STABILITY OF UNDERGROUND OPENINGS ADJACENT TO THE SINK HOLE AT THE NIOSH LAKE LYNN RESEARCH LABORATORY

D. R. Dolinar, T. E. Marshall, T. M. Barczak, T. P Mucho
NIOSH
Pittsburgh, PA

ABSTRACT

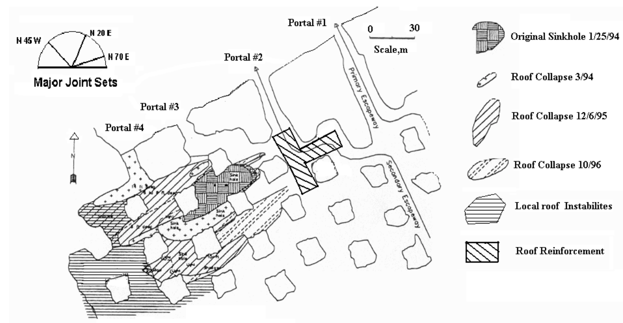
Over a portion of the older limestone mine workings at the NIOSH Lake Lynn Laboratory, a mining health and safety research facility, a large sinkhole formed caused in part by the intersection of several tightly spaced joint sets and unusual weather conditions. The anticipated propagation of the associated roof failures threatened to encroach upon a portion of the facility's secondary escapeway. To protect the escapeway, two cribs walls and over 80 cable bolts were installed. In addition, extensometers were installed to monitor roof movement in the escapeway. However, a large roof fall associated with the propagating roof failures in the sinkhole area partially destroyed one crib wall where the adjacent escapeway was not reinforced by cable bolts. Recent roof movement information indicates that this area is still active. Based on these measurements, steps have been taken to stabilize this portion of the escapeway. This paper discusses roof monitoring, the roof movement, the large roof fall, propagation of the roof failures, and the recent support measures undertaken to stabilize the escapeway.

BACKGROUND

The National Institute for Occupational Safety and Health (NIOSH) Lake Lynn Laboratory, a mining health and safety research facility is located along Chestnut ridge on the Pennsylvania-West Virginia border near Uniontown, PA. Stratigraphically, the facility is in the Greenbrier limestone of the Mauch Chunk Formation. This unit is composed primarily of a hard gray massive limestone that is interbedded with shales. The underground portion of the facility consists of the old mine works and the new research galleries.

The old mine works were part of an underground limestone mine developed in the mid 1960's and early 1970's from a highwall of a surface quarry escapeway (Mattes, et al., 1983 and Triebsch and Sapko, 1990). Approximately 2,300 m of entry was developed with rooms 15.2 m wide by 9.1 m high and 15.2 m wide pillars. In 1979, new galleries were developed from the older works to provide representative coal mine-sized openings for research. These galleries consist of about 2,300 m of 5.5 m wide by 2.0 m high entries. To provide access to the original underground quarry, four portals were driven into the highwall of the surface quarry. Portal 1 is used as the primary mine entrance while portal 2 functions as the secondary escapeway (figure 1).

In the late 1990's two hydrostatic chambers were constructed at the furthest extent of the old workings to allow for full scale testing of mine seals. In addition to providing access, ventilation and escapeways for the new research galleries, portions of the old mine are now being used for research studies. As underground limestone mining industry continues to expand, the importance of the old mine workings for conducting health and safety research related in large opening mines will also increase. Therefore the stability of the old workings is critical to facilitating this planned research.



On January 25, 1994 a large roof collapse occurred in the old workings near portals 3 and 4 and in close proximity to the highwall. The roof fall encompassed approximately 2,550 m^2 of entries and crosscuts. Over the collapsed area, the overburden thickness was approximately 30.5 m. Given the relatively shallow depth and the intersection of two significant joint sets observable through most of the roof rock units a sinkhole formed during the initial collapse. With time, the underground failure and sinkhole have expanded. Figure 1 shows the location of the original roof collapse and sinkhole as well the extension of the failure.

Several factors contributed to the roof collapse and the sinkhole formation. These factors have been discussed in previous reports but will also be summarized as follows below Zelanko, et al., 1996 and Iannachione et al., 1995). Two key elements were the joint systems and water. The dominant joint set is oriented at N 70° E nearly parallel with the highwall. Two less significant joint sets are oriented at N 20° E and N 45° W. All three sets of joints are near vertical. The three near vertical joint sets outlined large blocks of limestone in the roof of the collapsed area. Furthermore, surface and ground water had proculated through the joints and eroded the calcareous rock. These eroded joints were then in filled with low strength sandy clay, thereby, reducing the rock mass strength. Also, the shallow depth of the site and the proximity of the highwall resulted in low normal stresses across the joints (Iannachione et al., 2002).

Mine design in the form of excessive roof spans for the geologic conditions may also have been a factor. The rock mass rating using the Norwegian Geotechnical Institute Q system was 28.13 and using Bieniawski's Rock Mass Rating (RMR) was 67. These ratings indicate a good roof where the maximum span for a permanent opening based on the Q-system rating is 15.2 m. Although the entry spans were 15.2 m, intersection spans approached 22.9 m. The staggered pillar design employed also resulted in significant roof spans.

Finally, weather conditions may have triggered the roof collapse. Extremely cold weather resulted in a significant ice build up along the highwall and in the portal areas. A sudden warming trend then allowed snow to melt above the mine thus increasing the water level in the overburden. The highwall ice barriers could have restricted water flow from the overburden causing a build up of water pressure in the overburden and reducing the effective stress along the near vertical joint sets to a level that allowed the roof to collapse (Iannachione et al., 1995).

Although the roof collapse and sinkhole formed in the old workings, there was the potential that the failure could expand along the dominant joint set toward the nearby secondary escapeway. Expansion of the failure into the escapeway would seriously impact the operation of the facility. Therefore, a decision was made to reinforce and protect the secondary escapeway and to install instrumentation in the roof to monitor roof behavior. The design of the reinforcement was based on the structural and geologic conditions in the roof of the escapeway. The instrumentation would be used to determine whether the reinforcement maintained stability, and would ultimately identify the need for further action to protect the secondary escapeway.

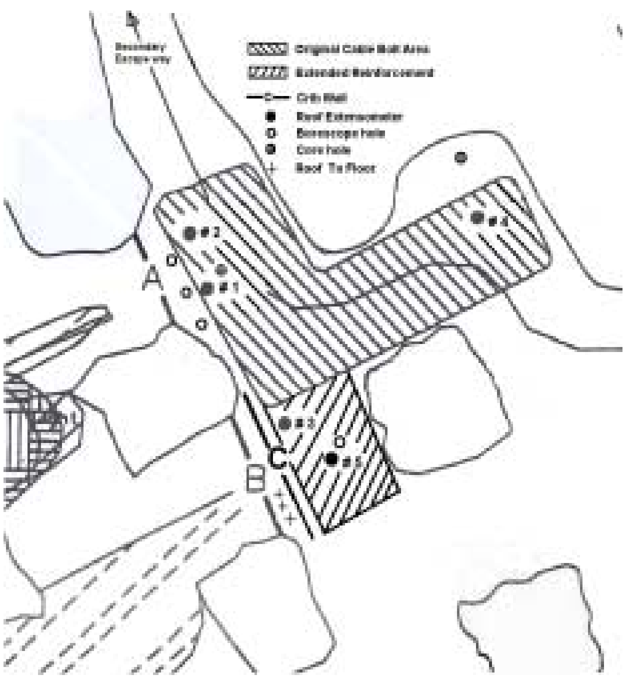


Figure 2. Location of roof reinforcement and instrumentation in the vicinity of the secondary escape way.

GEOLOGY AND STRUCTURE IN VICINITY OF SECONDARY ESCAPE WAY

To evaluate the geology and structure, two drill core holes (one core hole was angled 30 degrees from vertical) and several borescope holes have been drilled in and near the secondary escapeway (figure 2). A geologic column of the roof in the secondary escapeway based on the drill core obtained underground is shown in figure 3. The immediate roof consists of a competent gray limestone to a depth of 4.3 m. Above the limestone there are a series of red claystones and gray shales to a depth of 6.1 m. The claystones and shales are then overlain by 2.7 m of fined-grained limestone.

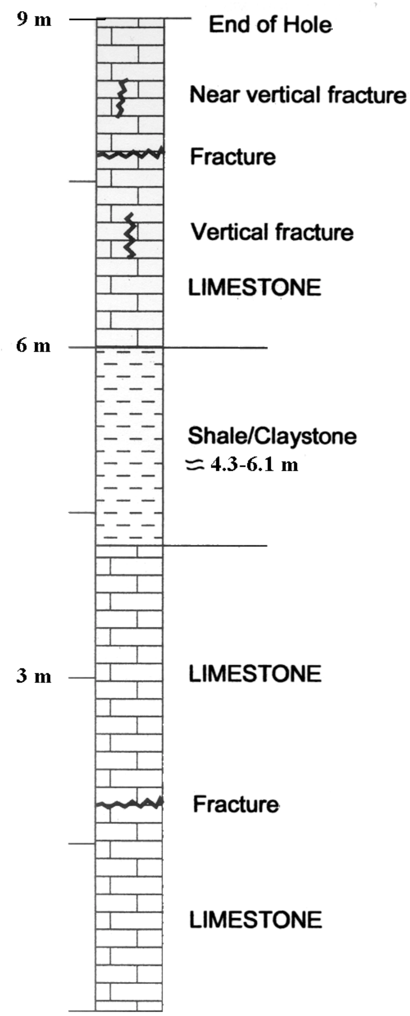


Figure 3. Roof geology observed in secondary escape way.

Several fractures and joints were observed in the core from both the lower and upper limestone sections. At a roof depth of between 0.9 and 1.2 m, sections of core were missing. This zone appears to correspond to a major separation along the bedding that was also noted in several observations holes in the secondary escapeway. In these holes the bedding separations and even open breaks in boreholes were observed at depths of 1.52 to 1.68 m in the mine roof (Molinda et al., 1996). In an adjacent area, the open fractures were noted at depths of 1.83 and 2.44 m in the roof.

The roof in the area of the secondary escapeway is heavily jointed with near vertical joints spaced between 0.3 to 1.83 m apart and orientated mainly N 70 E. These joints are nearly perpendicular to the orientation of the entries and parallel to the crosscuts and the highwall. However, some of the joints are inclined from vertical and intersect the adjacent near vertical joints, forming small wedges that are no more than 0.3 to 0.61 m wide that can fall out from the roof. Figure 4 shows these dominant roof joints crossing an entry adjacent to the secondary escapeway. In general, the joint density increases towards the portals and highwall. The vertical extent of most of these joints is not known but there appears to be several large joints or zones with several closely spaced joints with spacing between 6.1 to 9.1 m that may have a much greater vertical depth, possibly up through the shale. These large fractures and joints appear to be the main avenues for water seeping through the roof. The dominant joint and fracture system as well as the shale zone in the roof were taken into account in the design of the original reinforcement system installed in the escapeway

Figure 4. Dominant joint set (N 70 ˚E) that is nearly perpendicular to the entry and parallel to the high wall.

REINFORCEMENT OF SECONDARY ESCAPE WAY

Because of the potential for the failure to extend along the dominant joint set oriented at N 70 ˚E, the secondary escape way was reinforced in July of 1994 (figure 1). The design of the reinforcement systems developed to protect the secondary escape way focused on three elements: (1) isolating the secondary escape way from the collapse, (2) reinforcing the secondary escape way roof structure and (3) reducing the inflow of water through the roof. Three support remedies were then used that included crib walls, cable bolts and polyurethane grout (Zelanko et al., 1996).

Two concrete crib walls were built in crosscuts between the sink hole and secondary escape way and across the dominant joint set to act as breaker walls (figure 2). Each wall was about 15.2 m long, 9.1 m high, and 1.8 m wide, and composed of six four-point concrete cribs. The walls consisted of pre cast reinforced concrete sections that were stacked as four point cribs interlocked at specified positions. The walls were topped at the each of the crib corners with 1.83-meter-high timber crib packs. Grout bags were placed on the top of the timber cribs and pressurized to lock the crib walls in place resulting in about 4,270 kN of active force being applied to the roof.

To provide reinforcement along the dominant joint set, fully grouted tensioned cable bolts were also installed in the secondary escape way behind crib wall A (figure 2). The bolts were made from a 1.5 cm diameter, 7-strand cable with a minimum breaking strength of 258 kN. A total of 81 cable bolts were installed on a 3-m offset row spacing. To intersect the dominate near vertical joint set, the cables were installed at a 30-degree angle from vertical with half the cables angled inby and half outby from the portal in an alternating row pattern. A cable length of 9.1 m was used to allow anchorage into the upper limestone and the suspension of the lower limestone and shale layers from the upper limestone. Prior to grouting, the cables were pre-tensioned to 200 kN.

To reduce the permeability and increase the strength of the roof, polyurethane grout was pumped into the roof at five locations. These locations were the most highly fractured zones in the secondary escape way and were near crib wall A. A total of 2,470 kg of grout was injected.

INSTRUMENTATION

To monitor long term stability in the reinforced secondary escape, four multi-point wire extensometers were installed in the roof in mid 1995. Three of the extensometers (1, 2 and 4) were placed in the reinforced section of the secondary escapeway and one (3) in the unsupported entry adjacent to crib wall B (figure 2). A fifth extensometer would be installed during 2002, in the entry adjacent to crib wall B and inby extensometer 3. The roof anchors were placed at depths of 8.23, 7.01, 6.1, 3.05, 2.13, and 0.3 m. The 8.23-meter anchor was above the cable bolt depth and used as the reference point for analysis of roof movement. The extensometers were read with a data logger so a continuous record of roof movement was obtained.

1996 ROOF FAILURE

In October 1996, a roof fall occurred adjacent to the sinkhole near crib wall B that extended along the dominant joint set and encompassed an area from 6.1 to 15.2 m wide and 45.7 m long (figures 1 and 2). Most of the roof fall was only a few feet thick. However, one large block of rock estimated to be 4.3 to 5.6 m thick, 5.6 m wide and 9.1 m long weighing an estimated 4,500 kN fell from the roof and damaged the crib wall. The roof cavity where the large block fell extended to a depth of about 6.1 m possibly through the shale. Upon falling, the large block rotated upon striking the floor and a corner of the rock impacted the lower portion of the crib wall with a blow resulting in the collapse of one half or three crib sections of the wall (figure 5). However, the wall was not designed to take a dynamic side load from a 4,500 kN rock falling nearly 9.1 m. The impact was sufficiently violent to knock concrete crib blocks 15.2 m into the entry. Although the wall did partially fail, it did break the roof failure and prevented any extension of the roof fall into the entry beyond the crib wall.

Examination of the fall revealed that the block fell out between two near vertical joints orientated N 70 E that extended up through lower limestone to the shale. Further, the area was partially surrounded or isolated by collapsed roof. This partial isolation further reduced any confining forces along the joints that failed. Water was also present along the joints and roof in the area further weakening the shale above the immediate limestone roof and reducing the joint cohesion over time.

The roof instrumentation gave no clear indications that the large block of rock was about to fall as the wall was positioned between the failing roof and the extensometers. Also, none of the extensometers were located between the joints that outlined the large block. Approximately, 0.25 cm of movement had occurred at extensometer 3 located in the unsupported zone closest to the failure, however this movement had occurred nearly 8 to 12 months prior to the failure with most of this movement less than 2.1 m into the roof.

Figure 5. Crib wall B damaged by fall of large rock, October 1996.

LONG-TERM MONITORING OF INSTRUMENTATION

Of the four original roof extensometers, only extensometer 3 has shown any significant roof movement. Extensometer 3 is located in the unsupported entry near crib wall B and in the vicinity of a major joint from which water has continuously dripped.

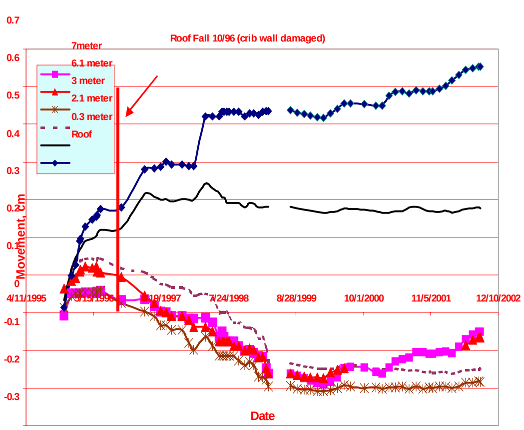


Figure 6. Roof movement measured at extensometer 3 between October 1995 and August 2002. Note: the gap in the data from March to July 1999 resulted from damage to the extensometer.

Figure 6 shows the roof movement at the various anchor horizons for extensometer 3. On the figure, the roof movement is for an anchor at a given roof depth with respect to the 8.1 m reference anchor. Between November 1995 and August 2002 over 0.63 cm of total movement of the roofline was detected with about 0.25 cm movement between 0.3 and 2.1 m movement and 0.25 cm between the roofline and 0.3 m. Through 1999, accelerated movements were initiated in November 1995, between late October 1996 and March 1997 and mid March 1998. There appears to be a seasonal pattern to the movement with the accelerated movement initiating in the fall or winter. Through March 2000, a fairly large amount of movement occurred but the movement was relatively shallow in the roof with most of the movement below 2.1 m. Since March 2000, there has been less than 0.13 cm of roof movement. However, most of this movement has been between 3.0 and 6.1 m into the roof, clearly much deeper than the previous movement.

Through mid 2000, the other extensometers 1, 2, and 4 have shown little or no movement. Roof movement, if any, has been less than 0.025 cm, which is near the detectable limits of the instrumentation. In general, the area that has been supported with cable bolts behind crib wall A has remained stable.

DECISION TO EXTEND THE REINFORCED ZONE

Beginning in March and April of 2000 a pattern of roof movement was observed at extensometer 3 that would lead to the decision to reinforce the entry adjacent to the damaged crib wall. Figure 7 shows the roof movement from July of 1999 to August of 2002. Again, the roof movement is for an anchor at a given roof depth with respect to the 8.1 m reference anchor. Although nearly an order of magnitude smaller, these roof movements were occurring at much greater depths than previously detected (figure 6). During this time period, only about 0.12 cm of movement was measured. However, most of this movement occurred between 3.0 and 6.1 m into the roof, the depth of the October 1996 roof collapse that partially destroyed the crib wall.

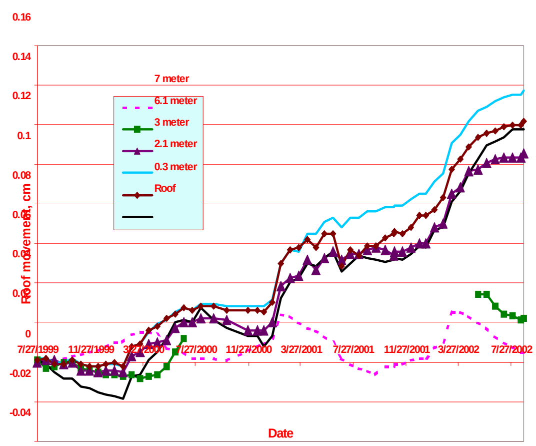


Figure 7. Roof movement measured at extensometer three between July 1999 and August 2002.

For three consecutive years, the roof movements were initiated in late January to mid-February. After each period of accelerated movement, there was a period with little or no movement. This movement pattern suggests that overall roof conditions are still stable. However, continued seasonal movement could result in a destabilization of the roof and eventually a large roof fall (Zavodini, 2000). Therefore, because of this pattern of deeper seasonal movement, a decision was made to develop and initiate a roof reinforcement plan to support the section of entry adjacent to the damaged crib wall.

ROOF SUPPORT DESIGN

Although the area in the entry behind the damaged crib wall is technically not part of the secondary escape- way, continued extension of the fall along the direction of the dominant joint set would extend the failure to the escape- way. Further, the entry provides direct access to the hydrostatic chamber area and the old mine works. Therefore, a three-phase reinforcement plan was devised to support the entry behind the damaged crib wall. This plan included the construction of a new crib wall, and the installation of both roof bolts and cable support. Figure 2 shows the extended support area adjacent to the secondary escapeway. Because the reinforced secondary escapeway had remained stable and that crib wall B actually broke the roof fall, the design of the reinforcement for the new area was based to a large extent on the previous successful design.

Another component of the plan was the installation of more instrumentation during 2002 to monitor roof behavior and add a further element of safety as work progressed in the entry that was to be supported. A fifth extensometer was placed in the center of the entry between the two joints along which the 1996 rock fall occurred (figure 2). Further, three roof to floor convergence monitors were installed just 1.52 m inby the new crib wall toward the roof fall in the crosscut (figure 2).

Crib Wall

A new crib wall (C) was built across the entrance of the cross cut just behind crib wall B (Figure 2). Figure 8 shows the completed crib wall that extends for a length of 21.3 m and covers two major joints, spaced 18.3 m apart. The crib wall was built on a steel reinforced concrete pad that was either 0.3 or 0.6 m thick depending of the floor elevation. A total of 9 interlocking 4-point cribs were erected with the height of the concrete crib portion of the wall being 6.71 to 7.01 m. The crib blocks were made from steel reinforced concrete were 0.61 m by 0.3 m and 1.83 m long.

Figure 8. New crib wall C covering the crosscut in front of damaged crib wall.

Sixty-centimeter diameter thin-walled steel containers that were 1.22 to 1.68 m high were placed on the corners of each four-point crib and pumped full of a 1,000 psi compressive strength grout. Prestressing cells were then placed on the filled containers. The cans are a stiffer system than the wood timber packs used on the original walls. Further, the cans will minimize any long-term creep and moisture affects that can occur with the wood cribs. These prestressing cells were 66 cm in diameter and consisted of two thin metal sheets that were welded around the perimeter to form a circular pancake cell. An inlet valve allowed the cells to be pressurized with a polyurethane resin. The grout-filled wall was then pressurized specially designed flat jacks placed on top of each can. Each flat jack was pressurized to between 0.34 and 0.69 MPa resulting in a total load of between 3,560 to 4,000 kN being applied to the roof and crib wall. Once the resin hardened, the cells provided a rigid support system.

During wall pressurization the data from the roof to floor convergence monitors and extensometers 3 and 5 were examined. When the wall was pressurized, no movement was detected on any of the instruments.

Roof Bolts and Mesh

The roof bolts and mesh system installed prior to the cable bolts was designed to provide support to the immediate roof and prevent the fall of surface material from the roof. An area of approximately 280-m square of entry adjacent to the new crib wall were supported with a 2.22 cm, grade 60, fully grouted torque tension bolts installed on a 1.52 m by 1.52 m pattern. A two-piece resin system was used with a fast set on top and a slow set resin on the bottom. This resulted in the upper 0.91 m providing anchorage for the lower tensioned portion of the bolt. The applied bolt torque was between 28 and 41 m-kg. Depending on the depth of the major bedding plane separation in the immediate roof, the length of the bolt varied from 2.44 to 3.05 m as the system was designed to be anchored above the main bedding fractures that ranged in depth from 1.52 to 2.44 m into the roof. In conjunction with the bolts, a 4-gauge welded wire mesh with a 10.2 by 10.2 cm opening size was installed to provide surface control and to minimize the risk of injury to workers from minor rock falls.

Cable support

Thirty fully grouted cables bolts were installed in the entry adjacent to the new crib wall. A Dywidag Z-Bulb cable bolt was used that consisted of a no. 6 (1.52 cm diameter), 7-wire strand, ASTM Grade 270 cable with a minimum breaking strength 260 kN. To minimize corrosion, the cable bolts were galvanized. This cable bolt was different than that used in the original installation where a thread bar was used to tension the cable because the manufacturer no longer makes the older thread bar type of cable bolt. With the new system, a barrel and wedge anchor head were used to tension the cable and to secure the bearing plate.

The cable bolts were 9.1 m long and installed on a 3-m row spacing with alternate rows offset by 1.52 m. This pattern resulted in 5 rows of cables across the entry with 6 cables per row. Approximately 18.3 m of entry adjacent to the new crib wall was reinforced with cables. The cable bolts were angled at 30˚ from vertical with alternating rows of the cables angled inby and outby from the portal. Again the system is designed to reinforce the dominant joint set with at least 2 cables bolts intersecting each joint. Essentially, the design of the cable bolt pattern is similar to the pattern used in the original cable bolt installation in the secondary escapeway.

The cables were installed in a 4.13 cm hole. However, the initial anchor was developed from a resin grout cartridge placed in a 3.49 cm hole along the upper 2.4 m of the bolt. Three Z-bulbs anchors created along the upper 1.5 m of the cable were used to increase the anchorage capacity in section where the resin cartridge developed the initial pre-tensioned anchor. A 1.2 m long steel tube was placed along the lower portion to stiffen the cable during insertion of the cable through the grout cartridge. The resin anchor was allowed to set over night before the cable was tensioned to between 89 to 98 kN using a cable-tensioning unit. The cables were tensioned against 20.3 by 20.3 cm, high capacity (grade 5), domed bearing plates. Two small holes were drilled in the dome of the bearing plates to allow for the breather and grout tubes to be installed for the grouting operation. Once the cable had been tensioned, the remainder of the hole and cable were fully grouted using a pumpable cement grout.

DISCUSSION

A large roof fall did occur in the fall of 1996 that destroyed a portion of crib wall B. The entry adjacent to the crib wall was then reinforced about six years later. However, during this time, the data from the instrumentation installed in the roof was continuously evaluated to determine if any changes occurred in the roof behavior.

Prior to 2000, although there was a fairly large amount of movement at extensometer 3, this movement was relatively shallow, below 2.1 m. The movement would occur as a sudden accelerated movement followed by a period of deceleration and finally by a period of little movement. Extensometer 3 was also near a major joint and apparently was detecting the loosening of the lower roof around the joint. Further, this movement occurred seasonally usually in the late fall or early winter. Because of the pattern and timing of roof movement, the cause of the movement appears to be external. The external stimuli for this movement probably is related to the seasonal conditions and changes that occur in the late fall or early winter.

Beginning in 2000, the pattern of roof movement at extensometer 3 changed. Although the movement was an order of magnitude less, the movement was much deeper into the roof and appears to involve the roof up through the shale-claystone at a depth between 4.27 to 6.1 m. Proximity of extensometer 3 to the major unsupported joint is also of concern where the deeper movement could indicate that the joint was now becoming active. Though instability near a major fracture may reflect just the local instability in the vicinity of the joint, continued movement near such joints could destabilize larger blocks of rock leading to the extension of the roof fall into the entry.

This latest roof movement pattern has also been seasonal with the initial accelerated movement occurring from late January to mid February for three consecutive years, from 2000 to 2002. The time frame is similar to when the main collapse occurred that caused the sinkhole formation. The periods of accelerated movement are again probably the result of external stimuli that is related to the winter season. However, it is not known whether this movement is related to ice and the change in the effective stress due to increased water levels such as may have occurred in the original failure. With the formation of the sink hole, the area should be well drained since there was no ice builds up near the collapsed wall or extensometer 3. However, this period does correspond to the coldest time of the year and to the coldest time in the mine. January/February temperatures near this location underground average between 7.2 to 10 ^{o}C while summer temperatures average about 18.3 ^{o}C. Although the change in temperature within the area underground is seasonally only 8.3 to 11 ^{o}C, this area appears to have little confining stress across the joints. Therefore, any contraction of the rock mass due to lower temperatures would further reduce any small confining stress across the joints (Hooker and Duvall 1971). For the present, the roof movement pattern suggests that the roof conditions are still stable. After each period of accelerated movement, the movement decelerates and there is a period with little or no movement. Essentially, the forces that trigger the movement are of limited duration and once reduced, allow the roof to again reach equilibrium.

Whatever the cause though, continued seasonal application of the external stimuli could ultimately lead to the destabilization of the main roof and eventually, a large roof fall similar to the one that damaged the crib wall. Further, to eliminate or reduce the affects of the external stimuli that are causing the roof movement would be difficult, whether it is water and ice or temperature. Therefore, because of this pattern of deeper movement, a decision was made to develop and initiate a roof reinforcement plan to support the section of entry behind the damaged crib wall.

In designing the reinforcement system for the entry adjacent to the secondary escapeway, a 3-component system was developed where each component in the reinforcement plan contributes to maintaining the stability and function of the entry. The new crib wall acts as a breaker to prevent the extension of the failure along the dominant joint set into the entry and toward the escapeway. The roof bolts and wire mesh provide support to the immediate roof in between the cable bolts and also surface control. Finally, the cable bolts provide reinforcement along the dominant joint set behind the wall. In addition, the continued monitoring of the instrumentation will provide a basis for evaluating the reinforcement effectiveness and long-term roof stability.

During installation and pressurization of the new crib wall, roof to floor convergence meters and extensometers 3 and 5 detected no roof movement. This lack of movement indicates that there were no major separations that were closed near the wall during pressurization. Further, locally the application of a 3,560 kN force can be significant, but the effects dissipate rapidly with distance. At extensometer 3, located approximately 3 m from the wall, where significant lower roof movement had occurred, there was no roof movement from wall pressurization.

The data from the instrumentation in the reinforced escapeway shows that little or no roof movement has occurred and therefore the area has remained stable since the summer of 1994 when the escapeway was reinforced. This provides strong evidence of the success of the design of the reinforcement system. Even though crib wall B failed, the wall did act as a breaker to the fall where the wall performed it's main function, to prevent the extension of the fall into the entry. Therefore, the design of the new reinforcement system for the entry could be based to a large extent on the previous design used in the secondary escapeway.

CONCLUSIONS

The original reinforcement system consisting of cable bolts and crib walls have maintained the stability of the secondary escapeway with little or no movement being detected despite the proximity of the sinkhole and major joint structures in the roof that extend from the failure. Further, even though a crib wall was partially destroyed by a large rock fall, the crib wall did break the fall and prevent the fall from extending into the entry. Therefore, the original reinforcement design and system can be considered successful.

Ultimately, based on roof movement detected by the roof instrumentation, a decision was made to extend the reinforcement into the entry behind the damaged crib wall. If the entry was left unprotected, the failure could extend across the entry and up to the secondary escapeway. Therefore, the instrumentation installed to monitor roof movement was a valuable tool on which decisions concerning the stability of the entry and the need for further reinforcement were made. Although there was a fairly large amount of movement in the lower roof, it was the much smaller, but deeper, roof movement that triggered the decision to install more reinforcement.

Because of the success of the original reinforcement, the design of reinforcement in the entry was based to a large degree on that original system. Since the performance of the original reinforcement was successful, the new system should provide adequate protection to the entry and secondary escapeway. Further, continued monitoring of the roof instrumentation will provide a measure of the performance of the reinforcement and an evaluation of the stability of the area adjacent to the sinkhole.

REFERENCES

Delanco, J.C., Mucho, T.P., Iannacchione, A.T. and Barczak, T.M., 1996, “Stabilization of a Large Shallow, Potentially Unstable Underground Opening,” Proceedings of the 2nd North American Rock Mechanics Symposium: NARMS’96, Montreal, Quebec, Canada, June 19-21, pp. 155-161.

Hooker V. and Duvall, W., 1971. “In Situ Rock Temperature Stress Investigations in Rock Quarries,” USBM RI 7589, 11 pp.

Iannacchione, A.T., Dolinar, D.R., and Mucho, T.P., 2002, “High Stress Mining Under Shallow Overburden in Underground U.S. Stone Mines,” Proceedings of the First International Seminar on Deep and High Stress Mining, Nov. 6-8, Perth, Australia.

Iannacchione, A.T., Mucho, T.P. and Prosser, L.J. 1995, “Ground Control Problems in the Underground Limestone Industry,” Proceedings of the Environment, Safety and Health Forum (National Stone Association), Nashville, Tennessee, October 22-24, 24 pp.

Mattes, R., Bacho, A. and Wade, L., 1983, “Lake Lynn Laboratory: Construction, Physical Description, and Capability,” USBM IC 8911.

Molinda, G.M., Monaghan, W., Mowery, G. and Persetic, G.F., 1996, “Using Ground Penetrating Radar for Roof Hazard Detection in Underground Mines,” NIOSH RI 9625, pp. 14.

Triebsch, G. and Sapko, M., 1990, “Lake Lynn Laboratory: A State of the Art Mining Research Laboratory,” Proceedings International Symposium on Unique Underground Structures, Denver, CO, June 12-15, pp. 75-1 to 75-21.

Zavodini, Z, 2000, “Time-Dependent Movements of Open-Pit Slopes. Slope Stability in Surface Mining,” W.A. Hustrulid ed., SME, Littleton, CO, pp. 81-87.