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**An Alternative Foundation Solution
State Route 22, Section A02 – Lewistown Bypass**

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**AN ALTERNATIVE FOUNDATION SOLUTION
STATE ROUTE 22, SECTION A02 – LEWISTOWN BYPASS**

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ABSTRACT: This paper details the Value Engineering efforts to design and install micropiles as an alternative to the as-designed driven and pre-drilled H-piles situated over karst terrain. A total of 295, 178 mm (7-in) diameter micropiles with an axial compression capacity of 890 kN (200-kips) were installed for the bridge structures to replace 511 driven and pre-drilled H-piles with an axial compression capacity of 445 kN (100-kips). In karst areas where cutter and pinnacle formations must be penetrated, micropiles are readily advanced through these formations and socketed into competent bedrock. Innovative grout mix design and construction techniques minimize grout loss within cavernous material to develop a competent bond zone. Seven compression load tests were conducted to verify the design bond stress values. This paper also presents bond stress values from micropile load tests conducted on similar projects situated over karst in an effort to assess the current state of practice.

INTRODUCTION

The Northern Lewistown Bypass project, located in central Pennsylvania, is the improvement and construction of nearly 19 km (12-mi) of highway with an estimated construction cost of \$130 million. The project is being built in three construction contracts. The third and central construction phase, Section A02, consists of approximately 7 km (4.5-mi) of new four lane, divided, limited access highway, including ten bridge structures.

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The majority of the grade separation structures within Section A02 were designed using deep foundations consisting of steel H-piles. The original design required the use of deep foundations consisting of steel H-piles to be driven and / or pre-drilled beyond the highly weathered, highly permeable, interconnected epikarst zone to end-bear on the underlying limestone. In an effort to assess pile driveability and preclude bearing the pile on a pinnacle or cavernous material located within the epikarst region, the contract documents required an exploratory drilling program prior to the start of pile driving. The results of the exploratory drilling indicated that the epikarst contained variable layers of weathered rock, clay seams and voids below the proposed pile tip elevations. Therefore, the driven and predrilled H-pile tip elevations would need to be lowered an average of 3.05 to 3.66 m (10 to 12-ft) from the original elevations noted in the bid documents. Based on the revised pile tip elevations, the General Contractor determined that the driven and pre-drilled H-piles would not be economically feasible. It was proposed to replace the steel H-piles with micropiles, since the extra drilling required to reach the suitable material using small diameter micropiles is much more cost-effective than using large diameter drilling. The value-engineering effort involved seven abutments and one pier for four bridge structures; the remaining structures were constructed in accordance with the contract documents.

GEOLOGIC CONDITIONS

The project site is situated in the Appalachian Mountain Section of the Ridge and Valley Physiographic province. This section is characterized by long, narrow ridges and broad to narrow valleys with some karst. Geologic structure consists of open and closed plunging folds having narrow hinges and planar folds (Map 13). Rocks exposed along the project site are of Devonian and Silurian Period, Paleozoic Era. (Map 340). Boring log information contained in the Contract Documents indicates the presence of limestone, fossiliferous limestone and calcareous to carbonaceous shale underlying the four proposed structures. The project area contains intensely folded bedding planes that generally exhibit a relative dip ranging from 70 to 90 degrees. Table 1 provides a generalized summary of the subsurface conditions at each site.

TABLE 1. Subsurface Information

Structure Name (1)	Typical Subsurface Conditions		Notable Features from the Subsurface Investigation (4)
	Bedrock Material (2)	RQD (3)	
1A	Limestone and calcareous shale	0 to 57	Solution vugs, massive clay seams, voids up to 2.6-m (8.6-ft)
2A	Limestone and fossiliferous limestone	27 to 75	Massive clay seams and 0.8-m to 1.5-m (2.6 to 5-ft) voids
3A	Limestone and carbonaceous shale	0 to 49	Very soft rock strata
5A	Limestone and calcareous shale	3 to 91	Solution vugs, clay seams and voids

Additionally, the exploratory test data indicated a highly variable top-of-rock condition, typical of karst. Figure 1 indicates the top-of-rock contours beneath the substructure footprint for Structure 5A, Abutment No. 2. Note the various pinnacle and cutter features located beneath the abutment footprint. Additionally, the presence of solution vugs, clay seams and voids

complicated the value-engineering proposal, both in terms of defining an acceptable bearing stratum as well as minimizing grout loss and avoiding the contamination of nearby residential groundwater wells.

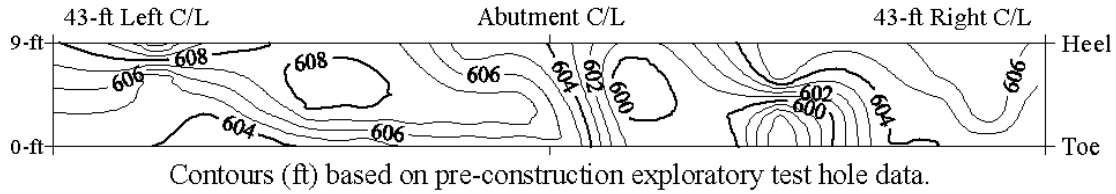


FIG. 1. Top-of-Rock Contours Beneath Abutment No. 2 Footprint, Structure 5A

MICROPILE DESIGN

The project included the redesign of the substructure foundations to minimize the number of micropiles needed. The intent of the redesign was to maintain the as-designed pile cap size and configuration for each substructure unit, while only revising the pile layout and minimizing changes to the footing reinforcement. Table 2 provides a brief description for each of the four bridges and the results of the redesign effort.

TABLE 2. Value Engineering Summary

Structure Name	Structure Type	Substructure Unit	Number of Piles	
			Original Design HP12x74	Value Engineer 178 mm (7-in) O.D. Micropiles
(1)	(2)	(3)	(4)	(5)
1A	32.6-m (107-ft) single span	Abutment 1	55	28
		Abutment 2	76	38
2A	32.9-m (108-ft) single span	Abutment 1	43	25
		Abutment 2	63	35
3A	42.4-m (139-ft) single span	Abutment 1	49	30
		Abutment 2	79	53
5A	21.3/37.8/37.4-m (70/124/90-ft) three span	Pier 2	72	50
		Abutment 2	74	36

Structure 5A presented an additional value engineering design challenge. Due to the change in the geological conditions along the length of the bridge, only the foundations for Pier 2 and Abutment 2 were redesigned. The original design required all vertical H-piles at each of the two fixed piers. For the value engineering design, batter micropiles were used at Pier 2 and the vertical H-piles remained at Pier 1. Since this change had the potential to significantly change the overall longitudinal stiffness of the bridge and therefore loads at each pier, an analysis was performed using COM624P (Lateral Load Analysis) and GTStrudl (Finite Element) programs.

Structure 5A was modeled as a 2-D frame using GTStrudl beam elements. The longitudinal spring constant for the Pier 1 H-pile foundation was determined using COM624P. Spring constants were determined from this pile analysis for both static and cyclic loading for various

loadings. Minimum and maximum spring constants were both considered in the analysis to bound the solution. Pier 2 was modeled as fully fixed at the foundation since batter piles along both pier axes were proposed. Connections at the abutments, which in reality are elastomeric expansion bearings, were modeled as freely moving except along the vertical axis. Loading for the FEA model considered longitudinal forces, as determined by the original designer, and thermal forces applied as a uniform temperature change using GTStrudl commands. For the thermal analysis, the piers were given a modulus of elasticity of 1/3 the normal modulus. Reducing the modulus accounts for the slow application of the thermal load and cracking of the concrete. Reactions, as determined using the FEA model, were used as input to the PENNDOT Pier analysis program, PAPIER, to check the adequacy of the pier as originally designed. This analysis showed that no modification of the pier or footing were required using the micropile configuration.

Micropiles are usually assumed to transfer their load to competent rock through grout-to-rock skin friction without any contribution from end bearing; since the movement to mobilize frictional resistance is significantly less than that needed to mobilize end-bearing. Although the rate of load transfer to competent rock is higher at the top of the bond length, the value considered when determining the grout-to-ground skin friction is the average value over the entire bond length (FHWA, 2000). The dependence on skin friction is considered geotechnical equivalent in tension or compression. Based on this information, a literature search incorporated both micropile and tieback grout-to-limestone skin friction data to establish a preliminary design value, that was to be confirmed by load testing. Table 3 summarizes the literature search.

TABLE 3. Review of Published Grout-to-Limestone Skin Friction Values

Source (1)	Bond Stress, kPa (psi)		
	Ultimate (2)	Allowable (3)	Avg. Allowable (4)
FHWA, 2000	1035 – 2065 (150–300)	415 – 825 (60 – 120)	620 (90)
AASHTO, 1996	910 – 1220 (132 – 177)	365 – 490 (53 – 71)	425 (62)
PTI, 1996	1035 – 1380 (150 – 200)	415 – 550 (60 – 80)	480 (70)

The literature search indicated an overall average allowable bond stress of 510 kPa (74 psi). Historic load test data from 21 previous load tests conducted in similar karst conditions across the Commonwealth indicate ultimate bond stress values that range from 683 to 2020 kPa (99 to 293 psi), with an average value of 1296kPa (188 psi) or a working bond stress of 517 kPa (75 psi). However, these micropiles were not loaded to failure, consequently, the ultimate bond stress value is unknown. The design team agreed on a working bond stress value of 482 kPa (70 psi), which was to be confirmed or refuted by site specific load test data. Figure 2 presents a typical micropile design. Specifically, the micropile design for this project

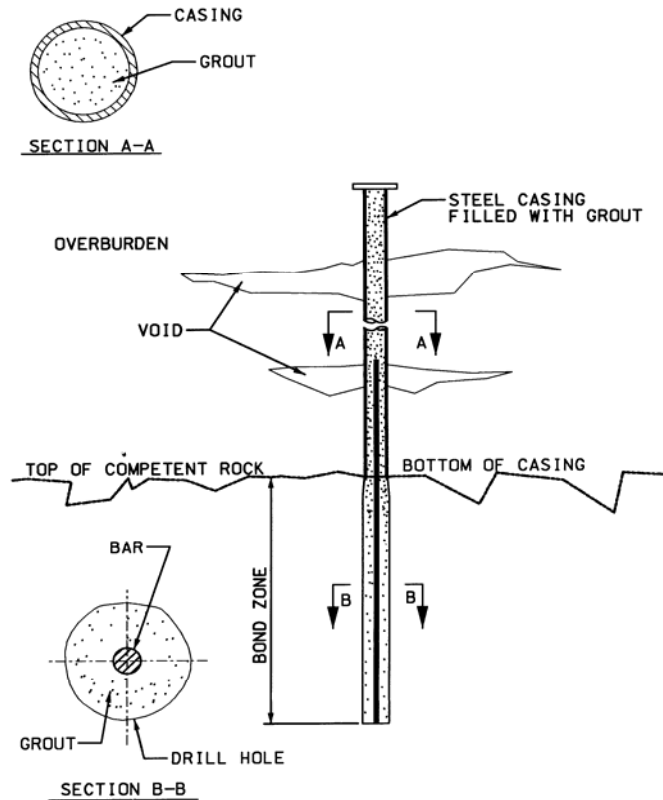


FIG. 2. Typical Micropile

considered a 0.178 m (7-in) diameter casing with a 0.197 m (7.75-in) diameter rock socket. The casing wall thickness was 0.013 m (1/2-in) with a No. 20 all-thread bar for the central reinforcement. The minimum rock socket embedment was 3.05 m (10-ft).

CONSTRUCTION

The ability to install micropiles in karst topography is a major advantage in their use as a foundation system. Since the performance of the pile is dependent on the bond between the grout and the competent rock, the integrity of the grouted bond zone in karst is of primary importance. Site-specific installation techniques must be selected with this in mind. This capability is gained principally by the optimal selection of drilling and grouting techniques.

In the opinion of the authors, micropiles are best installed in karst using rotary eccentric percussive duplex drilling. This method uses an inner rod and an outer casing, with the spoils flushed inside the casing. The bit on the inner drill rod is equipped with a down-the-hole hammer. The hammer bit is specially designed to open up during drilling to a diameter slightly larger than the outside diameter of the drill casing (see Figure 3).



FIG. 3. – Eccentric Hammer Inside the Casing

This bit provides a slightly oversized hole through obstructions or rock and thereby allows the simultaneous advancement of casing. Compressed air is used to drive the hammer and also acts as the drilling fluid to lift the cuttings. This drilling method is used in soils containing large amounts of obstructions such as cobbles, boulders or demolition waste and is very effective in advancing a drill casing through highly fractured rock zones in karst. Because near intimate contact between the casing and the surrounding soil and rock is constantly maintained, this method is highly effective for micropile installation in karst.

The response of the down the hole hammer indicates whether rock of sufficient quality is penetrated. Figure 4 indicates the difference between the top-of-rock contours from the exploratory test hole data and the as-built micropiles. This data shows the significance of the drilling techniques used in determining the actual bond zone.

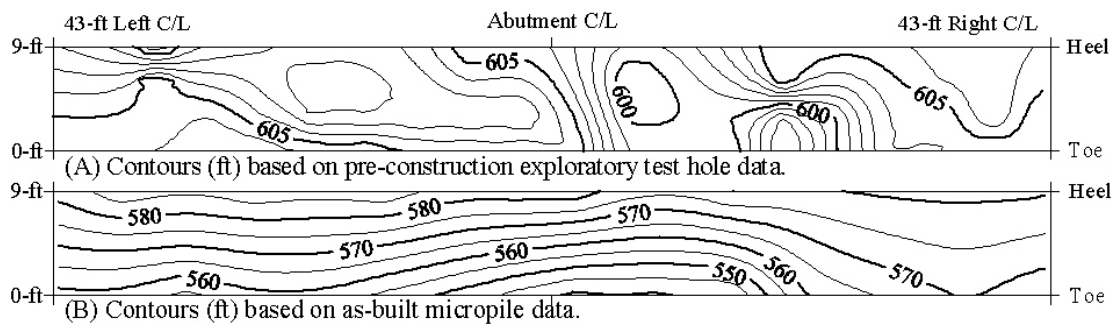


FIG. 4. Top-of-Rock Contour Comparison Beneath Abutment No. 2 Footprint, Structure 5A.

Final micropile tip elevations were determined during the drilling process. If during drilling the bond zone, voids or fractures were encountered that sum to less than 0.15 m (6-in) in total length, the bond zone would remain unchanged. If voids or fractures sum to greater than 0.15 m (6-in) but less than 0.30 m (1-ft) in total length, the bond zone would extend an additional 0.30 m (1-ft). However, if voids or fractures sum to greater than 0.30 m (1-ft), bond zone drilling would extend until either of the above referenced scenarios were satisfied.

Once a competent bond zone is established, the casing is withdrawn to the top of the bond zone and the micropiles are tremie grouted by lowering a grout tube to the bottom of the drill casing and rock socket. Grout was pumped through the tube as the tube is slowly removed from the hole. As the grout fills the drill casing or hole, it displaces the drilling fluid and / or water. Tremie grouting is primarily used where the micropile bond zone is founded in rock. When working in highly broken and fractured rock or in voided, karst situations, grout loss is possible and may warrant testing for a sealed bond zone. In order to greatly reduce excessive grout takes in the fractured rock, an anti-washout additive is used in the neat cement grout mix. Once the grout level stabilizes in the bond zone, the centralized reinforcing steel is placed.

LOAD TESTING

A total of seven load tests were conducted to confirm or refute the value engineering design and construction procedures. Load testing was in accordance with ASTM D1143 – Section 5.6 – Quick Load Test to a maximum test load of 1780 kN (400 kips). Pile head displacement vs. applied load graphs were developed for each load test in accordance with FHWA, 1992. The failure load was determined by the Offset Limit method (FHWA, 1992). Figure 5 presents a pile head displacement vs. applied load graph for a project micropile that crossed the failure criterion; this micropile was bonded in carbonaceous shale. Figure 6 presents the pile head displacement vs. applied load graph for a project micropile that did not cross the failure criterion; this micropile was bonded in limestone. Table 4 presents a summary of the load test data from the project site.

Structure 3A - Abutment No. 1

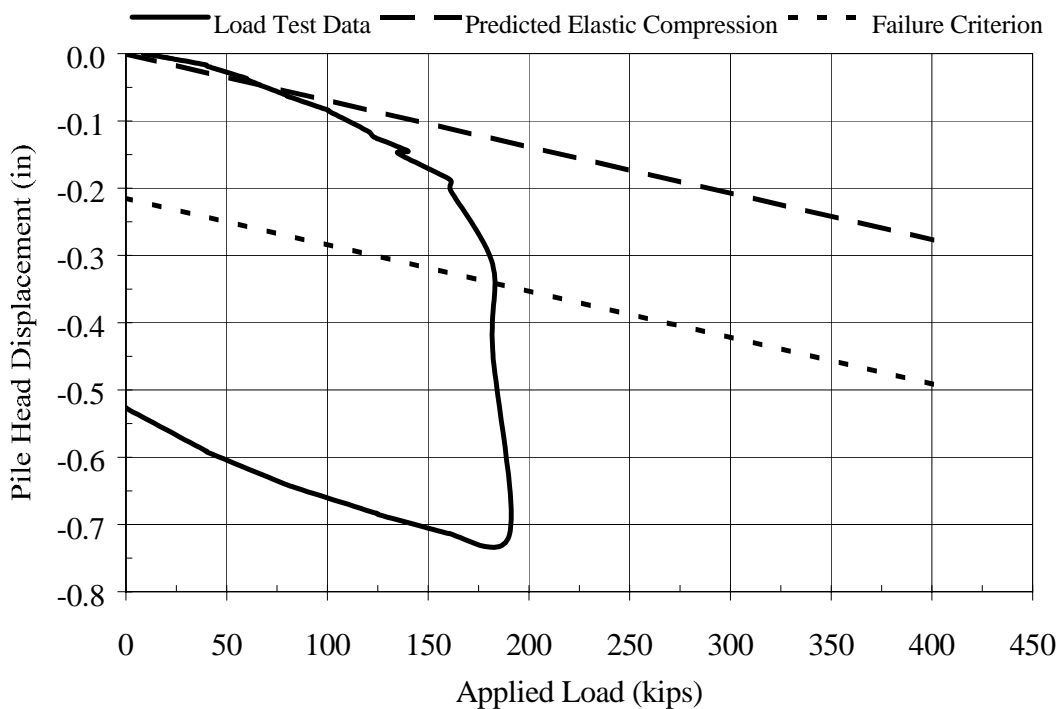


FIG. 5. Pile Head Displacement vs. Applied Load

Structure 1A - Abutment No. 1

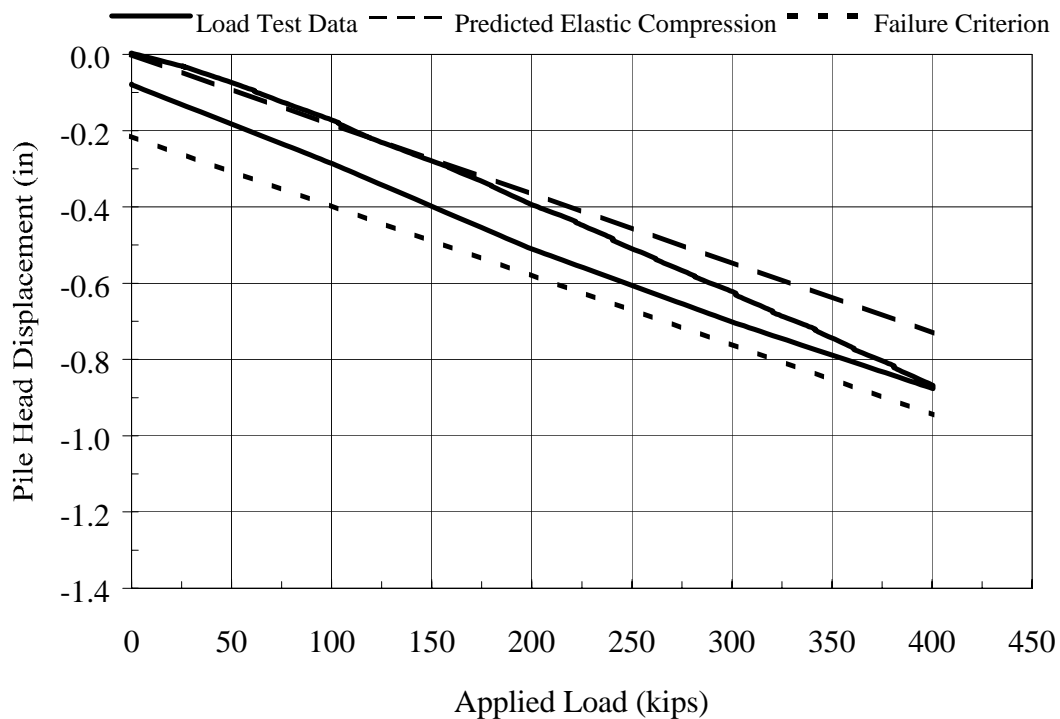


FIG. 6. Pile Head Displacement vs. Applied Load

**TABLE 4. Summary of Project Micropile Load Test Data
Maximum Test Load = 1780 kN (400 kips)**

Structure Name (1)	Predicted Elastic Compression at Maximum Test Load (mm (in)) (2)	Failure Criterion at Maximum Test Load (mm (in)) (3)	Compression at Maximum Test Load ^(a) (mm (in)) (4)	Residual Compression (mm (in)) (5)	Calculated Ultimate Bond Stress ^(b) (kPa (psi)) (6)
1A / 2 A	Abutment No. 1 – Bonded in limestone with shale interbeds				
	18.57 (0.731)	24.03 (0.946)	22.25 (0.876)	1.98 (0.078)	>965 (140)
	Abutment No. 2 – Bonded in limestone				
3A	Abutment No. 1 – Bonded in carbonaceous shale				
	7.04 (0.277)	12.50 (0.492)	18.14 (0.714)	13.36(0.526)	207 (30)
	Abutment No. 1 (re-test) – Bonded in carbonaceous shale				
	14.05 (0.553)	19.51 (0.768)	19.10 (0.752)	2.59 (0.102)	>207 (30)
	Abutment No. 2 – Bonded in limestone				
	18.29 (0.720)	23.75 (0.935)	17.50 (0.689)	3.00 (0.118)	>965 (140)
5A	Pier No. 2 – Bonded in shaly limestone				
	15.77 (0.621)	21.06 (0.829)	16.46 (0.648)	1.78 (0.070)	>965 (140)
	Abutment No. 2 – Bonded in limestone				
	21.54 (0.848)	27.00 (1.063)	26.16 (1.030)	1.35 (0.053)	>965 (140)
(a) The micropile load-movement curve for Structure 3A, Abutment No. 1 test pile exhibited a plunging type failure and crossed the failure criterion at 180 kips, as shown on Figure 5.					
(b) When the calculated bond stress indicates “>”, the load-movement curve did not cross the failure criterion, consequently the ultimate bond stress is not known, but is at least equal to the indicated value; typical example is shown as Figure 6.					

CONCLUSIONS

Micropiles are applicable for new bridge construction requiring deep foundations. This deep foundation system is particularly advantageous in difficult ground conditions, such as boulders, obstructions, mine voids and karst. Installation techniques enable a production micropile to also serve as a verification hole by observing hammer response. Design codes often require increased element redundancy and / or decreased element resistance for deep foundations over karst to account for uncertainty during construction. Micropiles provide an opportunity to reduce this redundancy while providing a verification process during construction. Innovative construction techniques and grout mix design minimize grout loss in karst. Moreover, load test data performed prior to installing production piles provide data to confirm and / or refute the design and installation process. This project demonstrated how prudent design, construction and inspection techniques further advanced the development of micropile technology.

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