Rockfall Hazard Mitigation at the TH-53 Bridge, Virginia, Minnesota

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Prepared for the 67th Highway Geology Symposium, July, 2016
Acknowledgments

The authors thank The Minnesota Department of Transportation, Parsons Transportation Group, Hoover Construction, Pacific Blasting and Demolition, and Kiewit Infrastructure Group for their roles and support on the project. The authors also thank the following individuals for their contributions to this project:
Tim Siegel – Dan Brown and Associates
Nathan Glinski – Dan Brown and Associates

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ABSTRACT

The Trunk Highway 53 (TH-53) Relocation Project near Virginia, Minnesota, includes a new bridge across the currently inactive Rouchleau Mine Pit, one of many open pit iron ore mines on the Mesabi Range. Rockfall hazards associated with the existing highwall on the east side of the Rouchleau Pit were assessed and mitigated to ensure construction worker safety and long-term performance of the bridge.

This paper provides a brief geologic background and describes the process of assessing and mitigating rockfall hazards at the TH-53 bridge site. Rockfall hazards were primarily assessed on the basis of observations made during site visits, including: geologic feature mapping, assessment of existing talus, identifying rockfall sources and travel paths, and run-out distance assessment via trial rolling of rocks. The influences of the local geologic conditions of the Biwabik Iron Formation at the site and the highwall geometry result in the potential for several mechanisms of rockfall hazard at this site, and the extreme northern climate contributes to the hazard through freeze-thaw conditions which initiate rockfall events. Rock bounce heights and velocities were analyzed using the Colorado Rockfall Simulation Program (CRSP).

Mitigation elements for the protection of workers constructing the east pier of the bridge, which is located on a bench cut into the eastern pit highwall, include an attenuator fence system and combined wire mesh and cable net drapery covering portions of the highwall face. A soil berm will provide long-term protection for the eastern bridge pier. Details of the mitigation system and its construction are described, including: element selection and sizing, site-specific details, anchor design, challenges to construction, and quality control.
INTRODUCTION AND BACKGROUND

Trunk Highway 53 (TH-53) southeast of Virginia, Minnesota, is being realigned to allow for future open-pit iron ore mining of land along the existing highway corridor. The new alignment follows the E-2 option shown in Figure 1. The new section of highway includes a three-span bridge across the currently inactive Rouchleau Mine Pit. The bridge is approximately 1,132 ft long with two abutments and two intermediate piers. The bridge is shown in elevation in Figure 2 and Figure 3 is a conceptual rendering of the bridge.

Construction of the 190-ft high pier column from the base of the East Highwall (circled in Figures 2 and 3) required excavation of a work pad into the toe of the highwall. Personnel and equipment needed to construct the pier and its foundations are therefore exposed to a significant construction-phase rockfall hazard. There is also a potential long-term rockfall hazard to the pier. The rockfall protection system employed a combination of cable net drapery with wire mesh backing, an attenuator fence, and catchment berms. The cable net and wire mesh drapery are suspended from two different levels, the top of the highwall and from an intermediate bench. The top of the lower drapery panels are suspended from an elevated top cable to form a 6-ft high attenuator fence. Berms were constructed around the perimeter of the Pier 1 work area, and the base of Pier 1 will be protected by a berm upon completion.

Figure 1 – Project Location and TH-53 Realignment.
GEOLOGY

Geologic Setting

The project is located in the Virginia Horn area of the Central Mesabi Iron Range. The Mesabi Range is a narrow belt of iron-bearing rocks in the Superior Upland physiographic province of northeastern Minnesota. The bedrock unit of interest at the bridge site is the Biwabik Iron Formation. Rocks of the Biwabik were formed between 1.85 and 1.93 billion years ago as sediments deposited in a shallow marine environment on the northern edge of the northward-migrating Animikie basin.
Stretching roughly between the cities of Grand Rapids and Babbitt, the Biwabik Iron Formation is approximately 120 miles long and between 0.25 to 3.0 miles wide, as shown in Figure 4 (1). According to Severson et al. (1), the Biwabik Iron Formation is around 730-780 ft thick in the Virginia Horn area. The formation is subdivided into four units referred to as (from bottom to top): Lower Cherty member, Lower Slaty member, Upper Cherty member, and Upper Slaty member. The cherty members are typically characterized by a granular (sand-sized) texture and thick-bedding (beds ≥ several inches thick). The slaty members are typically fine-grained (mudsized) and thin-bedded (≤ ½ in thick beds). “Slaty” is a local mining term indicating parting parallel to bedding in thin-bedded rocks and is not necessarily indicative of metamorphism or slaty cleavage (1). The cherty members are largely composed of chert and iron oxides with zones rich in iron silicates, while the slaty members are generally composed of iron silicates and iron carbonates with local chert beds. Both cherty and slaty iron-formation types are interlayered at all scales, but one rock type or the other predominates in each of the four informal members, and they are so named for this dominance. The repetition of the major cherty and slaty members is interpreted by geologists as being the result of transgressive and regressive ocean events.

The beds of the Biwabik Iron Formation generally strike approximately N75°E and dip gently south-southeast (2). The major exception to this orientation is the Virginia Horn, a reverse S-shaped bend in the central part of the formation near the cities of Virginia and Eveleth. The Virginia Horn is thought to be a broad, low-dipping anticline–syncline couplet (3). Although, the exact deformational processes resulting in the Virginia horn have never been definitively established (4).

Figure 4 – Map of the Mesabi Range (Biwabik Iron Formation) with the Duluth Complex shown to the east (1).
Local Geologic Conditions

The bridge site is located in the central part of the Virginia Horn “S”. Sub-horizontal bedding plane dip angles typically range from zero to 20 degrees with local variation. Dip direction varies but is predominantly northwest. Joints not associated with bedding planes are predominantly sub-vertical, typically dipping between 70 and 90 degrees. Two or three sub-vertical joint sets exist at locations across the site. The sub-horizontal bedding planes and sub-vertical joints form blocks. The blocks vary in size depending on the spacing of the bedding planes and the nature of the bedding plane contacts. Generally, the slaty layers are comprised of smaller blocks or chips and the cherty layers are comprised of larger blocks of several feet in dimension, as shown in Figure 5.

The intact Biwabik rock is both strong and dense. Uniaxial compressive strength averaged 21,300 psi based on 117 laboratory tests. The average unit weight of the 117 test specimens was 191 lb/ft$^3$. The density and strength of lab samples varied greatly.

![Figure 5 – Exposed Cut Face showing Variations in Block Size.](image)
Geologic conditions at the site result in rockfall through localized toppling, wedge, and block failures. Less durable slaty strata ravel, undercutting the more blocky strata. The primary processes driving localized instabilities are freeze thaw and hydraulic pressure.

Geologic Investigation

Geologic conditions at the site were assessed through several means: visual inspection of exposed cut faces (highwalls), manual strike and dip measurements, vertical and inclined rock core borings with optical and acoustic tele-viewer scans, and photogrammetric joint mapping. Rock core specimens were selected for laboratory testing for density, P-wave velocity, uniaxial compressive strength, and elastic modulus. Much of this geologic and geotechnical information was collected and used for other aspects of the project, including bridge foundation design and rock slope stability analysis, but provided valuable information on rockfall that supplemented the field and photogrammetric investigations.

ROCKFALL ASSESSMENT AND HAZARD MITIGATION

Conditions at the East Highwall changed significantly over the course of construction operations, largely in response to staging of blasting events to create a notch at the top of the highwall where the east abutment of the bridge is to be constructed. Mitigating the rockfall hazards required constant reassessment and revision throughout the construction process. The major steps in the process are described below chronologically.

Preconstruction Assessment and Recommendations

Prior to any clearing, grading, or construction, the East Highwall at the bridge location appeared as it does in Figure 6. Benches were covered in talus and trees were present on the larger benches. Five engineers from Dan Brown and Associates descended and ascended the wall face on ropes during a site visit in April of 2015. During this site visit, observations were made and recorded about rockfall paths, particle size ranges of talus, and the behavior of test rocks rolled from the highwall crest. Most of the rolled rocks stopped on a large bench about halfway up the slope due to the energy absorption of the talus and the natural barrier provided by the trees.

Based on the site observations, the planned mitigation scheme was to construct a cable net attenuator on the large intermediate bench with wire mesh and cable net drapery extending down to the Pier 1 work pad. This system is a hybrid of a flexible rockfall fence and unsecured drapery. At its upper end the drapery is elevated by anchored posts. Rocks impacting the suspended panels are slowed and redirected beneath the drapery, allowing them to move beneath the drapery in a controlled manner with low kinetic energy. The attenuator catches rockfall initiating from above and contains rockfall initiating from below.
Figure 6 – East Highwall Face, Existing Conditions Prior to Construction Operations.

The Colorado Rockfall Simulation Program (CRSP), Version 4, was used to evaluate the effectiveness of the attenuator concept, assess the attenuator location, and to determine the appropriate post height. The program CRSP simulates rocks tumbling down a slope, taking into account slope profile, rebound and friction characteristics of the slope surface, and rotational energy of falling rocks. Rock shape is idealized as being spherical, cylindrical, or disk-shaped. Rockfall can be generated from any location on the slope, and the program output consists of rock velocity, kinetic energy, and bounce height at each location along the slope. The percentage of generated rocks passing each point is also provided. The CRSP modeling methodology is described in further detail in the CRSP User Manual (5).
CRSP was used to analyze three slope profiles corresponding to (a) the centerline of the bridge alignment, (b) the northern edge of the Pier 1 work area, and (c) the south edge of the Pier 1 work area. Each profile was established on the basis of surveyed topographic contours. The CRSP User Manual provides guidelines on ranges of the input parameters for various materials (5). For the preconstruction analyses, the values presented in Table 1 were used to represent the slope face materials. Each section of the slope surface was idealized as consisting of either glacial till overburden, bare rock, or talus slope (covered with rock debris). The required parameters include surface roughness, tangential coefficient that accounts for frictional interaction between falling rock and the slope surface, and the normal coefficient which accounts for rebound of rocks bouncing on the slope surface.

Table 1 – CRSP Input Parameters.

<table>
<thead>
<tr>
<th>Slope Surface Geomaterial</th>
<th>Category in CRSP User Manual</th>
<th>Surface Roughness (ft)</th>
<th>Tangential Coefficient, R_t</th>
<th>Normal Coefficient, R_n</th>
</tr>
</thead>
<tbody>
<tr>
<td>Glacial till overburden</td>
<td>Firm soil slopes</td>
<td>0.20</td>
<td>0.70</td>
<td>0.16</td>
</tr>
<tr>
<td>Rock surfaces</td>
<td>Bedrock; hard surfaces</td>
<td>0.33</td>
<td>0.95</td>
<td>0.80</td>
</tr>
<tr>
<td>Talus</td>
<td>Talus</td>
<td>1.00</td>
<td>0.80</td>
<td>0.16</td>
</tr>
</tbody>
</table>

For each of the three profiles, CRSP was first used to analyze rockfall on the slope with no catchment system in place. The objective was to determine the locations where bounce height is minimized, which provides guidance and insight on where to locate the elevated cable net attenuator for optimum effectiveness. After several trial runs it was determined that elevation 1415 ft, which would be close to the west edge of the large intermediate bench, yields predicted bounce heights for all three profiles with maximum values less than 6 ft and average bounce heights of less than 1 ft. These results are consistent with our observations from rolling rocks, in which none of the rolled rocks passed the point corresponding to elevation 1415 ft. Note the rock rolling exercise was not a rigorously planned investigation with measurements of rock velocity, etc., but rather an informal exercise intended to provide general information.

Next, a 6-ft high barrier was placed at elevation 1415 ft and the CRSP analysis was performed again. In each case zero rocks (out of 100 rocks generated) passed the barrier (Table 2). Based on these results, a cable net attenuator suspended from 6-ft high posts on the bench at elevation 1415 ft was recommended for protecting workers in the vicinity of Pier 1. It was planned to install the attenuator after scaling the upper portion of the East Highwall for the protection of workers constructing the attenuator.
Table 2 – CRSP Results, 2-ft Diameter Spherical Rock and Input Parameters in Table 1.

<table>
<thead>
<tr>
<th>Slope Profile Analyzed for Rockfall by CRSP</th>
<th>Bounce Height at EL 1415 ft</th>
<th>Result of 6-ft High Attenuator @ EL 1415 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge Centerline, Including Abutment Excavation</td>
<td>0.77 3.07</td>
<td>No rocks passing attenuator</td>
</tr>
<tr>
<td>North Edge of Work Pad Excavation</td>
<td>0.69 5.09</td>
<td>No rocks passing attenuator</td>
</tr>
<tr>
<td>South Edge of Work Pad Excavation</td>
<td>0.69 3.55</td>
<td>No rocks passing attenuator</td>
</tr>
</tbody>
</table>

Upper Rockfall Protection

Prior to installing the attenuator system, partial excavation for the East Abutment by blasting and scaling of the East Highwall above the attenuator bench were performed in July and August, 2015. Blast rock directed down the face of the highwall filled the benches to capacity with rock debris and removed the majority of trees that had previously acted as a partial barrier (Figure 7). Rocks launched from the crest now traveled significantly further than observed in the preconstruction test rolls. Additionally, it was observed that scaling alone was likely not sufficient for reducing the rockfall risk to personnel cleaning the benches, constructing the attenuator, and excavating the Pier 1 work area.

In response to the altered conditions, draping the upper portion of the East Highwall was deemed the most safe and feasible means of providing the necessary rockfall protection. Recommendations for covering the upper portion of the slope with combined cable net and wire mesh drapery were provided to the contractor and the upper portion of the slope was covered down to the attenuator bench, as shown in Figure 8.

Pier 1 Excavation and Attenuator Construction

Pier 1 is founded on a bench excavated into the toe of the East Highwall by blasting and mechanical removal of blast rock in October and November, 2015. This work was performed in coordination with the attenuator construction. The attenuator posts and support cables were installed and drapery panels attached but not unrolled. Immediately following the Pier 1 excavation work, the attenuator drapery panels were deployed down the slope, extending to newly excavated Pier 1 work area, as shown in Figure 9. Berms were also constructed along the north, east, and west sides of the Pier 1 work area to create a catchment for rocks exiting from beneath the drapery.
Figure 7 – Site Conditions after Initial Round of Blasting and Scaling.

Figure 8 – Upper Rockfall Drapery.
Upper Rockfall Protection, Removal and Replacement

The drapery on the upper portion of the East Highwall was removed for final excavation of the East Abutment in January, 2016. The excavation work near the slope face was initially planned to be performed using only mechanical means or small controlled blasts; however, a least one blast resulted in significant rock debris being sent down the slope. Although the attenuator was not designed for such an event, it performed well, with relatively little damage. The post-blast conditions are shown in Figure 10. The following damage was observed during a site visit following the blast:

- Severed top cable wires
- Severed support cable wires and strands
- Completely severed lacing cables
- Dented posts with chipped paint
- Torn double twist mesh
- Rocks protruding through the drapery
Figure 10(a) – Attenuator Bench after East Abutment Excavation Blast.

Figure 10(b) – Damaged post support cable.
Figure 10(c) – Rock protruding through drapery.

Figure 10(d) – Impacted post.
During the blast, some rocks overtopped the attenuator, coming to rest in the Pier 1 work area. Prior to allowing construction operations to resume at Pier 1, the effectiveness of the attenuator was evaluated through a more rigorous program of trial rock rolling. Five rocks between approximately 6 and 18 inches in maximum dimension were rolled from multiple locations along the crest of the East Highwall. Additionally, some larger rocks were rolled using an excavator from two locations. Several trial rocks hit a small sloped bench above the attenuator bench that launched the rocks over the attenuator. This was primarily the case at the northern end of the attenuator, where the attenuator bench narrows and the attenuator fence is located closer to the upper highwall face.

Reinstalling the previously installed drapery panels with some additional panels to increase the coverage area to the north was the safest, fastest, most feasible means of providing the necessary rockfall protection to personnel working in the Pier 1 area. Recommendations for reinstalling drapery along the upper portion of the East Highwall were provided to the contractor and the upper portion of the slope was covered down to the attenuator bench. Recommendations regarding repair of the attenuator and lower drapery were also made.

ROCKFALL PROTECTION ELEMENTS

Combined Double Twist and Cable Net Drapery

Given the wide range of rock sizes observed at the site, combined wire mesh backed cable net drapery was selected on the basis of guidance by Muhunthan et al. \( \text{(6)} \). The wire mesh provides small opening sizes needed to contain small fragments of rock and prevent erosion while the cable net provides the strength and weight per unit area required to restrain rocks exceeding 2 ft in dimension. Figure 11 is a photo showing the drapery product used for this project. Note that the double-twisted wire mesh is both galvanized and PVC coated, while all other steel components and hardware, including the cable mesh, are galvanized for corrosion protection.

Attenuator Posts

The top cable for the attenuator is elevated by 6-ft high anchored posts on 20-ft center-to-center spacing. The anchored post spacing is based on analysis of debris loads and snow load as recommended in Muhunthan et al. \( \text{(6)} \). The posts are W8x48 sections welded to foundation base plates, which are bolted to footings consisting of 24-inch diameter, 8-ft deep holes backfilled with concrete. The posts are connected to the anchors by cables at the top and bottom. This member size and the cable connections comprise a detail based on experience with cable net attenuator systems that were field tested and subject to direct impact (to the post) from falling rocks as described by Arndt et al. \( \text{(7)} \). The posts have axial and flexural resistances substantially greater than the service loads transmitted by the drapery. The additional strength allows the posts to remain serviceable after sustaining a direct impact, which actually occurred at some of the posts when blast rock was sent down the highwall. The posts are painted for corrosion protection.
Anchors

Anchors consisting of ¾-inch diameter cable centered in a hole drilled perpendicular to the ground surface and backfilled with grout were used to support the attenuator posts and to hang the upper drapery. The anchors are spaced 20 ft center-to-center. Anchors installed entirely in rock have a minimum 3-inch diameter drill hole to a minimum depth of 6 ft. Anchors installed in soil or in a mixed profile of soil over rock have a minimum 5-inch diameter hole to minimum depth of 10 ft or 6 ft into rock, whichever is less. Anchor spacing is based on analysis of debris loads and snow load as recommended by Muhunthan et al. (6). The anchors are design for an ultimate pullout resistance of 24 kips and a factor of safety of 2.5. Proof tests were conducted on nine of approximately fifty productions anchors to a pullout load of 24 kips.

Quality Control

Careful inspection of the rockfall protection elements was performed after each significant installation. This included repelling and climbing along the seams between panels for close visual inspection, as shown in Figure 12. The inspection included checking for:
• Correct materials and corrosion protection
• Proper drapery alignment and coverage
• Correct size of cables and cable clips
• Correct number and orientation of cable clips
• Correct lacing of seams
• Correct cable tension

LONG-TERM ROCKFALL PROTECTION

The primary function of the drapery and attenuator is to provide a safe working environment during bridge construction. During construction, the drapery and attenuator will be maintained by the contractor. After construction, these elements will remain in place, but the owner does not plan on maintaining them. A 10-ft tall berm will be constructed around the base of Pier 1 to serve as long-term rockfall protection. CRSP analyses indicate that such a berm will provide adequate protection without any reliance on any of the other rockfall protection elements. The final configuration of the drapery and attenuator is shown in Figure 13.

Figure 12 –Drapery Inspection by DBA and MnDOT Personnel
SUMMARY AND CONCLUSIONS

A case history is described illustrating the successful application of rockfall mitigation technologies to provide worker safety under continually changing site conditions. Construction of the TH-53 Bridge across an iron ore open pit on the Mesabi Range in northern Minnesota required rockfall protection for workers involved in constructing a 190-ft high pier column at the base of one of the mine highwalls, while multiple stages of blasting were being conducted at the top of the highwall for abutment excavation. Coordinating the installation, removal, and reinstallation of rockfall protection elements with the construction sequence was critical to maintaining safe working conditions. CRSP modeling was found to be a useful evaluation tool; however, field observations and test rock rolling proved to be the best methods of performance evaluation. This case history also demonstrates the resiliency and robust nature of the rockfall protection system used at this site. The attenuator fence and drapery were subjected to extreme loading during an excavation blast. Some damage occurred, but the system remained intact and serviceable with relatively minor repairs.
REFERENCES


