HIGH CAPACITY MICROPILES IN MINED GROUND FOR BRIDGE SUPPORT: A CASE HISTORY OF INVESTIGATION, DESIGN, CONSTRUCTION AND PERFORMANCE: PAPER 4 LOAD TESTING

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ABSTRACT

This is the fourth paper in the series describing the various stages of the micropile works conducted for new bridge piers in Joplin, Missouri, for both Verification Tests, conducted preconstruction, and Proof Tests, conducted on production piles. The paper therefore describes the installation and testing (in two cases to failure) of 4 full-scale Verification Test Piles, and of 16 production piles, at the rate of one test per bridge bent. All tests were in tension. All the Proof Test results indicated minimal debonding and a very stiff performance to the Test Load of 1.2 times design load.

1. INTRODUCTION

Details of the site investigation and assessment, design, and construction of the high capacity micropiles for the Missouri Department of Transportation's bridges in Joplin, Missouri are provided in the three companion papers (References 1-3). This paper describes the background to, and the details of, the tensile load testing of 4 preconstruction "Verification" piles, and the similar testing of 16 service piles ("Proof Tests"). It will be borne in mind that the geology of the site was very "chaotic," and major efforts, in terms of pretreating by grouting the locations of the micropile groups, were undertaken to explore and prepare the rock for production piles.

2. TECHNICAL REQUIREMENTS

2.1 The Specifications

The minimum anticipated cased and bonded zone requirements, and minimum load requirements, were summarized in the Plans (Table 1, Figure 1). It was noted that the grout/rock bond assumptions underlying these plans and the Geotechnical Baseline Report were to be verified by load testing a minimum of 4 preproduction sacrificial Verification piles "and modifications by the engineer to the production micropile lengths made as required." The contractor was to design the load test system, modify the pile top connection to accommodate the testing equipment, and the proper execution of the

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Table 1. Foreseen Schedule of Pile Length.

MICROPILE DATA																				
BRIDGE NO.	A6140		A6149								A6150			A6165			VERIFICATION			
BENT NO.	1	2	1	2	3	4	5	6	7	8	2	3	4	1	2	3	1	2	3	4
Elev. A	301.000	299.000	299.500	293.500	298.500	299.000	298.500	298.000	299.000	296.500	296.000	296.000	296.000	297.000	295.700	298.500	---	---	---	---
Elev. B ***	299.500	297.800	298.300	292.300	297.300	297.800	297.300	296.800	297.300	295.300	294.800	294.800	294.800	295.500	294.500	297.000	302.000	302.000	301.000	303.000
Elev. C	296.000	294.000	294.000	292.300	295.000	293.000	295.000	295.000	295.000	294.000	292.000	286.000	292.000	294.000	293.000	295.000	296.000	296.000	292.500	299.000
Elev. D	293.000	291.000	291.000	290.300	293.500	290.000	291.000	294.000	294.000	293.000	289.000	285.000	289.000	291.000	290.000	292.000	292.000	292.000	289.500	290.000
Elev. E	285.000	275.500	281.000	284.300	287.500	284.000	281.000	288.000	288.000	287.000	284.000	279.000	279.000	279.000	278.000	280.000	288.000	288.000	286.500	287.000
Threadbar Diameter - mm	63.5	63.5	63.5	63.5	63.5	63.5	63.5	63.5	63.5	63.5	63.5	63.5	63.5	63.5	63.5	63.5	63.5	63.5	63.5	63.5
Design Compression Load - kN	1 423	1 891	1 437	1 649	1 556	1 271	1 480	1 565	1 510	1 439	827	827	1 784	1 846	1 035	1 846	---	---	---	---
Design Tension Load - kN	0	744	569	154	297	213	247	485	590	307	130	130	472	925	163	925	1 105	1 105	1 105	1 105
** Micropile (193.7mm) Cased Length (meter)	68.3	86.3	137.9	28.2	50.7	100.9	82.1	38.2	44.5	31.9	74.0	123.0	73.6	115.4	58.1	127.6	---	---	---	---
** Micropile (193.7mm) Bonded Length (meter)	81.0	189.7	182.6	75.2	75.2	75.2	125.3	75.2	75.2	75.2	61.2	73.4	121.6	291.6	146.9	291.6	---	---	---	---
Lateral Pile Bending Required (Yes/No)	Yes	No	Yes	No	No	No	No	No	No	No	Yes	Yes	Yes	Yes	Yes	Yes	---	---	---	---

** These quantities are included in the estimated quatities table on Sheet No. 2.

*** Elev. B represents the top of existing ground for the verification load tests.

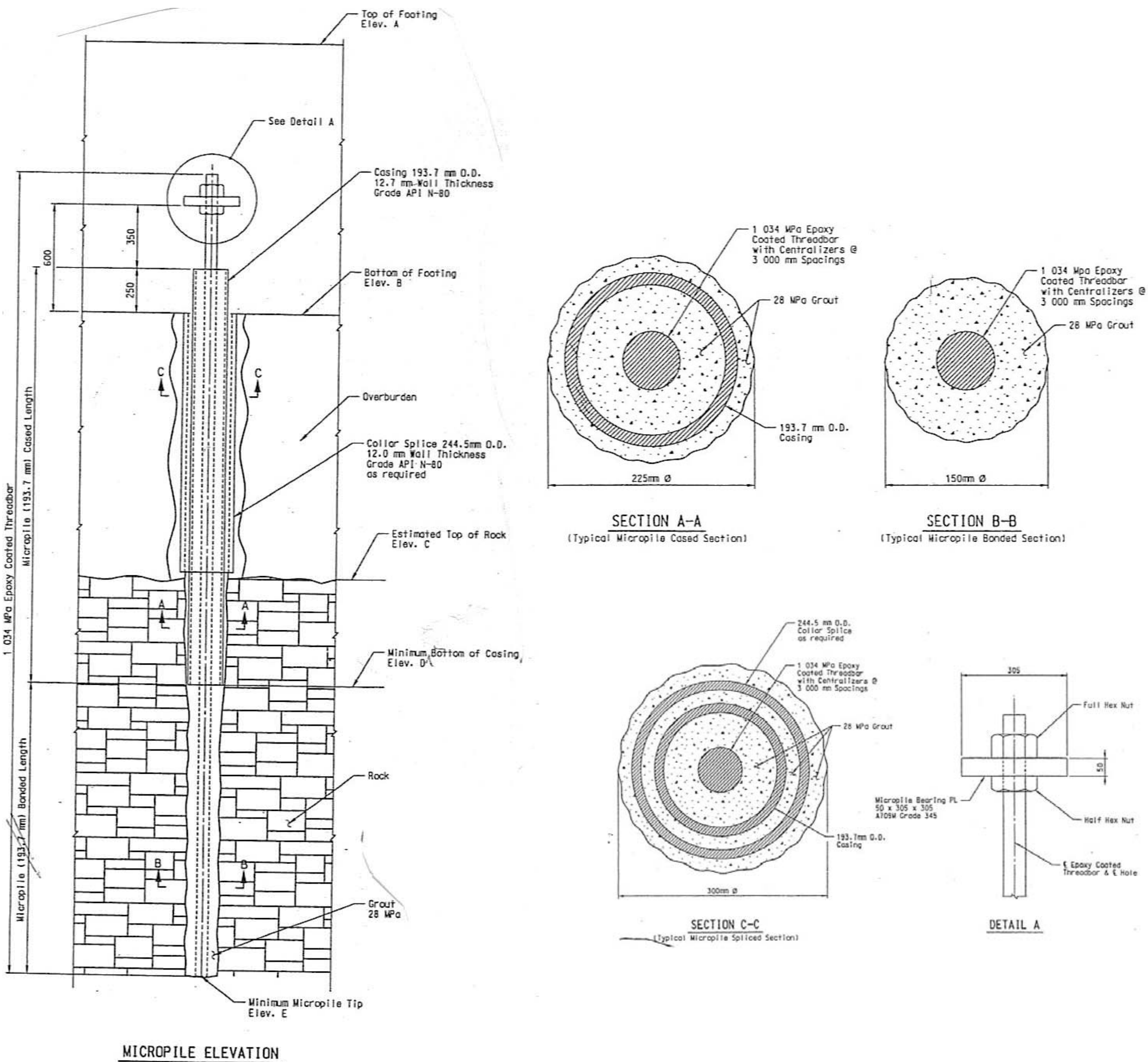

Figure 1. Typical Micropile Detail.

testing, and to collect all load/movement data. Proof testing of the production piles to 1.2 Design Load (DL) was to be conducted at the rate of one pile per each of the 16 bents (pier support structures) "to validate the contractor's quality control during construction."

2.1.1 *Verification Load Tests*

A minimum of 4 vertical piles were to be installed in different conditions as summarized in Table 1. One of the piles (VP2) was to be located in an area subjected to pregrouting. The construction means, methods and materials were to be identical to those foreseen for the production piles, except that the cased length above the bond length was to be constructed to prevent load transfer to the surrounding ground above the bond length. The maximum loads were not to exceed 80% of the structural capacity of the steel in tension. The actual test load was to twice DL shown in the plans or to rock/grout bond failure. The test method was incremental cyclic with intermediate maxima (and 10-minute creep holds) at 0.5, 0.75, 1.00, 1.25, 1.50 and 1.75 times DL. The creep test at 2.00 DL was to be run for 60 minutes. There were no acceptance criteria, i.e., no pass/fail concepts.

2.1.2 *Proof Tests*

The foreseen schedule for the 220 production piles is shown in Table 1, wherein the crucial elevations A through E are identified in Figure 1. The precise pile to be tested in each bent was to be chosen by the Engineer. The test was simple incremental loading in steps of 0.25, 0.50, 0.75, 1.00 and 1.20 DL with a 60-minute creep hold at 1.20 DL.

The acceptance criteria were:

1. Failure does not occur at 1.20 DL. (Failure was defined as inability to apply additional load.)
2. At test load, the apparent debonded length (calculated from the elastic extension) shall not exceed 50% of the bond length.
3. At the end of the 1.20 DL creep test, the creep rate shall not exceed 1 mm per log cycle (1-10 minutes) or 2 mm per log cycle (6-60 minutes). The creep rate shall be linear or decreasing throughout the creep test period.

Failure would involve testing another pile in the same bent, and modifications would be considered involving down-rating to 50% of the maximum load attained, post-grouting, or replacement with piles installed by different methods.

As for the Verification Tests, the Proof Tests were paid for on a lump-sum-for-each basis.

2.2 The Contractor's Submittal

The submittal was in conformance with the specifications and the pile details shown in Figure 2 were proposed. The contractor further detailed the following:

- Install each pile in accordance with standard means and methods.
- Prepare test frame as shown in Figure 3.
- Place support system for test frame consisting of timber cribbing.
- Lay test frame across timber cribbing and center over pile. Test frame consisted of 2 each W 36 x 300 A 36 steel beams, 24 feet in length.
- The steel reinforcing bar was extended from the top of the pile through the test beam.

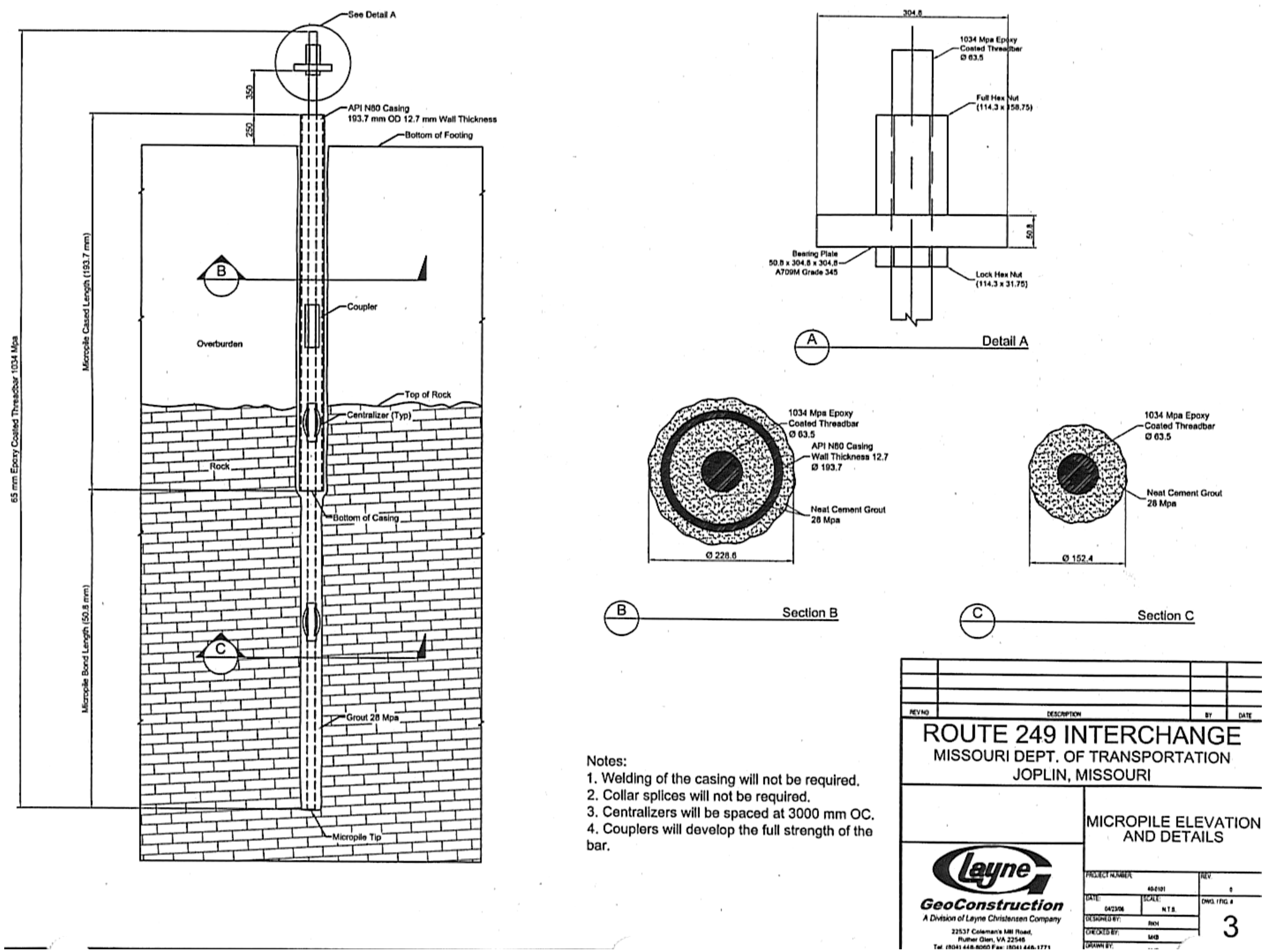

Figure 2. Pile Detail Proposed by Contractor.

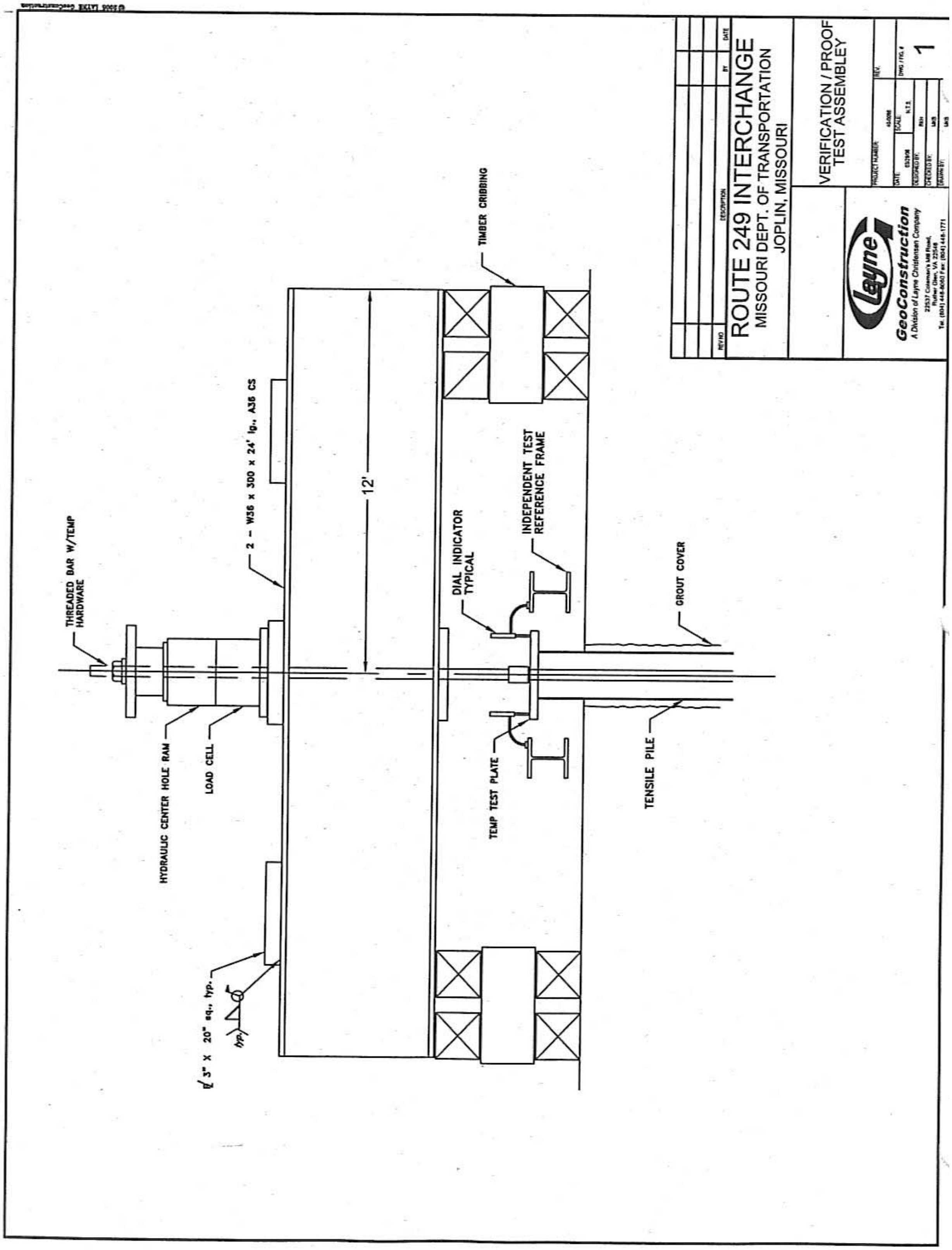

Figure 3. Detail of Tension Load Testing Frame.

Figures 4 and 5 show the testing underway in the field.

Figure 4. Load testing in progress.

Figure 5. Load testing in progress.

- Align hydraulic center hole jack over bar on top of frame. The jack was a 620-ton capacity Simplex. Calibration records were provided.
- Place temporary plate and hardware on top of ram to tension bar.
- Set 3 dial gauges at 120º intervals, accurate to 0.001 inch at top of casing (later increased to 4 gauges).
- Perform test in accordance with ASTM D3689 Quick Test, as modified by the specifications.

3. RESULTS OF VERIFICATION TESTS

3.1 Construction

For each pile, a 193.7 mm o.d. casing was installed through the overburden and upper rock horizon, the hole having been predrilled with a 229 mm diameter down-the-hole hammer. In addition, the 63.5 mm diameter Grade 1,034 MPa thread bar was debonded from the grout in the cased length to promote efficient transfer of the tensile stress into the bond zone, which alone was to be tested. Table 2 indicates the relationship between the Verification and Production piles.

Table 2. Relationship of Verification Test Piles to the As-Designed Production Piles.

As-Designed Micropile Conditions									
Bridge Number	Bent Number	Compressive Load (kN)	Bond Zone			Average Working Bond Stress (Mpa)	Ground Type	Verification Test Pile Number	Geologic Description from Baseline Ground Conditions
			Begin Elevation	End Elevation	Length (M)				
6140	EB-1	1423	293	285	8	0.38	2	1,2	Weak, Weathered Limestone for top 3m, Improving Rock Quality Below with Voids Encountered
	2	1891	291	275.5	15.5	0.26	2	1,2	Broken, Confused and Chaotic Limestone/Chert
6149	EB-1	1437	291	281	10	0.31	2	1,2	Weak, Weathered Limestone, Shale, and Sandstone for top 6m, Improving Rock Quality Below
	2	1649	290.3	284.3	6	0.58	2	3	Good to Excellent Quality Limestone, Occasional Poorer Quality due to Thin Bedding or Brecciation
	3	1556	293.5	287.5	6	0.55	2	3	Good to Excellent Quality Limestone, Occasional Poorer Quality due to Thin Bedding or Brecciation
	4	1271	290	284	6	0.45	2	3	Poor to Fair Quality Limestone
	5	1480	291	281	10	0.31	2	1,2	Weak, Weathered Limestone and Confused Ground
	6	1565	294	288	6	0.55	2	3	Good to Excellent Quality Limestone, Occasional Poorer Quality due to Thin Bedding or Brecciation
	7	1510	294	288	6	0.53	2	3	Good Quality Limestone
	8	1439	293	287	6	0.51	2	3	Good to Fair Quality Limestone
6150	2	827	289	284	5	0.35	2	1,2	Moderately Weathered Limestone and Shale, Good Rock Quality
	3	827	285	279	6	0.29	2	1,2	Weathered Limestone and Coal/Shale for top 2m, Improving Rock Quality Below
	EB-4	1784	289	279	10	0.38	2	1,2	Weathered Limestone and Lesser Amounts of Coal/Shale for top 6m, Rock Quality Below is Poor
6165	EB-1	1846	291	279	12	0.33	2	4	Weak Shale, Highly to Moderately Disturbed, Minor Sandstone and Limestone
	2	1035	290	278	12	0.18	2	4	Weak Shale, Highly to Moderately Disturbed, Minor Sandstone
	EB-3	1846	292	280	12	0.33	2	4	Weak Shale, Highly to Moderately Disturbed, Minor Limestone

The designed dimensions of the Verification piles consistent with Table 1 are shown in Table 3.

Table 3. Designed Verification Pile Dimensions and Ground Conditions.

	VP1	VP2	VP3	VP4
(B) Ground Surface Elevation (m)	302	302	301	303
(C) Est. Top Elevation (m)	296	296	292.5	299
(D) Min. Bottom of Casing Elevation (m)	292	292	289.5	290
(E) Min. Tip Elevation (m)	288	288	286.5	287
Foreseen General Rock Classification	Confused limestone and shale. No pregrouting of rock mass.	Confused limestone and shale. Pregrouting of rock mass.	Solid limestone.	Weak shale.

The actual as-built dimensions of the verification piles are shown in Table 4:

Table 4. Actual Verification Pile Dimensions and Ground Conditions.

	VP1	VP2	VP3	VP4
Ground Surface Elevation (m)	302.0	302.0	301.0	303.0
Actual Top of Rock Elevation (m)	293.0	292.6	296.0	297.0
Actual Bottom of Casing Elevation (m)	292.0	292.6	289.5	290.0
Actual Pile Tip Elevation (m)	288.0	288.2	286.4	287.0
Butt Elevation* (m)	302.3	302.3	301.3	303.3
Summary Rock Classification	Relatively sound limestone	Pregrouted confused shales and clay	Relatively poor quality limestone	Shale
Grout Volume	0.80 m^3	0.45 m^3	0.49 m^3	0.49 m^3

* Elevation of plate (below beam and jack) on which movements were measured during testing.

Note: 1) Bond zones were each 152 mm in diameter.
2) Each bond zone contained one coupler on the bar.

Pretreatment of the rock mass surrounding the VP2 was conducted in the pattern shown in Figure 6. Details of the pretreatment are summarized in Table 5:

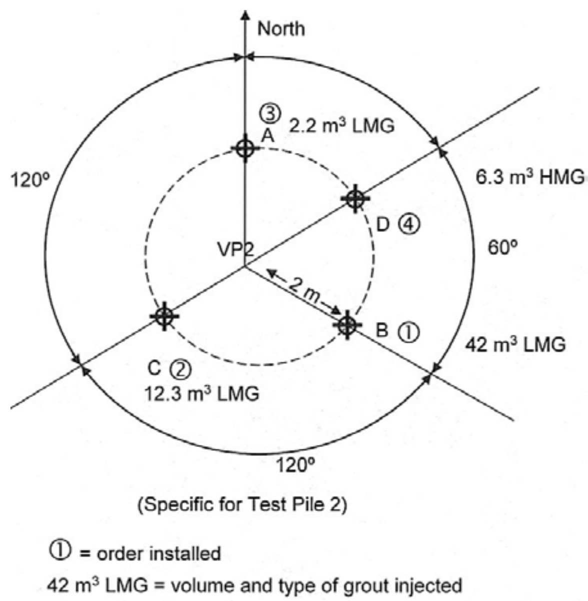

Figure 2. Layout of Pretreatment Holes for Verification Test Pile 2

Figure 6. Layout of Pretreatment Holes for Verification Test Pile 2.

Table 5 . Pretreatment Details for VP2.

PRE-TREATMENT HOLE	DRILLING			GROUTING	
	DATE	DESCRIPTION		DATE	DESCRIPTION
VP2B	05/16	0 – 6 m	clay and shale overburden	05/17	42 m^3 of LMG. Casing: 14 m (88 mm slump)
		6 – 9 m	clay		
		9 – 13.5 m	void		
		13.5 – 15.0 m	limestone		
VP2C	05/18	0 – 5.55 m	overburden	05/18	12.27 m^3 of LMG. Casing: 15.25 to 10.1 m (100 mm slump)
		5.55 – 8.65 m	poor rock		
		8.65 – 15.48 m	confused rock with several voids (each less than 1 m)		
		15.48 – 16.00 m	limestone		
VP2A	05/19	0 – 5.95 m	clay and shale overburden	05/19	2.2 m^3 of LMG. Casing: 15 to 6 m (100 mm slump)
		5.95 – 8.58 m	confused/sandstone		
		8.58 – 9.08 m	void		
		9.08 – 13.35 m	clay		
		13.35 – 15.00 m	limestone		

(continues)

<table>
<tr><th rowspan="2">PRE-TREATMENT HOLE</th><th colspan="3">DRILLING</th><th colspan="2">GROUTING</th></tr>
<tr><th>DATE</th><th colspan="2">DESCRIPTION</th><th>DATE</th><th>DESCRIPTION</th></tr>
<tr><td rowspan="5">VP2D</td><td rowspan="4">06/10</td><td>0 – 6.11 m</td><td>overburden</td><td rowspan="5">06/13</td><td rowspan="5">12 – 15 m, 1,623 gallons of HMG
9 – 12 m, 2 gallons of HMG
(Total 6.3 m^3)
WCR = 1.0, plus viscosifier</td></tr>
<tr><td>6.11 – 9.50 m</td><td>clay</td></tr>
<tr><td>9.50 – 13.27 m</td><td>shale/confused</td></tr>
<tr><td>13.27 – 15.02 m</td><td>limestone</td></tr>
<tr><td colspan="3">(communicated to hole 8 m to southwest)</td></tr>
</table>

Thereafter, on June 14, 2006, the Verification Test Pile hole itself was drilled. This provided the following data:

DEPTH	GROUND CONDITIONS
0 – 9.41 m	clay, shale and overburden
9.41 – 10.0 m	weathered limestone
10.0 – 15.0 m	shale and clay, caving
15.0 – 16.0 m	limestone

The hole was cased to 10 m, overdrilled to 16 m and then pregrouted through the rods with a neat cement grout to stabilize the hole. The following day, the hole was redrilled to a depth of 14 m and the bond zone remained open. The pile was then installed.

3.2 Analysis

Table 6 summarizes the data obtained during testing:

Table 6. Details of Verification Pile Testing.

PILE	6149B1			6165B1
	VP1	VP2	VP3	VP4
Max Load (kN)	2,210 kN	2,210 kN	2,210 kN	2,088 kN
Elastic Movement at Max Load	32.5 mm	34.92 mm	35.30 mm	N/A
Permanent Movement after Max Load	2.82 m	12.90 mm	2.600 mm	N/A
Creep 1-10 Minutes	0.248 mm	2.248 mm	0.057 mm	N/A
Creep 6-60 Minutes	0.387 mm	3.247 mm	Not Available	N/A
Comment on Load-Movement Diagram	Very linear, repeatable, no failure	Debonding to 1,105 kN, linear above, very close to failure at TL	Very linear, repeatable, no failure	Linear to 1,934 kN, but abrupt failure at 2,088

(continues)

PILE	6149B1			6165B1 VP4
	VP1	VP2	VP3	
At Test Load				
Calculated Debonding	-0.5 m	3.3 m	-1.3 m	N/A
Average Rock/Grout Bond	1.16 MPa (Test Bond)	1.15 MPa (Failure Bond)	1.54 MPa (Test Bond)	1.46 MPa (Failure Bond)
Comparable Working Bond of Production Piles	0.26 to 0.38 MPa	0.26 to 0.38 MPa	0.45 to 0.58 MPa	0.18 to 0.33 MPa

The following comments are apposite:

- VP 1 and 3 were tested to 2.00 DL (2,210 kN) without any indication of failure or imminent failure. They provided maximum average test bond values of 1.16 and 1.54 MPa, respectively.

- VP 2 reached the maximum test load of 2,210 kN but analysis shows it had likely failed at or just below this load. This load corresponds to an average ultimate rock-grout bond of 1.15 MPa.

- VP 4 reached a maximum load of 1.89 DL (2,088 kN), although it had likely failed at around 1.75 DL (1,934 kN). The maximum test load corresponds to an average ultimate bond value of 1.46 MPa.

- In the cases of VP 1, 3 and 4 there was little evidence of significant progressive debonding between the bar and the grout. However, it must also be noted that, despite attempts to totally disassociate structurally the bar from the grout in the cased zone, a certain amount of load (possibly up to 20% based on analysis of the elastic movement data) was shed in this region, above the bond zone. All the average bond values quoted above assume that all the load was transferred into the bond zone: they are therefore, in fact, over- estimates of the average rock-grout bond stress mobilized in the bond zone itself by the same amount, i.e., about 20%.

- These tests indicated that even in the poorest ground conditions, the apparent factor of safety which could be achieved at the grout-rock interface against failure was most likely in excess of three.

4. RESULTS OF THE PROOF TESTS

A summary of the proof testing results is provided in Table 7. The key findings, relative to the acceptance criteria noted in Section 2.1.2, above, are as follows:

- Every micropile reached the test loads (equivalent to 1.20 DL) of between 986 and 2,269 kN.

- At test load, the debonded length was exceptionally small to the extent that in only one case (TP 14) was it possible to conclude that the point of debonding may even have exceeded the cased length. In all the other cases, the piles apparently debonded only a short distance (1.1 to 4.3 m into the casing). In all cases, the load-movement curve was exceptionally linear attesting to minimal progressive debonding.

- Every pile comfortably satisfied the creep criterion at test load between 1 and 60 minutes. Only two piles (TP 6 and 14) exceeded a 1 mm creep amount.

5. FINAL OBSERVATIONS

The fundamental challenge of this project was to provide, in a verifiable way, a deep foundation system demonstrably capable of functioning in a very wide variety of ground conditions. This variety ranged from hard limestones to “chaotic” assemblages of shale, sandstone, weathered limestone and clay.

The Verification Test Program provided tested, and ultimate, average rock/grout bond values which proved invaluable in both verifying foreseen pile designs in many bents, but also in requiring bond zones to be lengthened in other areas.

The Proof Tests, conducted at a frequency of one per production bent, confirmed that the production piles performed in a manner consistent with design requirements. Indeed, their exceptionally stiff behavior, due to limited load transfer lengths through the upper cased pile section, will assure that service movements of the finished bridge piers (Figure 7) will be minimal.

Table 7. Details of Proof Tests

Location		Construction Data					Test Data			
Pile	Bent	Inclina-tion	Cased Length (m)	Bond Length (m)	Rock in Bond Length	Test Load (kN)	Elastic (mm)	Perm (mm)	Debonded Length (m)	Creep 1-60 Mins. (mm)
TP 1	6150 B4	Vertical	6.0	7.1	Lmst	2,148	5.77	2.92	1.76	0.29
TP 2	6140 B1	Vertical	6.5	8.4	Lmst	1,708	7.05	1.53	2.75	<0.2
TP 3	6150 B2	12º	6.1	4.9	Lmst	986	2.72	0.34	1.37	<0.1
TP 4	6150 B3	11º	10.3	6.0	Lmst	986	2.20	0.29	1.17	<0.1
TP 5	6149 B5	17º	6.8	10.2	Weath Lmst	1,791	6.83	0.55	2.43	0.52
TP 6	6149 B6	17º	3.3	6.6	Sandy Lmst	1,880	3.50	1.70	1.14	1.7
TP 7	6149 B3	17º	4.3	6.3	Lmst	1,867	4.50	1.50	1.50	<0.2
TP 8	6149 B3	17º	4.3	12.0	Broken Lmst	1,867	4.88	0.86	1.62	<0.1
TP 9	6149 B4	17º	8.3	6.3	Sstn Broken Lmst Clay	1,522	6.51	1.59	2.68	0.27
TP 10	6149 B1	Vertical	7.6	10.0	Broken Lmst	1,724	8.33	0.74	3.10	<0.2
TP 11	6140 B2	11º	7.3	15.8	Clay Sstn Broken Lmst	2,267	8.50	2.60	2.37	0.44
TP 12	6165 B3	Vertical	5.2	11.9	Shale	2,208	15.34	3.09	4.34	0.61
TP 13	6149 B7	17º	3.7	6.5	Lmst	1,820	3.22	1.90	0.85	0.29
TP 14	6165 B1	Vertical	4.7	12.0	Shale	2,215	23.60	6.45	4.82	1.05
TP 15	6149 B8	17º	5.8	12.9	Broken Lmst Clay	1,727	10.03	3.45	3.59	0.64
TP 16	6165 B2	11º	5.1	12.0	Shale	1,254	7.19	2.65	3.65	0.48
TP 17	6149 B2	17º	2.5	6.1	Lmst	1,969	4.34	0.85	1.16	0.34

Figure 7. View of Piers being Built above the Micropile Foundations.

6. REFERENCES

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