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ABSTRACT

Expansion and growth in the southwestern region of Missouri necessitated the expansion of State Route 249 and the construction of a new interchange to provide service to the Joplin, Missouri area. The project is located above a former lead and zinc mine in Jasper County, Missouri and includes a five bridge interchange connecting State Route 249 and US Route 171. The variable subsurface conditions, both natural and manmade prompted the design team to use ground improvement via grouting and small diameter micropiles to provide support for several of the bridge foundations on the project.

The scope of work included mine shaft closures, 17,070 m (56,000 ft) of overburden and rock drilling, 3,400 m (11,155 ft) of micropiles, 400 m^3 (524 cy) of balanced/stabilized high mobility grouts, and over 6,800 m^3 (8,900 cy) of low mobility grout. The selection of the grout used was based on the actual subsurface conditions. Low mobility grout (LMG) was used in voided conditions and for closure of the mine shafts encountered during the excavation. High mobility grout (HMG) was used in fractured rock with the goal of improving the mechanical properties of the rock underneath the future bridge footings and controlling grout volumes during micropile installation. The split spacing method was utilized for both LMG and HMG holes.

Geology of the project consisted of extremely variable bedrock with strong to very strong limestone, chert, breccia, extremely weak shale, and weak to strong sandstone in conjunction with the activities associated with the mining disturbance (such as partial filled vertical mine shafts, shallow and deep mine horizons, modified hydrology including artesian conditions). Real time monitoring and recording of all drilling and grouting parameters was conducted to assist in the evaluation of in-situ geological properties of the site in order to modify the ground improvement and micropile program as necessary.

This paper will discuss the design and execution of the ground improvement and micropile program. The project is an excellent example of the use of multiple ground improvement and foundation support techniques combined with real time data analysis to provide a foundation support solution for a complex geological environment.

BACKGROUND

Geology

The project area is situated within the Ozark Plateau physiographic province, a gently uplifted plateau of nearly horizontal sedimentary rocks. As the area is on the far west flank of the Ozark Dome, the dip is gently to the west – northwest at about 3 m per kilometer. The plateau has been eroded to form a topography of rolling hills.

Structurally, the area is controlled by the northwest – southeast trending Joplin Anticline and parallel east adjacent Webb City Syncline. References indicate the mineralization of the area appears to be confined to the synclinal areas.

Bedrock is of the Lower Pennsylvanian and Mississippian Age. The lowermost rock is the Reeds Springs Formation, composed of nearly equal parts of chert and limestone. The chert is bluish to tan, nodular and irregularly bedded. Chert can make up one third to two thirds of the formation. The formation averages 30 to 45 m thick in the project area.

The predominant controlling feature of the geology of the site is the brecciation of the bedrock and the "Cornfield Bar". The basal breccias are the "confused" or "broken" ground and consist of broken, angular chert lying on the slopes and bottom of the formerly solutioned, collapsed valleys. The chert is the residual component of the solutioned cherty limestone. It is in this porous, confused ground zone that most of the mineralization of the area has occurred. Areas of confused ground can extend nearly throughout the rock column of the project area, from the bedrock surface to over 35 m deep near the top of the Grand Falls Chert Member.

The "Cornfield Bar" feature has a large influence on the project, controlling the location of the broken and confused ground as well as location of the shale bedrock. The confused ground reaches nearly down to the sheet ground in the area of the Bar, so called because it is barren of mineralization. The width of the bar varies from 15 to 90 m, with the location of the bar in direct relationship to the location of the Cherokee Shale.

Mining

The Tri-State mining district, so named for its location at the junction of Missouri, Kansas and Oklahoma, was formerly one of the largest lead and zinc producing districts in the world. Major minerals mined were sphalerite (zinc sulfide) and galena (lead sulfide).

Present day evidence of mining on the right of way for the proposed project include, chat piles, mine shafts closed by Missouri Department of Natural Resources, and occasional surface depressions.

After reviewing historic mine maps and MoDOT documentation, an exploratory program consisting of a large tracked backhoe excavated several suspected mine features. Most of the features were designated as shafts or prospects. The excavations revealed the shafts were filled randomly with the onsite tailings, metal debris and trash. Vegetation, such as trees and bushes, was found at nearly every suspected location and are a good indication of the presence of a possible shaft.

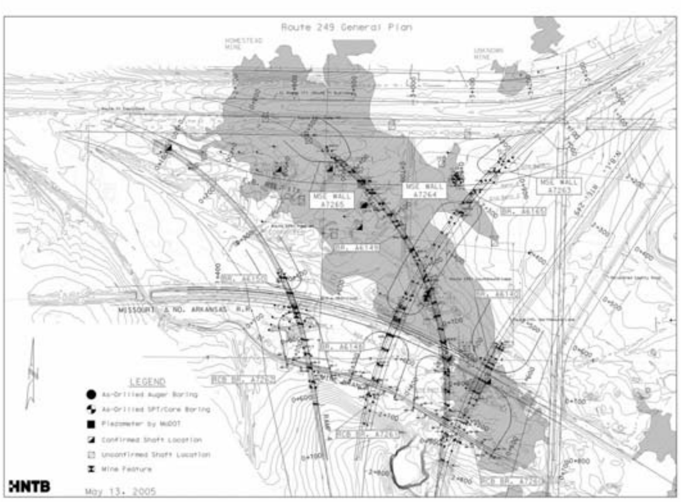

Fig. 1. Bridge Layout with Shaded Abandoned Mine and Suspected Mine Shafts

Some of the shafts had timber cribbing for support while others did not. Openings were typically on the order of 1.5 by 2 meters.

CONSTRUCTION

Phylosophy

One of the conclusions of the geologic/geotech investigation was the chaotic and "confused" nature of the subsurface at the site. During the course of several years, many borings were taken in an attempt to characterize the site. The characteristics of the subsurface were known to change drastically between boreholes located less than a meter apart. Drilling additional holes during the design phase might not provide further useful design information. Therefore, during design, the subsurface was classified design into zones of ground type. The subsurface characterization, as well as the design of ground improvements would be continued during the construction phase by drilling and treating the encountered mine voids and highly disturbed ground. The subsurface would be logged at each drill hole and treatment recommendations made in real time.

The production drilling equipment would include the use of monitor while drilling (MWD) as well as the real time observation of drilling and logging of the hole by a geologist or geotechnical engineer employed by the engineer. Grouting, with both low mobility and high mobility grouts would also be electronically recorded and monitored by the field inspection personnel.

The selection of ground treatment type was based on actual subsurface conditions encountered at hundreds of production holes rather than a few exploration holes taken during design. Low mobility grout (LMG) was used in voided conditions, all areas of mass ground treatment, and for closure of mine shafts. High mobility grout (HMG) was used in the fractured rock and foundation treatment to limit the use of grout during micropile installation. The use of real time observation was used successfully to modify the ground treatment and micropile installation in a seamless effort.

QA - Inspection

The QA/QC requirements from the specifications are summarized in Table 1.

Table 1: QA/QC Table.

ITEM/ACTIVITY	QA/QC	PURPOSE
Drilling	• Verticality, location and depth.	Assure holes in intended plan location.

ITEM/ACTIVITY	QA/QC	PURPOSE
	• MWD (i.e., real time monitoring and recording of major drilling information, e.g., penetration rate, strata changes, drill actions, flush characteristics, hole stability.	Each production hole will act as an investigation hole, to reduce geological uncertainty and demonstrate progressive improvement of the ground at each location during treatment. This will also confirm the suitable bearing horizon for the micropiles.
Grout Materials	• LMG – slump, cube strength • HMG – bleed, specific gravity, Marsh Cone, cube strength.	To ensure that accurate and consistent batching and proportioning is being undertaken.
Grouting Process	• For all grout injections the following parameters will be recorded: pressure, rate of injection, volume (per stage per unit length). Also recorded will be observations on interhole connections, breakouts, pressure irregularities, delays.	This data will permit the team to analyze the incremental performance of each phase of grouting (i.e., via use of Reduction Ratios, etc.) leading to logical decisions as to intensity of treatment (e.g., more or fewer holes, mix type selection).
Micropile Capacity	• Minimum four pre-production verification pile tensile load tests in the different ground types. • Minimum one proof load test per bent on productions piles.	Verification tests will confirm basic design assumptions. Proof tests will demonstrate consistency of installation quality of each main structural location.

Analysis of these data in real-time was particularly important on this project to assure that a responsive treatment was provided at each structure location, notwithstanding the provisions of the Specifications.

The successful implementation of this concept required: The full engineering cooperation between the owner's representative (HNTB) and the specialty foundation contractor (Layne GeoConstruction); and the on-site presence, guidance and participation of the owner's representative (HNTB).

HNTB provided a team for the construction engineering and inspection which consisted of a resident engineer and an inspector (geologist or engineer) for each drilling or grouting operation. The scale of the operations required a staff of a resident and four to five inspectors. Layne normally ran two or three rigs drilling and two rigs grouting.

Prior to the drilling of any hole, the field inspector responsible for logging the hole reviewed the GBR information to determine the expected elevations of rock-head and features within the hole. Part of the initial site set-up involved ensuring that all the relevant information was available onsite in an easy to search format to allow the inspectors to easily find this information.

The holes were logged during drilling by an HNTB inspector independently of Layne. Once the hole was drilled, logs completed by HNTB and Layne were reviewed and compared to the design intent of the plans and specifications as well as the GBR and Geotechnical Design Report. The automatic parameter recorder data submitted by Layne was checked to ensure that the automatic parameter recorder data and the manual log were consistent.

The holes were drilled utilizing rotary percussion drilling techniques with a down-hole-hammer, so the number of ground types included in the drill logs were limited to match the sensitivity of the drilling system. The rig inspector logging the hole characterized the material penetrated and recorded it using the following catagories:

- overburden;
- shale;
- chaotic, poor quality limestone, chert breccia;
- hard competent limestone;
- void/filled feature.

The log contained space for instantaneous penetration rates, flush comments and general comments.

Holes were used either for Low Mobility Grouting (LMG), High Mobility Grouting (HMG), or Micropiles. Each of these operations had a separate log format where information recorded was specific to the type of operation being performed.

The quantity of data being generated required a well organized filing system be utilized. Binders of the hard copies of all information were kept by area, with tabs for each hole in that area. Each tab contained the drill log, any inspector's notes, the automatic parameter recorder log and any grouting data (grouting logs). The hard copies were then scanned and kept in an electronic file. At regular intervals the files were distributed to the design team for review.

Construction Procedures

Construction began with the installation of the four design verification piles. The pile locations were placed in the three previously identified types of ground plus one test pile was installed in ground that had undergone pretreatment via grouting. The verification piles were placed in good limestone, broken and confused ground, treated broken and confused ground and shale.

Installation methods for the verification piles were to be those anticipated for the production piles. In each case an unbonded length was constructed in the overburden casing to prevent bonding of the reinforcing bar above the top of the bond zone. There was no acceptance(pass/fail) criteria for the verification piles. These piles were installed and tested to confirm design assumptions adjust assumptions for production pile installation, if necessary.

Ground Treatment

Mass ground treatment was undertaken in the areas where formerly mined (both shallow and deep) ground were thought to exist. The purpose of the mass treatment was to reduce the risk of ground loss under bridge approach embankments and in previously identified poor ground in the vicinity of bridge foundations. A typical foundation treatment grid for mass treatment can be seen in Fig. 2.

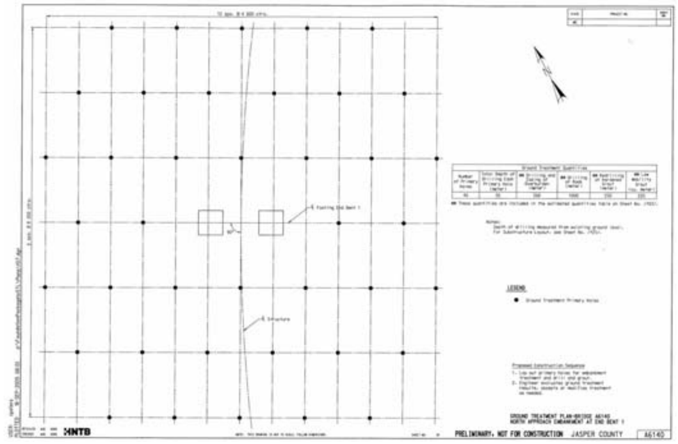

Fig. 2. Typical Foundation Treatment Grid

The treatment consisted of a pattern of holes generally four by four meters to a designated depth. The holes were drilled using rotary percussion drilling technique utilizing a down the hole hammer. Holes were logged in real time using a monitor while drilling (MWD) recording system. The MWD system provided drilling depth, advance rate, thrust pressure and rotary torque. The information from the MWD system was recorded on electronic media as well as printed in real time using a field printer located on the side of the drilling machine. The electronic media was downloaded at the end of the shift and the information transferred to a report format for submission to HNTB. All holes were also logged in real time utilizing traditional visual inspection by field inspectors who observed drill depth, rate of advancement, drill cuttings/lithology, hammer behavior, air return, rod torque, presence of groundwater and voids. This information was summarized in a written log and compared with the MWD log for inconsistencies.

The contractors MWD data and the inspectors' field logs were then utilized to formulate the grouting treatment to be used in the specific area drilled. An exception being the mass ground treatment holes that were all to be grouted with a low mobility grout. The intent of the mass ground treatment being to explore and fill mine voids to prevent massive ground loss.

The low mobility grout consisted of a contractor designed mixture of sand, cement, fly ash, additives and water. LMG grout strength was specified as 28 day strength of 4 mPa with a slump of 150mm (6") or less. Several modifications in mix design were necessary at the beginning of the project to achieve the project strength and slump criteria as well as the pumpability and set time the contractor required. Type C fly ash was allowed due to material availability.

The LMG was furnished by a local concrete batch plant and brought to the site in transit trucks normally carrying between 4.5 and 6.1 cubic meters. The grout was pumped with standard concreted pumps in lifts of one meter using a pressure refusal criteria of 4 mPa at the drill rig.

Fig. 3. Typical Ground Treatment Process – One rig drilling, one grouting

The main purpose of the ground treatment program was to explore for mine voids and reduce the risk of collapse. The holes normally ranged in depth from 20 to 30 meters. Volume of LMG injected into holes without voids was normally less than a 1 to 2 cubic meters. When mine voids were encountered, injected grout volumes ranged from 5 to 235 cubic meters. In several holes grout was injected in 50 cubic meter increments. Holes were allowed to rest for approximately two hours after each 50 cubic meter increment was placed without achieving refusal criteria in a particular lift. The resting time and grout rate were varied and changes made at the discretion of the rig inspector. In almost all cases, the rest period resulted in achieving the refusal criteria with addition of smaller amounts of LMG.

MINE SHAFT REMEDIATION

Several vertical mine shafts were also on the project right of way. None of these shafts were open to the surface prior to construction. A few of the shafts were evident from observed surface expressions and were excavated in the exploration phase and included in the contract documents. Other shaft locations were taken from mine maps obtained from historic sources. The contract documents included multiple suspected shaft locations that were to be explored by backhoe during the construction phase. The shaft exploration cost was based on measured volumes of material excavated. Some of the listed shafts were located, some were not, and other shafts not anticipated by surface expression or mine maps were found during the site grading.

Due to the possibility of encountering open shafts and ground collapse, a crane was specified to be placed in the vicinity of each work area for worker safety.

TYPE 1 MINE SHAFT CLOSURE

Once a shaft was located, the area was excavated generally to top of rock with backslopes for a safe temporary work area. The contractor then placed timber crane mats over the shaft openings to facilitate the setup of the drilling machine. Mine shaft drilling was a separate contract item due to the inherent possibility of encountering a variety of possible materials which may have been placed in the shafts throughout its' lifespan. The shafts were generally 1.5 meters square.

Once the drill rig was safely positioned over the opening of the shaft, the drill string was advanced to elevations thought to be the previous mine floor. This was confirmed by drill string advancement through several meters of competent material. Some of the shafts were necked off with several meters of miscellaneous fill and then water filled, while other shafts were filled with miscellaneous, mostly soft fill to the bottom.

Upon completion of drilling, the drill string was withdrawn and grout casing drilled into the hole. Low mobility grout was injected through the casing as the casing was withdrawn in two meter stages to the project refusal criteria. Again the amount of grout and any periods of rest time were at the discretion of the rig inspector.

In addition to the first hole, two additional confirmation holes were placed one to two meters from the original location. The purpose of these secondary holes was to confirm the original grout placed and explore for stopes and adits which may have occurred off the vertical shaft.

In addition to the 3 holes used for shaft grouting, a series of three additional confirmation holes were located approximately 10 meters from the shaft in the direction of any nearby adjacent structure. Again these additional holes were designed to explore for any possible stopes or adits emanating off the main shaft.

A total of 12 shafts were found and treated using this method. The volume of LMG required to complete treatment of a shaft ranged from 3 to 313 cubic meters.

TYPE 2 MINE CLOSURE

Another type of shaft closure was designed to be employed at shaft locations on the right of way but not near any bridge structure. The purpose of these shaft closures was long term site safety.

These closures were known as Type 2 closures and consisted of excavating the area of the shaft to the top of rock and placing a plug of expanded polyurethane foam in the throat of the shaft and then placing an inverted cone of cast in place reinforced concrete to seal the opening. One Type 2 mine closure was installed on the project.

FOUNDATION TREATMENT

Each foundation unit was excavated to bottom of footing elevation and inspected for signs of mining activity. The limits of the excavation included the area of the footing construction and a one to three meter area around the outside of the footing. Following excavation a series of holes was laid out surrounding and covering the footing area. These holes were also drilled using rotary percussion drilling techniques to depths ranging from 13 to 55 meters. Again, the holes were logged using both electronic MWD methods and the traditional visual inspection provided by the rig inspector.

Based on type of ground anticipated, three levels of foundation treatment intensity were specified in the plans as low, medium, and high. The low intensity averaged three primary and two secondary holes for a two footing bent. The medium intensity averaged three primary, two secondary, and four tertiary holes for a two footing bent. The high intensity treatment averaged three primary, two secondary, four tertiary, and four quaternary holes per two footing bent. A typical layout for high intensity treatment can be seen in Fig. 4.

After reviewing the drilling logs, a treatment scheme was chosen based on the character of the rock and the number and size of any voids logged in the drill hole. The purpose of the foundation treatment was three fold, the first to investigate and treat for mine voids and unstable ground beneath the limits of the footing, the second to minimize the potential for runaway grout during the installation of the micropiles and the third to confirm the limits of micropile bond zone. In general, voids larger than 152mm were desired to be treated with low mobility grout while broken and fractured rock was to be treated with high mobility grout.

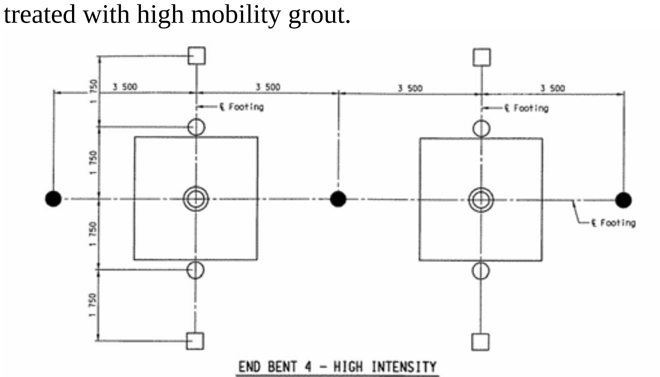

Fig. 4. Typical High Intensity Foundation Treatment Layout

The high mobility grout consisted of a balanced stable fluid grout designed by the contractor and composed of cement, fly ash, bentonite, welan gum and superplasticizer. There were three different grout mixes developed based on viscosity, A, B, and C. The grout was mixed at a central automated grout plant. The grout mixes ranged from a marsh cone time of 40 seconds to infinite. HMG was required to achieve a 28 day strength of 4 mPa (600 psi) High mobility grout was injected via a pneumatic packer utilizing upstage grouting methods where applicable. There were many instances where hole caving occurred due to the highly fractured and broken nature of the rock. In these instances the downstage grouting method was utilized until the hole reached total depth.

Prior to proceeding with micropile installation, the information from the entire group of foundation treatment holes was plotted, analyzed and compared to the depths, thicknesses and types rock materials assumed in the design of the micropiles. Upon completion of the analysis the overburden casing depths and micropile bond lengths were adjusted to reflect the actual conditions found during the foundation treatment. The overburden casing depth and micropile bond length were generally adjusted as a group at a single bent footing rather than on a pile by pile basis.

MICROPILES

The micropile design consisted of a cased length through the overburden and undesirable rock and a bonded length located within the competent rock strata identified in the foundation treatment phase. In cases where piles were designed with a lateral load component, the specification required the cased length be free of joints or an additional larger casing installed to provide an increased lateral load capacity. In all cases where a lateral load component was required the depth of the overburden casing was small enough to allow the contractor to install a single piece of casing. Overburden casing consisted of a, 193.7mm OD x 13 mm wall thickness, N80 Mill Secondary, flush joint threaded, steel casing. The casing tensile strength was a minimum 80 ksi.

The central steel reinforcement within the micropile consisted of an epoxy coated 65mm OD, grade 150 KSI threadbar. The epoxy coated threadbar extended from the bottom of the micropile bond zone up through the overburden casing and terminated within the footing. Two nuts and a plate were installed at the top of the threadbar within the footing to transfer tensile load to the micropile.

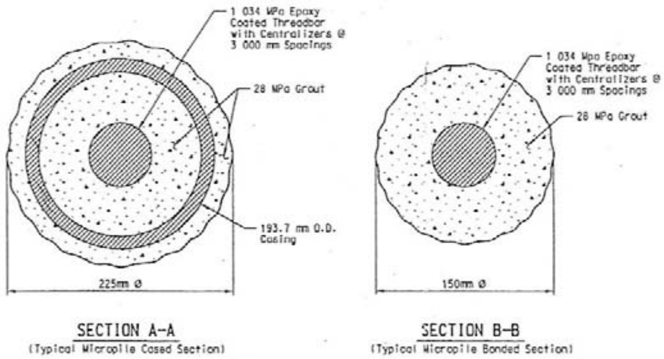

Fig. 5. Typical micropile cross section.

The initial micropile installation methodology consisted of open hole drilling utilizing a rotary percussive drilling technique and air flush. The overburden micropile casing was installed within a predrilled 245 mm hole to the planned depth of the casing. The casing was then capped and grout pumped down the casing and up the annulus until undiluted grout returned to surface.

Upon completion of the grouting the casing grout was allowed to cure for a minimum of one day. Open hole drilling techniques were again used to drill the grout from within the casing and extend the micropile bond zone to the desired depth. The bond zone diameter was 160 mm and the reinforcing bar was installed and grouted to the bottom of the hole. The bar was made up of stock 3 and 6 meter lengths and cut to final grade. The bars were joined with mechanical threaded couplers. PVC centralizers were placed on the bar at approximately 3 meter intervals.

Micropile drilling was electronically recorded utilizing the MWD system and traditional visual logging by the rig inspector. The rig inspector compared the rock conditions encountered with the bent specific micopile design assumptions and modified both the tip elevation of the overburden casing and tip elevation of the micropile bond zone accordingly. Micropiles were installed to plan length and depth or lengthened accordingly. In no case was the micropile shortened due to better than expected conditions.

Foundation treatment in several of the bridge bents was not entirely successful in solidifying the rock mass to prevent hole instability during micropile drilling. This prompted Layne GeoConstruction to implement an alternate micropile installation methodology. In bridge bents where this condition was found a cased drilling method was utilized in which the casing was advanced with rotary percussive drilling techniques to the bottom of the desired bond length. Upon completion of the casing advancement the drill tooling was withdrawn and the core reinforcing steel was installed with associated hardware. The hole was then grouted through a preinstalled tremmie tube until thick grout was seen exiting the top of the drill hole. At this time the casing was withdrawn to the desired elevation. Internal grout elevation was monitored as casing joints were removed and topped off as necessary. Grout was also injected through the drill head as casing was withdrawn. Micropiles in four out of the sixteen bridge bents were installed utilizing this system. Micropile grout takes in these bents were also significantly higher than anticipated due to the fractured nature of the rock and the hole instability.

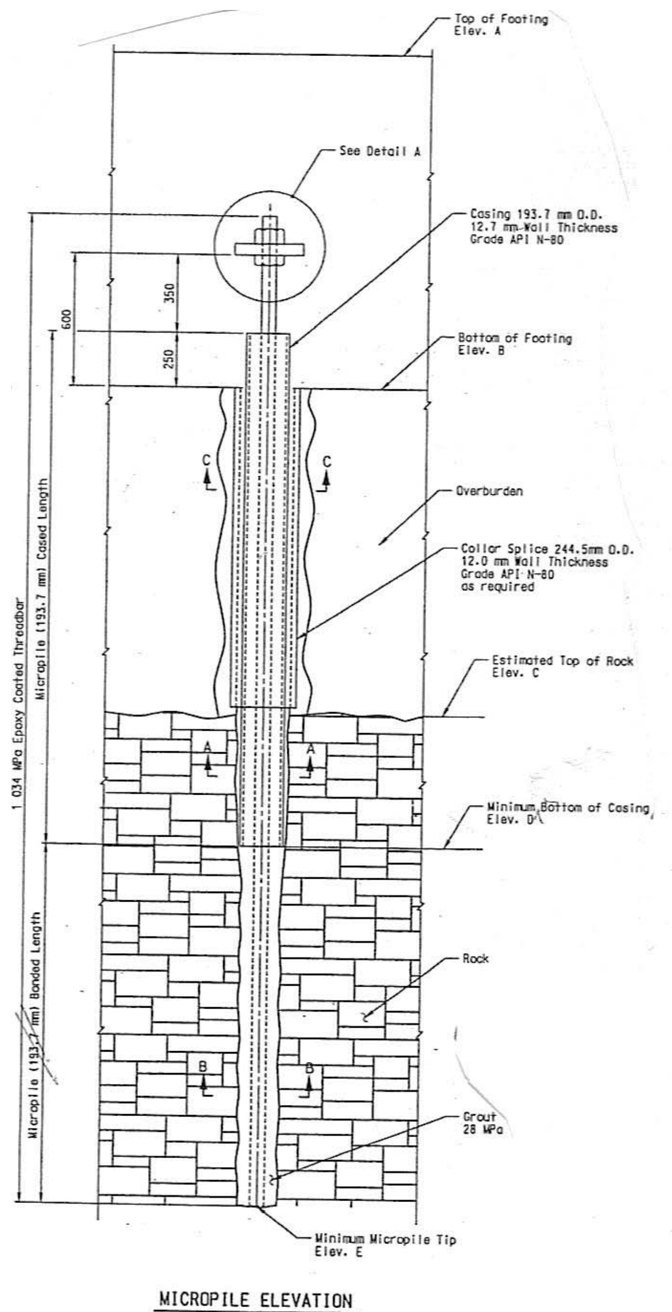

Fig. 6. Typical micropile elevation.

One micropile was selected for proof testing at each bent. Normally, the selection was based on a possible anomaly observed during the installation of the pile. Some of the possible anomalies were; high grout takes, low grout takes, hole instability, or difficulty inserting the bar. Micropile proof testing was conducted both on vertical and battered micropiles. Micropile proof testing acceptance criteria was as follows:

1. Failure does not occur at 1.20 DL.
2. At test load, the apparent debonded length (calculated from the elastic extension) shall not exceed 50% of the bond length.

Item	Plan Quantity	Actual Quantity	Over Under	% of Plan
Type 2 Mine Closure	3 ea	1 ea	-2 ea	33%
Mine Shaft Drilling	763 m	734 m	-29 m	96%
Micropile Proof Load Test	16 ea	16 ea	0 ea	100%
Micropile Verification Test	4 ea	4 ea	0 ea	100%
Overburden Drilling	1716 m	2221 m	+505 m	129%
Rock Drilling	16415 m	14765 m	-1650 m	90%
Micropile Bond Length	2016 m	2086 m	70 m	103%
Micropile Cased Length	1241 m	1309 m	68 m	105%
Redrill	4107 m	857 m	-3250 m	21%
High Mobility Grout	575 m³	385 m³	-190 m³	67%
Low Mobility Grout	3636 m³	6650 m³	+3015 m³	183%

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 liner or decreasing throughout the creep test period.

Fig. 5. Micropile Testing

CONCLUSIONS

The system of gathering and inspecting information from the drilling and grouting in real time reduced the risk associated with design of ground improvements and micropile installation in a very complex geologic and mined environment. The system helped control quantities on the project and allowed for adjustments to all aspects of the grouting and micropile construction without interrupting the work process.

Adjustments were made to both the foundation treatment hole layout and the micropile installation procedures to mitigate problems arising from the chaotic nature of the rock strata. Grout quantities for micropile installation in several foundation bents exceeded expectations due to the fractured nature of the rock. Additional payment was negotiated with the contractor for the overrun in grout quantities.

Verification test results confirmed the initial design assumptions of the project team. The analysis of the MWD data, conventional drill logs and grouting reports significantly assisted the project team in analysis of bond zone placement and in several foundation bents micropile bond lengths and bond placement we modified to account for insitu rock conditions. Proof testing results allowed the project team to confirm micropile design assumptions. Also, proof tests on micropiles with small anomalies confirmed the construction procedures utilized for the production installation operations.

Table 2. Contract Quantities

The planned contract quantities were originally estimated taking into consideration the conditions of the site as a hole and may have not been directly estimated at each footing. In the end the quantities varied greatly from hole to hole and bent to bent as the subsurface conditions were truly chaotic and confused. However, when the highly variable quantities were applied to unit costs, the total cost was within four percent of the original estimate.

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