TIED-BACK MICROPILE WALLS IN LANDSLIDE REPAIR

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In September 2004, Hurricanes Frances and Ivan caused the Pigeon River in western North Carolina to flood. This flooding eroded the embankment below Interstate 40 (I-40) and sent one of the eastbound lanes, along with the roadway shoulder, into the river below the embankment. North Carolina Department of Transportation (NCDOT) needed to reopen all the lanes of I-40 as soon as possible and considered this damage to be an emergency situation. NCDOT contracted Schnabel Foundation Company to design and build a repair that would keep this important interstate open and reestablish the eastbound lane. The obstacles included limited work room, the requirement of keeping the remaining lanes of I-40 in service, and producing a final product that would be relatively maintenance free and last at least 100 years. The proposed repair had to consider that the existing interstate was founded on a rock fill that was constructed from rock blasted off the adjacent mountain side. The selected design included two, tied-back micropile walls to address these obstacles and other issues. Each of the walls were constructed of 10.75 inch diameter micropiles and corrosion-protected tieback anchors installed through the existing rock fill into competent bedrock, concrete wales, and a reinforced shotcrete facing. This successful project was completed during the coldest months of the year and within the timeframe established by the State of North Carolina. This paper will discuss the design and construction of the repair and the many challenges that had to be overcome.

Introduction
North Carolina and many other states in the southeastern United States faced an onslaught of rains during the 2004 hurricane season. Hurricanes Charley, Frances, Ivan, and Jeanne each made a path through central and western North Carolina from August to September 2004. In September, three tropical storms (Frances, Ivan, and Jeanne) had a major impact on the total rainfall amounts over much of western North Carolina. The rainfall totals ranged from 10 to 25 inches, which was 200 to 500 percent above normal.

Interstate 40 (I-40) parallels the Pigeon River in the westernmost part of North Carolina. The massive water flow of the Pigeon River from Tropical Storm Ivan and releases from Walters Dam undermined the toe of embankments in numerous locations along I-40 on September 17, 2004 (Photo 1). This undermining eventually led to slope failures at almost every bend in the Pigeon River. From data accumulated by the U.S. Geological Survey, the Pigeon River had a flow of 19,800 cfs (cubic feet per second) on September 8th during Hurricane Frances and 17,100 cfs on September 17th during Hurricane Ivan. The daily mean gauge height of the Pigeon River had been approximately three (3) feet but on September 17, 2004 the gauge height increased to a maximum height of almost eighteen (18) feet.

The most notable slope failure was between Mile Marker (MM) 3 and 4 where a sharp curve occurred in the river (Photo 2). At that bend, a 200-ft long section of the I-40 shoulder had fallen into river. State troopers were alerted and arrived on site to close down the eastbound lanes of I-40 for the safety of the traveling public.

This paper discusses the challenges encountered while designing and constructing the repair of the slide between MM 3 and MM 4 using a tied-back micropile wall. Design of the tied-back micropile wall was performed by Schnabel Foundation Company. Construction of the tied-back micropile wall and the required roadway repair was completed by a joint effort of the North Carolina Department of Transportation (NCDOT), Schnabel Foundation Company,

Photo 1 - Effects of Tropical Storm Ivan

**Original Roadway Construction**

In 1955, a two-lane road was constructed between the mountainside and the Pigeon River from the Tennessee Stateline into North Carolina. The mountain was blasted and the rock mass was utilized to build the roadway embankment. In 1964, the road was widened to two-lanes each way to form I-40. From the Tennessee Stateline to MM4, I-40 is located between the steep mountainside and the river. The mountain had numerous slope stability issues that led to rockslides throughout the corridor. Rehabilitation of the pavement, rock slope stabilization, and widening of roadway shoulders occurred in 1982.

Photo 2 - Slide between MM 3 and MM 4

**Existing Site Conditions**

This section of I-40 is on the western edge of the Blue Ridge Belt and contain metamorphosed sedimentary rocks of the Snowbird Group. The Snowbird Group consists mainly of metamorphosed siltstones and sandstones with some quartzites and meta-graywacke.

The existing roadway embankment was constructed with small to large boulders blasted from the existing mountain side. The subgrade and pavement structure for I-40 was constructed above the embankment fill. The roadway elevations was approximately 65 feet above the Normal High Water elevation of the Pigeon River.

In general, the existing subsurface under I-40 roadway embankment fill consisted of silty sands and some gravel underlain by dense gravel, cobbles, and boulders. Below this layer was mostly cobbles and boulders comprised of quartzite and meta-graywacke fragments. Crystalline rock was encountered below the boulder layers and classified as meta-graywacke. The meta-graywacke at the site is a sandstone that contains a much higher percentage of feldspar, biotite, and other minerals than typical sandstone, which is composed mainly of quartz sand grains with a small percentage of other minerals. This difference in the meta-graywacke made the rock extremely hard and abrasive which made drilling extremely difficult.

The area near the slides has a variable thickness of soil over the rock from a few inches to numerouse feet. The rock can be highly weathered in areas and does have a tendency to fail or slide periodically. Much of the natural material on the lower portions of the slopes is colluvium with boulders ranging from a few feet to over 10 feet in diameter.

**Repair Options**

Several options were considered for the repair of the landslide. The options that were presented to the NCDOT for consideration included a tied-back soldier beam and lagging wall with either a precast or cast in place (C.I.P.) concrete facing, a temporary soil nail wall that would allow construction of a permanent Mechanically Stabilized Earth (MSE) wall, and a permanent tied-back micropile wall with a reinforced shotcrete facing. Through a selection process that considered safety, schedule, long term maintenance, innovation, environmental stewardship, and price; the tied-back micropile wall with a shotcrete facing was chosen.

In addition to the design of the tied-back micropile wall, the slope between the river and the base of the wall required protection from future flood events. The toe scour protection below the proposed wall was an innovative
concept developed by the Structure Design Unit, Geotechnical Engineering Unit, and Hydraulics Unit of the NCDOT. The concept is basically a magnified and expanded version of a typical wrapped-face fabric wall, but instead of using fabric, the design incorporated ring nets typically used for rockfall catchment fences and instead of using select backfill, 2-foot to 4-foot diameter boulders were used for encapsulation. The idea was to protect the toe from scour caused by future large floods. The system was designed to be flexible and to withstand impact from other large boulders carried by the waters during another major flood event. Details of the design of the toe scour protection are beyond the scope of this paper.

Figure 1 illustrates details of the design concept for repair of the landslide. The maximum height of the tieback wall was approximately 35 feet while the toe scour protect ranged from 15 feet to 25 feet high.

Wall Design
A tied-back micropile wall was chosen to address the extremely challenging drilling conditions and the access restrictions that maintaining the existing lanes of traffic imposed. The working space at the top of the slide was limited and the marginally stable bench precluded the use of large caisson drills that would be required to install 18 inch to 30 inch diameter holes to accommodate traditional H or W shaped steel soldier piles. A 10 ¾ inch diameter micropile was chosen because the material was readily available, it produced a section modulus that would limit the rows of tiebacks in the deepest section of the slide (approximately 35 feet) to 3 rows, and would work with the anticipated drilling systems. This size micropile could be installed using fairly small tracked drills that could safely work on the limited bench. The micropiles would be installed though the embankment fill into competent bedrock below the slide.

The major considerations that had to be addressed in the design of the tied-back micropile wall were corrosion considerations, influence of micropile joints on the integrity of the wall, potential earth pressure behind the wall and design of the shotcrete facing and concrete waler.

Corrosion Considerations
The micropiles consisted of 10 foot sections of mill secondary API drill casing with machined flush jointed threads. The casing had a minimum yield strength of 80 ksi. The strength of the micropiles was confirmed by coupons cut from representative pieces of the casing.

The design of the micropiles considered the potential corrosion that could occur during its 100-year design life. Due to the nature of the fill material that the micropile would be installed through, full and continuous grout cover around the micropile was not expected. It was assumed that the voids within the fill would allow air and water to migrate through the fill and therefore would present a corrosive environment. A corrosion rate of 0.08 mm per year which corresponds to recommendations in the FHWA –H1-97-013 for fill material was used for design. The inside of the micropile casing was filled with concrete, so the material loss due to corrosion only needed to be considered on the exterior wall of the micropile. This potential loss of material due to corrosion caused a reduction in the designed thickness of the micropile wall by approximately 50% which in turn reduced the long term section modulus of the micropile by more than 60%. The design of the micropile was based upon this long-term condition, although this may be considered conservative. The permanent reinforced shotcrete facing, attaches to the micropiles with headed studs and provides long term moment carrying capacity to the wall system. The permanent shotcrete facing was in place prior to the potential corrosion loss from the micropiles.

Influence of Joints
The micropile casings were installed in 10-foot sections. Each section was connected using a threaded joint. Sectional micropiles used for soldier beams are usually designed so that the joints are located away from areas of high bending moments. However at this site, the variable distance to bedrock and the extremely difficult drilling conditions made controlling the joint locations impractical.

It is very difficult to model the behavior of a threaded casing join in bending. Additionally, limited data exists on casing joint strength in bending. L.B. Foster Piling [Wilson, 2003] performed three bending tests on 7-inch casing joints machined with a proprietary thread design. The test results gave the ultimate moment
section modulus for the unthreaded section. The elastic section assumes the extreme fiber is just reaching yield, while the plastic section assumes the whole section has reached its yield strength. In both the elastic and plastic analyses, the section will continue to carry load after the moment capacity is exceeded. For example, using 0.66fy for a 7–inch pipe would give us an allowable moment capacity of approximately 68 k-ft. L.B. Foster’s 7-inch pipe could carry this at the joint without breaking, but it would be working at about 75% of its ultimate (brittle) strength. The joint would definitely be the weakest point in the micropile. Filling the micropile with concrete does little to increase the bending strength of the joint.

In the case of this project, the joint strength did not govern the micropile design due to several factors. Because of corrosion considerations, the micropiles had already been designed for loads that would not exceed 40% of the allowable load prior to significant corrosion loss. Furthermore, the micropile design did not take advantage of moment redistribution of the flexible wall [Peck, et. al.,1974] that would reduce the moments created along the micropiles. Finally, in the long term condition, the reinforced shotcrete facing will carry the vertical bending moments. The micropile casing was only required to carry high bending moments during construction prior to the installation of the permanent facing.

The main purpose of the micropiles is to transfer the vertical loads of the tiebacks and wall facing elements to the competent bedrock below the slide. There is no reduction of a micropile’s compressive strength at a joint location.

**Tieback Anchors**

Although micropiles have been hailed recently as a panacea for many geotechnical problems, the use of tiebacks were critical to resisting the sliding mass and preventing future movements. The main roles of the micropiles were to transfer the vertical loads imposed by the tiebacks and wall components to below the sliding mass and to provide support of the facing between wale locations. Tiebacks actively oppose the movement of the soil mass rather than behaving in a passive manner. As a result, a component of the tension in the tiebacks acts against the thrust of the potential sliding mass while another component of the tension increase(s) the normal stresses on slip surfaces in the soil [Morgenstern, 1986]. The tiebacks consisted of double-corrosion protected, 0.6-inch diameter, 7-wire, 270 ksi strand conforming to the requirements of ASTM A-416. The tiebacks were installed at an angle of 30 degrees from horizontal and were spaced at 10 feet center to center along the wales. The vertical spacing of the tiebacks was approximately 10 feet. The tiebacks developed their capacity in the competent bedrock beneath the embankment fill. The tiebacks were designed to accommodate an earth pressure originally based upon diagrams presented in FHWA-IF-
Ground Anchors and Anchored Systems. The soil parameters used in the design were those recommended by the NCDOT. These soil parameters were:

\[ \begin{align*}
\phi' &= 34^\circ \\
C' &= 0 \text{ psf} \\
\gamma &= 130 \text{ pcf}
\end{align*} \]

These values were probably conservative when considering the macro structure of the embankment fill, through which the critical failure surfaces passed and the apparent stability of the existing extremely steep slopes at and around the slide. The earth pressures determined from the original diagrams were then compared to a limit equilibrium analysis, which satisfied both moment and force equilibrium (Spencer’s Method). The limit equilibrium analysis demonstrated that to achieve a factor of safety of 1.5, the earth pressure determined using the original diagrams required a 15% increase.

The tiebacks were connected to the micropiles and integrated into the shotcrete facing using a C.I.P. concrete continuous wale (Figure 2). The wale was structurally connected to the micropile utilizing studs welded to the micropile. The main purpose of the studs was to transfer the vertical component of the tieback load to the micropile.

![Figure 2 – Tieback Connection Detail](image)

**Shotcrete Facing**

A permanent shotcrete facing was proposed for the tied-back micropile wall. The use of shotcrete was initially resisted by the NCDOT due to the fact that the construction of the wall was planned for the coldest months of the year. The major concerns were the feasibility of cold weather shotcreting and questions about the long-term durability. It is true that special considerations must be observed when installing a shotcrete facing during cold weather. However, the use of proper and careful construction techniques and a proper mix design will provide a durable wall facing that meets all strength requirements.

The shotcrete mix was proportioned to limit the amount of Portland Cement. This limitation was considered important due to the need to limit the thermal stress and potential shrinkage cracking of the shotcrete facing.

Thermal stress gradients and the resulting cracking during curing must be considered in shotcrete facing construction. Cracking due to thermal stress is described in Section 1.3.2 of ACI 224.1R "Causes, Evaluation and Repair of Cracks in Concrete Structures". It has been found that tieback wall facings are especially susceptible to thermal stress cracking because only one face is exposed. The other face is placed against wood lagging or the ground surface. The result is that the temperature of the shotcrete at the back of the facing is much higher than the temperature at the exposed surface. The temperature gradient induces relative contraction at the surface and subsequent cracking. The first option listed by ACI 224 to reduce thermally-induced cracking is to reduce the maximum internal temperature. This reduction can be achieved by reducing the amount of cement per unit volume, which in turn reduces the total heat of hydration produced during the curing process. (See ACI 207.4R-6 “Effect of Restraint, Volume Change, and Reinforcement on Cracking of Massive Concrete” for a discussion of chemical reactions and heat generation.)

The potential for drying shrinkage cracking must also be considered in shotcrete facing construction. According to ACI 224, drying shrinkage is caused by the loss of moisture from the cement paste constituent of concrete. As with thermally-induced cracking, tieback wall facings are vulnerable to this phenomenon because only one side of the facing is exposed. ACI 506-R-90 “Guide to Shotcrete” further adds in Section 1.7 that shotcrete has a greater than normal potential for drying shrinkage cracking.
due to the high cement factors that are typically used.

The extremely cold winter that was expected only exacerbated the potential problems. Therefore, the shotcrete strength was limited to 3,000 psi thereby limiting the cement content.

In addition to proportioning an appropriate shotcrete mix design, proper and careful construction techniques must be also followed. The recommendations provided by ACI 306R-88 “Cold Weather Concreting” were followed to ensure that a quality wall facing was constructed despite cold weather conditions. These recommendations include protecting the newly placed shotcrete with enclosures and/or insulating blankets until minimum compressive strength goals are reached in order to prevent frost damage (Photo 3).

Photo 3: Special precautions used during shotcrete installation

**Micropile Installation**

As previously noted, the site had restricted access, limited room for benches, and the work had to be completed while keeping the remaining lanes of I-40 open to traffic (Photo 4). These challenges contributed to the decision to utilize small-tracked drill rigs to install the micropiles. These rigs consisted of three tracked drill rigs (Hutte 605, DK 725 and DK 820) all tooled with rotary percussive duplex drilling systems. This type of drilling method involves the simultaneous advancement of the outer drill casing and an inner drill string. The inner drill string is attached to a down-hole hammer which is equipped with an under-reaming system. The under-reaming system allows the micropile casing to advance directly behind a specialized drill bit that is designed to cut a hole that is marginally larger than the outside diameter of the micropile casing while having the ability to retract through the casing once the casing has advanced to its final depth. Several under-reaming systems exist in the industry. These systems can consist of retractable wings, a drop off crown or ring bit, or an eccentric bit attached to the end of a down-hole hammer designed to fit within the inner diameter of the micropile casing. The following under-reaming bits were considered for use during installation of the micropiles:

- URG concentric under-reaming bit by Holte Manufacturing
- NUMA T240 SuperJaws® concentric under-reaming bit, and
- Stratex guide device with drop off rings concentric under-reaming bit.

Both the URG and NUMA bits use retracting “wings” that extend beyond the outside diameter of the micropile while advancing the system. After the required drilling depth is reached the wings are retracted and the hammer and bit are removed up through the inside of the micropile. The Stratex system uses drop off rings (or crowns) which are bits that fit over the body of a guide device (i.e. main bit body). After the required drilling depth is reached, the drop off ring is detached by pulling the drill rods up and pulling the ring off with the casing end. The main body of the bit was then pulled through the inner diameter of the casing leaving the micropile in place.

Photo 4: Micropile Installation

Compressed air was used to drive the down-hole hammer and flush the cuttings out of the
casing. The energy imparted by the hammer is directly proportional to the air pressure used to drive the hammer. Due to the extremely hard rock encountered, high pressure air (350 psi) was necessary.

Drilling through the existing roadway fill presented several challenges. Not only did the meta-graywacke boulders and bedrock prove to be very abrasive to the drill bits, but the rock matrix was sometimes so loose and discontinuous that the hammer was allowed to advance without expanding the under-reaming wings when the NUMA system was used. When the drilling system advanced in this manner, the casing would bind against the sides of the narrow hole. In addition to challenges created by the loose rock matrix, the bedrock sloped steeply towards the river below. Once the drilling system encountered the steeply sloping bedrock, the drilling system tended to walk downhill which created alignment problems and also caused binding of the micropile casing behind the down-hole hammer. This issue was exacerbated by a decision to install the micropile casing without the use of a drive shoe. A drive shoe attached to the end of the casing allows the hammer to impart a driving force to the casing itself. The use of a drive shoe attached to casing left in the ground becomes a fairly expensive and time consuming proposition. The difficulty in advancing the casing behind the hammer was addressed by slowing the drilling rate and attaching cutting medium to the end of the micropile casing.

**Tieback Installation**

Tiebacks were installed using crane supported platform drills and tracked drill rigs. The nature of the embankment fill and the hardness of the boulders and bedrock created challenges for both types of drilling rigs. Also, the large amount of voids and fissures interspersed throughout the fill made grouting around the tiebacks extremely challenging.

The tiebacks were designed to penetrate through the embankment fill and develop their capacity in the competent bedrock. Many different methods were tested while trying to find an efficient manner to install the tiebacks. Three types of rotary percussive duplex drilling systems were attempted. These systems were:

- Concentric under-reaming using a Numa SuperJaws® bit on a 4-inch down-hole hammer. This system was able to advance a cased hole to full depth, but it was slow and the extremely hard and abrasive meta-graywacke caused the bits and the wings to wear very quickly. The bit was advanced at an approximate rate of one foot every three minutes in the embankment fill and one foot every six minutes in the competent bedrock. These bits were wearing very quickly and had to be refurbished often.

- Concentric under-reaming using a drop off ring (crown bit). Although this system advanced fairly quickly through the embankment fill, the bit slowed significantly once competent bedrock was encountered (approximately one foot every five minutes. The drop-off ring (crown), having an approximate diameter of six inches, was powered by a 4-inch down-hole hammer. The difference in energy imparted on the bit when compared to 6-inch down-hole hammer is significant. Also, the drop off rings did not easily pull off after drilling though the extremely hard rock.

- Eccentric under-reaming. This system had the same problems as the concentric under-reaming and tended to allow the casing to hang up more often.

The penetration rate of a down-hole hammer is dependant upon the size of the hammer, the size of the bit, and the air pressure used to run the hammer. The following equation can be used to compare the relative penetration rates of identical down-hole hammers equipped with bits of varying diameters.

$$\frac{(\text{Diameter Bit A})^{3/2}}{(\text{Diameter Bit B})^{3/2}} = \frac{\text{Penetration Rate Bit A}}{\text{Penetration Rate Bit B}}$$

This equation [Ingersoll-Rand, 1976] accounts for the dissipation of energy that occurs when the area of the bit increases. It is evident when examining this equation that when a larger bit is used on a smaller hammer that is designed to fit inside of the micropile casing, the penetration
rate is drastically reduced. This equation also illustrates why the penetration rate was less of a problem for the larger diameter micropiles.

After attempting these rotary percussive duplex drilling systems methods, a combination of more traditional approaches was used to install the tieback anchors. A hole was initially advanced through the embankment fill using a 6-inch down-hole hammer. After retracting the hammer, casing was advanced through the predrilled hole using a top-drive drifter on the drill head. Once the casing was advance to the top of bedrock, a 4-inch down-hole hammer with a traditional 4-inch bit was advanced through the casing and into the competent bedrock. Once this down-hole hammer was withdrawn, the tieback was installed and grouted. The casing was left in place to control the grout takes and provide additional corrosion protection around the unbonded zone of the tieback.

**Wall Performance**
The performance of the wall was determined by monitoring the movement of the wall at the top of the wall. The monitoring consisted of taking lateral measurements off of an established baseline to known points, which were nails set in the face of the shotcrete. The measurements were the perpendicular distances from these points to the baseline. Each measurement was compared to both the original measurements and all previous measurements at the respective points. This data was then evaluated to determine the total movement of the wall, if any, was out of expected ranges. The movement at the top of the wall ranged from 0.08% to 0.15% of the wall height between the baseline set up and wall. These movements were lower than the anticipated movements which ranged from 0.15% to 0.3% of the wall height.

**Conclusions**
The use of a tied-back micropile wall to stabilize a landslide in extremely difficult drilling conditions with limited access was competed within the timeframe expected and performed satisfactorily (Photo 5). The use of micropiles was successful in transferring the vertical loads of the tiebacks and the wall components to the competent bedrock below the slide. The use of tiebacks was critical to stabilizing the potential slide mass. Installing the micropiles and tiebacks through an embankment fill consisting of large cobbles and boulders in a soft soil matrix proved to be extremely challenging. The use of rotary percussive duplex drilling systems proved to be invaluable, but still presented challenges that had to be overcome. Although not actually implemented at the site, the use of drop-off rings or crowns would probably be the most efficient method of installing the large diameter micropiles through the difficult conditions presented by the embankment fill and the extremely hard bedrock. Drop-off rings were not as efficient in installing the smaller diameter tiebacks in the extremely hard meta-graywacke due to the ratio of energy produced by the small hammer to the bit area. Multiple drilling passes, although inefficient seemed to work best.

![Photo 5 – Tied-back Micropile Wall with Permanent Shotcrete Facing](image)

**References**

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