Field Trip Stop and Drive-By Site Descriptions
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Drive-By Site A

Soil Nail Walls along Kancamagus Highway in Albany, NH

Three soil nail walls were constructed from 1996 to 1998 along the Kancamagus Highway in Albany, New Hampshire. The Kancamagus Highway, designated a National Scenic Byway, is a 35-mile long stretch of NH Route 112 that cuts through the White Mountain National Forest. The walls reach a height of up to 22 feet and range in length from 425 to 600 feet. The soil nail walls replaced deteriorating wood crib walls that supported steep hillside cuts. Subsurface conditions encountered within the wall excavations included fill materials from the original crib walls and a natural glacial ice contact deposit consisting of gravelly sands with cobbles and boulders. The sites also had high groundwater levels.

The walls were constructed in a top to bottom sequence, with the excavation and wall construction occurring in 5 to 7 foot lifts. A shotcrete facing with a geocomposite drain system placed behind the shotcrete was utilized for each wall. The shotcrete facing ranged in thickness from 6.75 to 11.5 inches and was reinforced with either 0.5 inch diameter reinforcing bars or steel wire mesh. The geocomposite drain extended the full height of the wall on a 5 foot center to center spacing (Figure 1). A temporary dewatering system was required during the construction of each wall to lower the high groundwater levels.

![Figure 1 - Typical soil nail wall section (Haley & Aldrich, 1997)](image-url)
The soil nails consisted of epoxy coated 1.25 inch diameter threaded reinforcing bars, with a minimum of 1.5 inches of grout cover. The soil nail lengths ranged from 20 to 50 feet and were generally placed on a 5 foot vertical and horizontal spacing. The design load of the nails was typically 40 kips. The soil nails were installed prior to placement of the shotcrete on the first wall and after placement of the shotcrete through blockouts on the remaining two walls (Figure 2).

To minimize the freeze-thaw effects behind the shotcrete, two layers of rigid extruded polystyrene insulation with a total thickness of 6.25 inches were placed over the shotcrete facing (Figure 3). In the past, cracking and displacement of the nails caused by ground freezing and thawing have precluded the use of these types of walls in cold climates (Haley & Aldrich, 1997).

A rough timber facing, which consisted of pressure treated 2 inch by 10 inch horizontal boards and 4 inch by 6 inch vertical posts, were attached to the wall over the polystyrene insulation (Figure 4). The timber facing provided an architectural wood face finish, which was requested by the US Forest Service (Figure 5) for aesthetic reasons. It was anticipated that the planking would have a shorter service life than the wall itself, requiring replacement of the facing in future maintenance. It is not known exactly how long the facing will last, but its replacement is expected to be relatively easy.

Geotechnical instrumentation was used to monitor conditions during construction of the walls and to verify whether the rigid insulation prevented ground freezing behind the wall. Slope inclinometer casing was installed prior to beginning the excavation on all three walls and was used to monitor lateral ground movements during and after construction. Vibrating wire piezometers were used to verify that the temporary dewatering systems had lowered the groundwater levels to an acceptable level prior to beginning the wall excavation. In addition, the piezometers were used to verify the long term performance of the horizontal drains installed on one of the walls.

Vibrating thermistors installed on the first wall behind the shotcrete verified that the rigid insulation successfully prevented ground freezing behind the wall. Vibrating wire load cells and strain gages were placed on three in-line soil nails on the third wall to determine if the actual nail loads were consistent with the predicted design loads.

One wall had permanent horizontal drains installed 50 feet into the hillside to lower the groundwater level and to reduce long term groundwater pressures on the wall. The horizontal drains consisted of two inch diameter slotted PVC pipes installed with a five degree upward angle. The addition of horizontal drains to reduce groundwater pressure, allowed the NHDOT to decrease the length of the nails to as short as 20 feet and reduce the thickness of the shotcrete.

Geotechnical design services were provided by Haley and Aldrich, Inc. of Bedford, New Hampshire for the first two walls and by the NHDOT Geotechnical Section for the third wall. Construction oversight was provided by the NHDOT Bureau of Construction. All three walls were constructed by Busby Construction of Atkinson, NH, in three separate contracts.
This soil nail wall design has provided a relatively maintenance-free, economical and environmentally friendly solution for locations with challenging site and subsurface conditions in a cold climate.

Figure 2. - Drilling Soil Nail (Cleary, 1996)

Figure 3. - Placing Insulation (Cleary, 1996)

Figure 4. - Attaching timber facing (Cleary, 1996)

Figure 5. - Soil Nail Wall with timber facing (Cleary, 1996)

References

Cleary, Thomas, 2000, Summary Paper of Three Soil Nail Wall Projects by the New Hampshire Department of Transportation along the Kancamagus Highway in Albany, New Hampshire, Bureau of Materials and Research, New Hampshire Department of Transportation.

STOP 1: Pemigewasset Scenic Overlook

By Brian Fowler and Wally Bothener
This stop lies immediately west of the height-of-land (870 m; 2,855 ft.) on the Kancamagus Highway (NH 112). It provides an opportunity to view scenery typical of the interior of the White Mountain region. The high peaks on the left are the East Peak of Mt. Osceola (1,267 m; 4,156 ft.) and Mt. Osceola (1,323 m; 4,340 ft.). The long Scar Ridge leads off to the right. To the immediate right of the Stop is Mt. Huntington (1,128 m; 3,700 ft.), and to the northwest and beyond are the higher peaks of Mt. Whaleback (1,093 m; 3,586 ft.) and Mts. Flume and Liberty (1,319 m; 4,328 ft. and 1,359 m; 4,459 ft., respectively). These peaks lie at the southerly end of the Franconia Ridge that forms the easterly side of Franconia Notch. Through the valleys below flow the East Branch of the Pemigewasset River and its tributaries.

Much of the region visible is underlain by the Conway and Mt. Osceola Granites. They dominate the Early Jurassic White Mountain composite batholith (see Eusden, et al., 2013, p. 78). They are the same age (~180 Ma) and differ primarily by the presence of one or two feldspars and hydrous or anyhydrous mafic minerals, respectively. They are roughly centered within overlapping ring dikes composed of finer-grained, porphyritic granites and/or syenites that may also preserve volcanic tuffs, breccias, and flows. These components testify to a long history of violent volcanic activity in the middle Mesozoic. Younger analogs include the Tertiary San Juan volcanic field of southwestern Colorado, the Pleistocene Yellowstone volcanics, and perhaps the recent Mt. Toba, Krakatoa, Mt. Mazama (Crater Lake), Mt. Pinatubo, and Mt. St. Helens activity in which parts or the entire volcanic edifice was removed during eruption.

All this terrain was at least twice buried by Pleistocene continental glaciation. The stoss and lee topography created by ice moving from northwest to southeast across the region is evident in steeper southeast-facing and gentler northwest-facing slopes. All these slopes are geologically young, having been exposed just since the departure of the Late Wisconsinan Ice Sheet 12,000 to 15,000 calendar years ago.

Of obvious interest are the numerous landslide tracks that scar the mountainsides. All those visible are debris avalanches that originated on 30\(^\circ\) and steeper slopes from super-saturation of collapsed moraine and bouldery-cobbly weathering detritus. Slides were initiated by heavy precipitation and/or melting events, with most occurring prior to human settlement of the region. These tracks have been subject to repeated but lower-volume avalanching as saturation events mobilize residual debris in their tracks and along their margins. The surfaces of their debris fans grade downslope from bouldery-cobbley debris to chaotically interbedded deposits of coarse sandy-silty gravel and minor amounts of stratified silty sand. The fans display stratigraphy typical of debris flows with the largest clasts on and near their surfaces. This stratigraphy is created by dispersive stresses within the debris streams during their rapid downslope movement.

Recent movements include those created by Tropical Storm Irene in 2011. This extraordinary storm furnished the region with 6 to 8 inches of rain in an 8 to 10-hour period. This deluge remobilized practically all of these tracks and created numerous new tracks in many previously unaffected drainages. We may be able to observe one of these new tracks from the highway later in the trip on the northwesterly side of Mt. Eisenhower (1,451 m; 4,760 ft.) near Bretton Woods.
Drive-By Site B

Loon Mountain Bridge over East Branch Pemigewasset River in Lincoln, NH

On August 28, 2011, Tropical Storm Irene (downgraded from hurricane status as it entered the New England area) tracked northward, centered over the Connecticut River Valley, which is the border between New Hampshire and Vermont. Being on the easterly side of this fast moving storm, NH was spared the worst effects of the storm. Nonetheless, the state still experienced high winds, heavy rain and flash floods. As the storm passed over the higher elevations of the state, the water was rung out of the clouds with up to 7 inches falling in a 12 hour period in the White Mountain National Forest region. Damage to the region was extensive, including a number of roads and bridges being washed out. For a short period of time travel east to west in the region was not possible because of many closed roads. Fortunately, nobody was injured in the flooding.

A major bridge impacted by Tropical Storm Irene was the Loon Mountain bridge crossing over the East Branch Pemigewasset River in Lincoln, NH. The 1960’s era bridge was a three span structure that originally crossed a small lake created by a dam 1,000 feet down river. Thus, the bridge was designed for calm water conditions and had shallow spread footings on soil. A flood in 1973 destroyed the dam, and the impoundment drained. The bridge then experienced severe scour problems at the piers from the river, so they were retrofitted with sheet piles to deepen them.

Unfortunately, the stub abutments were never retrofitted. When Tropical Storm Irene hit, the river became a torrent, eroding about 30 feet of the northern river bank, leaving the north bridge abutment with virtually no support. The bridge stood for three days before collapsing. The span was removed, and a temporary one lane bridge was installed. Because of its history of problems, it was determined that a full bridge replacement would be pursued at a cost of 6 million dollars.

Map showing the track of Tropical Storm Irene on August 28, 2011, along the border of New Hampshire and Vermont.

Loon Mountain Bridge the day after the tropical storm event with 30 feet of the north embankment soil eroded away.

August 31, 2011, the north abutment falls.
STOP 2: Barron Mountain Rock Cut

By Richard Lane, Brian Fowler, Krystle Pelham

Rock Slide

On November 7, 1972, during the construction of Interstate 93 in Woodstock, NH, a rockslide consisting of 17,000 +/- cubic yards of rock buried a portion of the Interstate 93 northbound barrel. The site, which overlooks the Pemigewasset River, is located at the base of Barron Mountain between I-93 Exits 30 and 31. Immediately after the slide, all rock excavation in the area was ceased and an extensive redesign of the roadway was undertaken to include changes to the highway alignment, reconfiguration of the rock slope, construction of a massive concrete retaining wall, relocation of a segment of NH Route 175 along with construction of three new bridge structures over the Pemigewasset River, drilling of horizontal drains to reduce water pressure in the slope, installation of rock reinforcement to stabilize the rock cut and instrumentation to monitor for further movement (Haley & Aldrich, 1973a).

Site Conditions and General Geology

The existing northbound rock slope reaches a maximum height of 130 feet and is 600± feet in length. A rock bench, 90 feet above ditch elevation, extends along the rock slope except on the north end where the scar from the slide is located (Figure 1). The southbound barrel, constructed 30 feet below the northbound barrel, notches into bedrock in the median and rests on fill down slope. A concrete wall, built along the river, supports the toe of the steep embankment slope.

Slightly foliated gneissic rock composes the southern portion of the rock cut, grading into strongly foliated quartz-mica schist in the northern section. A large andesite dike intrudes the country rock in the middle of the cut with smaller basalt and pegmatite intrusions scattered throughout the rock slope. Several sets of joints crisscross the rock, some parallel to and others transverse to the foliation. Many of the joints in the schistose zone are filled with mylonite gouge with the presence of slickenside along some of the surfaces. Numerous fractures and offset features indicate a complex history of past tectonic events.

The 1972 rockslide occurred along a highly fractured mylonite zone, which ranged in thickness from 1/2 inch to 11 feet (Figure 2). This weakened zone of low strength material along the failure surface was oriented nearly parallel to the roadway alignment and dipped into the road at approximately 38 degrees (Fowler, 1976 and 1977). Additional mylonite zones with similar orientation were encountered south of the slide area during the early stages of the reconstruction requiring a second redesign of the rock slope. Water flowing along unfavorably oriented mylonite zones was most likely a major factor in triggering the slide.

Rock Reinforcement Installed to Stabilize Slope after Slide

Both active and passive reinforcements were installed at the Barron Mountain site (Figure 3). The passive reinforcement consists of 70 tendons, generally 50 to 60 feet in length (instrumented tendons were longer), installed with no anchorage assembly in three rows on a 10’ by 10’ grid pattern along the toe of the southern half of the northbound rock slope. An additional 30 tendons were installed on 8 foot centers in the upper portion of the rock slope. The tendons are 1.25
inches in diameter, Dywidag, Grade 150, continuously threaded, solid steel bars, which are encapsulated in cement grout along their entire length. In general, the tendons were installed at an upward angle of 25 to 30 degrees from horizontal (Haley & Aldrich, 1974). The primary purpose of the tendons is to prevent large-scale failures in the rock slope.

The active reinforcement consists of polyester resin grouted, pre-stressed rock bolts installed to secure existing blocks, to tie together the rock mass, to preserve the full gravity effect of the rock bench, and to prevent minor rock falls from reaching the highway (Haley & Aldrich, 1974). The rock bolts are 1 inch in diameter, Dywidag, Grade 150, continuously threaded, solid steel bars which are grouted along the anchor zone with polyester resin grout. A small number of the rock bolts are Bethlehem Steel, Grade 80, continuously threaded, solid steel bars (Haley & Aldrich, 1974). The pre-stressed rock bolts are end point anchorages secured with a bearing plate and nut at the rock face. The unbonded, free-stressing portion of the rock bolts is not grouted and is unprotected. The rock bolts ranged in length from 10 to 30 feet and were initially pre-stressed to 20 or 40 kips, depending on the grade of steel. A total of 200+ rock bolts were installed on the northbound and southbound rock slopes at the Barron Mountain site.

Field Instrumentation

Field instrumentation was installed to monitor the rock mass behavior and to collect data on the performance of the rock reinforcement (Haley & Aldrich, 1974). The instrumentation included three 150 foot deep six position rectilinear extensometers on rock tendons, three 80 foot deep four position rectilinear extensometers on rock tendons, two 50 foot deep two position mechanical extensometers on rock tendons, ten sets of temperature measurements on rock tendons using thermistors, and four sets of load cells on rock bolts. In addition, twelve vertical holes were drilled 50 feet into rock to serve as observation wells for monitoring water levels and to listen for subaudible rock noise.

Initially all the instruments were read weekly and the observation wells checked monthly. The data collected from the instruments was continuously plotted in an attempt to identify potential movement within the rock mass and monitor changes in stress levels in the rock reinforcement.
elements. After 18 months, instrument readings were reduced to four times a year. As time passed, instruments began to fail, readout wires were damaged by rock fall, wires were chewed by mice, and metal readout boxes corroded. Every spring and fall, detailed visual observations were conducted to include examination of cracks in the rock, water seepage, staining and condition of exposed portions of the rock reinforcement and grout. Although, no obvious trends in the instrument readings developed, loosening of bearing plates at several of the non-instrumented rock bolts and heavy seepage of water from a few tendons were observed. Continuous plots of the instrument readings were maintained until 1985, when the last of the instruments stopped working. Although visual inspections of the rock slope and the reinforcement were conducted periodically, there was no method for determining the actual condition of the existing rock reinforcement. Over time, corrosion of the metal reinforcing elements, particularly the unprotected segments of the rock bolts and the long-term reliability of the resin grouted pre-stressed bolts were becoming more of a concern.

**NHDOT Two Phased Research Study (2003-2004)**

The estimated design life of unprotected rock reinforcement systems is approximately 50 years based on service life and metal loss equations. The New Hampshire Department of Transportation (NHDOT) has been concerned with the longevity of the rock reinforcement system at the Barron Mountain rock cut given that more than half the anticipated design life has passed. To address this concern, the NHDOT undertook a research study to assess the existing condition of the rock reinforcement. The condition assessment followed the recommended practice from NCHRP 24-13 (NCHRP, 2002) and was performed in two phases implemented in the summer 2003 and fall 2004. McMahon and Mann Consulting Engineers, P.C. conducted both phases of the research.

The first phase (Fishman, 2004) involved the measuring of the corrosiveness of the surrounding environment and performing nondestructive testing (NDT) on selected reinforcement elements. Samples of weathered rock and groundwater were tested for pH, resistivity, moisture conditions, and sulfate and chloride ion concentrations. A rate loss model was used to determine potential metal loss from corrosion and to estimate the remaining service life of the reinforcement. The study utilized four NDT methods, recommended in NCHRP 24-13 (NCHRP, 2002), to assess the condition of the reinforcement elements. Two were electrochemical: half-cell potential measurements and measurement of polarization current; and two involved wave propagation techniques: the impact echo test and an ultrasonic probe. The electrochemical tests identify the presence of corrosion or the vulnerability of the reinforcement steel to corrosion. The wave propagation techniques assess the severity of the corrosion, diagnose the loss of pre-stress and the lack of grout cover, determine if the cross section had been compromised, and identify locations of potential bending or deformation in the metal bars.
The second phase (Fishman, 2005), which was a pooled fund study, used destructive testing to verify the results from Phase I. The techniques included lift-off testing of selected rock reinforcement and the physical, chemical, and metallurgical testing of steel and grout samples retrieved from exhumed reinforcement (Figure 4). The grout condition was evaluated by observing the coverage of the exhumed reinforcement, by the consistency of the grout, and by the physical properties of the grout mix. Bulk specific gravity and absorption were used to determine the effectiveness of the grout as a barrier against moisture and to manage the intrusion of elements that could cause corrosion. Examination of the exhumed metal elements consisted of visual observations of corrosion, measuring the pit depths and the loss of section. Samples of the exhumed metal reinforcements were subjected to tension tests to measure the percentage of elongation and to determine the corresponding stress-strain curves. Metallurgical tests included a spectrographic analysis to assess the metal composition, and a metallographic examination to observe the microstructure of the thread bar material. Destructive testing verified that the electrochemical tests correctly identified the presence of corrosion. The lift-off tests and direct measurements confirmed the echo test results. Measurements on exhumed rock reinforcement verified that the greatest loss of section was within the free length behind the anchorage assembly.

Figure 3 - Typical section with rock reinforcement (Fowler, 1976)
The tendons are in better condition compared to the rock bolts. In spite of the apparently high porosity of the cement grout, it appears to have protected the steel from significant corrosion to date. Many of the rock bolts have suffered a loss of pre-stress and some corrosion is evident. Thus, with respect to impacts on service-life, the rock bolts at this site are more vulnerable than the tendon reinforcements. Compared to loss of service from corrosion, results from the condition assessment revealed that loss of pre-stress is the bigger concern relative to remaining service-life. The condition assessment also revealed locations of increased corrosion activity. Thus, a sound technical basis was established for planning future maintenance and rehabilitation activities at the site, ultimately resulting in a cost savings to the NHDOT.

The Barron Mountain rock cut was a unique site for determining the effectiveness of these techniques because of the age of the reinforcement, the environmental conditions, the variety of installation procedures, and the use of different types of grout. The loss of measured cross section of the unprotected portion of the rock reinforcement was consistent with the predictions from the mathematical models for the service life of unprotected steel and with the observations from the NDT.

**Removal of Unstable Block and NDT Testing Along I-93 Southbound Median Rock Cut**

During 2005, an unstable block was removed from the existing median rock slope along the southbound barrel. The block (estimated dimensions 6 ft. by 7 ft. by 8.5 ft.) had become detached from the surrounding rock forming an open tension crack, 1 to 2 inches in width. Although the block had moved, it was still held in place by two 1-inch diameter resin grouted rock bolts. Rock bolts in the median cut were of similar design and installed with the same procedures as those along the northbound barrel. The bearing plates for rock bolts securing the block had buckled, indicating a high level of distress and a significant increase in load. Portions of the rock bolts that had secured the block were recovered for testing when the block was removed. One of the steel rock bolt bars was severely corroded where it had crossed an open joint. Measurements showed the bar had lost approximately 25% of its cross section (Figure 5).

![Figure 4 - Exhuming rock bolt by over coring (Lane, 2005)](image1)

![Figure 5 - Approximately 25% loss of cross section of rock bolt recovered from the southbound median (Lane, 2005)](image2)
McMahon & Mann Consulting Engineers returned to the Barron Mountain rock cut in 2007 to evaluate thirty-six existing rock bolts installed along the median rock cut of the southbound barrel. The work was conducted under the National Cooperative Highway Research Program (NCHRP 24-28), LRFD Metal Loss and Service-Life Strength Reduction Factors for Metal Reinforcing Systems in Geotechnical Applications (NCHRP, 2011). The evaluation consisted of conducting nondestructive tests (NDT) on three clusters of rock bolts scattered throughout the existing cut and testing of soil/rock infilling material. The NDT measurements included half-cell potential; corrosion rate; wave dispersion from sonic echo tests; arrival times from sonic echo tests; and arrival times from ultrasonic tests. Interpretations of the impact tests indicated approximately 30% of the bolts had experienced loss of pre-stress and the grout quality was questionable in 80% of the cases. These results were consistent with the findings from the two phased research study (2003-2004) completed on the existing rock bolts installed along the northbound rock cut.

**Retrofit and Remediation of Rock Reinforcement at Barron Mountain (2009)**

Although visual inspections of the rock slope and reinforcement were conducted annually there was no method for determining the actual condition of the existing rock reinforcement. More than half of the generally accepted 50-year service life had passed, and results of two phased research study conducted under a pooled fund study (TPF-5(096)) indicated that approximately 30% of the rock bolts may have suffered a loss of pre-stress. The research provided an effective method for identifying areas of possible corrosion, assessing the overall condition of the reinforcements and estimating remaining service life. As a result, in the summer of 2009 a remediation contract was advertised that was completed by Pacific Blasting and Demolition, Ltd. Over 200 resin grouted rock bolts were tested and approximately 32% of the rock bolts tested exhibited a loss of pre-stress and required replacement.

![Figure 6 – Rock bolt installation being performed in 2009 by Pacific Blasting and Demolition.](image)

**References**


STOP 3: Old Man Historic Site

Franconia Notch and the Old Man of the Mountain

By Brian Fowler

Franconia Notch is one of the best developed and famous glacial troughs in northeastern North America and the White Mountain region. Its fame arises not only from its impressive scenery but from the former presence on the cliffs of Cannon Mountain of the singularly famous Old Man of the Mountain natural rock “Profile”. On May 3, 2003, the Profile collapsed and fell about 825 ft. (250 m) onto the talus slope above the Interstate 93 “Parkway” in the Notch. The collapse resulted in the loss of a famous geologic landmark, the official emblem of the State of New Hampshire, and a sublime “old friend” to many who had visited its viewing site in the 198 years since it was discovered in 1805. On this 10th anniversary of its collapse, this stop will visit the Old Man Memorial Plaza & Geological Exhibit at its former viewing site where the geology and rock mechanics of its creation and failure can be reviewed and the implications of its collapse considered. Eusden et al. (2013) contains a general summary of these matters. Detailed discussion, along with the Profile’s nearly 200-year “human history”, appears in Fowler (2005).

Formation of the Notch

Franconia Notch (Figure 1) is a classic example of a glacial trough created by several lengthy episodes of erosion by thick continental ice sheets. Because their “plastic” ice masses sought the path of least resistance through the mountain range as they approached from the northwest, they first converged into its deeper pre-existing passes and then flowed through them continuously as they thickened and then waned during each episode. The last two of these ice sheets (the Illinoian and Late Wisconsinan) were sufficiently thick to bury the region’s highest peaks as shown by deposits of basal till on the summit of Mt. Washington at 6,288 ft. (1,900 m).

Most workers here agree that the last pulse of continental glaciation (Late Wisconsinan) arrived here about 25,000 calendar years ago, reached its maximum thickness about 18,000 years ago, started to recede from the highlands about 15,000 years ago, and had fully retreated from lowlands north of the highlands by about 12,000 years ago. Surficial geologic mapping shows this deglaciation process occurred in two ways. First the higher peaks were exposed as nunataks above the ice sheet as it downwasted around them, while the rest of the ice sheet gradually retreated northwestward around the mountain highlands.
Franconia Notch is joined in the region by Crawford and Pinkham Notches as the best developed examples of its glacial troughs. Together, they represent the deepest of the pre-glacial passes in the mountain front. The glacially-sculpted U-shaped cross-sections of these Notches are distinctive and best developed in Crawford Notch, which we will visit at Stop 5. This cross-section in Franconia Notch has been substantially modified by post-glacial talus infilling, particularly along its western slopes.

Figure 1 – View of Franconia Notch from the south.

The Old Man of the Mountain

The Old Man of the Mountain (Figure 2) was a delicately cantilevered, 7,200-ton rock mass created naturally by the combination of two actively persistent weathering processes that formed the Cannon Cliff after the departure of the last ice sheet around 12,000 calendar years ago. The first is the intense kaolinization weathering of potash-rich feldspars along the joint systems in its Early Jurassic Conway Granite pluton. The second is the easy mechanical excision of consequently loosened blocks by intense cycles of freeze-thaw wedging (presently 30-60 cycles per year). The effectiveness of these processes is evident from the enormous talus slope accumulated at the base of the Cannon Cliff, the most extensive in the region.
Three systems of joints in the rock mass were exploited by these processes. The first and second are subvertical and subhorizontal joints formed during the cooling of granite pluton (Figure 3), while the third is a combination of more closely-spaced, near-surface subhorizontal and subvertical joints formed during surface dilation as the weight of the last ice sheet was removed. This dilation opened easy paths for deep penetration of large volumes of rain, meltwater, and wind-driven cloudwater into the rock mass. This penetration in turn encouraged the rapid progress of kaolinization along the joints and a dramatic simultaneous increase in porosity and cleft water pressures within the intersecting joint systems (Figure 4). This ample supply of cleft water made it relatively easy for the active diurnal and seasonal freeze-thaw cycling to mechanically excise joint-bounded blocks destabilized by the weathering. In these ways, the formation of the strikingly human Profile was a serendipitous consequence of just which of these destabilized blocks was excised and in just what order so that the Profile and the delicate cantilevering needed to perch it on the cliff was created and temporarily (from a geologic point of view) preserved (Figure 5).
Figure 3 - Joint distribution and rock mass bounding joints in the pre-collapse rock mass. Chin block (now missing) creates lowest shadow.

Figure 4 - Intersecting weathered joints in the pre-collapse rock mass. Chin block on the right behind the vertically hanging rope.
Figure 5 - Delicately cantilevered pre-collapse rock mass. The triangular chin block is to the right of the vertically hanging rope.

Figure 6 - Post-collapse residual rock mass after Profile toppled forward (left). Note broken steel turnbuckles on residual Forehead (topmost) slab.
On May 3, 2003, as had been predicted by earlier studies (Fowler, 1982, 2005), the structural base upon which the cantilevering relied failed and the frontal portion of the rock mass that included the Profile's particular blocks toppled forward and off the cliff. This event created a significant social and tourist-economy loss for the State. In response, numerous proposals were put forward to physically replace the Profile in front of the cliff using various constructed combinations of rock and/or lighter-weight artificial materials tensionally anchored to the residual rock mass. Such proposals rely on the assumption that this residual mass is in sufficiently sound structural condition to accept this sort of foundation (Figure 6), but post-collapse evaluations show this assumption is not appropriate.

These evaluations strongly suggest the bulk structural integrity of the residual mass has been, and continues to be, seriously compromised by the same intense processes that formed and then destroyed the Profile. They show that deep penetrative weathering and buildup of cleft water pressures along its joints, combined with intense freeze-thaw excision, continue unabated reducing its bulk strength, resistance to various modes of block displacement, and its capability to serve as a foundation resource for tensional anchorage.

Immediate post-collapse and recent continuing observations (Fowler, 2005, 2009) show these deteriorated conditions continue to develop deeply within the residual rock mass. From the residual cliff inward, significant deterioration is observed to depths of at least 20 ft. (6 m), while observations on the mountainside behind and above show these conditions to depths of more than 50 ft. (15 m), especially in the vicinity of the compound subvertical bounding joint systems at the margins of the residual mass (Figure 3). Increasing cleft water pressures are particularly problematic in this rock mass because there is no feasibly reliable way to reduce or eliminate them by sealing or draining, as shown by many attempts to do so on and above the original Profile over the many years before its collapse.

Figure 3 shows these seriously deteriorated internal and bounding joint combinations. When the cantilevered blocks that comprised the Profile collapsed, the rearward portion of the similarly deteriorated rock mass behind did not fall simultaneously. This permitted the residual mass to remain tenuously perched on the cliff. However, partial or even complete collapse of this mass could now easily occur as a combination of sliding and toppling within and along these weathered joint systems.

In any case, these geotechnical circumstances lead to the conclusion that successful long-term active and/or passive reinforcement of this dynamically impermanent rock mass will be very difficult and probably impossible to achieve. This leads informed geotechnical and public policy observers to generally agree that investment in any project proposed to be founded upon it will represent imprudent professional and political risk.
Old Man Memorial Plaza
The Old Man Memorial Plaza recreates the remarkable experience of seeing the Profile high above on the cliff. The Plaza and its exhibits have been constructed by the Old Man of the Mountain Legacy Fund in cooperation with the NH Geological Survey using entirely private donations and the proceeds from the sale of personalized pavers on the Plaza floor. Pavers are of three sizes that are custom-engraved with personal messages; many are still available. Information about the Plaza, its exhibits, history and geology of the Old Man, purchase of pavers, and Fund activities is available at: http://www.oldmanofthemountainlegacyfund.org/.

References


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LUNCH: Cannon Mountain

Lunch will be served at the Peabody Slope Base Lodge of the famous Cannon Mountain Ski Area owned and operated by the State of New Hampshire, Division of Parks & Recreation. The ski area was constructed in the early 1930’s by the Civilian Conservation Corps and others as a way to provide jobs during the Depression and to tap into the newly-popular automobile-based recreation industry then emerging. Cannon was one of the first ski areas of its size in North America and was the first in North America to feature transportation of tourists and skiers to its highest point, the summit of Cannon Mountain (4,100 ft.), via the aerial tramway. The Tramway is celebrating its 75th year of operation this year. In 1938 when it opened, it immediately became a sensation in the skiing world, drawing much appreciated attention to northern New Hampshire’s recreation opportunities, along with providing hearty skiers with up to 2,300 feet of elevation drop. Many of the earliest competitive alpine skiing events were held here at Cannon because of its steep and highly technical terrain. Many championship skiers have learned and perfected their technique on its challenging often windblown and icy slopes, the latest of these being Bode Miller of Olympic and World Cup fame.
STOP 4: Carroll Visitors Center

Introduction
By Woody Thompson
This stop is located in Twin Mountain village at the former site of the Twin Mountain House (1868-1960). It was one of the “grand hotels” that catered to White Mountain tourists from the 1800s through the mid-1900s (Figure 1). Most of those hotels have been lost to fire or demolition. (Later today we’ll see one of the great survivors – the Mount Washington Hotel at Bretton Woods.) Note the information kiosk at the visitor center, which provides interesting details about this part of New Hampshire.

Figure 1 - View looking north at the Twin Mountain House in the 1800s (part of a stereoscopic view published by Kilburn Brothers, Littleton, NH). Note the cross section of an esker ridge to left of hotel and the Ammonoosuc River in foreground.

We are overlooking the Ammonoosuc River valley just south of here (Figure 2). This is one of several rivers that flow generally west out of the White Mountains and empty into the Connecticut River on the New Hampshire-Vermont border. Many Quaternary scientists have investigated the Ammonoosuc River basin, starting with Louis Agassiz in the mid-1800s.
(Thompson, 1999). The appeal of the region to 19th century geologists was fueled by easy railroad access and popularity of the mountain scenery with tourists and artists. There were numerous controversies over the extent and types of glaciers that affected the White Mountains, and the manner in which they developed and subsequently retreated. Details of the glacial chronology and its relation to climate change continue to be studied today.

![Image of Twin Mountain village](image)

**Figure 2 - Google Earth view looking south across Twin Mountain village, showing the stop at Carroll visitor center at the junction of Routes 3 and 302. The gravel pit at bottom-center edge of photo is the esker location shown below in Figure 3.**

The glacial features in the Ammonoosuc valley tell us much about the retreat of the last glacial ice sheet from the White Mountains. As the climate warmed toward the end of the Ice Age, the Laurentide Ice Sheet began to melt. This caused the glacier to become thinner, while at the same time its southern margin retreated back toward Quebec. Some of the rock debris carried by the glacier was simply released from the melting ice and not carried any farther. This material is called “till”, and it forms a widespread blanket over New Hampshire.

Just north and west of here, there are clusters of bouldery till ridges (moraines) that were heaped up at the margin of the glacier when it was briefly reenergized during its overall retreat. These deposits include the Beech Hill Moraines marked on Figure 4. Recent research has shown that they formed during a brief interval of cold climate called the “Older Dryas event” that occurred about 14,000 years ago (Thompson et al., 2009).
Other glacial sediments were transported by meltwater streams that originated within or upon the ice sheet. The coarser material (gravel) usually was left closest to the glacier, either as subglacial tunnel fillings (eskers) or as stream deposits laid down a short distance beyond the ice margin (outwash). Sand, silt, and clay tended to be carried greater distances and sometimes came to rest in temporary glacial lakes. Both eskers and glacial lake deposits occur here in the Ammonoosuc River valley and will be described below. Glacial sand and gravel deposits are very important to the New England economy. They offer well-drained building sites that are easier to excavate than till or bedrock. They are also important sources of construction aggregate, and many of them are high-yield aquifers.

Note: The following sections are modified from Thompson et al. (1999, 2002).

The Ammonoosuc Valley Esker System

Meltwater flow within the Laurentide Ice Sheet carved tunnels at the base of the glacier. The subglacial drainage often followed valleys, but the water was confined under pressure and could actually flow uphill over topographic barriers beneath the ice. Today we can tell where some of those tunnels were located because they became choked with sand and gravel. After the ice melted away, the tunnel-filling sediments were left behind as ridges called eskers.

Figure 3 - West side of esker ridge, just north of Twin Mountain. W.B. Thompson photo.
The Ammonoosuc esker system originates just north of Twin Mountain village (Figure 3) and follows the valley eastward. It merges with another esker entering from the north and continues to Crawford Notch. The esker ridge is discontinuous, and parts of it are concealed in the forest. A convenient place to visit the esker is the Eisenhower Wayside Park, located on U.S. Route 302 between Bretton Woods and Crawford Notch. A cross-section of the ridge is seen from the parking lot, and you can hike up a short path to the top and follow the ridge crest back into the woods.

**Glacial Lake Ammonoosuc**

In New England, much of the meltwater from the ice sheet poured into temporary lakes that formed in front of the glacier as it retreated northward. Many of those lakes resulted from the ice damming valleys that sloped toward the glacier margin. At any given time, the water level in each ice-dammed lake was controlled by the elevation of the lowest gap in the surrounding hills through which the lake water could escape. As the ice sheet retreated from a river basin, the lake level in that valley would drop whenever a new and lower outlet was uncovered by melting of the ice. Eventually the lake would completely empty and disappear when the ice sheet left the basin.

We are not certain who was first to recognize the former existence of glacial lakes in this part of the White Mountains. It may have been Warren Upham, who worked with State Geologist Charles Hitchcock on the monumental geological survey of New Hampshire in the 1870s. Upham’s observations led him to propose that a lake had existed in the upper Ammonoosuc River valley, in the vicinity of today’s Bretton Woods community (Upham, 1878). This was the water body that James Goldthwait (1916) named “Lake Ammonoosuc”. It resulted from damming of the river basin by the last ice sheet when it receded west and north from the valley. As the ice margin withdrew, numerous lake levels developed through the process described above (Thompson et al., 1999).

In 1930, Richard Lougee assisted Goldthwait in the gravel inventory funded by the New Hampshire Highway Department. Lougee was assigned to map several 15-minute quadrangles in the White Mountains. He identified many of the glacial lakes that were dammed by the margin of the ice sheet as it withdrew from this area and temporarily blocked the normal stream drainage. Lougee also realized that the earliest stage of Lake Ammonoosuc (the Crawford stage) drained eastward through Crawford Notch.

Following the Crawford stage, glacial retreat enabled Lake Ammonoosuc to drain southwest into the Gale River valley in Franconia. The water escaped through five progressively lower outlets called “spillways”. The lake levels corresponding to these spillways are known as the Gale River stages of Lake Ammonoosuc (G1-G5 in Figure 4).

The widest and deepest phase of Lake Ammonoosuc may have been Gale River 2 stage. The log for a well located just west of Twin Mountain village shows a contact between thick glaciolacustrine clay and the underlying till at an elevation of 401 m (Flanagan, 1996).
Comparison of this lake-bottom elevation with the nearby G2 spillway elevation of 445 m indicates a local water depth of at least 44 m.

Figure 4 - Map from Thompson et al. (1999) showing outlets for the Gale River stages of glacial Lake Ammonoosuc (arrows G1-G5). The thick gray lines mark successive positions of the glacier margin. B = Beech Hill Moraines. C = Carroll Delta.
Figure 4 also shows recessional positions of the glacier margin that correlate in time with the Gale River and later stages of Lake Ammonoosuc. These ice margins were inferred from the orientation of nearby moraines, together with ice blockages of the valley that would have been required to hold the lake at elevations corresponding to the known deltas and spillways.

Lougee (1940) published the elegant block diagram reproduced in Figure 6. It shows the ice margin lying against the northwest flanks of Beech Hill and Cherry Mountain when the Carroll Delta was built into Lake Ammonoosuc. This delta is located a short distance north of our stop, on US Route 3. The elevation of its upper surface indicates that it was deposited during the Gale River 2 stage of Lake Ammonoosuc. The Carroll Delta and associated esker deposits have been important sources of sand and gravel for many years. The Twin Mountain Sand & Gravel pit (now owned by Pike Industries) has been worked at least since the mid-1900’s (Figure 5).

**Figure 5** - View looking west at an exposure of the Carroll Delta in the Twin Mountain Sand & Gravel pit. The former lake level is marked by the contact between the nearly horizontal fluvial beds (top) and the sloping foreset beds below. The glacier margin stood at the north edge of the delta, and the delta built southward (R to L) into Lake Ammonoosuc. W.B. Thompson photo.

Today we can still see the spillway channels eroded by water draining from glacial lakes in the White Mountains. Most of them have flat floors cut into glacial till, and are now occupied by swampy wetlands. The channels are most clearly visible when leaves are off the trees. A good example is the G2 spillway (Figure 4) that drained the overflow of the Gale River 2 stage of
Lake Ammonoosuc. This channel crosses US Route 3 in several places southwest of Twin Mountain village.

Water entered Lake Ammonoosuc not only from the melting glacier but also from the early Ammonoosuc River and smaller streams draining the surrounding mountains, as shown by Lougee's diagram. The position of the lake shifted westward – and its surface elevation dropped – as the ice margin retreated down the valley toward Bethlehem. Eventually it drained completely and the upper reach of the Ammonoosuc River joined with the lower part of the river that flows southwest from Littleton. Following the disappearance of Lake Ammonoosuc, flood plain and alluvial fan deposits have accumulated on the old lake floor.

**Figure 6.**
Glacial Lake Ammonoosuc - Gale River 2 stage  
(after Lougee, 1940)

**Cog Railway**

The steam engine and passenger car displayed next to the Carroll visitor center (Figure 7) were recently retired from the nearby Mount Washington Cog Railway. This famous tourist attraction is the world’s oldest mountain-climbing cog railway, having been completed in 1869. The New Hampshire legislature approved plans to build the railway, though they thought it was an impossible task. One legislator suggested that it “should not only be given a charter up Mount Washington but also to the moon”!
Similar early cog railways were built on Pikes Peak, Mount Rigi (Switzerland) and other mountain locations. They are called “rack railways” because they have a cog wheel on the engine that meshes with a center rack rail on the track. The siding switches are thus quite complicated! Mt. Washington’s “Cog” operation has included several generations and designs of steam engines, much to the delight of both tourists and rail fans. Each engine has a name, and they are listed along with their current status in a Wikipedia article about the Cog: http://en.wikipedia.org/wiki/Mount_Washington_Cog_Railway.

In the early 1900s, regular train service carried passengers right to the base of Mount Washington, where they could simply cross the station platform and board the Cog to the summit! Starting in 2008, new diesel-fueled Cog engines began to replace the coal-burning steam locomotives. A single daily steam train still climbs the mountain, with most of the other old locomotives kept in reserve.

![Cog steam engine and passenger car on display next to the Carroll visitor center.](image)

**Figure 7** – Cog steam engine and passenger car on display next to the Carroll visitor center.

**References**


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STOP 5: Mt. Washington Scenic Overlook

By Brian Fowler

If the weather is clear on the day of the field trip, this stop will provide us with a spectacular view of the entire Presidential Range. The summit of Mt. Washington (6,288 ft.; with its buildings, towers, and the Mt. Washington Cog Railway ascending its slopes) dominates the view with Mts. Clay (5,533 ft.), Jefferson (5,712 ft.), and Adams (5,774 ft.) to its left. To its right are the peaked summit of Mt. Monroe (5,372 ft.), the subdued ridge of Mt. Franklin (5,001 ft.), the dome of Mt. Eisenhower (4,760 ft.), and the long ridge of Mt. Pierce (4,312), named for the only NH “native son” to serve as a U.S. President: Franklin Pierce. This view is one of the most-often photographed scenes in the White Mountains, especially if graced with some snow and early-Autumn foliage below timberline.

The peaks to the left of Mt. Monroe, except for Mt. Clay, are underlain by high-grade and thus highly erosion-resistant metasedimentary members of the Devonian Littleton Formation. Those to its right are underlain by the lower-grade and less-resistant metasedimentary members of the Silurian Rangeley Formation (see Eusden, et al., 2013). As indicated at Stop 3, Mt. Washington was thinly covered by the last ice sheet (Late Wisconsinan), suggesting that here at Stop 5, there was about 4,500 feet of ice overhead at the Last Glacial Maximum.

Any stop at this location needs to address the spectacular Mount Washington Hotel in the foreground, one of the last of the “grand hotels” in the White Mountains. Opened in 1902 and operated more or less continuously since, the Hotel and its elegant “turn-of-the century” ambiance have played host to many famous (and some infamous) guests, along with the Bretton Woods International Monetary Conference in 1944 that established the World Bank and set the stage for the economic recovery that followed World War II. The Hotel and its surrounding Resort are today operated by Omni Hotels & Resorts. It was recently host to the record 1,150 attendees at the Annual Meeting of the Northeastern Section of the Geological Society of America.
As we proceed south from Stop 5 to Stop 6 (Willey House Site), we will pass through the initial spillway of ancestral Glacial Lake Ammonoosuc (described in Stop 4). This spectacular spillway is the narrow cleft we will pass through about ½ mile south of the Appalachian Mountain Club’s Highland Center at the height of land on US Route 302. The cleft has been substantially modified since immediate post-glacial time by roadway and railway construction through it since the early 19th Century, but it is clear an enormous volume of meltwater was funneled toward and through it when the ice sheet had begun to move north of the area. Evidence of this can be seen from the right-hand sides of the buses, just after passing through the cleft, in the form of the huge but now wholly abandoned plunge pool located immediately south of and below the cleft.

References

STOP 6: Willey House

Crawford Notch and the Willey Slide
By Brian Fowler

As indicated at Stop 3, Crawford Notch is one of the deepest best developed glacial troughs in the region (Figure 1). As with Franconia Notch, its deeply developed glacially eroded cross-section is distinctive, being the result of several episodes of continental glaciation passing through and eroding its flanks.

Figure 1 - Crawford Notch looking south from Mt. Willard (873 m; 2,865 ft.). The debris fan of the Willey Slide lies above and to the right of the south end of the pond visible to the left of the highway in the center of the photograph (elev. ~ 400 m; 1,320 ft.). The highway curves to the left and then over a combination of this debris fan and several others from similar avalanche tracks to its north.

The Willey Slide occurred during a torrential rain event in 1826 and was responsible for the deaths of all the members of the unfortunate Willey Family who had homesteaded on the floor of the Notch at the eventual location of its debris fan. The story of this tragic event was made famous at the time by Nathaniel Hawthorne in his short story entitled “The Ambitious Guest”
(1835), but it has been revisited since by numerous authors. Interested readers can consult Google for this extensive listing of publications. Here we discuss the general rheology of the slide and its deposits, but a bit of the story is needed to properly set the stage.

Upon hearing the noise of the approaching debris avalanche above their house, the family decided to evacuate and seek refuge at locations beyond what they anticipated would be the track of the avalanche. They did so but were quickly buried, along with their farm hands and livestock, by two debris streams that separated around “a large boulder” located just above the house (Hawthorne, 1835). Had they stayed in the house, they would have been spared by this division of debris streams and tragedy would have been averted. All of their remains were subsequently recovered except those of one child who still “rests in peace” somewhere beneath the surface.

Recent STATEMAP surficial geologic mapping of the immediate area for the U.S. and N.H. Geological Surveys (Fowler, 2012) has identified the specific features of this avalanche track and helps better establish what actually happened during the slide. The debris avalanche followed a single track downslope from its initiation point about 1 km (1/2 mi.) and about 500 m (1,650 ft.) above on the slopes of Mt. Willard. At a point approximately 150 m (500 ft.) above the house location, a series of subtle bedrock promontories separated the fast-moving debris flow into the two streams that moved past the house to the north and south. This subtle “divide” occurs on the slope well above and behind the “large boulder” assumed to have spared the house. Work in the area shows this track system has since been frequently (and recently) remobilized by comparatively small-volume debris flows with detritus following the same tracks as the 1826 slide. A short walk up and behind the observation platform at the “large boulder” shows these two debris flow channels and their abutting levees. The slopes laterally below the platform show the mixed bouldery-cobbly and till-based debris that comprised the original and later slides.

Publicity about the “tragedy of the Willey Slide”, along with the more or less contemporaneous Owl’s Head Slide to the north in Jefferson, NH and the region’s scenery, aroused great public interest. This curiosity encouraged regional railroad firms, along with local hotel and livery services, to develop excursions and tours of these and other sites during the mid to late 19th Century. This was the beginning of the now robust automobile-based tourist economy in the White Mountains.

References

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Drive-By Site C

US Route 302 Bridge over the Sawyer River in Harts Location, NH

Another bridge impacted by Tropical Storm Irene in 2011 (described in Drive-By Site B for the Loon Mountain Bridge) was the US Route 302 Bridge over the Sawyer River in Harts Location, NH. The bridge, built in 1990 to replace an older structure, was a modern, single span bridge with a 95 foot wide opening. The bridge superstructure consisted of a concrete slab on steel girders. The bridge substructure consisted of full height concrete abutments with spread footings founded on alluvial sands and gravel. There was no history of severe flooding or problems with the bridge. Just downstream is a single span railroad bridge with granite block abutments, constructed over 140 years ago.

Normally, the 9.1 mile long Sawyer River has small flows in a cobble and boulder lined channel that is about 15 feet wide and runs only a few feet deep. On the day Tropical Storm Irene hit, the river nearly overtopped the bridge. The main force of the water was directed at the north abutment because of a bend in the river about 300 feet upstream. The Sawyer River collects runoff from the White Mountains to the west and south, which have significant height, so runoff concentrations were rapid and fast moving. As a result of the flash flooding, the bridge suffered severe scouring effects. Both of the abutments were undermined by the rush of water, and a large quantity of material behind the north abutment was lost. The approach slab fell, buried utilities were cut, and the road was severely damaged. The abutments settled 6 to 18 inches from scour holes formed below them. The abutments and footings cracked from the settlement, the girders twisted and the deck cracked. The highway bridge was a total loss with no way to salvage any part of it. The railroad bridge just downstream suffered some minor scouring at its south abutment, but was spared any major damage because it was shielded by the highway bridge. The scouring at the railroad bridge was quickly and easily repaired by filling the scour hole with concrete.

Figure 1 - US Route 302 bridge the day after the tropical storm event. Note the large dip in guardrail from undermined abutment.

Figure 2 - A close up of the scour that occurred behind the north abutment with the guardrail suspended in the air, the utilities cut, and approach slab fallen into the hole.
As a consequence of the damage to the bridge and along other portions of the highway, US Route 302 was closed for over two weeks. A temporary bridge was installed to allow the highway to reopen. The replacement US Route 302 Bridge is currently under construction through a design-build contract with Alvin J. Coleman & Sons of Conway, NH for a cost of 2.4 million dollars. The bridge design was performed by GM2 Associates of Concord, NH. The replacement bridge consists of a slab on steel girder deck, but the span has been increased to 135 feet to provide a larger hydraulic opening for the Sawyer River. The foundation depths have been extended 5 feet deeper than the previous bridge, but they still have a spread footing configuration. Very large rip rap stones, with a minimum dimension of 3.4 feet, were specified at the abutments as a scour countermeasure.

A deep foundation configuration was considered for the bridge in the pre-bid, conceptual design phase by the NHDOT, but subsurface conditions at the site were found to be very problematic. Test borings were drilled as deep as 121 feet without hitting glacial till or bedrock at the site – an unusual condition in NH. Only alluvial and glacial drift materials were encountered in the test borings, and these deposits consisted of sand, gravel, cobbles and boulders. Representative soil densities were difficult to obtain in the test borings that employed Standard Penetration Tests because of the cobbles and boulders, which were nested in several layers as much as 30 feet thick!

Driven piles were not possible at the site. Deep predicted scour depths made slender pile foundations, such as drilled micro-piles, unworkable because of potential unsupported pile lengths. Larger diameter drilled shafts were also impractical because of the numerous boulders present. After considering all alternatives and design standard requirements, a spread footing configuration was deemed the most economical solution by the NHDOT. The deeper spread footings protected by very large rip rap in conjunction with the wider bridge span were the recommended approach to provide cost effective scour resistance for the bridge.

References

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