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→ Cassiar

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Support at Cassiar Underground Mine

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Since 1952 and until the summer of 1990, a large orebody of chrysotile asbestos was exploited in the Cassiar open-pit mine. In March 1988, a decision was made to start development of the underground mine. A block-caving mining method was designed to extract the ore from the serpentinite host rock. A system of access and ventilation drifts has been developed, mostly in argillitic rocks, and production drifts have been laid out in the footwall of the orebody in serpentinite.

A lack of detailed local geological information resulted in the location of the extraction drifts on 1350 level, the first production level, within the incompetent, sheared serpentinite. The ground support on 1350 North level has not behaved as expected and, combined with the rather poor standards of ground-support installation, contributed to the many support failures that occurred in the early stages of development. The paper describes the measures taken to improve the support.

Based on the experience gained from the 1350 North level, the support system for the 1350 South level was revised and the quality of installation significantly improved. Together with better ground conditions, the result has been that only minor damage on 1350 South has been observed.

Introduction

Cassiar is located in northern British Columbia, approximately 100 km south of the Yukon border (Figure 1). The town, with a population of 1200, and the plant nestle in a glacially formed valley at an elevation of 1070 m, and are surrounded by rugged, mountainous terrain with peaks over 2000 m. Access to the community is via 15 km of paved road connecting Cassiar with Highway 37.

The Cassiar Mine provides the sole economic base for Cassiar, and the infrastructure and services, such as a hospital and a government office, for the surrounding region.

The average winter temperature in Cassiar is between minus 10 and minus 20°C. The winter season extends from October to May and, during that time, there is an overall snowfall of approximately 390 cm. During the summer months, the temperatures reach 25°C. The average annual rainfall is 31 cm.

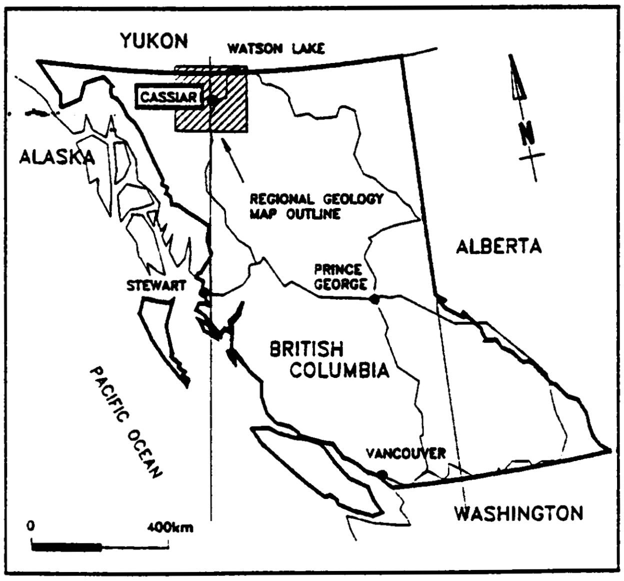


Figure 1. Location map of Cassiar

Cassiar is the only operating asbestos mine in British Columbia. The first ore was mined in the fall of 1952, and from then the Cassiar open pit was in more or less continuous production until the spring of 1990. The total ore extracted from the open pit is estimated at 20 Mt of first-quality fibre. In 1978, an exploration adit intersected another chrysotile-bearing serpentinite orebody, after which exploration drilling took place. The drilling results confirmed a large orebody containing 16 Mt of recoverable reserves, the deposit being open to the south and east at depth. Underground production started in the fall of 1990, and up to February 1992 more than 1110 kt of ore had been extracted.

Geology and Geomechanics

The regional geology of the Cassiar area was recently described in detail by Nelson[1], who concluded that the Cassiar area contains a thrust-repeated miogeoclinal succession structurally overlain by the Sylvester allochton (Figure 2).

The asbestos orebodies are contained in a fault-bounded slice of serpentinized ultramafite within the Sylvester allochton. The Sylvester Group is an ophiolitic assemblage of chert, and volcanic, clastic, and ultramafic rocks. Figure 3 shows a typical geological section through the orebody.

The footwall sheet of the McDame serpentinite, mostly chert and argillite, lies over the basal Sylvester thrust fault. Burgoyne[2] describes its basal part as highly deformed and sheared, marked by carbonaceous or graphitic argillite.

The McDame serpentinite, which hosts the orebody, dips easterly under McDame mountain, and is bounded on its hangingwall and footwall by thrust faults. The serpentinite consists of an outer mantle of older, darker serpentinite,

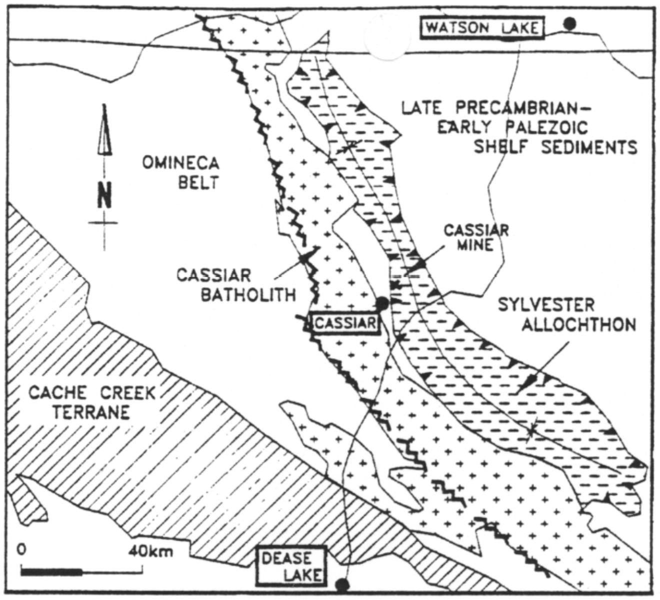


Figure 2. Regional geology of the Cassiar area

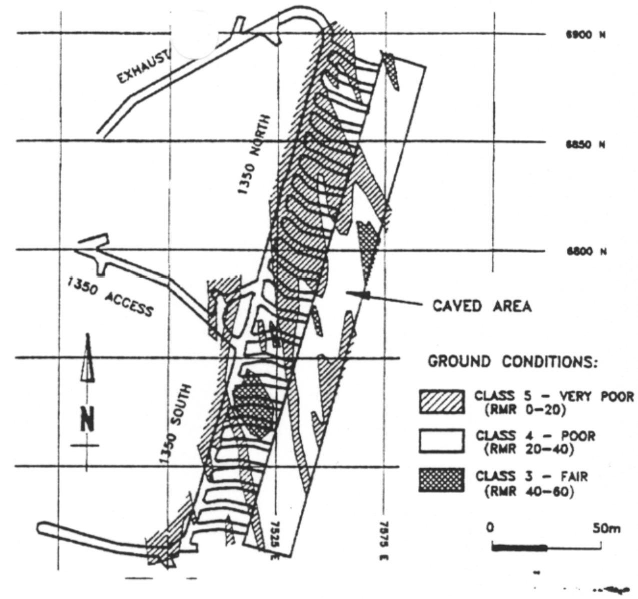


Figure 4. Ground conditions on 1350 level

with an overprinting green serpentinization of the core. This core contains most of the economic-grade asbestos. Dark, barren serpentinite of the outer mantle would lie in the class 3A–3B of the geomechanics classification[3]. The green serpentinite of the core is highly deformable and sheared, and it would lie in class 5A–4A. The orebody is 50 to 220 m thick and is intersected by numerous shear zones. The shear zones are 0,1 to 5,0 m wide and consist of light green, soft, lenticular shear fragments of serpentinite, magnesite, and talc. The presence of talc in the shear zones contributes greatly to the low friction surfaces of most of the discontinuities. The geomechanics ratings of such shear zones are usually 5B–5A. Figure 4 shows the generalized ground conditions on 1350 level.

The hangingwall consists of competent, highly fractured argillite and volcanic rocks, which are separated from the serpentinite by an alteration zone.

Estimates of the strength properties of the different intact rocks are summarized in Table I. The unconfined compressive strengths of specimens taken on 1350 level range from 30 to 50 MPa but, owing to the number of fractures and fibre seams, as well as the low friction surfaces on the joints, the rockmass strength is estimated as from 1 to 5 MPa.

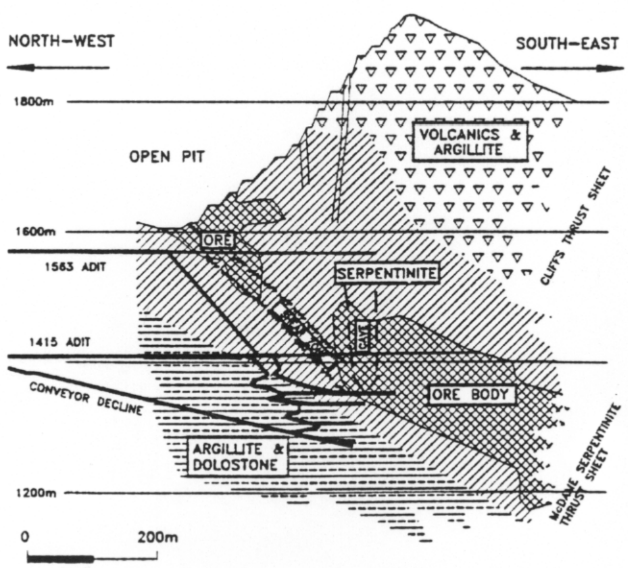


Figure 3. Generalized geological section through the orebody

Table I
Summary of rock strength

Rock type	Uniaxial compressive strength, MPa
Hangingwall argillites	~100
Serpentinite on 1350 level	30–50
Footwall argillitic chert	~150
Shear zones	<8

Mining

The production area is accessed by a system of adits, conveyor declines, and main access ramps developed in the footwall argillites (Figure 5). The deposit is mined by a modified footwall drawpoint method, with sublevels at 15 m vertical intervals and 20 m horizontal intervals. The sublevel development consists of a footwall extraction drift developed on strike, approximately 25 m from the footwall–orebody contact (Figure 6). Drawpoint crosscuts are taken off at a spacing of 10 m, and extend to the contact, where the drawpoint brow is established and heavily supported. The crosscut is then extended a further 20 m to provide for the undercut drilling and blasting required to initiate the cave (Figure 7). The drawpoint spacing of 10 m ensures the interaction of drawpoints along strike, resulting in an active draw trough with a maximum width of 15 m. The drawpoint positions are offset on alternate sublevels to maximize recovery, although the horizontal interval of 20 m is designed to provide an ore 'barrier' between successive levels that will protect the active draw trough from up-dip dilution. Some of the barrier ore is then recovered during a reclamation stage by retreat of the drawpoint to a 'secondary' drawpoint brow.

Caving is initiated by drilling, loading, and blasting in the undercut crosscuts, with pulling of the undercut swell between rings until the main, primary, drawpoint brow is reached, at which time continuous draw can begin. The rate at which draw is built up and the sequence of commissioning are carefully controlled to ensure that a

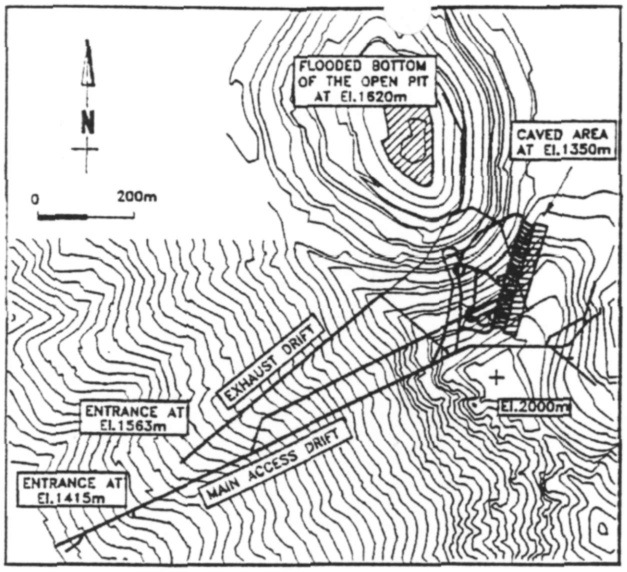


Figure 5. The layout of the mine

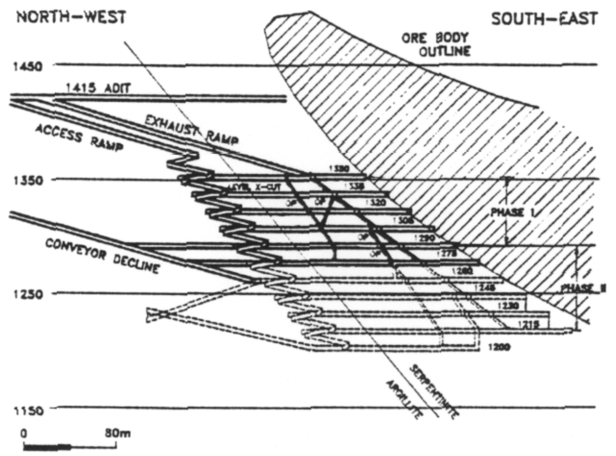


Figure 6. Schematic mine section

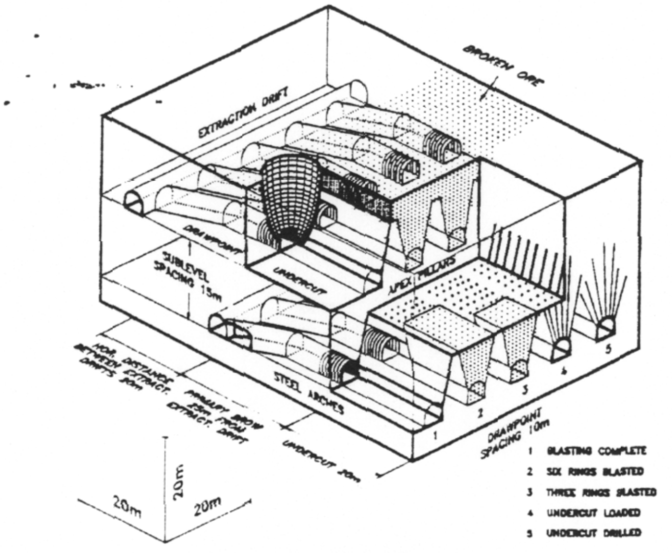


Figure 7. Layout of the footwall drawpoints

continuous cave is developed, and that good interaction between drawpoints is achieved before the design daily draw of 300 t per drawpoint is scheduled. The mine is designed to operate at 4 to 5 kt per day, providing 1 to 1.6 Mt of concentrator feed per year, and a depletion rate of 12 to 16 months per level. Ore is transferred to the centrally located orepass by 2,7 m^3 load–haul–dump (LHD) units, and is hauled to the portal of 1415 level by conveyors for transfer to the tramline delivery system.

Ground Support on 1350 Level

Support System for 1350 North

The design of the ground support was based on information gathered from the exploration development programme, as well as from block-cave operations in Quebec and Zimbabwe. The support philosophy was to reinforce the rockmass while allowing it to yield. The initial project highlighted the fact that the integrity of the openings can be maintained only if the lining has yielding characteristics and is tied into the rockmass reinforcement.

Since the start of development, the mine has experienced serious problems owing to the development contractor's low standard of support installation. Even after Cassiar crews had taken over, the mine staff did not realize that the work standards required on mines with poor ground conditions must be much higher than those accepted on mines with good ground. Production demands meant that certain recommended support procedures, such as floor support, were not implemented.

The development and the support system described below was adopted for the extraction drift and drawpoints on 1350 North production level (Figures 8 and 9).

(1) The length of advance was limited to between 1,5 and 2,4 m. In very poor ground, the round was spiled with fully grouted 22 mm Dywidags, 2,4 m long at +10 degrees and 0,35 m spacing, from the junction of the face to the back. A baby-arch blasting technique was used, with perimeter holes drilled at 0,46 m spacing.

(2) After blasting, the round was mucked out with attention to the ground condition. If the excavation was located in a very poor shear zone and sloughing of the walls occurred, the mucking operation was stopped, and a first layer of shotcrete was applied to stabilize the ground.

(3) The first 50 mm of shotcrete was applied after mucking and washing of the walls. No bolting was carried out before the shotcreting.

(4) After shotcreting, 22 mm, 2,4 m long, fully grouted Dywidag bolts were installed at a spacing of 1,0 m in all directions. Resin cartridges were used for the grouting of the Dywidag bolts on the extraction drift. The resin was replaced with a Portland cement grout, utilizing a 25 mm grouting tube placed at the toe of the hole. While the cement grout was setting, a new round was blasted.

(5) Welded-wire, 9-gauge steel mesh with 100 by 100 mm openings was installed over the entire shotcreted area, from floor to floor. The mesh was tightened to the shotcrete with a 150 by 150 by 9.5 mm plate and nuts, overlapping with the previous round.

(6) Finally, three rounds behind the face, a second layer of 50 mm shotcrete was applied.

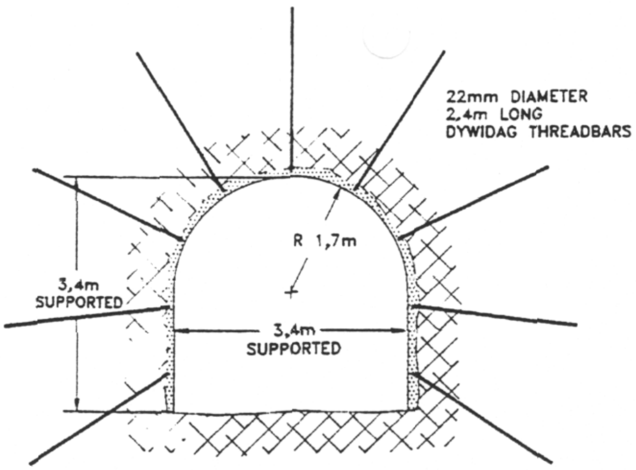


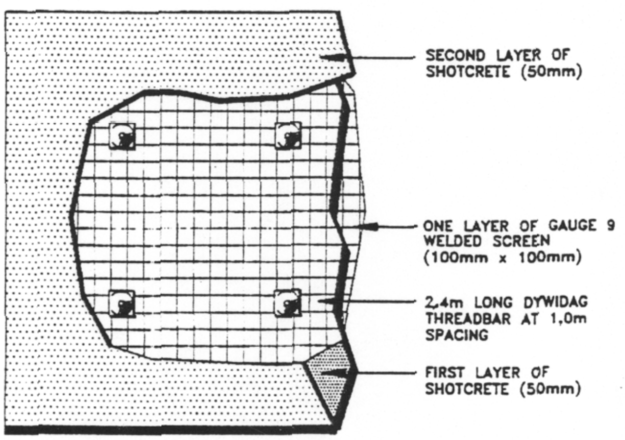
Figure 8. Layout of the rock support on 1350 North

Additional Support for 1350 North

The 65-degree corners at the drawpoint turnouts were additionally supported to cater for the increased span. Four to five fibrecore ropes (25 mm in diameter) were placed horizontally on the pillar, two on one side and three on the other. Both ends were grouted 3,0 m towards the centre of the pillar.

In addition to the standard support, the drawpoint brows were reinforced with steel arches and additional cables. Nine sets of rigid arches were installed at a spacing of 0,45 m. The arches were tied together with 25 mm bars and with top plates. The whole structure was tightened to the sidewalls with a system of 6 m long cable bolts penetrating the pillar and completely covered with shotcrete. The cables on the back were 8,0 m long and were angled back over the drawpoint.

Systematic measurement of the deformation was introduced. The rock-mechanics technician recorded the drift support failures and closely inspected any cracks on a weekly basis. Wire extensometer points were installed in every drawpoint to monitor drift closure. The locations of the extensometer anchors were chosen to represent different ground conditions. The objective was to determine the



NOTES:

5,0m LONG CABLE BOLTS CONNECTED WITH STRAPS WERE INSTALLED APPROX. 6–8 WEEKS AFTER SUPPORT COMPLETION

AMOUNT OF STEEL IS APPROX. 1,5 kg PER SQ.METRE

Figure 9. Detail of the original support

performance of the installed support system and to fully understand the reaction of the rockmass to the development and to the different stages of cave development.

Performance of the Support on 1350 North

Since 1350 North was the first area scheduled for production, it was logical to observe the performance and failure of the ground support in this area.

Shotcrete

The effectiveness of shotcrete is governed by the shooting of a minimum thickness of 5 to 10 per cent of the drift radius on the whole rock surface, and by the bond with the wall rock[4]. The minimum required thickness for 1350 level was 100 mm. All the shotcrete, of the dry type, was applied pneumatically. The composition of the shotcrete is given in Table II. No reinforcement fibres were used in the serpentinite support because these cannot be separated in the mill and thus may contaminate the product. Core and drill tests clearly indicated that, in the initial stage of development, the shotcrete skin did not meet the required minimum thickness. Debonding was another problem that was observed shortly after the shotcrete had been applied. The second layer of shotcrete was applied three rounds behind the face. Fibre, dust, and diesel fumes from equipment coated the first layer. It was very difficult to clean this surface before the second layer was shot. On many occasions, core tests found that shotcrete had been placed over rebound material and/or over muck that had not been removed from the wall or floor corner.

Although uniaxial compressive strength testing showed that most of the samples met the required strength (30 MPa), improper application resulted in many weak areas where failures were initiated.

Rockbolts

In the initial stage of ground support, only one type of rockbolt was employed: a threaded Dywidag bar, 22 mm in diameter and 2,4 m long. The yield strength of this rebar is 14 t, and the breaking strength 18 t. Originally, resin cartridges had been used for grouting. Resin with a setting time of 90 seconds was used, and developed at least 20 per cent of the ultimate strength within approximately 40 minutes. Although mine staff demonstrated the successful installation of the resin-grouted bolt in very poor ground on several occasions, the low standard of the development contractor's work was indicated by the pull-test failure of a great number of bolts.

As described by Martin[5], the problem lay specifically in

- overdrilling of bolt holes, resulting in oversize holes in weak ground so that the normal number of resin cartridges did not provide sufficient resin
- insufficient resin cartridges placed in a hole
- threadbars spun for an incorrect length of time (i.e. either too short to enable proper mixing of the resin, or too long with the result that resin was forced out of the hole).

Table II
Composition of the shotcrete

Ingredient	Description
Silica fume	12%
Accelerator	3%—used to reduce setting time
Type 10 cement	Used in temperature above −10°C
Type 30 cement	Used in temperature below −10°C

The problems described resulted [ ]ot only insufficient anchoring strength for the bolts but also a lack of grout at the collar.

The second major problem in the installation of bolts was the failure to tighten nuts after the grout had set up.

After repeated problems with the poor standard of resin-cartridge grouting techniques, a Portland cement grout with a water-to-cement ratio of approximately 1:3 was successfully introduced.

A third problem was the lack of bolt installation near the bottom of the wall. The support procedures specified a bolt spacing of only 1 m. When the last bolt had been placed 0.9 m above the floor, for example, no other bolt was installed to support the bottom of the wall.

Screen

Welded-wire 9-gauge steel mesh with 100 by 100 mm openings was placed over the entire shotcreted area after the Dywidag bolts had been installed. Experience showed that welded-wire mesh did not perform well in this type of environment in that, shortly after tensile cracks had developed in the shotcrete, the mesh failed. Owing to the uneven surface, the welded-wire mesh did not form well to the back or sidewalls of the openings. Unfortunately, the proper overlap of mesh at the bolts had been underestimated, as well as the proper installation at the bottom of the wall.

Corner Ropes

Corner ropes were installed to reinforce the critical part of the pillars. Because the ropes had not been properly tightened to the wall, they applied minimal restraint during the initial pillar relaxation. Shortly after installation, the shotcrete develope[ ] [ ]ge vertical cracks as a result of initial pillar deformation. Exposed ropes were showing signs of high tension. The corners in which four ropes had been installed retained the pillar well, but the corners with two ropes suffered extensive damage and failure of some of the ropes. As a result of such failures, all the corners were re-supported with four ropes.

Steel Arches

The rigid steel arches themselves have behaved fairly well. It was surprising that the arches functioned well and applied support pressure even after severe deformation (Figure 10). Part of the success was the interlocking of the arch structure with the top plates and connection bars, which prevented disintegration of the arch unit. The weakest point was the anchoring system. Arches were installed on the top of the concrete floor, so that no concrete was placed after the arches had been erected, which could block the legs from moving into the drift. The bottom channels were not bolted to the floor, and the cable grips, on which the whole anchoring system relied, failed. The combination of poor excavation and excessive pillar deformation often necessitated the slashing of the drift in order to install arches. Once considerable deformation and distortion of the arches had occurred, the connection bars pulled through the arch beam.

Several weak points were observed in the yielding sets that were tested in one drawpoint in Class 5A ground. Firstly, the legs and crowns had been joined with a jointing system consisting of a U-bolt and a flat clamp. This connection failed in the first three sets, where the yields exceeded 75 to 100 mm. Because of the shotcrete, which encases the whole structure, there was no access for the

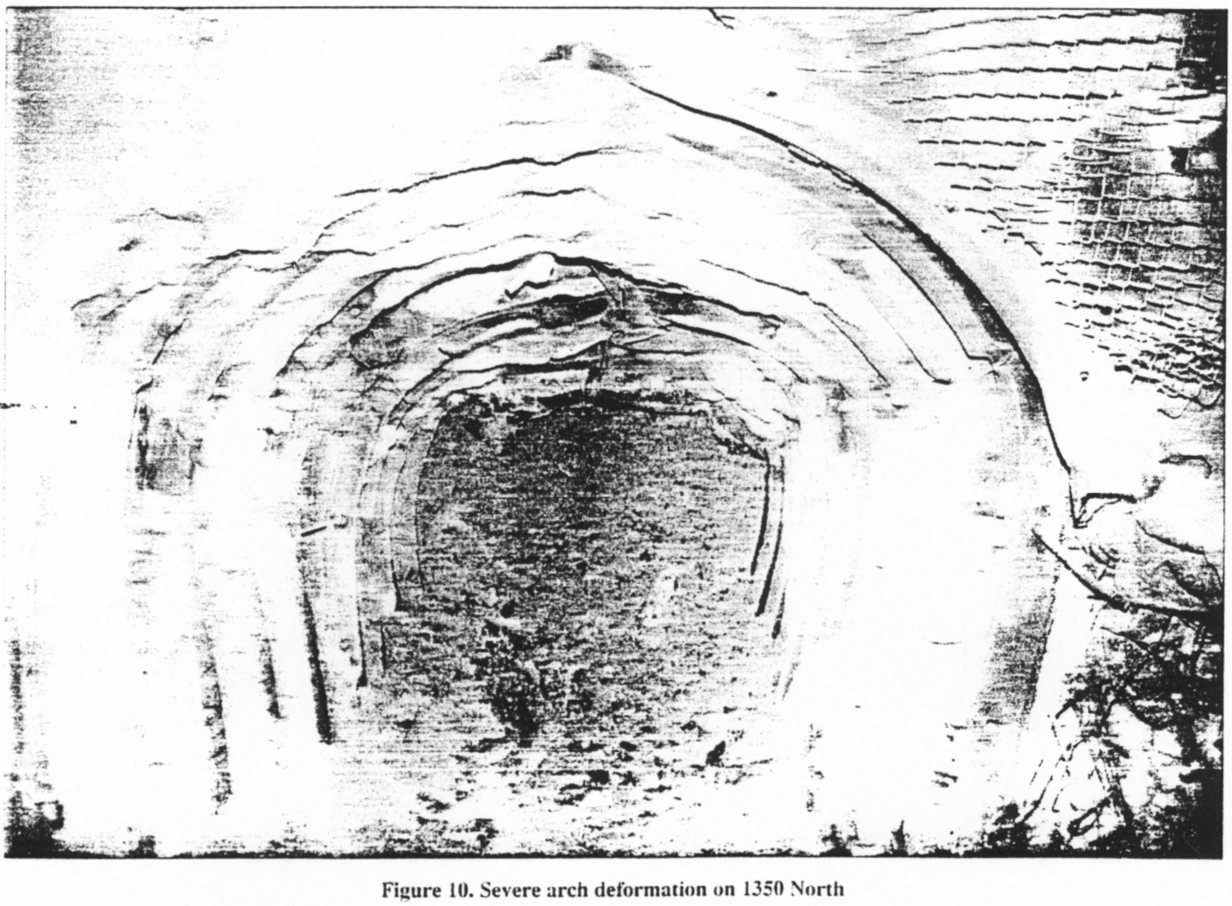


Figure 10. Severe arch deformation on 1350 North

resetting of the connecting bolts. Secondly, the spacer channels that linked the arches had been attached to them by J-bolts. Under severe distortion, the J-bolts failed. A third problem was posed by the anchoring system, which failed as described earlier.

Repair Techniques Used on 1350 North

Although the ground support was designed for the 11 to 15-month lifetime of the level, post-support failures of the drift were observed shortly after the excavation of the drawpoint began, and repairs had to be initiated in the early stages of development. Vertical cracks in the shotcrete appeared as a first sign of deformation. They were usually initiated at the bottom of the drift and progressed upwards. Once the cracks had reached 1 to 2 mm in width, the welded-wire mesh failed. Debonding of the shotcrete layers occurred, often close to the floor. Monitoring of the drift closure showed a rate of horizontal closure of approximately 5 to 10 mm per week.

Since no damage was observed on the back, repairs were concentrated on the sidewalls. The critical situation of pillar failure was recognized, and large-scale repairs and installation improvements were implemented immediately. All the loose shotcrete was removed, and the exposed rock was re-shotcreted. The bottom area of the walls was reinforced with additional Dywidags connected horizontally with 0,3 by 2,4 m welded straps. A system of 16 mm fully grouted cable bolts was installed through the pillars (Figure 11). There were 3 cables in the ring, and the rings were at a spacing of 2 m. Surface restraint was provided by vertical and horizontal welded straps overlapped at the bolts and tightened with 150 by 150 by 15 mm plates and cable grips (Figure 12). The whole steel structure was encased in shotcrete: firstly, to provide good contact between the straps and wall and, secondly, to protect the support from LHD damage.

Monitoring of the drift closure was re-established, and the horizontal movement decreased from between 5 and 10 mm per week prior to the installation of cable bolts to almost 0 mm after repairs (Figures 13 to 15). Development of the level continued, and the undercut was excavated and supported.

The first undercut was blasted in November 1990 in drawpoint no. 5. Shortly after the undercut blasting had

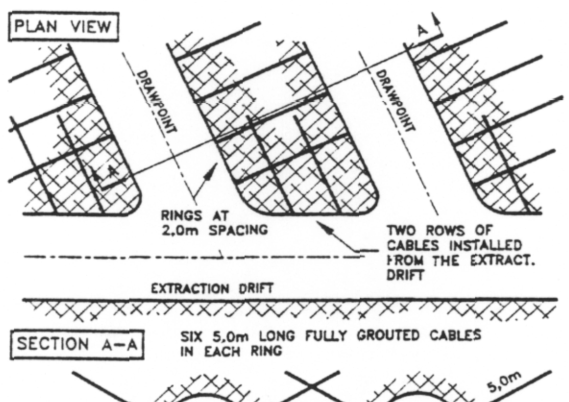


Figure 11. Pillar cable reinforcement

been completed the support in the drawpoints showed an increased load, plates deformed, and large-scale shear failure occurred at the spring line of both the sidewalls. Deformation increased rapidly after the shotcrete had cracked owing to the failure of the weld-mesh reinforcement. The main problem was complete failure of the cable grips. This meant that large slabs of reinforced shotcrete pulled loose from the bolts, or pulled shorter bolts with them. Some of the grips that had not slipped had become overloaded, and cable-bolt failure occurred.

Conditions deteriorated very quickly. Closure rates in some drawpoints reached over 100 mm per week. Most of the drawpoints became too narrow for the operation of equipment, as well as becoming unsafe for the crews. Only 10 to 20 per cent of the ore reserves had been extracted by the time that most of the drawpoints on 1350 North were shut down. Efforts were concentrated on completing the development of 1350 South and bringing it into production.

A second series of repairs was started several weeks after the drawpoints had been shut down. Loose shotcrete was removed, and some drawpoints were slashed to the proper width. Open ground was re-shotcreted and covered with welded-wire mesh, and 4,8 m long Dywidag bolts were installed at a spacing of 1,0 m.

In this advanced stage of pillar disintegration, long anchors did not help very much, and sidewall convergence continued at basically the same rate as before the repairs. Most of the plates holding the wire mesh were pulled through the shotcrete. Some bolts failed, and some of them anchored in shear zones were pulled out with the shotcrete.

The whole support system was re-evaluated and the support reaction re-assessed. The performance of the current support system indicated that the support had to have a greater yield capability. A combination of heavy chain-link mesh with split-set stabilizers at a spacing of 0,75 m was designed. Long Dywidag bolts were used to anchor the whole system deep into the rockmass.

The excessive deformation (Figure 16) and floor lift (Figure 17) in the north has meant that most of the drawpoints have had to be slashed and re-supported.

Although the closure rates did not slow down after the repairs had been made, the increased width of the drawpoints provided room for future deformation. The yielding capability of the chain-link mesh provided some degree of support pressure and a safe working environment by holding up the broken shotcrete.

In spite of the severe ground-support problems on 1350 North, most of the first-phase ore was extracted, and the installation of arches for the second phase has begun.

Support System for 1350 South

Based on experience from 1350 North, a more efficient support system was developed, and the standard of installation has improved significantly. A special effort was made to ensure the minimum of delay between the excavation and the support installation.

Figure 18 shows the basic support layout used on 1350 North. Horizontal straps were now integrated as part of the support (Figure 19), which increased the amount of steel in the shotcrete by approximately three times. After the drawpoint excavation had been completed, 4,8 m long fully grouted Dywidag bolts were installed at a spacing of 1,0 m in the middle of the existing pattern. Concrete reinforced with rebars was installed on the floor and, additionally,

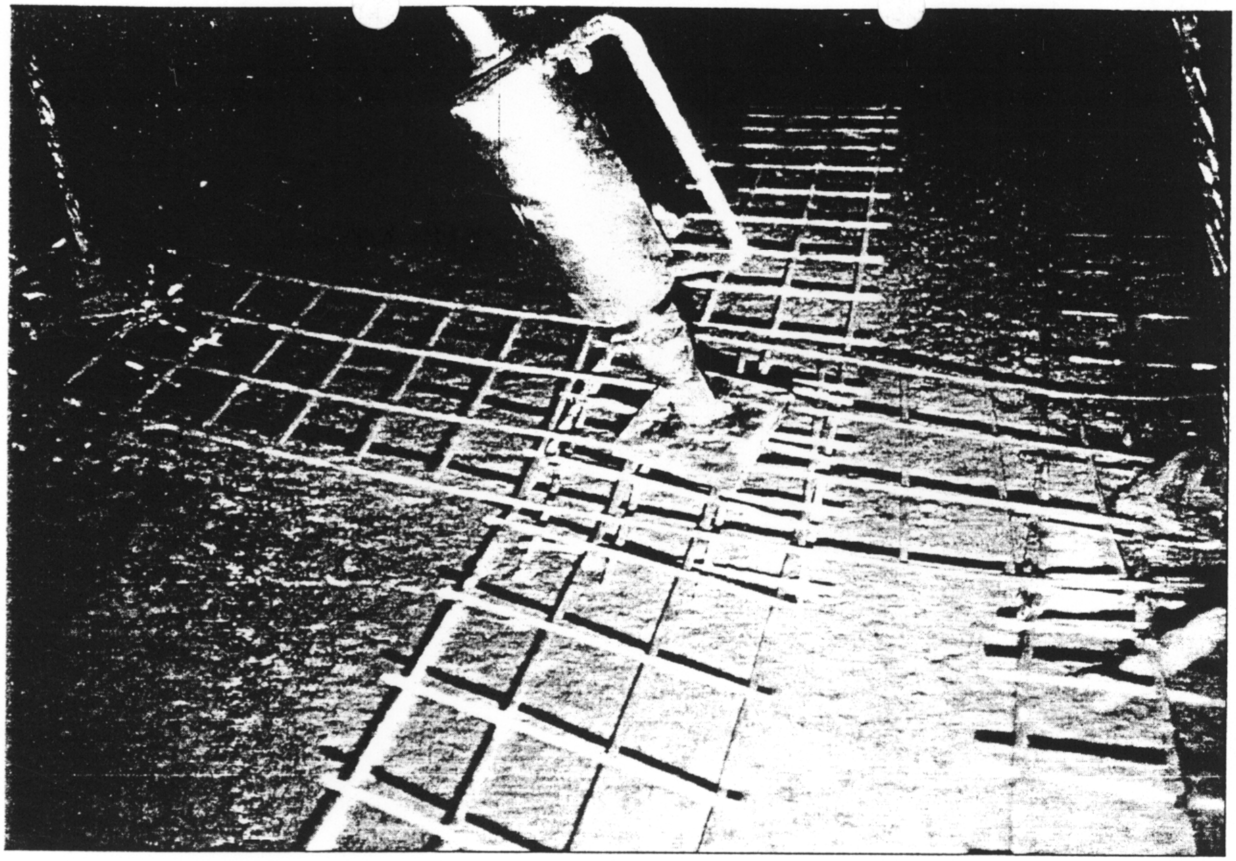


Figure 12. Detail of the horizontal and vertical strap system used for repairs at 1350 North. The cable grip is installed with a hydraulic jack

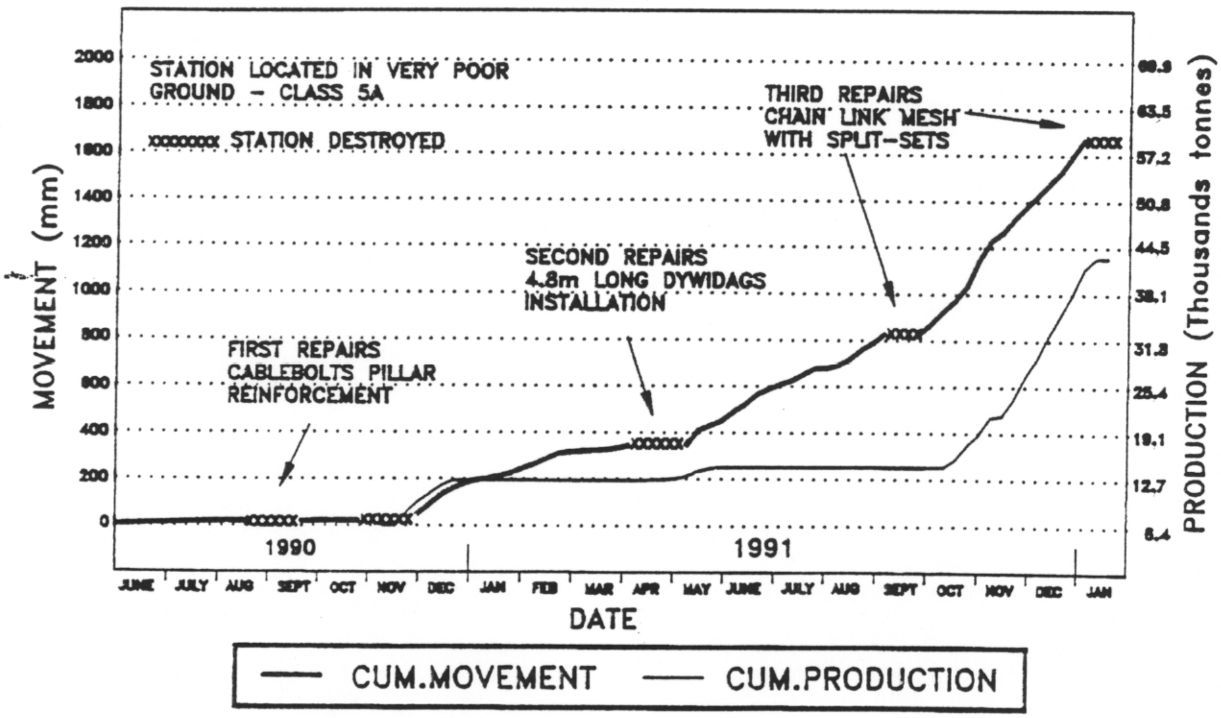


Figure 13. Cross-drift deformation, drawpoint no. 2

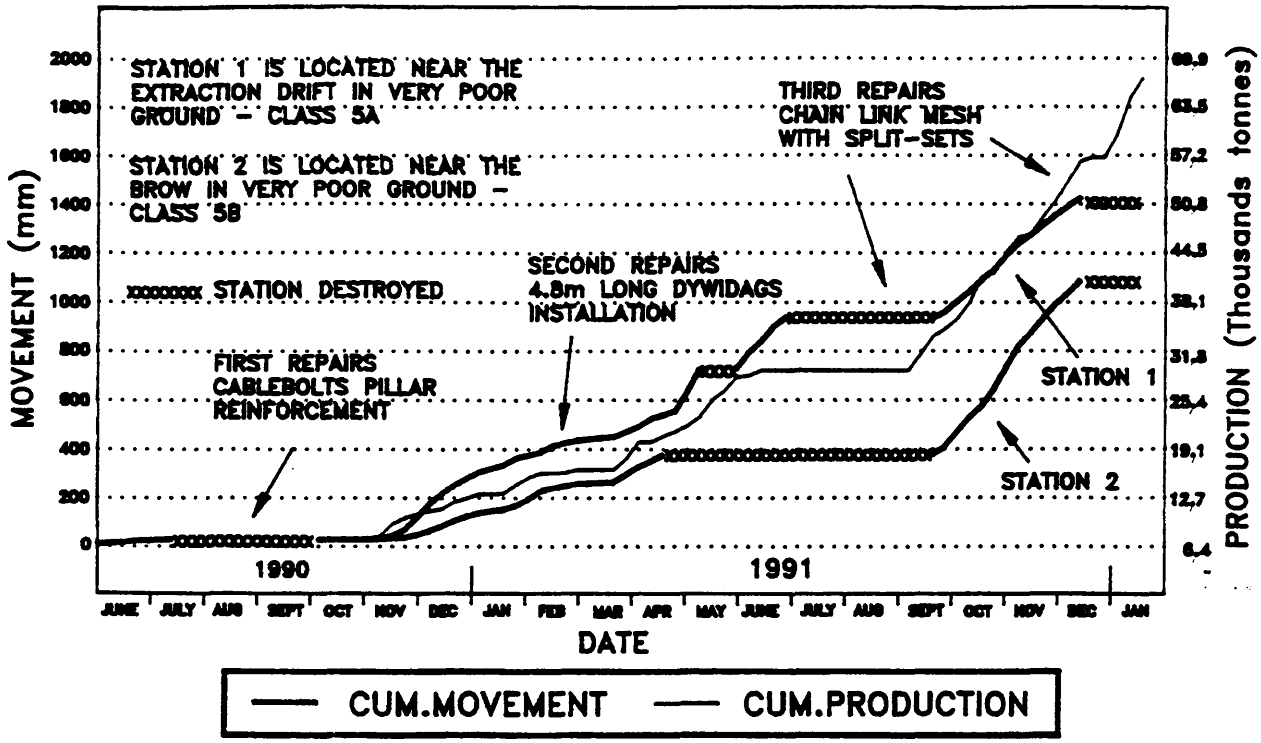


Figure 14. Cross-drift deformation, drawpoint no. 6

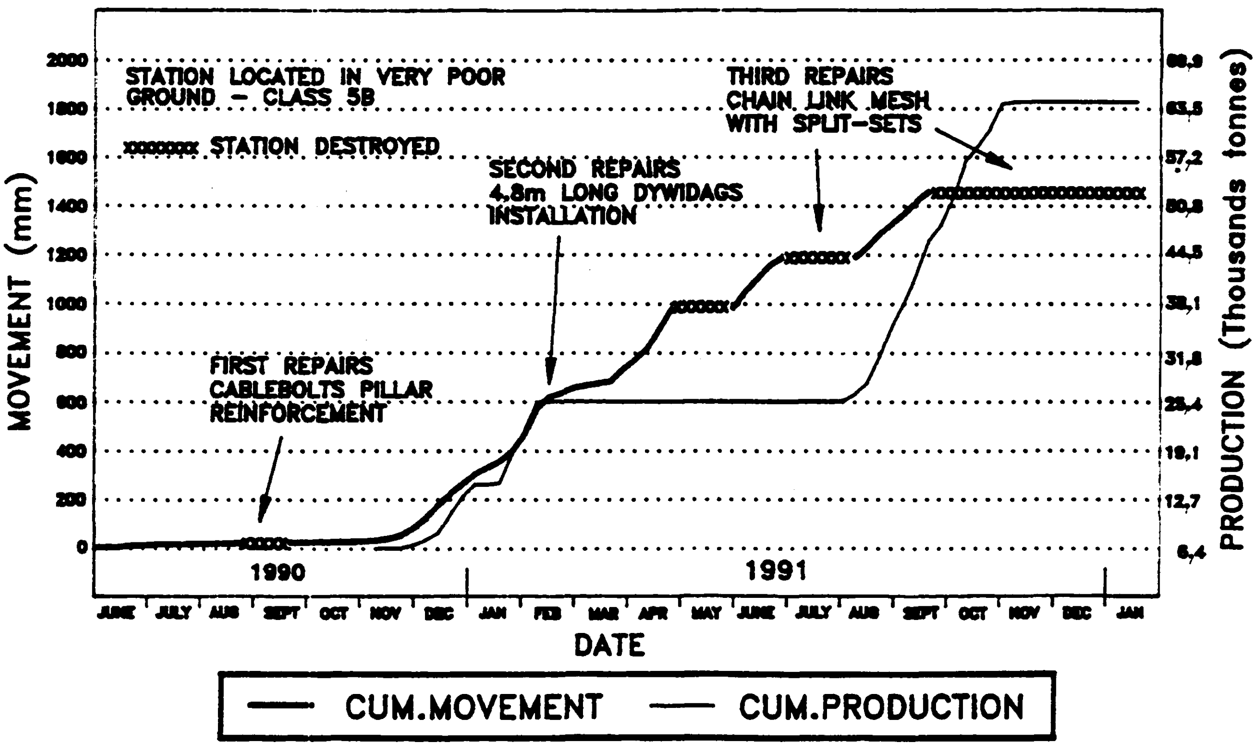


Figure 15. Cross-drift deformation, drawpoint no. 9

25 mm ropes were grouted 3.0 m into the floor. Figure 20 shows the convergence record of drawpoints supported with the new system.

Since some cracks were observed in the shotcrete lining, a vertical strap system was added to increase the support stiffness. The results of closure monitoring of the drawpoints with the described support system are shown in Figure 21. Because of its poor performance, welded-wire mesh was later replaced with one layer of 50 by 50 mm gauge-6 chain-link mesh (Figure 22). The chain-link mesh proved to be far superior to the welded mesh in its deformation capability (Figure 23).

Because of difficulties with the rope installation (namely, proper tensioning to the wall), the ropes were replaced with 15 mm 27 t cable bolts. Seven cable bolts were installed on each corner, which improved their distribution over the entire corner. The typical layout is shown in Figure 24. Cable bolts were connected with steel blocks and cable

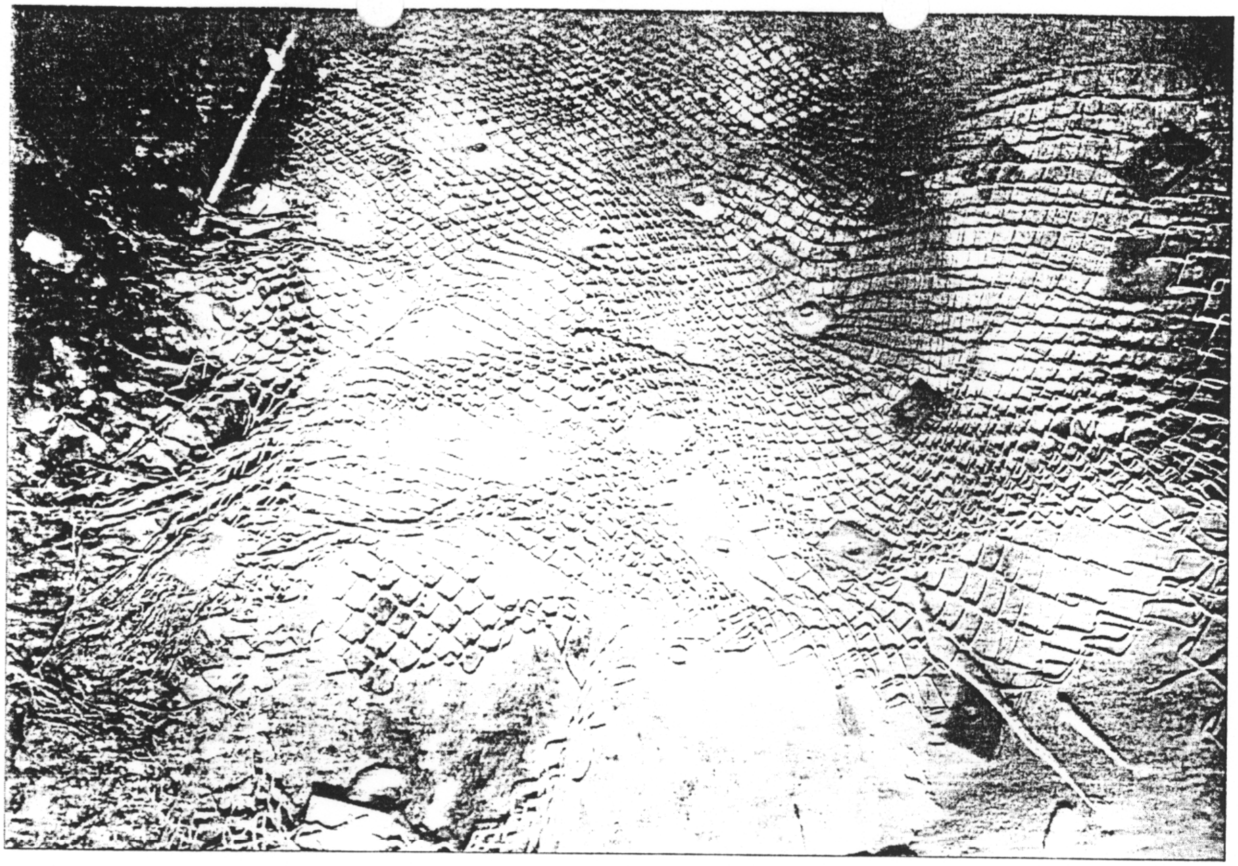


Figure 16. Extensive sidewall damage in drawpoint no. 9, 1350 North

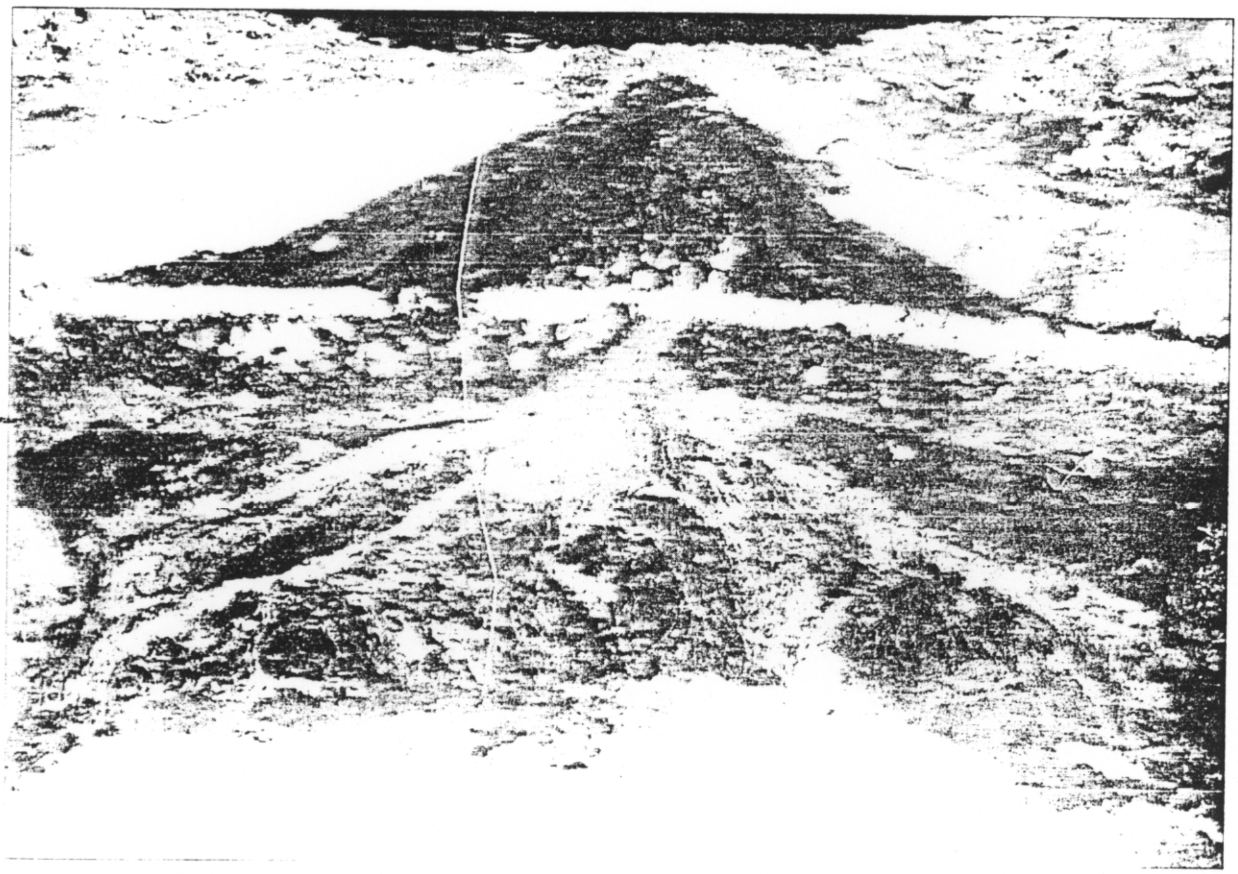


Figure 17. Floor heave in drawpoint no. 8, 1350 North, showing plastic deformation of the sheared serpentinite

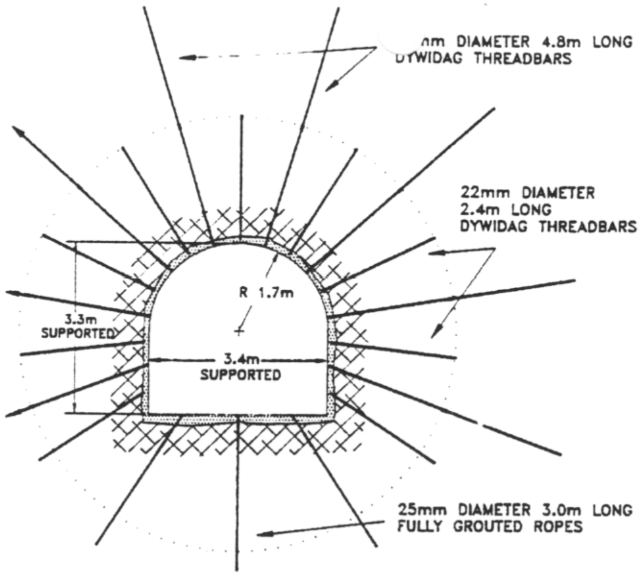


Figure 18. Layout of the rock support on 1350 South

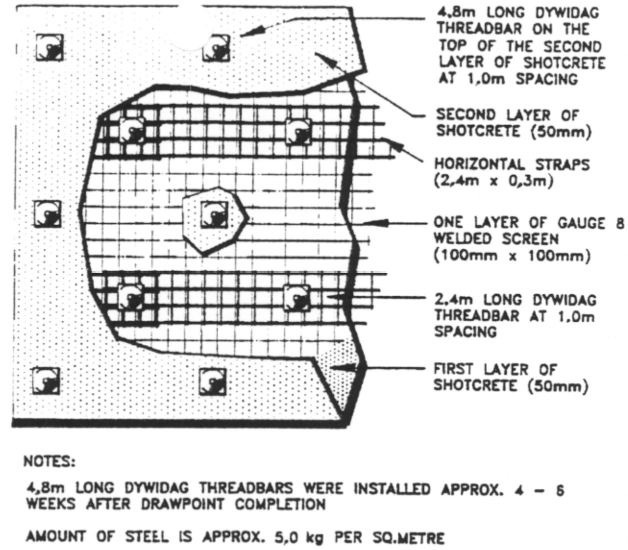


Figure 19. Detail of support lining—horizontal straps

grips, and were tensioned to approximately 3 t. Although there was no problem on 1350 South, this connection system failed under extreme pressure on 1350 North when used in repairs. Connection blocks were later successfully replaced with split-set cable slings.

Three types of steel arches were used as reinforcement in the drawpoint brows of 1350 South. All of them performed very well, and no failure of arches was observed. Compared with those for 1350 North, the anchoring techniques were significantly better. The cable bolts were replaced with 4,8 m long Dywidag bolts, and the floor channels were anchored to the floor with four 2,4 m long Dywidag bolts.

The first type was a rigid-arch system and is shown in Figure 25. The second type was a 29 kg yielding profile installed in the same manner as the rigid one. Later, during second-phase arch installation, the arch connection was redesigned and new connection brackets were successfully used (Figure 26). This new design provided a much stronger and simpler connection, as well as providing flexibility in the arch spacing.

The third type was a rigid arch with a flat back to provide a greater wearing surface at the brow.

A single layer of chain-link mesh was installed on the back over the final layer of shotcrete to provide protection for the crew against spalling shotcrete.

The primary draw was completed in January 1992, approximately 18 months after the drawpoints had been excavated. Not a single repair was required.

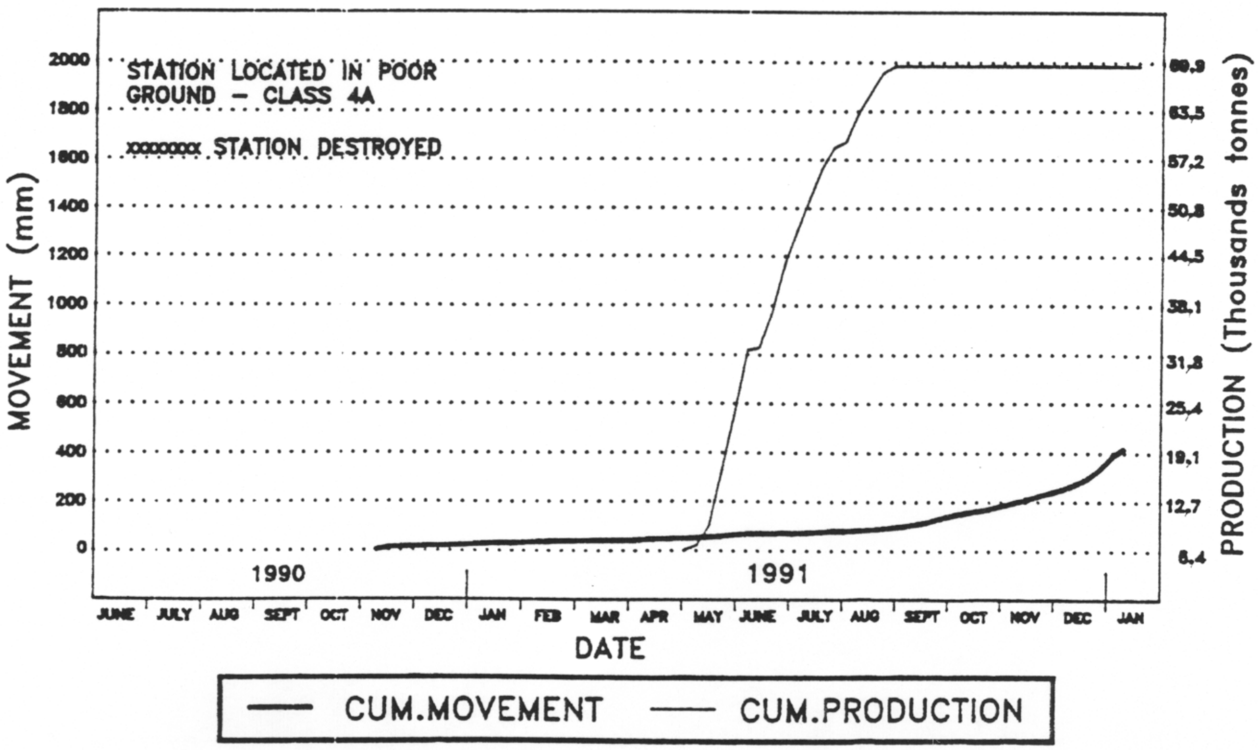


Figure 20. Cross-drift deformation, drawpoint no. 16

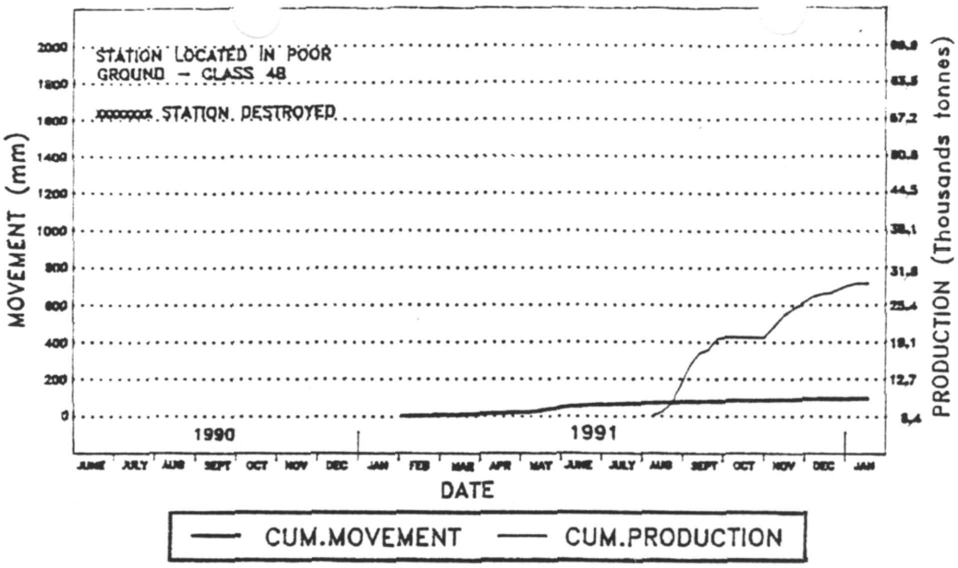


Figure 21. Cross-drift deformation, drawpoint no. 21

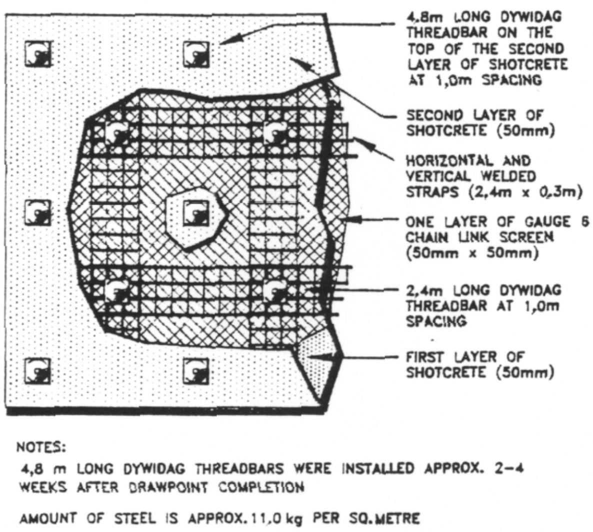


Figure 22. Detail of the support lining—horizontal and vertical straps

The Rockmass Response and Failure

It was assumed that the horizontal stress was twice the vertical stress[6]. In Cassiar's case, high topographic relief results in a rapid change in vertical stress over a relatively short distance. The maximum stress was estimated at 25 MPa, while the strength of the rockmass was only 1 to 5 MPa.

As the drawpoints on 1350 level were excavated, the horizontal stresses were cut off, which increased the vertical loading on the pillars. Magnified by the pillar's horizontal relaxation, the rockmass adjusted with small movements on discontinuities. The rockmass response in this environment has been described by Brumleve[7]. The deforming shear zones provided additional degrees of freedom for block movement as loading and unloading of the rockmass continued. Zones of competent rock started to dilate, which resulted in new loading on the shear zones. This cyclical interplay continued until the rockmass disintegrated.

The low standard of support and repeated slashing gave the pillars additional opportunity to deform. This caused gradual pillar disintegration, and the load-carrying capacity of the pillars was reduced significantly. Plastic flow of the weak rockmass into the openings was observed on many occasions (Figure 16).

Extreme vertical loading of the drawpoints on the north occurred after the cave zone had extended from 1350 level into the argillite. This was caused by massive wedge failure above the level. Figure 27 shows an idealized cross-section.

When closure readings and production are plotted against time, a relationship between the two can be seen. With increased production, there is a corresponding increase in movement and *vice versa*. The drawing process creates a low-density area in the column that allows the wedges to move into the cave area. This movement reduces the weight or friction along the shear zones, and the wedge increases the loading on the pillar below it. If no material is removed from the drawpoint, loose material consolidates and acts as a backfill against movement.

Conclusions

Although more than 90 per cent of the predicted ore reserves were extracted from 1350 level, the mine experienced major ground-control problems on 1350 North, the scale of which was not anticipated. On the other hand, the experience gained from 1350 North and back-analysis of the problems helped the mine to develop improved support techniques for 1350 South.

The factors that caused ground-support failure on 1350 North are as follows.

(1) Under-estimation in the initial ground-support design. Owing to the extremely poor ground conditions and the unfavourable orientation of the major geological features encountered on 1350 North, the original support design was not adequate. The support was not designed to withstand the pressure caused by massive wedge failure and, because of production demands, floor support was not installed.

(2) Poor performance of certain support elements. The cable grips failed when shotcreted over. Welded-wire mesh and welded straps proved to be inadequate where deformability of the support lining was required.

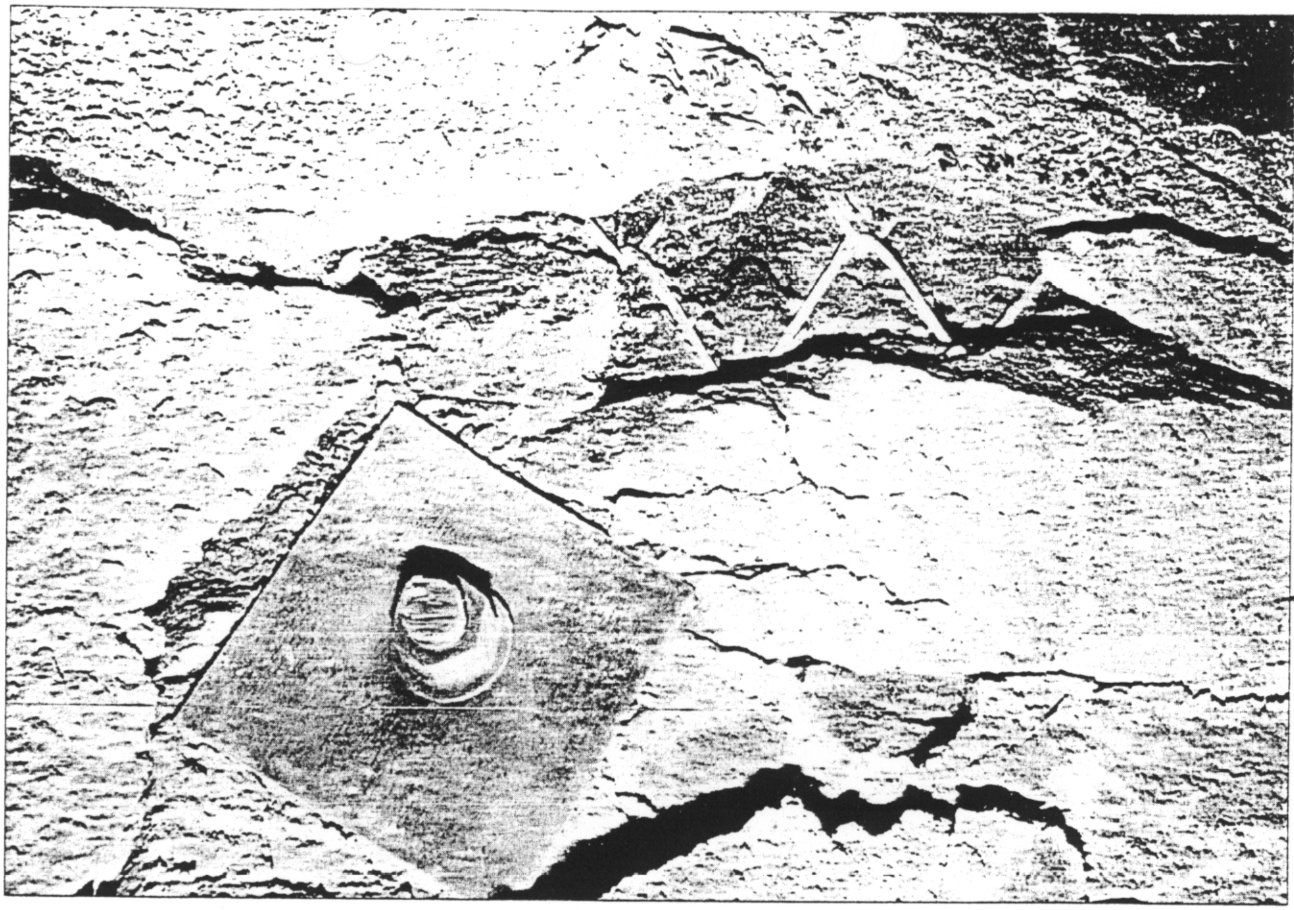


Figure 23. Intact chain-link mesh in cracked shotcrete lining

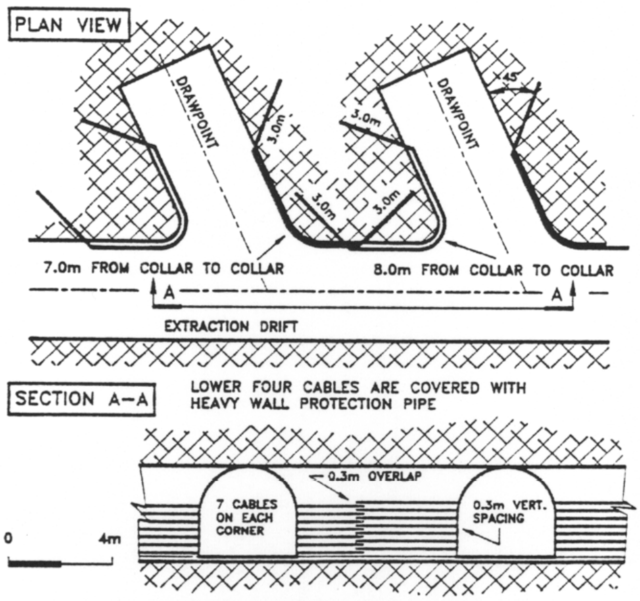


Figure 24. Pillar corner support

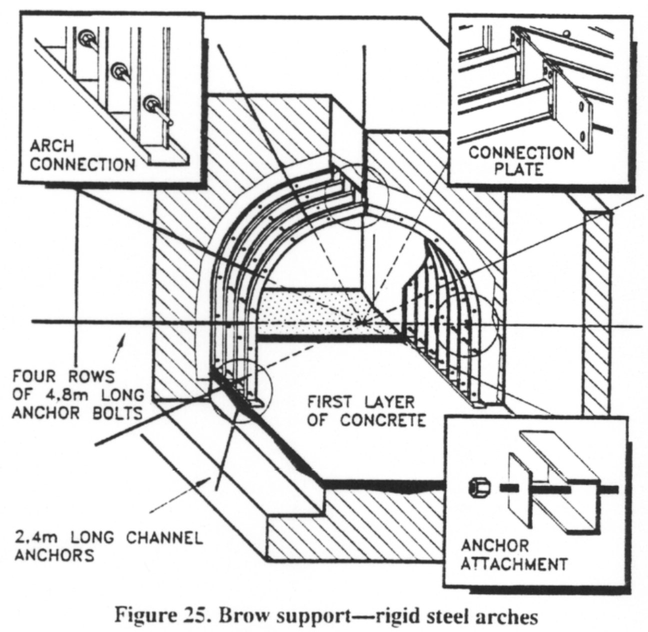


Figure 25. Brow support—rigid steel arches

(3) Low standard of development and support installation. The failure to follow correct support procedures and frequent delays in the installation resulted in weak points in the support lining, and subsequent failure. The correct blasting technique was not used consistently, causing disturbance of the surrounding ground, support damage, and overbreak (which creates irregularities on the walls). A low standard of survey control resulted in deviations from the original design and weakened pillars.

(4) Erratic mining environment. Ground-support problems related to production delays caused by a labour dispute, poor fragmentation, and unexpected water problems created an inconsistent mining environment. Repeated loading and unloading of the rockmass had a very negative impact on the stability of the excavations.

It is likely that overcutting of the extraction level would have reduced the toe stresses and load on the drawpoints but, owing to time and cost constraints, this concept of a separate overcut level was abandoned in the early planning stage.

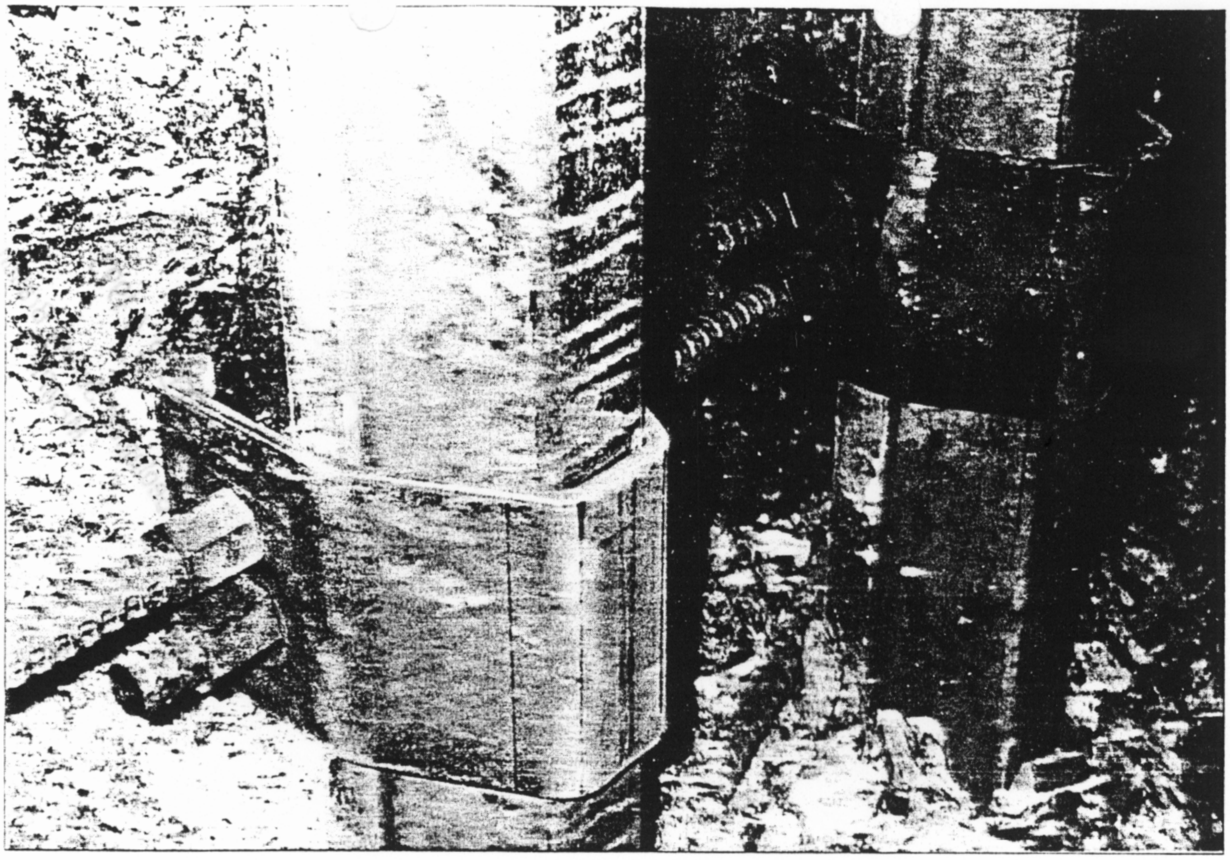


Figure 26. New arch-connection bracket

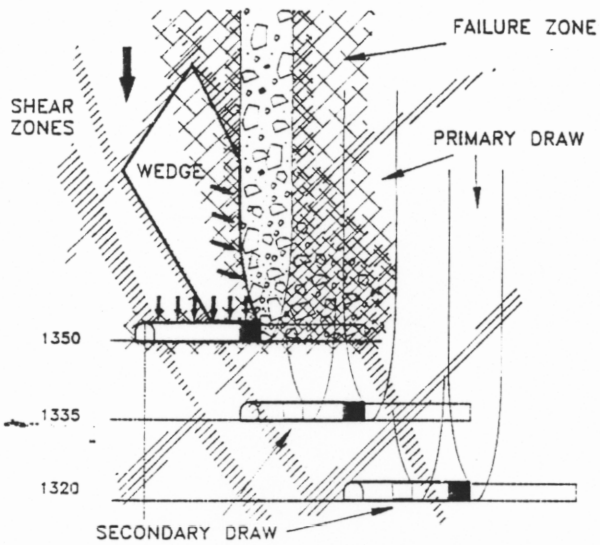


Figure 27. Wedge failure and drawpoint loading

It was found that, besides all the factors causing support failure, a support system in very poor ground conditions has to have a greater yield capability and still maintain the right support pressure. Although there are still some problems to be solved, the support system using shotcrete reinforced with heavy chain-link mesh, split-sets stabilizers, and long cross-pillar anchors, combined with large surface restraining plates, is the right direction to follow. This system provides the initial support pressure and has the deformation capability to allow the rockmass to become destressed without disintegrating.

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