1. Micropile Schedule from Bid Documents

Pier	Geologic Control Zone	Reinforce-ment Outside Diameter	Reinforce-ment Wall Thickness	No. Micropiles per Bell	Design Load Compression	Design Load Tension	Tension Test Loading	Max Allowable TVM/Perform-ance Pile TVM	Approx. Bedrock	Miniumn Length Bedrock Embedment	Top of Reinforce-ment Pipe	Tension Test Loading	Tension Test Loading
		(in)	(in)	(ea)	(kips)	(kips)	(kips)	(in)	(ft)	(ft)	(elev. ft)		
Pier 16-S	B	8.625	0.875	6	550	460	550	1.29	-125	20	-26.25	550	1.29
Pier 17-N	B	8.625	0.875	6	530	460	530	1.41	-142	18	-26.25	530	1.41
Pier 17-S	B	8.625	0.875	6	530	460	530	1.41	-136	19	-26.25	530	1.41
Pier 18-N	B	8.625	0.875	6	530	460	530	1.51	-152	17	-26.25	530	1.51
Pier 18-S	B	8.625	0.875	6	530	460	530	1.41	-141	18	-26.25	530	1.41
Pier 39-N	C	8.625	1.000	16	1,010	900	1,010	0.89/1.13	-70	55	-44.25	1010	0.89
Pier 39-S	C	8.625	1.000	16	1,010	900	1,010	0.92	-72	55	-44.25	1010	0.92
Pier 40-N	C	8.625	1.000	16	1,010	900	1,010	1.36	-96	48	-42.25	1010	1.36
Pier 40-S	C	8.625	1.000	16	1,010	900	1,010	0.97	-73	54	-42.25	1010	0.97
Pier 50-N	D	8.625	1.000	14	1,140	960	1,140	1.43	-89	51	-42.25	1140	1.43
Pier 50-S	D	8.625	1.000	14	1,140	960	1,140	1.49	-92	50	-42.25	1140	1.49
Pier 51-N	D	8.625	1.000	6	770	530	770	0.99	-75	30	-30.25	770	0.99
Pier 51-S	D	8.625	1.000	6	770	530	770	1	-76	29	-30.25	770	1.00
Pier 52-N	D	8.625	1.000	6	770	530	770	1.08/1.35	-63	28	-12.25	770	1.08
Pier 52-S	D	8.625	1.000	6	770	530	770	1.02	-59	29	-12.25	770	1.02
Pier 53-N	D	8.625	1.000	6	770	530	770	1.12	-62	28	-8.25	770	1.12
Pier 53-S	D	8.625	1.000	6	770	530	770	1.11	-61	28	-8.25	770	1.11
Pier 54-N	D	8.625	1.000	6	770	530	770	0.96	-49	30	-8.25	770	0.96
Pier 54-S	D	8.625	1.000	6	770	530	770	0.96	-47	31	-8.25	770	0.96
Pier 56-N	E	8.625	1.000	10	1,100	820	1,100	0.99	-39	47	-12.25	1100	0.10
Pier 56-S	E	8.625	1.000	10	1,100	820	1,100	0.91	-35	49	-12.25	1100	0.91
Pier 57-N	E	8.625	1.000	14	1,070	800	1,070	0.95	-55	52	-28.25	1070	0.95
Pier 57-S	E	8.625	1.000	14	1,070	800	1,070	0.95	-55	52	-28.25	1070	0.95
Pier 58-N	E	8.625	1.000	16	840	620	840	0.85/10.9	-78	45	-44.25	840	0.85
Pier 58-S	E	8.625	1.000	16	840	620	840	0.85	-78	45	-44.25	840	0.85
Pier 59-N	E	8.625	0.875	6	770	530	770	0.97	-74	30	-30.25	770	0.97
Pier 59-S	E	8.625	0.875	6	770	530	770	1.02	-77	29	-30.25	770	1.02
Pier 60-N	E	8.625	0.875	6	770	530	770	0.99	-69	30	-24.25	770	0.99
Pier 60-S	E	8.625	0.875	6	770	530	770	1.06	-74	28	-24.25	770	1.06

The average Contract free length of the “type A” Production Micropiles was 64 feet (range of 21 feet to 128 feet). For “type B” piles the average Contract free length was 54 feet (range of 11 feet to 118 feet). The Contract micropile bond lengths for “type A” micropiles averaged 30 feet and ranged from 12 to 55 feet. Contract bond lengths for “type B” micropiles averaged 21 feet and ranged from 10 feet to 45 feet. The maximum allowable total vertical movement for Performance and Proof Piles was re-calculated using as-built data prior to load testing. The recalculated total vertical movement was then used as the pass/fail criterion for the load tests.

In addition to the 463 micropiles installed overwater, a further 282 micropiles of slightly different design were installed primarily on land at the east abutment of the bridge using different equipment and installation methods. These piles are not discussed in this paper.

Subsurface Conditions

The subsurface profile along the alignment of the bridge typically consists of soft Bay Mud and other unconsolidated sedimentary deposits overlying Franciscan Formation bedrock. The Franciscan Formation is a *mélange* or tectonic unit produced by fragmenting and mixing of several rock types originally deposited in an offshore, deep-marine trough and then faulted into place. The Franciscan Formation is dominated by massive graywacke sandstone with lesser amounts of shale, chert, limestone, volcanic, and metamorphic rocks, all of which are intruded by serpentine. Along the bridge alignment, the depth to bedrock varies from about 15 to 40 feet below mean sea level at each end of the bridge to about 300 feet below mean sea level in the deepest portion of the bay. Typically, the depth to bedrock is less than 200 feet below mean sea level along most of the bridge alignment. Overlying soft/loose Quaternary sediments are absent near the east abutment and reach a maximum thickness of about 300 feet near the center of the bridge.

The nature of the Franciscan Formation bedrock encountered in micropile rock sockets consisted primarily of graywacke sandstone some shale, and lesser amounts of chert. Generally, the Franciscan Formation was of highly variable quality in terms of RQD, weathering, and strength. The majority of geotechnical test borings drilled in support of this project encountered rock with RQD’s significantly less than 50% and roughly half of the Franciscan formation encountered by the Micropiling Contractor was classified as an intermediate geomaterial (that is, an earth material transitional between soil and rock). The weak nature of the rock can be attributed to both mechanical weathering/shearing associated with fault movement as well as profound chemical weathering that may be related to paleoclimate changes in sea level. Compounding the issues of deeply sheared and/or decomposed Franciscan Formation is the fact that bedding and shear plane contacts are near vertical. This resulted in unpredictable and extreme variations in rock quality over relatively small horizontal distances (such as between adjacent micropiles). In addition, encountering a vertically dipping bed of poor quality rock during drilling typically indicated that one would remain in poor quality rock to the bottom of the rock socket simply based on the bedding geometry and thickness. The low strength intermediate geomaterial encountered was prone to caving which made it difficult to seat and seal micropile casings, achieve the Owner's micropile design load by specified tip elevation, and maintain an open rock socket through installation of steel reinforcement and grouting.

Subsurface lithology along the bridge alignment was determined from three different geotechnical investigations performed in the 1950's, 1995, and 2001. The subsurface conditions described in the bid documents were determined from borings drilled in the 1950's in support of the original bridge construction and from borings drilled in 1995/1996 by the Owner. The 1950 borings did not penetrate into bedrock more than a nominal depth and therefore essentially provided information only on the sediments overlying the Franciscan Formation. The 1995/1996 borings drilled by the Owner did penetrate bedrock but were widely spaced and extended well beyond the micropile specified tip elevations indicated in the Contract. Three of the 18 borings drilled along the 4.5 miles of bridge alignment were adjacent to piers scheduled for retrofit and the other borings were located at piers scheduled for CISS or CIDH piles. The three borings pertinent to micropiled piers extended on average 100 feet into Franciscan Formation. It is understood that the reason the depth of the 1996/1996 borings was to install accelerometers. No unconfined compressive strength tests were performed on rock samples collected from the 1995/1996 borings. It is understood that downhole pressure meter tests were attempted in some of the 1995/1996 borings but no reliable data were collected because of the poor condition of the rock. In 2001, 71 post-contract borings drilled by the General Contractor as required by the Bid Documents at locations where CIDH piles, CISS piles, and micropiles were proposed. At each pier scheduled for micropile retrofit, one boring was drilled on the north side of the pier within 5 feet of the footprint of the bell foundation and in line with the center line of the bell/diaphragm structure. The post-contact borings were subsequently used by the Contractor to prepare the Micropile Work Drawings. Following approval of the Working Drawings by the Owner, micropile construction commenced beginning with the installation of Performance Piles. The number of Performance Piles installed on the project is indicative of the complex site geology encountered along the RSR alignment.

Micropile Working Drawings

The Working Drawings for micropiling construction procedures covered drilling, spoils and slurry containment, grouting, Performance Pile platforms and load frames, Proof Pile testing, production piling work platforms, and micropiling templates. The Drawings provided tabulated elevations for top and bottom of existing concrete bell, steel reinforcement, 12 inch casing, and rock socket. Figure 5 illustrates the complexity and detail involved in a typical pier. In addition, the Drawings showed top of bedrock topographic contours based on H-pile tip data, a schematic of subsurface conditions based on the post-contract Log of Test Boring, and a plan view of the north and south bells with the location and identification number of each micropile. A separate piling drawing was done for each pier. Because of variations in bedrock topography beneath each bell, essentially every micropile had a unique geometry in terms of elevations of steel reinforcement and rock socket tip. In the

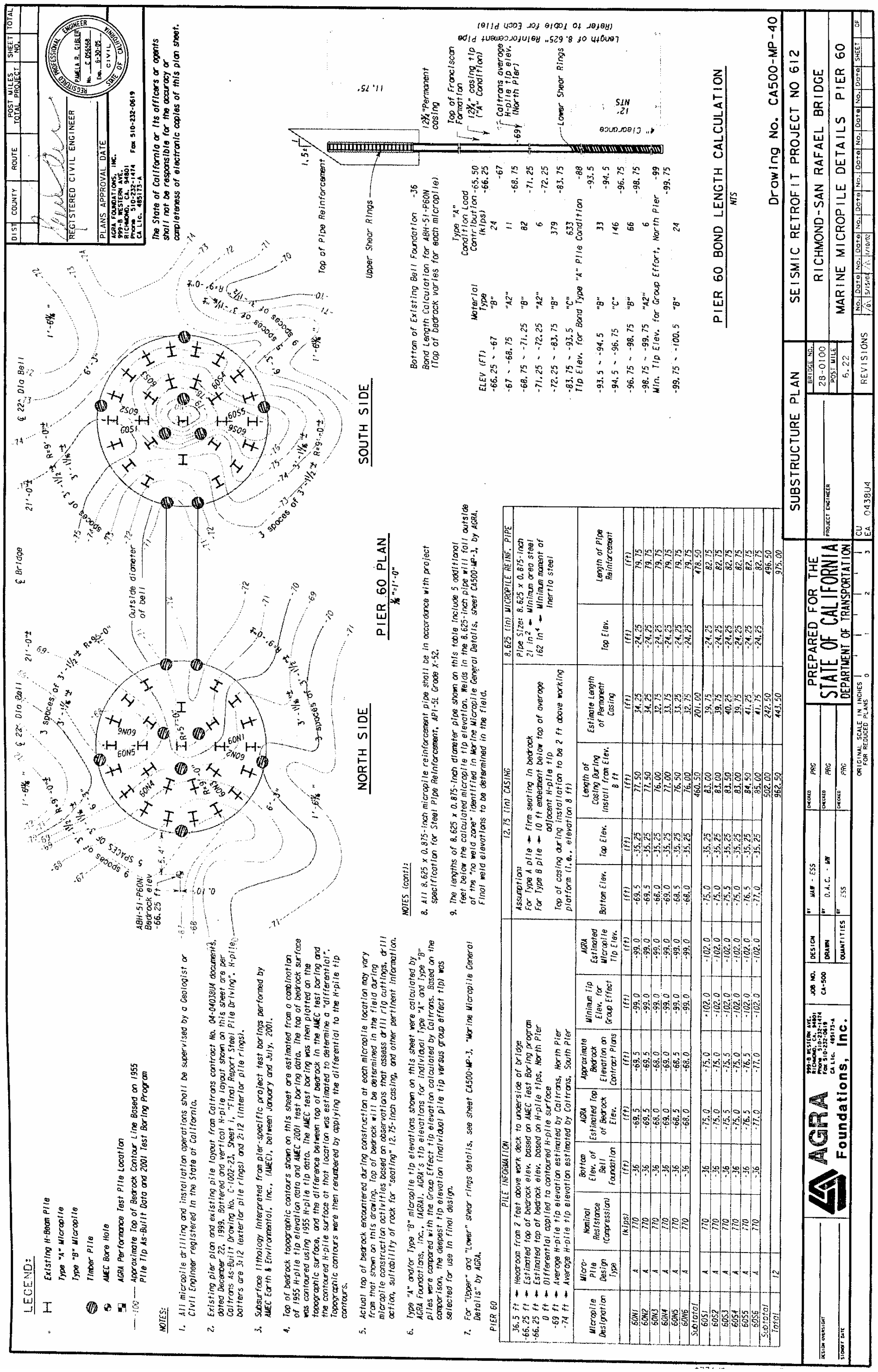

Figure 5. Typical Working Drawing: Pier 60.

field, the Micropiling Contractor used the approved Working Drawings as an estimate of micropile geometry. However, the actual subsurface conditions encountered ultimately dictated the as-built geometry of each micropile. For example, variations in where competent bedrock was encountered influenced where the 12 inch casing was seated while variations in rock quality influenced the length of the rock socket and also the length of the 8 inch or ATB reinforcement. The Working Drawings were useful in providing fairly accurate target elevations for anticipated construction. At the same time, the Micropiling Contractor had the latitude to modify the Working Drawing design for any of the micropiles based on actual subsurface conditions encountered.

Performance Micropile Installation and Testing

Performance Pile installation and testing is a standard component of a major, complex micropiling project and is undertaken to demonstrate or investigate the quality and adequacy of the design, the materials, and the construction. Performance piles are installed and tested in advance of the production piles to provide an opportunity to verify pile capacity, establish load-deformation behavior, identify causes of pile movement, and verify the appropriateness of the assumed rock shear strength used in design. Performance Pile testing is especially important when unusual ground conditions are encountered in the bond zone, where difficulties are anticipated in drilling and grouting, and where piles are unusually long. The Post Tensioning Institute and Federal Highway Administration guidance recommend that cyclical incremental load testing be conducted on Performance Piles (PTI, 1996, and FHWA, 1999). In instances where geotechnical data are minimal or absent, it is not uncommon to test Performance Piles to failure (i.e., begin to pull the pile out of the ground) for the purpose of obtaining the ultimate shear strength of the rock/ground interface in the bond zone. Such piles also allow the Contractor to demonstrate and optimize his specific means and methods.

The load test system used for Performance load tests (as well as Proof load tests) on the RSR project consisted of a frame equipped with a horizontal reaction beam and two 600 kip jacks at each end of the beam (Figure 6). The reaction beam had a vertical port at its center through which the pull collar on the micropile was positioned. A load cell was secured at the top of the pull collar so that the base of the load cell was flush with the top of the reaction beam. Initially, one extensometer was installed to measure total vertical movement of the micropile. Later in the testing program, an additional extensometer was included in each load test so that a total of two extensometers recorded elongation data. In addition, between 4 and 8 dial gauges were installed on the test frame, jacks, micropile, and reaction beam. The calibrated load cell and extensometers were provided by the Owner. The load test frame and setup, calibrated jacks, pressure gauges, and dial gauges were provided by the Micropiling Contractor.

All Performance Piles were tested in tension up to a maximum load of 120% of DWL using one cycle of loading. Incremental cyclical test loading was not permitted and no Performance Piles were tested to failure. The maximum Performance Pile test load was 1,212 kips at Pier 39. The maximum total vertical

Figure 6. Performance Pile Load Test Setup

movement pass/fail criterion was applied to both Performance and Proof Pile load test results.

Grouting

The Contract required a grout mix with six components (water, cement, fly ash, silica fume, super plasticizer, and retarder), a water to cement ratio (W/C) of 0.35, a Marsh Funnel flow time of between 15 to 35 seconds (water = 9 seconds), and a 28 day strength of 6,000 psi. Prior to installing the first micropile, mixing and testing of trial batches was undertaken to define a grout mix which satisfied the specification as well as to assess potential grout-related constructability issues that might arise during pile installation. For comparison purposes, the Micropiling Contractor also mixed and tested a standard grout consisting of only cement and water with a W/C of 0.45 with the intent of convincing the Owner to use a more conventional mix. During the trial batching, the Contract grout exhibited unusual properties among which were high viscosity, erratic cube appearance and compressive strength results, and lack of temperature increase up to 6 hours after commencing mixing.

Despite the concerns of working with a grout that appeared to be experimental in nature, the prescribed grout mix was used in construction. Recognizing that the grout mix was highly sensitive to very small variations in the six components that make up the mix, the Micropiling Contractor was exceptionally careful that the high shear grout plants used in construction were clean of solids and liquids prior to mixing, water calibration gauges on the grout plants were accurately set, fresh water used in the mix was of moderate temperature, the first batch of grout was always wasted to purge the grout plant of impurities, weathered or damaged bags of cement were not used in the mix, and a visual check of the grout was made prior to placement to identify if obvious signs of lumping and segregation were present. Still, problems with grout thixotropy were experienced when either the mix water, ambient air, or bagged cement were too hot or too cold. Detailed notes were taken on the foregoing and the Owner was notified of the problem in real time. Had any of the micropiles failed during proof testing, the data documenting grouting problems were available to assess and correlate to the test failure. In addition to the internal QA/QC described, measurements of grout flowability and unit weight were taken for each micropile at the beginning, middle and end of first stage grouting as required by the Contract. Three sets of cube samples were also collected at these intervals for unconfined compressive strength testing. All tested cube samples met or exceeded the specified 28 day strength. Post grouting was attempted on an experimental basis twice but failed because of the rapid strength gain in the placed grout and the strong lateral confinement of the Franciscan Formation.

Documentation of Bedrock Conditions

The QA/QC specifications in the Contract required that a Drilling Operation Log be prepared for each micropile. The Drilling Logs were to include the elevation of first bedrock encounter, tabulated drill rates, and classification of the drill cuttings before and after rock was encountered. The Contract also required that the Drilling Logs carry a stamp of a Registered Geologist or Engineer prior to submittal to the Owner for review. The Micropiling Contractor was free to develop a format that suited his purposes as long as the foregoing were included in the Operation Logs.

To complete the Drilling Logs, the Micropiling Contractor assigned a field engineer or field geologist to each platform. Because the Micropiling Contractor was installing micropiles on up to three piers 24 hour per day, a total of six geologists/engineers were employed to document construction activities. The Drilling Logs documented lithology encounters, drill rates, drill action, quality of Franciscan Formation, and encounters of H-piles, wood piles, tree logs, and scour protection. The Logs included elevations of the top, bottom, and thickness of lithologic layers, top of Franciscan Formation, top of rock suitable for seating the 12 inch casing so as to effect both seat and seal, working shear load per layer, and cumulative shear load with depth. Cuttings were described typically on 0.5 to 10 foot intervals (depending on drill rate) and included percent sand/silt/gravel, grain size, color, Unified Soil Classification symbol, rock type, degree of weathering, and degree of fracturing. This information was assessed in real time to determine where rock suitable for seating and sealing the 12 inch casing was encountered and assign shear strength values to variable quality lithology encountered in the rock socket, as shown in Figure 5.

All samples of cuttings were bagged and labeled with micropile number, date, time, sample elevation, and initials of the sampler, and the cuttings were stockpiled on-shore when micropiling at a pier was completed. Cuttings samples and Logs were reviewed daily by the Micropiling Contractor's Registered Geologist prior to forwarding the logs to the State. If casings were artificially seated by plunging the casing tip into a shallow grout volume to effect a seat and seal, a Log was made of the lithology and elevations drilled, volume of grout placed, and elevation of artificial seating. The majority of the piles had both a bond zone and steel reinforcement length that was unique to each pile as well as longer than the specified minimum length of bedrock embedment and reinforcement length used for bidding purposes. A consequence of non-uniform pile lengths was that reinforcing steel could not be manufactured in advance of drilling since it was never known at what elevation the design load would be achieved until drilling of the micropile rock socket was completed.

In addition to completing the Micropile Drilling Operation Logs which mainly pertained to documenting top of bedrock, casing seating elevation, and cumulative shear load, the field engineers/geologists were also responsible for recording all other construction activities. This included concrete bell core logging to identify the elevations at which degraded concrete and loose rebar were encountered; H-pile and wood pile encounters; grout QA/QC and volumes placed; photographs of recovered core; heat numbers for welded piles; and quantities of equipment damaged or destroyed during coring and drilling activities with photographs as appropriate.

Selection of Grout/Rock Ultimate Shear Values

Only three of 18 pre-bid geotechnical borings drilled in 1995/1996 were drilled near piers scheduled for micropiling retrofit (Piers 2, 7, and 59). Of the three borings pertinent to micropiling, all were located at least 100 feet from the footprint of the belled foundations. No geotechnical laboratory tests were performed on Franciscan Formation samples from these borings, and downhole pressure meter testing in the test bores was not successful. As a result, site-specific rock strength data were not available to evaluate the shear strength of the grout/rock interface in the micropile bond zone. Lacking such site-specific data, the Micropiling Contractor undertook a lengthy review of published literature to develop the ultimate shear values used on the RSR project. No specified factors of safety (FS) or other design data on the bond zone was provided prebid in Contract and so a FS of between 2 and 3 was used to calculate working shear stress values.

Using the foregoing and assuming the Franciscan Formation could be divided into four general types of lithology based on weathering, fracturing, and RQD, the Micropiling Contractor developed working shear values to assign to earth materials encountered in the bond zone (Table 2). Applying these values to the lithologic information in the 2001 post-construction LOTB's, it was anticipated that typical, average working shear values for the project would range from 29 psi to 62 psi. The Micropiling Contractor assumed that these shear values would be field-verified in the Performance Pile load tests either through incremental cyclical loading or loading Performance Piles to failure. However, this was not the case as Performance Piles were subjected to only one cycle of loading (i.e., load pile to maximum load in 10 incremental steps and then decrease the load to zero in 4 decremental steps). In the end, the assumed working shear values were used to track accumulated load in all micropile rock sockets. Based on the evaluating Performance and Proof Pile testing data collected during the RSR project, the overall working stress was estimated to range from 33 psi to 88 psi, averaging 61 psi at test load.

Table 2. Micropile Rock Socket Working Shear Values Based on Literature Review

Earth Material Type	General Description	Working Shear Strength (psi)
Lithology "A1"	Very soft, decomposed or highly sheared rock, unconfined, encountered at top of bedrock surface	0
Lithology "A2"	Very soft, decomposed or highly sheared rock, confined, encountered below top of bedrock surface	14
Lithology "B"	Medium rock, moderately to highly fractured, moderately to highly weathered.	73
Lithology "C"	Hard rock, predominantly fresh, slightly fractured	144

Supplemental Geotechnical Test Borings

Two supplemental geotechnical test borings were made at the Pier 40 South Bell in June 2003 as part of a QA/QC program to confirm the working shear stress values for problematic, interbedded, decomposed graywacke and chert. The two south bell borings were in addition to the 2001 post-bid geotechnical boring drilled at the Pier 40 north bell by the Contractor. At Pier 40, there was a 23 foot difference between the top of bedrock at the north and south bells, and at the onset of the project it was suspected that the elevation difference was due to lithology variations between the bells. The two supplemental borings were drilled at micropile locations 40S06 and 40S12. The borings were advanced and logged using a CP/HQ wireline system and a Diedrich D-120 drill rig, the same equipment implemented to complete all of the 2001 post-bid Contract borings. Unconsolidated-undrained triaxial testing was conducted per ASTM D2850-95 on a total of 10 samples, nine of which were soft decomposed greywacke, and one of which was clay with chert. The samples used in triaxial testing were collected between elevations -114 and -144 in both borings. Point load test samples were selected from the remaining footage of recovered material. These included testing of “irregular lumps”, and cylinder-shaped diametral samples. Some of these samples deformed plastically during point load testing because of their extremely soft/decomposed condition, and so were eliminated from consideration. Point load tests were performed on irregular lump samples collected between elevation -155 and -178 feet and on diametral samples between elevation -120 and -173 feet. The triaxial test results showed that working shear could range from 7 psi to 25 psi, average 17 psi. Point load test results for irregular lumps showed that the working shear could range from 5 psi to 47 psi, average 23 psi. Results for diametral samples were not used because the majority of these samples preferentially broke long fractures or bedding planes, thereby resulting in artificially low unconfined compressive strengths. Ultimate shear values were measured directly from the triaxial tests. Ultimate shear values from point load test data were calculated as 10% of ultimate strength values determined from the laboratory data. Working shear was calculated by applying a FS of 2.5 to ultimate shear values to calculate. The foregoing was the only geotechnical testing of rock strength testing performed for the entire project.

The bid documents indicated the anticipated average working shear values at Pier 40 of about 42 psi to 49 psi based on a 1,010 kip design load and rock socket lengths of 48 feet (north bell) and 54 feet (south bell). In essence, the test results indicated that the working shear strength of the Franciscan Formation at Pier 40 was about half of what was anticipated at bid time. In using the working shear strength determined from the additional test borings at Pier 40, rock sockets were 33% longer on the north bell and 50% longer on the south bell. In addition, the 12 inch casing was embedded 715% deeper on the south bell.

As a result, materials and labor usage increased dramatically at this pier because of deeper rock sockets, longer lengths of casing and reinforcement, and grouting volumes On the north bell, the primary issue was the need for longer rock sockets because of the presence of weak graywacke. Casing seat and seal were not an issue because of the 10 foot thick layer of fat clay encountered on the top of decomposed bedrock at this bell. Significant increases in casing length did not occur at this bell because top of bedrock was encountered an/near the elevation shown on the Working Drawings. Lastly, rock socket caving was not an issue because the decomposed graywacke had enough clay content to maintain an open hole. Therefore, while there was a need for longer reinforcement steel on the north bell in order to meet design load requirements, if was not necessary to stabilize the rock sockets or artificially seat the casings using grout.

The Pier 40 south bell was more complicated because the decomposed graywacke was overlain by and interbedded with decomposed, pervasively fractured chert. The fracture spacing resulted in rhombic-shaped, course sand to pea gravel sized clasts that could be drilled through rapidly (similar to drilling through a very dense course sand). However, this material was extremely prone to caving, was too weak to seat the casing in, and did not facilitate the development of a casing seal. To resolve these issues, the rock socket was drilled until the onset of caving at which time it was backfilled with grout until the top of grout stabilized at about 5 to 10 feet above the tip of 12 inch casing. It was not unusual for a hole to be drilled and grouted multiple times as it was advanced deeper into the Franciscan Formation, or for each partially drilled hole to take two to three times its theoretical volume of grout before stabilizing. The large grout volume consumed was due to fracture porosity in the chert, a larger diameter hole created from caving, and leakage to the ocean at the tip of the 12 inch casing. As an example, the Pier 39 Performance Pile (which encountered decomposed, highly fractured chert similar to Pier 40) was drilled and grouted 13 times to various depths before a micropile could be installed. The lack of seal in the weak, caving chert was an on-going problem at the Pier 40 south bell that ultimately was solved by drilling a 14 inch diameter rock socket that was fully cased with a 12 inch casing. The reinforcing steel was installed in the fully cased hole and then the casing pulled up during grouting. To prevent grout washout, the tip of the 12 inch casing was embedded on average 36 feet into the Franciscan Formation.

Load Test Data Analysis Methodology

A number of technical issues related to micropile testing became apparent during the first performance test at Pier 58 (micropile P58PP01). Among these were questions related to maximum load and elongation recorded by the Owner versus those recorded by the Micropiling Contractor, movement of the bridge and/or load test frame reference beam on the due to tidal action and bridge traffic, and the reliability of engineering survey data taken using various permanent and temporary bench marks on the bridge, the reference beam, and the Performance Pile. At maximum load during the testing of P58PP01, the difference between the load cell and the jack pressure readings were on the order of 20% with the load cell always reading the lower value. Considering that the Performance Pile test load at Pier 58 was 120% of 840 kips (or 1,008 kips), a 20% discrepancy results in a load of 1,210 kips being applied to the micropile or an overloading condition of about 200 kips. The overloading attributable to the 20% discrepancy between the load cell and jack pressure could easily be enough to over stretch a pile and erroneously fail it. In fact, the first Pier 58 Performance Pile (P58PP01, <u>Table 3</u>) was failed by the Owner because it exceeded the allowable total vertical movement, regardless of the fact that it was overloaded. As a result, the Micropiling Contractor was directed to install and test a second Performance Pile at Pier 58 (P58PP02). Although the situation of the failed Pier 58 Performance Pile was later resolved, a different approach to data collection and analysis was immediately implemented on every subsequent load test. In addition, modifications were made to the physical setup of the test which included the use of two rather than one extensometers by the Owner to monitor micropile elongation, modifications to the reference beam by the Micropiling Contractor to ensure that it was completely independent of the work platform and the load test frame, and the use of additional dial gauges to examine movement of the jacks, reference beam, and top of 12 inch casing. The Micropiling Contractor also began recording the elongation where the jack load showed that the maximum Contract test load had been achieved. The next and last step in incremental loading would be where the load cell showed that the maximum Contract test load had been achieved.

During all load tests, the data were reduced and the load-displacement curve graphed in the field to identify and resolve problems at the time of testing. As appropriate, these data were further evaluated in the office to include micropile composite action and Bay Mud adhesion to the exterior of 12 inch casing. The Micropiling Contractor also provided the Owner with a written report summarizing the pressure jack and dial gauge data, and addressing in writing any issues of inconsistencies between the data sets, on a pile-by-pile basis.

<u>The Issue of Two Data Sets</u>

By virtue of having two independent means of collecting load test data, the Micropiling Contractor and the Owner ended up with two sets of data for the each test, and the two data tests were always different to some extent, and often made the difference between passing or failing a pile. The Owner used only the load cell/extensometer data to evaluate micropile total vertical movement. They did not

Table 3. Summary of Free Length and Percent Rock Socket Debonding Based on Jack Pressure and Dial Gauge Data

					Pre-Load Pile Condition		No Adhesion or Composite Action		Composite Action Only, No Adhesion		Both Composite Action and Adhesion	
Pile Number	Pile Type	Load Test	Reinforcing Steel	Area Steel (in^2)	Free Length (ft)	Percent Debond Rock Socket	Free Length (ft)	Percent Debond Rock Socket	Free Length (ft)	Percent Debond Rock Socket	Free Length (ft)	Percent Debond Rock Socket
01N03	A	Proof	CRIP	19.640	31.00	0%	37.72	18.28%				
01S07	B	Proof	CRIP	19.640	37.86	0%	22.54	0.00%	30.68	0.00%	30.62	0.00%
02N02	B	Proof	CRIP	19.640	51.85	0%	49.85	0.00%	64.92	52.49%	68.83	68.19%
02S04	A	Proof	CRIP	19.640	37.25	0%	36.17	0.00%	45.89	21.87%	47.10	24.94%
03N05	A	Proof	CRIP	19.640	52.25	0%	44.92	0.00%				
03S02	A	Proof	CRIP	19.640	53.75	0%	44.05	0.00%				
04N02	A	Proof	CRIP	19.640	68.95	0%	46.44	0.00%				
04S05	A	Proof	CRIP	19.640	73.50	0%	48.72	0.00%				
05N02	A	Proof	CRIP	19.640	88.75	0%	77.92	0.00%				
05S06	B	Proof	CRIP	19.640	98.30	0%	74.16	0.00%	104.50	42.91%	109.30	76.12%
06N02	A	Proof	CRIP	19.640	108.75	0%	73.45	0.00%				
06S02	A	Proof	CRIP	19.640	108.65	0%	75.83	0.00%				
07N02	A	Proof	CRIP	19.640	108.25	0%	58.82	0.00%				
07S01	B	Proof	CRIP	19.640	108.73	0%	85.18	0.00%				
08N02	B	Proof	CRIP	19.640	95.65	0%	61.14	0.00%	97.37	12.20%	105.12	67.16%
08S05	B	Proof	CRIP	19.640	80.15	0%	53.38	0.00%	79.74	0.00%	83.93	22.43%
09S02	B	Proof	CRIP	19.640	82.86	0%	49.09	0.00%				
09S05	B	Proof	CRIP	19.640	78.65	0%	54.05	0.00%				
10N02	B	Proof	CRIP	19.640	113.75	0%	70.02	0.00%				
10S05	B	Proof	CRIP	19.640	83.75	0%	58.63	0.00%				
11N02	B	Proof	7/8-in Walled	21.304	108.85	0%	59.55	0.00%				
11S02	B	Proof	7/8-in Walled	21.304	85.65	0%	48.22	0.00%				
12N02	B	Proof	7/8-in Walled	21.304	94.35	0%	50.99	0.00%				
12S04	B	Proof	7/8-in Walled	21.304	82.50	0%	50.63	0.00%				
13N02	A	Proof	7/8-in Walled	21.304	82.30	0%	54.48	0.00%				
13S05	A	Proof	7/8-in Walled	21.304	75.30	0%	39.08	0.00%				
14N02	A	Proof	7/8-in Walled	21.304	86.65	0%	56.53	0.00%				
14S05	A	Proof	7/8-in Walled	21.304	79.75	0%	50.94	0.00%				
15N02	A	Proof	7/8-in Walled	21.304	94.75	0%	58.62	0.00%				
15S02	A	Proof	7/8-in Walled	21.304	86.50	0%	52.08	0.00%				
16N02	B	Proof	7/8-in Walled	21.304	102.65	0%	64.24	0.00%				
16S02	B	Proof	7/8-in Walled	21.304	104.75	0%	64.83	0.00%				
17N05	B	Proof	7/8-in Walled	21.304	123.65	0%	55.40	0.00%				
17S05	B	Proof	7/8-in Walled	21.304	113.00	0%	48.70	0.00%				
18N02	A	Proof	7/8-in Walled	21.304	126.60	0%	59.79	0.00%	120.60	0.00%	117.73	0.00%
18S02	A	Proof	7/8-in Walled	21.304	117.60	0%	66.75	0.00%	112.53	0.00%	122.13	29.42%
39N14	B	Proof	1-in Walled/ATB	28.910	33.70	0%	46.60	27.28%	52.88	40.55%	53.94	42.79%
39S04	A	Proof	1-in Walled/ATB	28.910	34.50	0%	48.93	28.87%	55.34	41.68%	56.54	44.08%
40N12	A	Proof	1-in Walled/ATB	28.910	71.00	0%	66.58	0.00%	89.39	38.31%	93.16	46.17%
40S10	A	Proof	1-in Walled/ATB	28.910	59.60	0%	86.49	35.67%	97.90	50.80%	97.90	50.80%
50N04	A	Proof	1-in Walled	23.995	47.55	0%	71.37	44.57%	78.35	57.62%	81.65	63.80%
50N11	A	Proof	1-in Walled	23.995	45.75	0%	48.81	5.37%				
50S11	A	Proof	1-in Walled	23.995	54.45	0%	48.39	0.00%				
51N02	A	Proof	7/8-in Walled	21.304	42.15	0%	34.61	0.00%				
51S02	A	Proof	7/8-in Walled	21.304	43.80	0%	39.98	0.00%				
52N02	A	Proof	7/8-in Walled	21.304	51.51	0%	39.53	0.00%	55.05	11.52%	56.24	15.39%
52S06	A	Proof	7/8-in Walled	21.304	44.95	0%	48.05	10.00%				
53N05	A	Proof	7/8-in Walled	21.304	60.20	0%	43.08	0.00%	58.14	0.00%	59.13	0.00%
53S02	A	Proof	7/8-in Walled	21.304	43.14	0%	47.73	11.81%	58.63	39.86%	60.66	45.08%
54N02	A	Proof	7/8-in Walled	21.304	33.45	0%	45.50	31.58%	47.26	36.20%	48.28	38.87%
54S05	A	Proof	7/8-in Walled	21.304	39.50	0%	45.04	15.39%	57.21	49.19%	58.82	53.67%
56N02	B	Proof	1-in Walled	23.995	35.45	0%	38.60	8.39%	45.29	26.21%	45.93	27.91%
56N04	A	Proof	1-in Walled	23.995	31.46	0%	48.47	40.27%	52.86	50.65%	53.86	53.02%
57N05	A	Proof	1-in Walled	23.995	28.27	0%	45.19	33.92%	45.93	35.39%	46.59	36.71%
57N11	A	Proof	1-in Walled	23.995	30.30	0%	48.26	32.79%	51.15	38.06%	52.04	39.69%
57S07	A	Proof	1-in Walled	23.995	31.54	0%	62.54	59.08%	58.10	50.63%	59.80	53.87%
58N05	A	Proof	1-in Walled	23.995	50.65	0%	57.78	25.15%	67.65	59.96%	70.65	70.55%
58S12	A	Proof	1-in Walled	23.995	38.20	0%	43.19	10.81%	48.20	21.65%	49.20	23.81%
59N02	A	Proof	7/8-in Walled	21.304	50.35	0%	48.29	0.00%				
59S05	A	Proof	7/8-in Walled	21.304	49.45	0%	58.07	26.68%				
60N02	A	Proof	7/8-in Walled	21.304	45.30	0%	42.53	0.00%				
60S05	A	Proof	7/8-in Walled	21.304	56.30	0%	38.93	0.00%				
0AN04	A	Proof	CRIP	19.640	31.95	0%	24.95	0.00%				
0AS05	A	Proof	CRIP	19.640	27.75	0%	30.06	6.24%				
02PP01	A	Performance	CRIP	19.640	47.62	0%	37.99	0.00%	47.46	0.00%	48.08	1.58%
13PP01	A	Performance	7/8-in Walled	21.304	86.96	0%	59.01	0.00%	86.66	0.00%	91.08	19.20%
39PP01	A	Performance	1-in Walled	23.995	42.87	0%	49.02	15.16%	58.24	37.90%	59.59	41.23%
51PP01	A	Performance	7/8-in Walled	21.304	40.46	0%	47.60	20.74%	54.54	40.87%	59.24	54.51%

assess debond or creep, elastic movement, and permanent set. The Micropiling Contractor collected and evaluated both data sets to calculate total elongation, permanent set, elastic elongation, permanent set, rock socket debond length (or free length), and creep at maximum load.

For the six Performance Pile tests and 27 Proof Pile tests, the Micropiling Contractor also evaluated the impact of Bay Mud adhesion (ranging from 0 psf to 500 psf on casing exterior) and the composite action of grouted 12 inch casing and reinforcing steel on load test results. Both load cell/extensometer and jack/dial gauge data were evaluated. The first step in the analysis was to calculate debond length or free length based on elastic elongation recorded and then solve for total free length "L" using PL/AE. Given calculated free length and knowing the total length of the pile, a calculation of rock socket debond was made. The second step in the analysis to calculate free length by breaking the micropile into 5 segments as follows:

- L1=pile length between top of 8 inch reinforcement to top of first stage grout,
- L2=pile length between top of first stage grout and tip of 12 inch casing,
- L3=pile length between tip 12 inch casing and assumed point of fixity (i.e., length of debonded rock socket),
- L4=pile length between tip of 12 inch casing and tip of 8 inch reinforcement, and
- L5=pile length between mud line and top of first stage grout.

Using elastic elongation measured in the field at peak loading, L3 was solved for with Bay Mud adhesion (using 250 psf and 500 psf) and without Bay Mud adhesion (0 psf) by iterating to find the point of fixity. The compilation of 31 load test analyses show that the calculated rock socket debonded length increases when composite action and adhesion are taken into consideration (Table 3 and Figure 7). Using the jack/dial gauge data, the average rock socket debonding is 7% when adhesion and composite action are ignored. With composite action only, rock socket debonding increases to 29%. With composite action and 250 psf adhesion on the 12 inch casing, debonding increases further to 37%. Note that the maximum load values used to calculate the debond values in Table 3 represent "overloading" conditions of up to 20% and the debond lengths at maximum Contract load measured using jack pressure would be correspondingly less. Rock socket debonding of 50% is considered by the industry to likely represent pile failure, confirmation of which should be made in conjunction of review of creep rates, maximum elongation, and a graph of the load-elongation curves for incremental and decremental loading. However, since the Owner used only total vertical movement as the pass/fail criterion for the tested piles, the magnitude of rock socket debond length was not a factor in the Owner's evaluation of pile performance. With the exception of the P58PP01 Performance Pile, no Performance Pile or Proof Pile was failed by the State and no production piles required remediation based on load test results. This occurred even though some piles technically did not satisfy the Owner's pass/fail elongation criterion. At piers where load test data indicated exceedence of the elongation criterion, Production Pile rock sockets were extended five feet deeper and/or a #20 steel ATB was added down the center of the 8 inch diameter reinforcement steel.

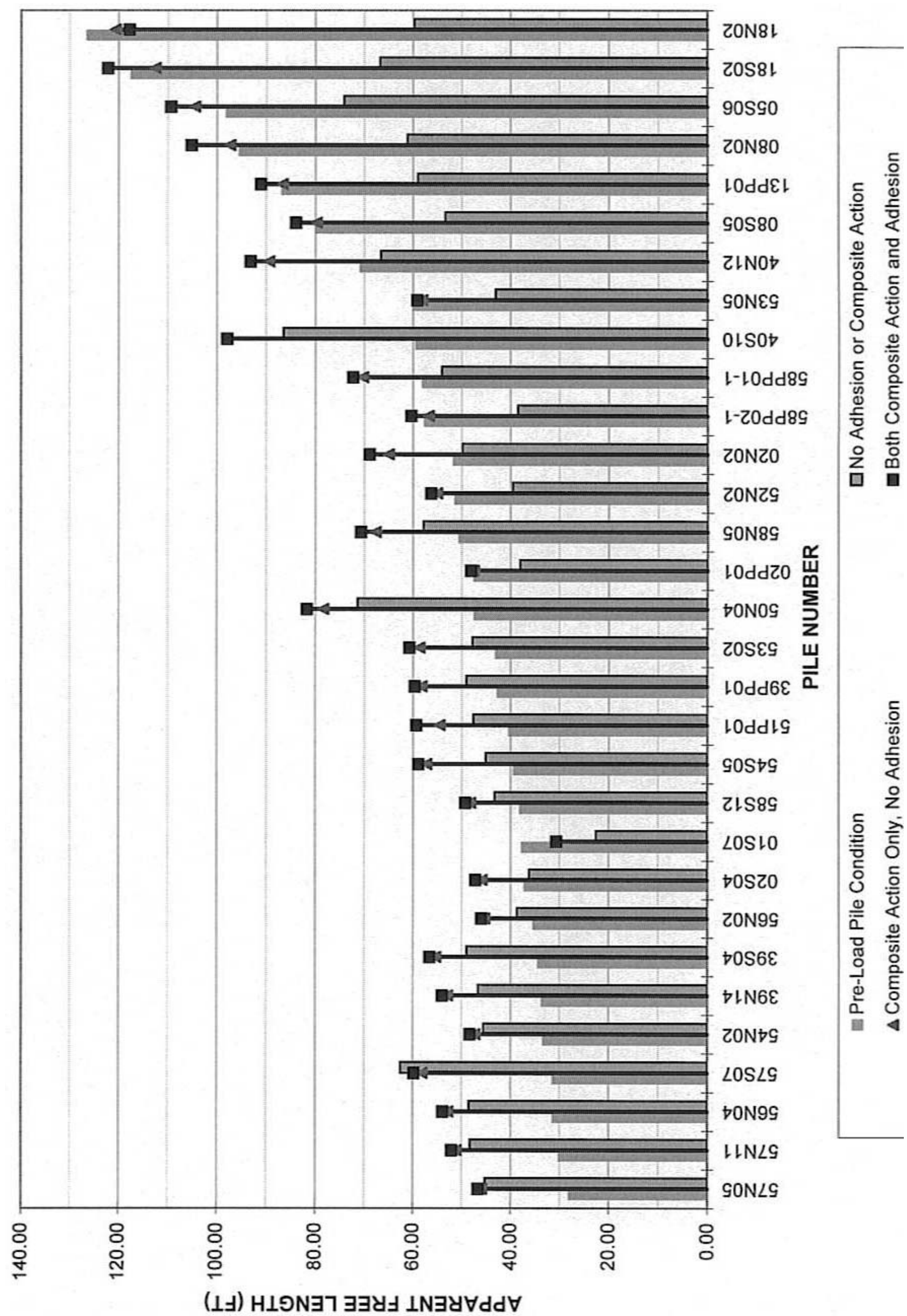

Figure 7. Micropile Apparent Free Length Based on Pressure Jack and Dial Gauge Data

Summary

The important QA/QC measures exercised on this project are as follows:

- Working Drawings with tabulated micropile geometry data for each pile were a valuable reference document for engineers, drillers, and platform foremen in the field. They prevented mistakes in micropile installation, ensured that the minimum required Contract specifications were achieved, and helped field staff identify when an anomalous situation was encountered. For example, the Working Drawings for each bell indicated where the top of the bell, bottom of the bell, and top of bedrock were anticipated to be encountered. If the elevation of bell top encountered during drilling did not match with the Working Drawing, the drillers immediately knew to check the plumbness of the conductor casing and drill string. If the length of cored concrete extracted did not match the theoretical, the field crew knew to look for evidence of degraded or segregated concrete in subsequent coreholes. The fact that no micropile was installed to the wrong dimension on this project is in part attributable to the Working Drawings which became a reference tool as much for the drillers and foremen as for the platform engineer.
- Field Engineers/Geologists at each platform documented factual data, provided interpretation of such on a real time basis, and informed senior staff of issues arising in the field. Drillers and foremen contributed to the notes on a regular basis as appropriate but otherwise were able to focus on their jobs without being encumbered by detailed note taking.
- Load test data analysis was performed in the field as data were collected to identify and address discrepancies with the Owner. The data were further evaluated in the office to assess elastic movement, total movement, and apparent free length and prepare a written report summarizing the results. At problematic piers, finite difference analysis was performed to examine the role of adhesion and composite action on pile performance.
- Given the absence of geotechnical rock strength testing data, a thorough review of published literature was completed to assess the range in unconfined compressive strength for various rock types and correlate these data to site conditions identified from LOTB's and rock core samples. At one pier, additional test borings were drilled by the Micropiling Contractor to further investigate the strength parameters of the rock.
- The grout design was experimental in nature. Since the Owner was not inclined to alter the mix design, the Micropiling Contractor was very careful to document pre-construction and construction grouting data and issues.

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