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The Treatment by Jet Grouting of a Bridge Foundation on Karstic Limestone

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Abstract

During construction activities for Pier 3 of the new Bill Emerson Memorial Bridge, over the Mississippi River at Cape Girardeau, MO, major karstic features were found in the limestone bedrock. After review of various construction options, it was decided to use 3-fluid jet grouting to treat the features in the foundation rock mass from its surface elevation of about El. 250 feet (76 m) to El. 200 feet (61 m), over a footprint of 67 feet (20 m) by 109 feet (33 m). The work was conducted from the deck of a cofferdam, through about 70 feet (21 m) of water, placed sand, and river bed sediments. A key to the success of the project was the very intense QA/QC and verification programs adopted.

Introduction

Missouri Route 74 adjoins Illinois Route 146 on a structure over the Mississippi River at Cape Girardeau, Missouri (Szturo and Chuaqui, 2000). This structure, completed in 1928 is a through truss structure which is not only functionally obsolete, but insufficient for anticipated traffic volumes. The Missouri Highway and Transportation Commission elected to replace this bridge with a new 4-lane crossing located immediately downstream of the

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existing bridge. With a minimum horizontal clearance of 800 feet (244 m), the conceptual structure types considered were limited to through truss spans, trussed arch spans, and cable-stayed spans. Pier location considerations were an integral part of the study and the location of substructure elements as well as the effect on the superstructure span lengths (ultimately a cost consideration) were key to the recommendation to design and construct a 3-span, symmetrical cable-stayed navigation unit. At the time of the conceptual studies (completed in October, 1991), there were no subsurface investigations, but the foundation requirements were estimated based on the information shown on the original bridge plans. Borings were taken at each pier location in early 1992 and the logs and subsequent laboratory testing were the basis for the project Geotechnical Investigation Report of April, 1992, from which the following data are synthesized.

Site Geologic Information

The project is located on the eastern border of the Ozark Uplift and the Southwestern border of the Illinois Basin. At the bridge site, the Mississippi River is located at the extreme western edge of the alluvial plain. Bedrock outcrops on the west bank, with the City of Cape Girardeau constructed adjacent to the bank. To the east (Illinois), the alluvial plain extends approximately five miles. At the east bank, bedrock is approximately 100 feet below the surface. Just south of the site, the coastal plain deposits of the Mississippi delta begin and extend for hundreds of miles to the Gulf of Mexico.

The bridge is located approximately 50 miles (80 km) north of New Madrid, Missouri. The New Madrid region has been the most seismically active region of central and eastern North America. Historical data indicate that the famous 1811-1812 events created surface wave magnitudes of around M 8.5. Between 1813 and 1990 over 23 earthquakes having magnitudes of 4.5 or greater have been documented in the New Madrid area. Considering that M 8 or larger events are anticipated every 550 to 1,200 years, the design earthquake is essentially a repeat of the 1811 and 1812 events.

The Ordovician bedrock at the site comprises mainly the Plattin and overlying Kimmswick Formations. The Kimmswick outcrops in the bluffs to the west of the bridge site and consists of coarsely crystalline, nearly pure limestone. Heavy solutioning is readily visible in the outcrops of the Kimmswick in the bluffs. The solutioning of the bedrock in this area is not cavernous but appears as widened joints filled with various materials. The initial borings taken at Pier 3 in the river found limestone of the underlying Plattin Formation, a very hard and brittle limestone containing various amounts of chert. As evidenced in a nearby quarry, the Plattin Formation is several hundred feet thick.

Geologic maps of the area indicate faulting, the majority inferred, since most of the bedrock is obscured by surficial soils, urban development and the alluvial plain. Most of the faults are considered normal, although some movement is thought to have been strike-slip. The faults are considered inactive, although some are thought to have occurred near the beginning of Holocene time.

At the onset of preliminary design, a geotechnical investigation was undertaken to characterize subsurface conditions at each of the proposed pier locations to MODOT standards. Where borings indicated unusual conditions, additional borings were requested and accomplished. Solutioning, karstic conditions and poor rock conditions were noted in minor amounts at areas near the West Abutment, Pier 4, and occasionally at the east approach structures. At Pier 3, however, the borings taken indicated limestone of excellent quality with near perfect recovery and RQD.

Pile Foundation Construction Procedures and Challenges

The west abutment is founded on drilled shafts with rock sockets in the underlying Plattin Limestone Formation. The west tower pier (Pier 2) for the cable stayed main span is founded on a massive spread footing (approximately 60 ft (18 m) x 100 ft (30 m) x 12 ft (4 m)) bearing on limestone bedrock, also of the Plattin Formation. During construction of these structures minor solutioning of the bedrock was noted, as anticipated, but did not pose significant problems. The mid river tower (Pier 3) was planned to be founded on a 67 ft (20 m) x 109 ft (33 m) dredged caisson with a 5-foot (1.5-m) key excavated into the underlying bedrock. This dredged caisson was to be sunk inside a sand-filled sheet pile cofferdam (Figure 1). Pier 4 was also to be constructed on a smaller dredged caisson with a rock key. Piers 5 through 14 were to be founded on groups of drilled shafts with rock sockets extending into the underlying bedrock.

As the first step of construction of Pier 3, the Contractor elected to blast the required 5-foot (1.5-m) keyway into the underlying bedrock, which was found at about El. 250 ft (76 m). The blasting plan called for a perimeter of closely spaced pre-split holes along the perimeter with series of production blasts in the interior. The closely spaced pre-split holes were to form the neat line of the keyway, and were drilled from a barge anchored on location in the river (approximate El. +320 ft (97.5 m)) with a template for the locations secured to the edge. Variations in the surface elevation and quality of the bedrock and problems with the percussive drilling were noted immediately. Some locations along the perimeter indicated a lack of competent rock and interconnectivity between the pre-split holes. The drilling and blasting operations were halted and a supplementary exploration program of coring the bedrock was undertaken comprising 23 additional borings.

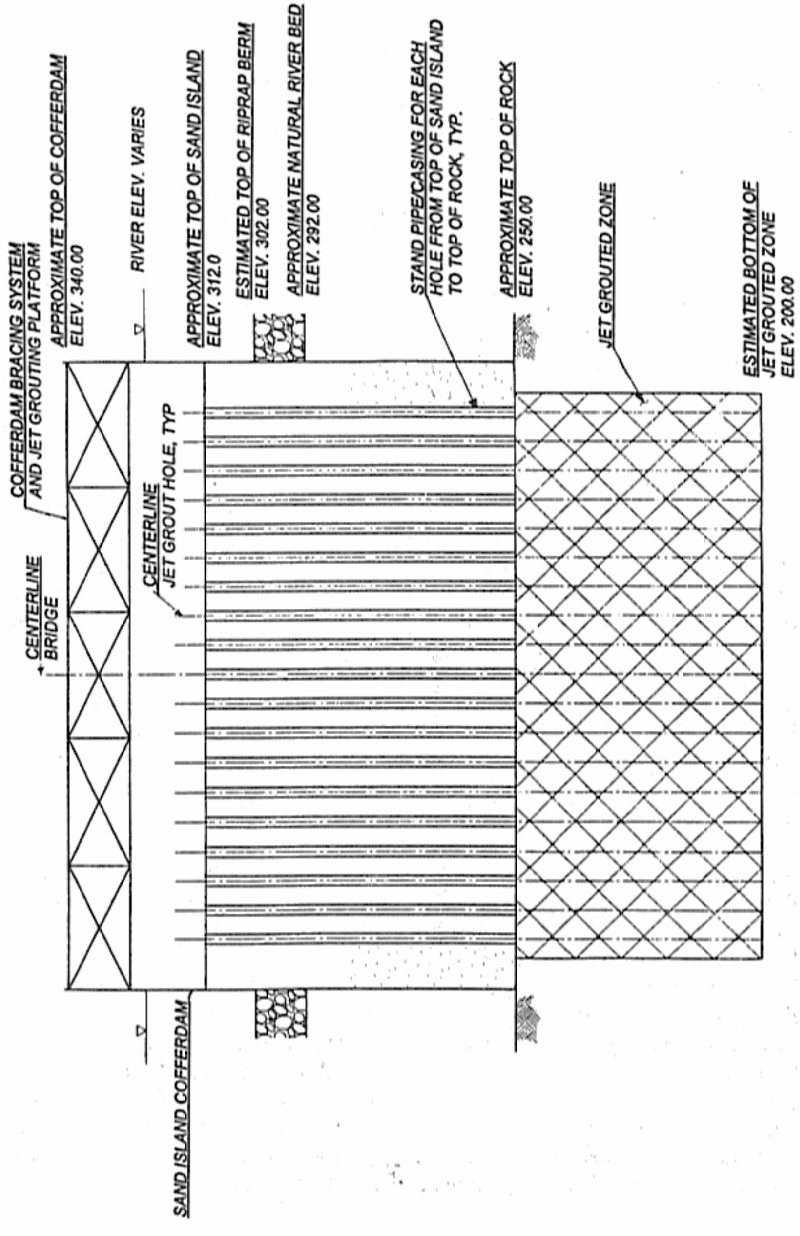

Figure 1. Elevation of treatment, from cofferdam, through water and sand island.

Most of the cores (NQ triple tube) extended 50 feet (15 m) into the rock, i.e., to about El. 200 ft (61 m), and geophysical logging was also utilized. A large number of significant karstic features were found to exist throughout the extent of the foundation, and their occurrence seemed to diminish with depth.

The primary joint set in the limestone had a strike of 5°, the secondary of 90°, and the tertiary of 45°. Most of the solution features were associated with the vertical primary joint set. Analysis of the additional cores indicated that interior half of Pier 3 would be located on competent rock and the west and east quarters on solution widened joints. These features revealed no voids or open cavities but had a filling of residual clays, with various amounts of chert. Some of the filling also consisted of alluvial material scoured and replaced in the rock, or rock in various stages of deterioration.

Possible Solutions

Possible alternative solutions for the Pier 3 foundation included a) increasing the depth of the foundation; b) increasing the plan area of the foundation; c) replacing the dredged caisson with a tremie footing; d) replacing the dredged caisson with drilled shafts; e) adding micropiles beneath the dredged caisson; and f) improvement of the founding limestone with ground treatment. On grounds of cost, performance, and constructability, option (f) was pursued.

Ground treatment was considered superior to any of the structural modifications because the treated karst would allow the limestone bedrock to act as anticipated during the design earthquake, and to minimize (or even eliminate) changes in the structural behavior of the bridge. Both compaction grouting and permeation grouting were considered inappropriate for this site and the project goals, and therefore various jet grouting concepts were pursued in order to systematically locate the karstic features and to largely replace them with an engineered material. Jet grouting had the further advantages of improving the bearing capacity of the foundation without modifying the stiffness of the bridge and impacting its calculated response to various loading cases; of providing the Contractor with flexibility of scheduling (i.e., before or after foundation construction); and of being apparently less costly then the other alternatives.

Aspects of the Jet Grouting Option

General

Since physical inspection of the rock surface was practically impossible, there was a very strong emphasis on process verification, and on the selection of appropriate treated karst properties. Structural analyses indicated a peak bearing pressure of 29 tsf (2.8 MPa), so, to provide a 25% increase to that, in the upper elevations, a treated karst strength of 500 psi (3.4 MPa) minimum was specified. These analyses, coupled with what was inferred about the vertical intensity of the solutioning dictated a maximum depth of treatment of El. 200 ft (61 m) (Figure 1).

Jet Grouting

Three-fluid jet grouting was specified since the goal was to remove as much of the karstic clay as possible from the foundation mass, and to provide a minimum 500 psi (3.4 MPa) soilcrete strength. In addition the larger diameter columns capable of being created by three-fluid methods would reduce the amount of drilling required in the logistically challenging conditions. Three-fluid jet grouting would also be advantageous in helping to quantify the degree of clay removal, and, vitally, would ease slurry expulsion by reducing the slurry viscosity (Burke et al., 1989). This contractual requirement of the Contractor was strongly emphasized in the Specifications. Given the extremely variable nature of karstic limestone terrains, it was not realistic to believe that all the clay could be located and removed. However, treated columns were conceived as providing stiff lateral "buttressing" between the competent, unweathered elements of the rock mass, so preventing these slabs moving horizontally significantly during a seismic event.

Specification and Procurement

In order to permit prospective contractors to most appropriately select and utilize their own particular expertise, many of the key operational decisions were left to the bidders themselves as summarized in Table 1.

Column Confirmation Testing

The specification required that a column confirmation test (CCT) be performed in a structurally less critical zone in Pier 3, along the centerline of the future bridge, to confirm that the Contractor's proposed means and methods would meet the performance and acceptance criteria established in the specification.

Hole Layout

Since the precise locations and extent of features in a karstic limestone are largely unpredictable, a systematic approach to treatment geometry was necessitated. A regular distribution of holes was shown on the plans as a common basis for bidders (Figure 2). However, provision was made to install

Table 1. Summary of specification requirements and responsibilities.

FACTORS OR CRITERIA ESTABLISHED IN SPECIFICATION	ITEMS OR FACTORS FOR CONTRACTOR TO SELECT/EXECUTE
Extent and depth of treatment (between El. 200 ft (61 m) and 250 ft (76 m))	Study all available data
Contractors' experience levels	Provide staff, equipment and instrumentation
Three fluid jet grouting to be used	Meet listed performance criteria
Locations of test area	Submittals for means and methods
Standards for materials	Quality control testing
Grout mix to be stable (low bleed)	Design of methods and grout mix
Strength acceptance criteria (500 psi (3.4 MPa))	Selection of jetting parameters
Minimum acceptable length of lateral treatment (3 ft (0.9 m)) from any drill hole	Methods for spoils control and measurement
Need for acceptable test areas prior to production grouting being permitted	Execution, scheduling, and sequencing for all facets of the program
Selection of coring locations	Coring
Core acceptance criteria	Grouting and geophysical testing area (CCT)
Locations for primary holes	Wet grab sampling
Need for ensuring efficient spoils venting	Automatic parameter recorder
	Quality control and grouting records

the columns in a Primary-Secondary-Tertiary sequence, with the exact locations of the higher order holes determined in the field based on progressive analysis of drilling and grouting data from the Primary holes.

Column Confirmation Testing and Limited Production Grouting

The Contractor attempted three column confirmation tests (CCT-1 through CCT-3) that for various reasons did not adequately demonstrate his ability to meet the performance criteria to the satisfaction of the Owner. A fourth column confirmation test (CCT-4) was executed on a force account basis with procedures and parameters established by the Owner and his consultants with input from the Contractor. The results and observations from this test were sufficiently promising that limited production grouting based on modified CCT-4 procedures was initiated. Limited production was confined to the areas

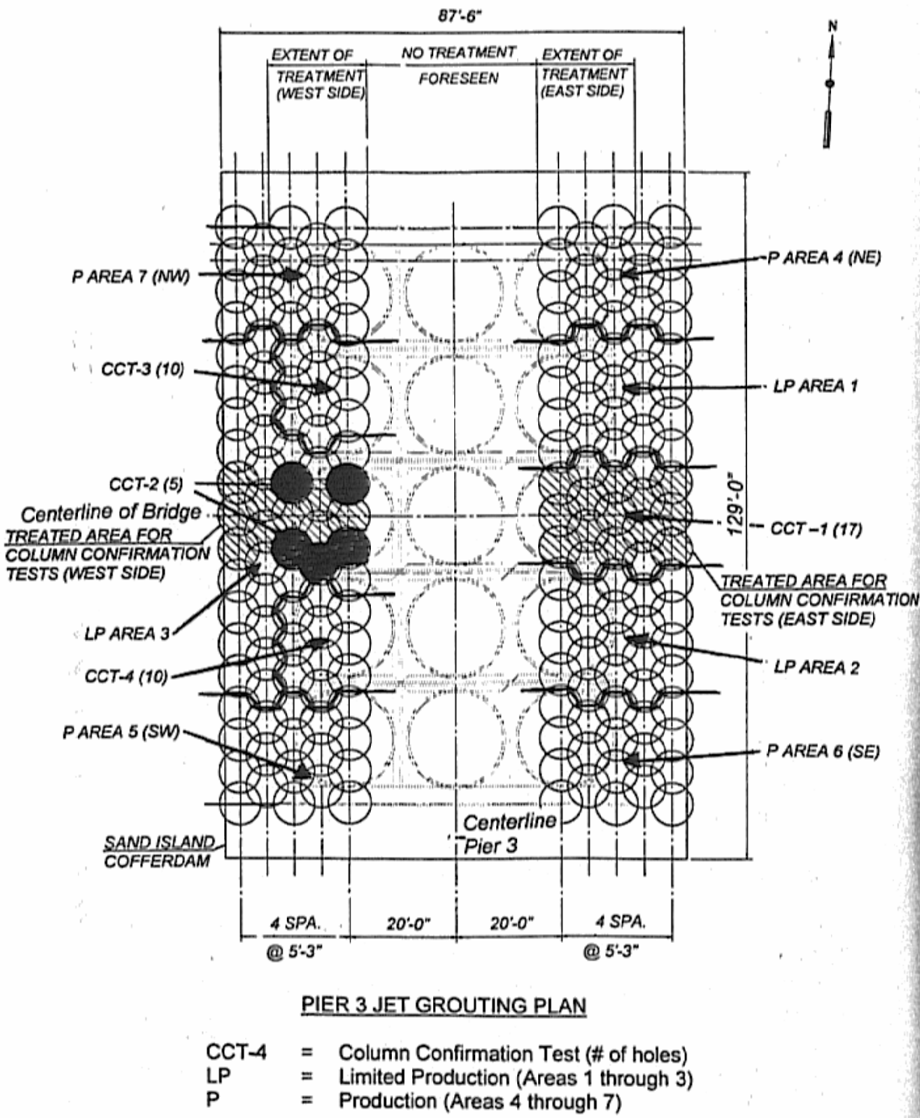

Figure 2. Conceptual grout hole layout, and CCT area locations.

adjacent to the CCT testing zone but still away from structurally the most critical areas, the corners of the footing.

The original strength criterion of 500 psi (3.4 MPa) was based on an analysis of peak bearing pressure at El. 250 ft (76 m). Since the bearing pressure would dissipate rapidly with depth, the strength criterion below El. 240 ft (73 m) was relaxed to 200 psi (1.4 MPa) for subsequent production grouting. This change was made to help the Contractor's progress and to save money.

Verification coring followed closely behind limited production grouting and, after adequate ground treatment had been confirmed by the coring, the Contractor was allowed to start full production grouting in the structurally critical corners also.

Means and Methods Employed to Execute the Grouting

The drilling and grouting operations were conducted from an existing platform over the cofferdam. Key elevations are shown in Figure 1. Barges were used for the other support operations, including cement storage, grout batching, grout pumping, and spoils containment.

Steel casing of 12¾-inch (324-mm) diameter was drilled to top of rock, or to a maximum depth of El. 240 ft (73 m). The drilling method was rotary (i.e., an open ended casing with a cutting shoe and water flush). A down-the-hole hammer then drilled a 9-inch (229-mm) hole through the formation. The drilling progress was carefully logged to identify the location of karstic features. The hammer was then removed from the hole and an 8-inch (203-mm) i.d. PVC pipe was installed to top of rock or the bottom of the casing. Once the PVC pipe was installed, the steel casing was removed.

The purpose of the PVC pipe was to provide a pathway for the spoils from the jet grouting to vent at the working platform. If top of rock was not encountered prior to El. 240 ft (73 m) or the PVC did not reach top of rock, a short zone below the bottom of the PVC was jetted prior to full depth treatment ("downstaging"). This was done to help ensure that sand or alluvium would not collapse into the hole and inhibit spoils venting during grouting.

Each hole was then redrilled with the jet grout rig, which was equipped with specialized three fluid jet grouting rods and a monitor. The upper nozzle (air/water) was spaced approximately 3 feet (1 m) above the lower nozzle (grout). Once the hole was redrilled to full depth the jet grouting operations commenced. The monitor was then rotated and lifted at controlled rates.

The jet grouting parameters finally selected were

- Air Pressure – 10 to 15 bar (1 to 1.5 MPa);
- Water Pressure – 350 to 450 bar (35 to 45 MPa);

- Water Flow – 70 to 80 l/min;
- Grout Pressure - 350 to 450 bar (35 to 45 MPa);
- Grout Flow – 310 to 375 l/min;
- Lift Rates
  - Limestone – 5 seconds per 1.5 inch (4 cm) lift;
  - Feature between El. 200 ft (61 m) to 240 ft (73 m) – 35 seconds per 1.5 inch (4 cm lift);
  - Feature El. 240 ft (73 m) to 250 ft (76 m) – 48 seconds per 1.5 inch (4 cm) lift in feature.

These parameters resulted in an applied specific energy of 250 MJ/m below El. 240 ft (73 m) and 330 MJ/m above El. 240 ft (73 m).

The grout had a water:cement ratio of 0.8 by weight, with super-plasticizer at 1.25% by weight of cement and hydrated bentonite at 3% by weight of water (to provide volume stability if injected into large features). During water jetting, a clay dispersant was used at 0.75% by weight of cutting water to enhance the efficient removal of clays and ease the routine venting of spoils from the formation.

Verification Methods

The verification of the grouting effectiveness was of the highest priority. A rigorous QA/QC program was initiated at the beginning of the project and maintained throughout to document and eliminate variations in the grouting parameters and methods. For production and limited production grouting, "column evaluation sheets" were developed to provide a method of quickly communicating the Engineer's field observations and comments to the Contractor. A column evaluation sheet was filled out for each column installed and a copy provided to the Contractor when the column was completed. For each specific process or phase of the column installation, the work was evaluated with respect to being in general conformance with the specification and the Contractor's approved procedure (based on field observations).

A spoils testing program was implemented: samples were taken from the spoils line at regular intervals during grouting and tested to determine cement content based on the heat produced to neutralize the soil-cement sample. The clay content and water content could then be calculated.

Potential Impact of Non-Conformant Work

The distinction between non-conformant versus non-acceptable work was often debated during the execution of the work. Non-conformities were recorded in grout rheology, parameter recording, wet grab sampling, record

keeping, standpipes, dispersant usage, operational parameters, and spoils venting. It was always felt that inefficient spoils venting was the most critical, as earlier noted by Bell (1993), Xanthakos et al. (1994), Sugawara et al. (1996), and ASCE (1997). If spoils cannot vent easily and are difficult to remove, back pressure builds up in the formation, which reduces the effectiveness of the jetting resulting in treatment of limited diameter and highly variable composition. Further, ASCE (1997) stated, "the European code of practice for jet grouting supports the need to assure a continuous flow of spoil return to the surface during jetting. It is very important to understand that control of the return spoil ensures pressure control of the in situ erosion environment so that no energy is wasted hydrofracturing the soil in lieu of eroding the soil."

The coring from limited production columns confirmed that columns constructed with adequate venting of spoils produced better ground treatment than columns constructed with poor venting of spoils. Columns that were grouted with poor spoils venting over large sections of feature were recommended for replacement. In total, 10 of the approximately 190 columns installed during production and limited production grouting were replaced due to concerns arising from poor spoils venting.

Verification Program

As part of the first Column Confirmation Test (CCT1), pre- and post-grouting seismic surveys were conducted. The seismic testing was not intended to be part of the acceptance criteria per se. Rather the data were intended to illustrate the overall improvement of the foundation mass (i.e. reduction in distribution of significant low velocity clay filled karst) as a result of the jet grouting. Originally conceived as being a global demonstration to be conducted over the entire foundation, the seismic testing program was actually used as a local verification of the area treated during CCT-1 only. It provided evidence that low velocity zones had been transformed into higher velocity zones by the jet grouting and that the treatment was providing general benefit to the foundation.

Wet Grab Sampling

The Contractor was required to obtain samples of the treated soil by wet grab sampling from the freshly jet grouted holes at locations and depths selected by the Engineer. Cubes were prepared and tested for unconfined compressive strength in accordance with ASTM 109 at 7 and 28 days. The anticipated sampling frequency was one sample per column during limited production grouting and two columns per week during production grouting.

The specification required the average compressive strength of all treated soil samples tested to be greater than the design strength (500 psi (3.4 MPa) later relaxed below Elevation 240 ft (73 m) to 200 psi (1.4 MPa)). Samples with less than 70 percent of the design strength would indicate non-conformance. Of the 56 columns installed during limited production, 42 wet grab samples were retrieved. Compressive strengths for wet grab samples were generally high with only one 28-day strength under 500 psi (3.4 MPa).

The number of wet grab samples required was reduced to two per week when the Contractor started working in the corners as per the specifications for production work. Of the wet grab samples obtained during production, the strengths continued to be high, with most 7-day breaks above 500 psi (3.4 MPa).

Verification Coring

The Contractor was required to obtain samples of the treated soil by coring and to test these samples for unconfined compressive strength in accordance with ASTM D2938-86. The specification required the use of triple tube and wireline methods, and required that the coring methods and treated ground conditions to be such that 90% recovery be obtained as part of the acceptance criteria. It was anticipated that 10% of the columns would be cored during production. An estimated four samples of core were to be selected by the Engineer from each hole for testing. The specification required the average compressive strength of all treated soil samples tested to be greater than the design strength. Samples with less than 70 percent of the design strength would indicate non-conformance. The core locations were selected by the Engineer.

A total of 13 holes (LP1 through 13) were cored in the limited production areas. Core locations for LP1 to LP9 were selected to intersect columns which had been grouted in conformance with the specifications. The goal was to determine if adequate treatment was being achieved when conforming procedures had been followed. The total amount of feature that would have been encountered in Holes LP1 through LP9, if drilled prior to grouting, was 44%. The post grouting results showed only 9% untreated feature or 4.5 feet (1.4 m) in 50 feet (15 m).

Core locations for LP10, LP11 and LP 13 were selected to intersect columns that had been grouted with some significant variation to the approved procedure (non-conformance). The goal was to investigate the effect of these variations on the quality of the ground treatment. Core holes LP10 and LP11 were located to intersect columns assessed to have poor venting of spoils. Core hole LP12 was selected to intersect a column installed through a smaller diameter PVC pipe. Core hole 13 did not intersect any feature (i.e., recorded solid rock), while the two holes with poor spoils venting (LP10 and LP11) if

cored prior to grouting would have encountered 44% feature. The post grouting results showed 22% untreated feature.

Even with best available coring practice, the coring process disturbed the cores making it difficult to select samples of sufficient length for testing. In addition, handling also damaged some of the samples, which likely had sufficient unconfined compressive strength but sheared easily along the axis perpendicular to the core. However, of the samples that were tested, only one sample did not meet the 500 psi (3.4 MPa) criterion above El. 240 ft (73 m), and all samples met the 200 psi (1.4 MPa) criterion below El. 240 ft (73 m). The average core recovery for holes LP1 through LP13 was 89.4%. The drill penetration rates and composition of return flush were studied in areas of core loss to classify the material being penetrated. The reason for core loss was for the most part due to soft untreated material, but on some occasions the core loss was due to chert grinding the core within the core barrel or mislatching during coring. The grouting records for neighboring columns, as well as the distribution and elevation of soft zones were all reviewed prior to releasing the Contractor to perform production grouting in the corner areas.

A total of 10 holes (P1 through P10) were cored in the production areas in strategically located positions (Tables 3 and 4). Any zones showing untreated clay were carefully analyzed to determine if remediation was warranted. Of the core holes with less than 90% recovery:

- Areas around P3 and P4 were remediated by installing an adjacent additional jet grout column.
- Areas around P7 and P7A were judged unnecessary to be remediated based upon the elevation and distribution of the soft material.
- P8 was not remediated based upon some of the core loss being ascribed to chert grinding the grouted core, and also upon the elevation and distribution of the soft material.

Excluding data from P3 and P4 (since these areas were later remediated), the cores showed 56% limestone, 35% treated feature, and 9% clay, identical results to the Limited Production area data. All but one of the core samples met the compressive strength criteria.

Conclusions

The verification data indicated that the Pier 3 foundation had been improved to an acceptable standard. These data included wet grab samples that met the specified strength criteria, core recovery that on average met the 90% recovery criteria, core with less than 10% overall untreated feature, and core break data that met the specified strength criteria. In addition, the observations made during column installation and geophysical testing were also consistent with

Table 3. Production Column Coring Data

HOLE	AREA*	LIMESTONE (M)	UNTREATED CLAY (M)	GROUTED MATERIAL (M)	TOTAL LENGTH (M)
P1	4	10	0	3	13
P2	4	4	0	9	13
P3	6	4	4	6	14
P4	6	3	4	6	13
P5	5	3	0	12	15
P6	5	4	0	11	15
P7	7	1	3	6	10
P7A	7	6	3	2	11
P8	7	2	3	11	16
P9	7	6	0	8	15
P10	6	6	2	7	15
Total		49	19	81	150
%		33%	13%	54%	100%

*Production area as shown in Figure 2.

Table 4. Production Column Recoveries

CORE HOLE	AREA*	DEVIATION (DEGREES)	RECOVERY (%)
P-1	4	Less than 1	96
P-2	4	1.0 - SW	95
P-3	6	2.0 - N20E	66
P-4	6	2.0 – N80W	89
P-5	5	1.0 – S	96
P-6	5	0.5 – W	95
P-7	7	0.5 – S	82
P-7A	7	1.75 – NE	
P-8	7	0.5 – SW	80
P-9	7	0.5 – NE	99
P-10	6 angled	24 – S10E	90

*Area as shown in Figure 2.

adequate ground treatment. These included verification of generally consistent parameters and procedures, and spoils testing. The authors believe that of all the factors influencing the quality and extent of treatment, those processes influencing the efficiency of spoils returns during grouting were by far the

most significant. In this regard, the selection of appropriate borehole and drill tooling diameters, injection parameters and mix design (for both grout and jetting water) was critical.

The next contract has been let and construction of the bridge is currently underway, including the completion of the Pier 3 structure.

Acknowledgments

The Missouri Department of Transportation is the Owner of the project. Geosystems, L.P. (formerly ECO Geosystems, Inc.) were specialty subconsultants to HNTB Corporation for the ground treatment and also provided construction inspection services. Nicholson Construction Company were the jet grouting contractors. HNTB Corporation performed structural studies, preliminary and final design, preparation of plans, specifications, and cost estimates for the new bridge.

Special thanks are due to John F. Szturo (HNTB), Marcelo Chuaqui (MC Grouting, Inc.), and the field and headquarters staff of MODOT for their excellent contributions to the work and to the contents of this paper.

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