Grouting with the MPSP Method at Kidd Creek Mines, Ontario

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Introduction

One of the decade's few advances in the methodology of rock grouting is the multiple packer sleeved pipe system. MPSP overcomes the problems posed to efficient treatment in the face of collapsing, voided or highly variable ground conditions. The method has been used with distinction to form grout curtains in association with several high dams in Asia and Europe, while it can also be used to improve the mechanical properties of masses when used in 'consolidation' grouting applications (Bruce & Gallavresi, 1988).

Most recently, it has been adpoted in an entirely different environment – to stabilise highly variable weakly cemented rock fill, approximately 800m underground in an operational copper mine in northern Ontario. This particular case history is not only of interest and relevance to the underground engineering community, but it also provides a clear demonstration of the workings of the method in general. The program featured work in two distinct locations and phases, and the paper describes how observations from the first were used to optimise operational parameters in the second phase.

The MPSP method

Background

The fissure grouting of stable rock masses has been long practised and very well documented. Houlsby (1990), for example describes the techniques of downstage, upstage, and circuit grouting. The relative advantages and disadvantages are summarised in **Table 1.**

However, there are often conditions in which neither stage grouting method can be relied upon to provide an effective and reliable treatment. For example, in downstage schemes, the presence of very fissured, granular or fragmented rock (eg sugary limestone) may result in caving of the stage after drilling, and before grouting can be executed. Thus, in the worst case, only the uppermost part of that stage will be treated, and the lower section will remain ungrouted and most probably cause similar problems during later redrilling operations. Likewise, the presence of such strata, voids and/or soft infill zones will prevent upstage grouting being practical: packers will be very difficult to 'seat' efficiently, and may permit grout bypass upwards, leading to ineffective treatment or trapped packers, and usually both.

Such unstable geological conditions reflect more typically the problems associated with soils grouting, for which the standard hi-tech approach is the well known tube à manchette (sleeved pipe) system. A fundamental feature of the tube à manchette operation is the necessity to rupture the sleeve grout, thus permitting egress of

* The MPSP method was devised by the Rodio Group of Companies, which controls the rights to its use. Nicholson Construction is the licensee in North America.

	Downstage	Upstage
ADVANTAGES	1 Ground is consolidated from top down, aiding hole stability and packer seating and allowing successively higher pressures to be used with depth without fear of surface leakage. 2 Depth of the hole need not be predetermined: grout take analyses may dictate changes from foreseen, and shortening or lengthening of the hole can be easily accommodated. 3 Stage length can be adapted to conditions as encountered to allow 'special' treatment.	1 Drilling in one pass. 2 Grouting in one repetitive operation without significant delays. 3 Less wasteful of materials. 4 Permits materials to be varied readily. 5 Easier to control and programme. 6 Stage length can be varied to treat 'special' zones. 7 Often cheaper since net drilling output rate is higher.
DISADVANTAGES	1 Requires repeated moving of drilling rig and redrilling of set grout: therefore, process is discontinuous and may be more time-consuming. 2 Relatively wasteful of materials and so generally restricted to cement based grout. 3 May lead to significant hole deviation. 4 Collapsing strata will prevent effective grouting of entire stage, unless circuit grouting method can be deployed. 5 Weathered and/or highly variable strata problematical. 6 Packer may be difficult to seat in such conditions.	1 Grouted depth predetermined. 2 Hole may collapse before packer introduced or after grouting starts, leading to stuck packers and incomplete treatment. 3 Grout may escape upwards into (non-grouted) upper layers or the overlying dam, either by hydrofracture or bypassing packer. Smaller fissures may not then be treated efficiently at depth. 4 Artesian conditions may pose problems. 5 Weathered and/or highly variable strata problematical.

Table 1. Major advantages and disadvantages of downstage and upstage grouting of rock masses.

grout into the surrounding soil. However, in all but the softest rocks, the lateral restraint offered by a rock mass is sufficient to prevent the sleeves from opening and allowing the flow of grout into surrounding fissures. In addition, the nature of the system – involving the use of a stable cement-bentonite sleeve grout – essentially plugs off, for a short but often critical distance, those fissures which are intersected by the borehole, thus further circumscribing the possible effectiveness of the system. In this regard, the finely fissured soft shales very successfully treated at Grimwith dam, Yorkshire (Bruce, 1982) could well be regarded as an upper limit for rock mass quality in terms of effective tube à manchette grouting.

It was against this background of providing high quality treatment of ground which would otherwise frustrate the effectiveness of these conventional methods that Rodio developed the MPSP system.

Installation and operation

MPSP owes much to the principle of the tube à manchette sytem, in that grouting of the surrounding rock is effected through the ports of a plastic or steel grout tube placed in a predrilled hole. However, unlike tube à manchette, no sleeve or annulus grout is used. Instead, the grouting tube is retained and centralised in each borehole by concentric collars-fabric bags inflated in situ with cement grout. These collars are positioned along the length of each grout pipe, either at regular intervals (say 3m to 6m) to isolate standard stages, or at intermediate or closer centres to ensure intensive treatment of special or particular zones. The system permits the use of all grout types, as dictated by the characteristics of the rock mass and the purpose of the ground treatment.

The typical construction sequence is as follows **(Figure 1)**:

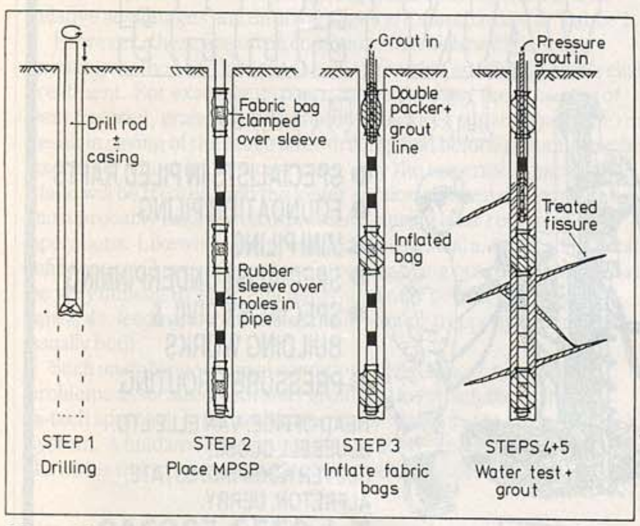


Figure 1

1 The borehole is drilled by fastest available method (usually rotary percussive) with water flush to full depth. Temporary casing may be necessary to full depth also, as dictated by the degree of instability of the rock mass, Typically borehole diameters are 100 mm–150 mm.
2 The MPSP is installed. Pipe details can be varied with requirements, but a typical choice consists of a steel pipe, 50mm od, with each length screwed and socketed. Each 5m pipe would have three 80mm long, 4mm thick rubber sleeves equally spaced along the length, protecting groups of 4mm diameter holes drilled in the pipe. A concentric polypropylene fabric bag is sealed by clips above and below the uppermost sleeve in each length and is typically 400mm to 600mm long **(Photograph 1)**. For short drill holes, plastic pipes of smaller diameter may be used. The temporary drill casing is then extracted, and any collapsing material simply falls against the outside wall of the MPSP tube.

Photograph 1: Fitting fabric bag over plastic MPSP.

3 Starting from the lowermost pipe length, each fabric bag is inflated via a double packer positioned at the sleeved port covered by the bag. A neat cement grout is used at excess pressures of up to $200kN/m^2$ to ensure intimate contact between bag and borehole wall. The material of the bag permits seepage of water out of the grout, thus promoting high early strength and no possibility of later shrinkage. Clearly the choice of the bag material is crucial to the effective operation of the sysem: the fabric must have adequate strength, a certain elasticity, and a carefully prescribed permeability.
4 Water testing may then be conducted if required, through either of the two free sleeves between collars, again through a double packer. Tests show that a properly seated fabric collar will permit effective 'stage' water testing at up to $400kN/m^2$ excess pressure.
5 Grouting is executed in standard tube à manchette fashion from bottom up via the double packer (usually of the inflatable type.) The grouting parameters are chosen to respect target volumes (to prevent potentially wasteful long-distance travel of the grout) and/or target pressures (to prevent potentially dangerous structural upheavals).

The following additional points are especially noteworthy regarding the MPSP system. First, it is clear that, if a hole has been grouted once, it generally cannot be regrouted: some of the pressure grout will remain in the annulus outside the tube and so form a strong sleeve grout preventing the opening of sleeves in contact unless a very weak grout mix were used. (The system does, however, allow different stages *in the same hole* to be treated at different times.) Thus the MPSP system adopts the principles of stage grouting, where split spacing methods are used: the intermediate secondary holes both demonstrate the effectiveness of the primaries and intensify the treatment by intersecting incompletely grouted zones. Analyses of water test records, grout injection parameters, reduction ratios and so on will dictate the need for further intermediate grouting phases.

Second, in addition to the technical advantages of the system, there are significant logistical and work scheduling attractions. For example, the drilling and installation work can proceed regularly at well known rates of production, without requiring an integrated effort from the grouting crews (as in downstage grouting). In addition, the secure nature of the grout tube prevents the possibility of stuck packers, which is an unpleasant but unavoidable fact of life in upstage grouting in boreholes in most rock types. Grouting progress is therefore also more predictable and smoother, to the operational, technical and financial advantage of all parties concerned.

A third point relates to the straightness of the borehole and thus the integrity and continuity of the ground treatment. The temporary drill casings used in the hole drilling operations (1) are typically thick-walled and robust. They therefore promote hole straightness, whereas the uncased boreholes common in stage grouting in rock, and drilled by relatively flexible small-diameter rods, are known to deviate substantially, especially in cases where fissures and/or softish zones in the rock mass are unfavourably located or oriented. By way of illustration, at Metramo Dam, Italy, the maximum deviation recorded in a test block of 150 cased holes, each 120m long, was 1.5% with the great majority being less than 1%.

Grouting of backfill: Kidd Creek Mines Timmins, Ontario

Background

As described by Yu and Counter (1983) Kidd Creek Mines, near Timmins, northern Ontario, routinely used consolidated rockfill to fill mined openings underground such as stopes and pillars. The minesite produces 4·1 million tons of copper, zinc, lead and silver ores annually from its No 1 and No 2 underground mining operations.

No 1 mine was developed with a shaft to a depth of 930m in 1973 to recover the ore between the floor of the open pit and a depth of 792m. Stoping widths are normally 18m, the heights varying from 90m-135m and lengths from 22m to 65m. Vertical rib pillars between stopes are typically 24m wide and sill pillars are about 30m thick. Pillar recovery is implemented after filling the mined stopes with the consolidated backfill. This permits total recovery of the ore with a minimum dilution, while also maintaining safe working conditions.

No 2 mine was developed from a second shaft, sunk to a depth of 1530m in 1978, for recovery of ore beneath No 1 mine.

A total of 2·5 million tons of backfill are required annually to fill the mined-out areas completely. About 80% has been consolidated material, consisting of crushed mine rock – generally less than 150mm in maximum dimension – mixed with a cementitious binder before being gravity fed into the voids. The balance has been unconsolidated rockfill or sandfill.

As part of the ongoing expansion of the mine, it will be necessary to backfill at greater depths, and to facilitate this opertion, vertical holes 630mm in diameter, drilled through previously backfilled areas, were projected. Typically a raise boring machine is used first to drill, by rotary methods, a pilot hole. This is then reamed out to full diameter in the raise boring.

However, early tests in the older filled areas where these holes were required showed that the degree of consolidation of the backfill was highly variable, and in some cases almost non-existent. As a result, conventional drilling was exceptionally difficult, and led to massive overbreak in the fill – always a major safety concern underground. Clearly some form of additional ground stabilisation

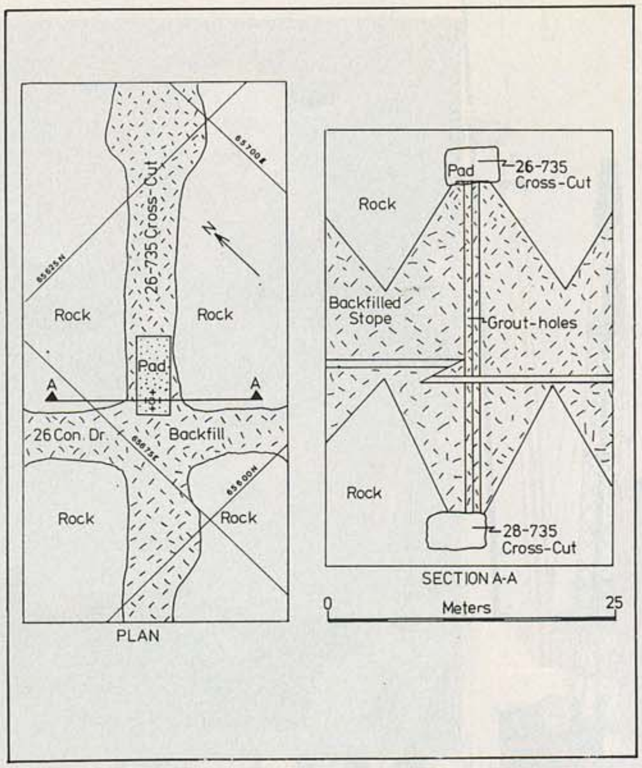


Figure 2

was required prior to raise boring, and a research programme was put in hand.

This programme consisted of grouting and then raise boring through two distinct backfilled stopes. In each case, the work was conducted through 60m of fill, from the 790m level. The first trial was conducted at the 735 cross cut (**Figure 2**) and, in addition to being the proving ground for the system, equipment and materials, this test also permitted the grouted zone to be explored by subsequent down-drilling of the 250mm diameter pilot hole, but upreaming of the 630mm diameter reamed hole.

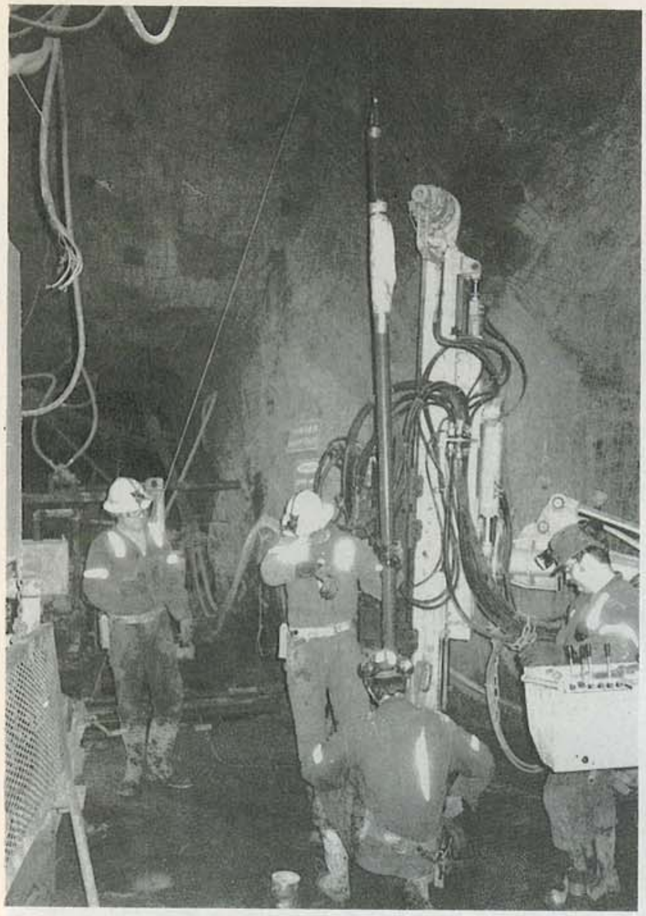
The second test site was located at the same level, but at the 660 cross cut – some 75m to the north. This test incorporated modifications based on the first test, but also demonstrated the competence of the grouted fill with respect to up-drilling of the pilot, but down-reaming of the final bore – a far more severe test of the concept.

The headroom at each test site was about 4m, while access to the working locations restricted maximum equipment widths and heights to about 2.5m. As noted below, the properties of the fill did vary somewhat within and between the two stopes treated. However, the fill was generally highly variable in composition and competence, with a bulk unconfined compressive strength of about 5000kN/m^2, although the strength of the andesite aggregate itself was over 300 000kN/m^2.

First test (735 cross cut)

System selection

Drilling of grout holes. The unstable ground demanded the use of a continuous contemporary steel casing, full length. This could be advanced only by some form of duplex method (Bruce 1989), but would normally require the considerable power of a large drilling rig to reach full depth in such difficult conditions. However, given the restrictions placed on drill size (and so, in a general way, on drill power), the Krupp double head drilling system was used. This was mounted on a Krupp DHR 80 diesel hydraulic track rig, with special short mast (**Photograph 2**). The lower part of the double head rotates the outer casing slowly (but to the benefit of high torque) in one direction, while the upper part simultaneously rotates the inner drill rod (with drill bit and hammer) in the opposite direction. The



Photograph 2: Short mast Krupp DHR 80 drill rig, with double head, inserting MPSP into cased hole, foreground.

cutting action of the system is thus enhanced, even though each component, ie outer casing and inner rod, is rotating relatively slowly. The system also discourages drill hole deviation as a result of this contra-rotation in association with the use of the tough thick walled casing (11mm). The dynamic boundaries of the annulus between rods and casing also help prevent blockages caused by drilling debris being returned to the surface.

Grouting method. The easiest method of grouting such conditions is simply to pump grout through the casing as it is slowly extracted. However, another method was necessary here since the highest degree of control over the grout placement and procedure was required; and having to use 1m long casing lengths would severely interrupt such grouting operations, possibly leading to blockages in the lines, or worse, accidental cementing of the drill casing in the hole. A grouting method independent of the drill casing was therefore necessary, and the MPSP system appeared to be ideally suited to the role. This was the first use of the system in North America.

Grouting concept. A pattern of four grout holes was arranged around the position of the subsequent raise bore **(Figure 3)**. Grouting was intended to stabilise the ground in this vicinity to permit the raise bore to proceed quickly and safely. A cement-based grout was considered most appropriate, bearing in mind the materials available in the mine; the suspected nature of the fill; and the intended purpose of the grout in situ.

It was also decided to drill and grout the first two holes (1 and 2) as primaries, before commencing with the other two (3 and 4). In this way, any beneficial effect of the primary treatment would be noted in the secondary activities.

Drilling

The hole positions were carefully laid out on a specially prepared level concrete pad, 7m by 3m in plan, cast on the fill and ranging from 100mm to 250mm in thickness. The outer drill casing was 133mm in diameter, and the inner drill carried a 100mm down-the-hole hammer.

As is typical in such programmes, the drilling of the first hole proved very problematic, and consumed a longer period then foreseen. The drilling confirmed that the fill was generally very loose, and contained frequent very large, very hard rock boulders. However, with adjustments to drilling techniques and hardware, and improvements to air flush characteristics, the holes were drilled with progressively increasing ease.

Penetration times, torque requirements and flush return characteristics were continuously measured, permitting an assessment of gross changes in the fill characteristics every metre. **Table 2** summarises the drill production data, the major points being:
1 the significantly faster drill production after the primary grouting and
2 the reduction in the number of problem zones encountered by the two later holes.

These features can be seen in detail in **Figure 4** which compares penetration rates for holes 1 (primary) and 4 (secondary). At the same time, casing torque requirements decreased, while flush characteristics improved in the two later holes: both features indicative of improved ground conditions.

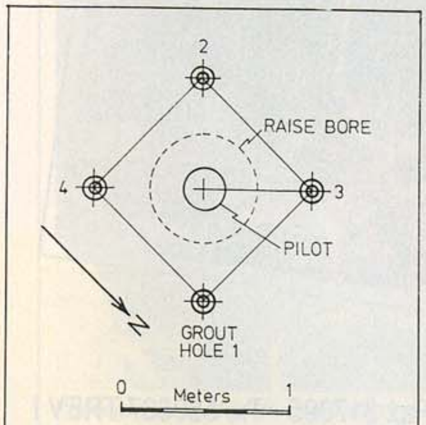


Figure 3

Hole	Drill Days on the Hole	Actual Drilling Times: On the Hole (mins)	Actual Drilling Times: Penetration (mins)	Penetration Rates (mins/1 m casing): Min.	Max.	Mean	Depths of Major Obstructions
1	6-1/2	2538	899	3	60	15.2	17, 19, 25, 33, 34, 43 m
2	3-1/4	1115	846	4	50	14.3	11, 12, 35, 36, 37, 38, 45-47 m
3	2-1/4	805	663	4	43	11.2	36, 48, 58 m
4	2-1/4	787	658	5	29	11.2	53 m

Table 2: Drill production analysis.

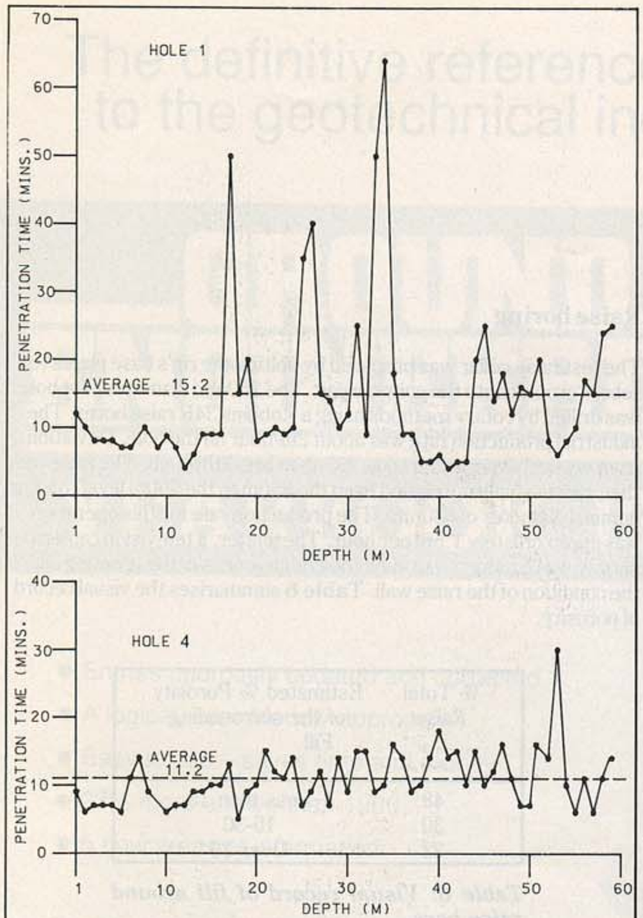


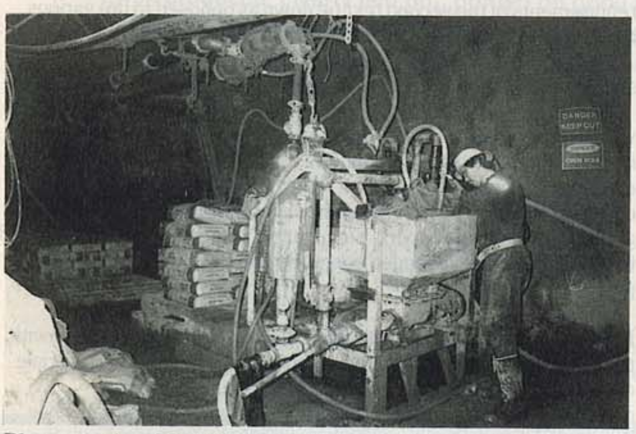
FIGURE 5: COMPARISON OF DRILLING PENETRATION RATES HOLE 1 (PRIMARY) AND HOLE 4 (SECONDARY)

Figure 4

Hole	Length (m)	Deviation Direction	Distance (cm)	Angular Deviation (%)
1	59.2	East	6	6/5920=0.1%
2	59.2	Northeast	125	125/5920=1.1%
3	59.4	Southwest	32	130/5960=0.5%
4	59.6	West	130	130/5960=2.2%

Overall Average: $\frac{293}{23,740} = 1.2\%$

Table 3: Summary of drill hole deviations.



Photograph 3: Modified colloidal mixer/pump unit, where bagged cement was added to surface mixed mine slurry.

As shown in **Figure 2,** each of the four holes was designed to 'break out' into the drive at the lower 850m level. This allowed the deviation of the holes to be measured with great accuracy by precise mine survey methods **(Table 3).**

Careful consideration of the drill monitoring data suggested that hole 2 had kicked off from vertical after glancing a large boulder or slab at a depth of 45m-47m, while it was probable that over-crowding of the drill system by an overzealous and relatively inexperienced operator caused the deviation of hole 4. Overall, however, the holes proved remarkably straight, given the extremely onerous drilling conditions, and in all four cases the deviation did not hinder any subsequent operation (eg extraction of casings, placing of pipes).

Placing MPSP pipes

Plastic pipes of 72mm od in 3m lengths were used as the grouting pipes. Each length had rubber sleeves at about 1.5m intervals. Every fourth sleeve was fitted with a 600mm long fabric bag to provide therefore 6m stage lengths in the ground. The bags were capable of expansion of up to 190mm in diameter, and so ensured a good seal with the ground upon inflation.

After placing of the MPSP in each hole, and the extraction of the steel drill casing, the bags were inflated via a Rodio double packer, using carefully controlled volumes of neat cement grout.

Grouting

In order to reduce the amount of bagged dry materials to be transported and handled underground, the 'base' of the cement grout mix was one of the mine's standard formulations. These grouts are prepared under the highest quality control at the surface, and pumped through kilometres of 100mm or 150mm diameter steel lines to the point of usage underground.

In this case, the slurry was pumped to a large storage tank, near the special mixer/pump to be used for the trial (**Photograph 3**). The design of the mix is shown in **Table 4.** Early tests confirmed that, while it was easily pumpable and had a long setting period, it was too fluid and unstable (ie high bleed capacity) to use in this particular application. Thus, dry cement was added to this slurry at the test site, in a Colcrete colloidal mixer, in a proportion providing the thickest mix capable of routine mixing and pumping **(Table 5).**

The grout was then pumped by a Moyno progressive cavity pump through the flexible injection line and inflatable packer. In each hole, grouting was conducted from the bottom up, successive stages being designated primary and secondary and treated in that order to allow an extra degree of data analysis and control. Flow rates and volumes were regulated by manually controlled valves on the grout circulation line system.

Based on estimated grout travels and theoretical ground porosity, each stage was injected with 2000 litres of grout. Early on it became clear that the grout was flowing very freely, with considerable amounts, from the lower stages especially, draining into the opening at the 850m level up to 4m or 5m radially distant from the hole breakthrough location. To try to stop this leakage, and to help localise the effect of the grouting, sodium silicate solution was added from the adjacent hole during grouting. When encountering each other, a very rapid or flash set occurs in the ground, the exact time depending on the composition and relative amounts of each component.

By the end of the grouting, over 76 000 litres of cement grout and 9 000 litres of sodium silicate solution had been injected. At each level, there was a slight reduction in rate of flow, and a slight increase in pumping pressure through each successive phase of grouting, highlighting a certain degree of tightening up in the ground. Typical grout flow rates were 20 litre/min to 30 litre/min with pressures of about $1500kN/m^2 – 2000kN/m^2$. However, these values reflected less on the properties of the ground than the hydraulic characteristics of the pumping hardware and ancillaries, in which the limiting diameter was 11mm in the 60m long flexible grout line.

Consideration of the grout seepage patterns into the 850m level led to the conclusion that the grout was not necessarily remaining local to the points of injection and so not filling completely the voids it encountered. Given the very open nature of the fill, and the characteristics of the grout used, this was scarcely surprising. Instead, it was felt that the grout was in general passing down through the fill, and thorougly coating the aggregate en route. The grout, when set, would therefore be gluing adjacent blocks together, as opposed to filling completely all the voids between them. This analysis was supported by the much improved drilling performance after the grouting of the first two holes.

Grouting was therefore considered complete when all the stages had been grouted once, and preparations were made to commence raise boring.

Component	Weight (kg)	SG	Yield (litres)
Portland cement	4272	3.1	1378
PFA	2091	2.7	774
Water	4759	1.0	4759
Mix	11,122		6,911

WSR = 4759/4272 + 2091 = 0.75
Bulk Density = 11122/6911 = 1.61

Table 4: Composition of one 'batch' of standard mine mix (Recipe 4).

One tank of slurry (60 litres capacity)		+	Added dry cement	=	Injected mix
PC	37 kg	+	30 kg	=	67 kg
PFA	18 kg		–	=	18kg
Water	41 kg		–	=	41kg
Total	96 kg	+	30 kg	=	126 kg

For the injected mix WSR = 0.48
Bulk density = 1.82
Bleed = 19% (after 2 hours)

Reaction time with equal volume of sodium silicate solution = 50 sec

Cube strength = $12\,000N/mm^2$ at 28 days.

Table 5: Composition and properties of injected mix. (Note: The volumes and weights cited are consistent with the batch capacity of the storage and mixing tanks of the grout plant, and reflect site practice.)

Raise boring

The test raise collar was prepared by bolting the rig's base plates to rebars grouted into the concete pad. The 250mm diameter pilot hole was drilled by rotary methods using a Robbins 34R raise borer. The industrial production rate was about 2m/hour on the hole. Deviation from vertical was measured as 2% upon breakthrough. The raise was then successfullly upreamed from the 850m to the 790m level to a final nominal diameter of 630mm. The production rate for this operation was approximately 1.5m per hour. Thereafter, a television camera survey was conducted to view the effectiveness of the grouting and the condition of the raise wall. **Table 6** summarises the visual record of porosity.

% Total Raise	Estimated % Porosity of the surrounding Fill
48	Less than 15
30	15-30
22	Over 30

Table 6: Visual record of fill around raise bore.

These observations suggested that the grouting had reduced the porosity quite substantially in places, but that elsewhere the fill was still relatively open structured, though stable. This confirmed the feel for the gluing action of the grout, obtained during the injection phases. In addition, it was possible that the rigid plastic pipes, at relatively close centres, had contributed an in situ reinforcing effect to the larger fill blocks, helping to stitch them together somewhat and so forming a more stable framework in situ.

Second test (660 cross cut)

Amendments to system

Based on the execution and performance of the drilling and grouting undertaken in the first test, and the more arduous reaming requirements of the second test (ie down as opposed to up) various amendments to the methodology were made:

Hole geometry: Six peripheral holes were installed, supplemented by a seventh, central hole **(Figure 5)**.

Grout mix design: The peripheral holes and the bottom of hole 7 were to be grouted with a more viscous, stable cement-based grout, incorporating bentonite. The aim was to restrict flow and improve the efficiency of void filling (as distinct from interblock glueing). The central hole was to be grouted with a higher strength, neat cement grout, for additional support. Stage target volumes were also changed to accommodate the changed geometry and the anticipated backfill porosity at this location.

Equipment: The opportunity was taken to experiment further with the details of the drilling method, although the fundamental principle of double-head duplex was retained. The grout station was overhauled to provide higher volume and pressure potential, while the grout delivery lines were increased in diameter to facilitate the faster pumping of more viscous grouts.

As in the first test, the grouting was conducted in two phases: holes

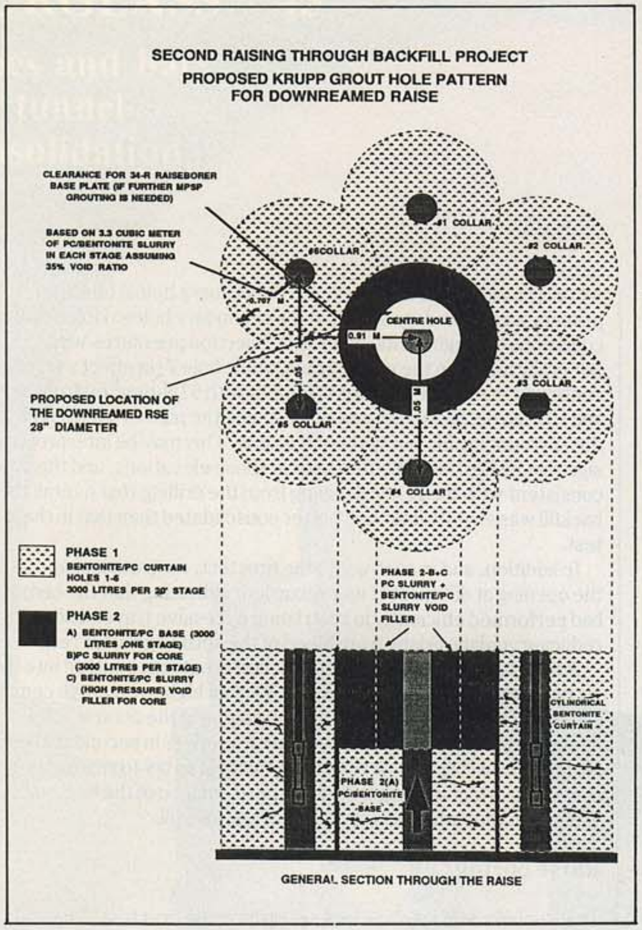


Figure 5

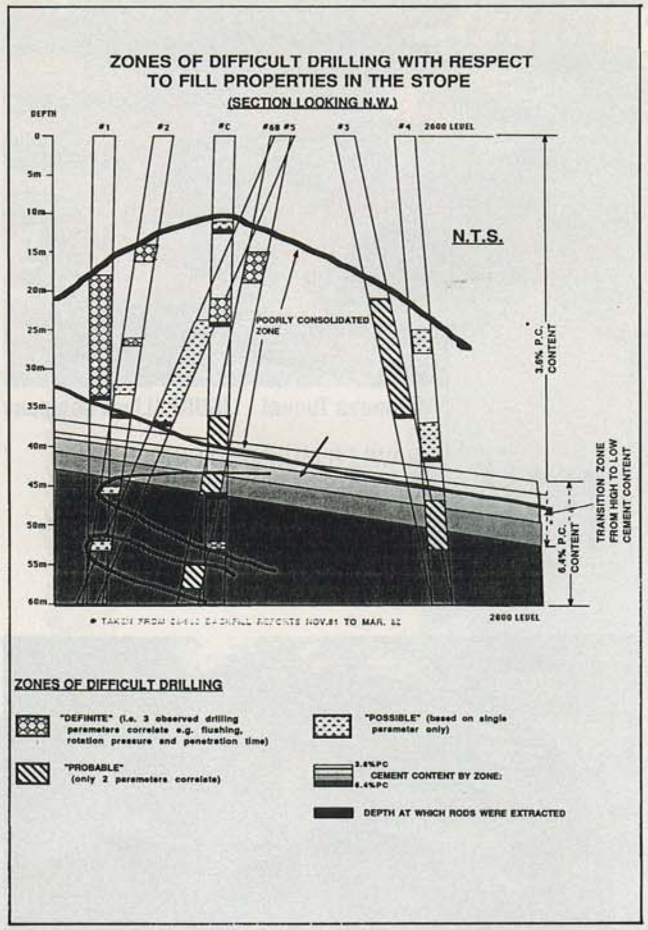


Figure 6

1,3 and 5 **(Figure 5)** drilled and grouted as primaries, followed by holes 2, 4 and 6 as secondaries, a and finally hole 7 (central).

Drilling

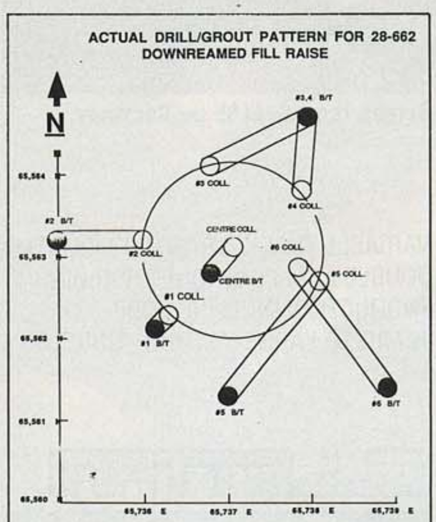
The test raise collar was positioned in the centre of the 28-662 stope about 15m away from the original backfill truck dump point, and downslope from the peak of the fill cone. Very high torques were necessary to rotate the casing to depth in all holes and with all experimental modifications to the drilling methodology. However, best performance was obtained with the lost crown system (Bruce, 1989), which gives an oversized hole (140mm diameter) without the need for an eccentric drill bit.

Particular difficulties were encountered in penetrating zones of unconsolidated backfill **(Figure 6),** mainly in the region where the original slurry mix design changed from a slurry to aggregate ratio of 6.4% (lower part) to 3.6% binder ratio (upper part). In most holes the casing had to be temporarily withdrawn and the cutting tools replaced before full drilling depths could be attained.

The secondary holes were generally drilled substantially faster than the primaries, and with slightly easier drilling demands, reflecting the benefits of the primary grouting. The exception was hole 6 which had to be abandoned, possibly as a result of an artificial obstruction in the fill, and was replaced by hole 6b. As evidenced by the degree of hole deviation **(Figure 7),** this replacement hole was fortunately placed to give a more uniform hole pattern in situ. Close records of drilling parameters were maintained for each hole, including torque, thrust, flush return characteristics, and penetration rate.

Grouting

The peripheral and plug grouting was executed with a bentonite enriched slurry developed from field experiments. Each 100 litre batch of 60% PD mine slurry had almost 6kg of bentonite added at the colloidal mixer. The target volume was 3000 litres per stage, and this was achieved in most cases, at pumping rates of up to 60 litre/min. However, as a result of grout migration between and along holes, certain sleeves could not be opened to permit injection. Such stages



HOLE NO.	DEPTH	DEVIATION %
1	60	0.42
2	60	1.83
3	60	2.08
4	60	1.50
5	60	2.92
6	60	2.98
CENTER	60	0.70

Figure 7

were typically secondary stages in the primary holes, and both primary and secondary stages in the secondary holes. Hole 7 had tight conditions throughout its lower half. Injection pressures were generally higher in the secondaries, while hole 7 (in effect a tertiary) continued this trend. Various attempts with $51N/mm^2$ hydroblastic and EB cap detonations were made inside the pipe to open a path to the surrounding fill, but without success. This may be interpreted as a sign that fill has been grouted tight at those elevations, and this was consistent with the general feeling from the drilling that overall the backfill was slightly finer and better consolidated than that in the first test.

In addition, and in contrast to the first test, no grout leakage into the opening at 850m level was recorded, indicating that the bentonite had performed efficiently in restricting excessive travel, and in reducing voidage arisng from bleed of the setting grout at rest.

A total of $160m^3$ of bentonite enriched grout was injected into the six perimeter holes, while a further $30m^3$ of higher-strength cement grouts were injected into (the upper) stages of the central hole. Pressures were higher and injection rates lower in secondary holes. No sodium silicate was necessary in this test to try to restrict grout travel, again highlighting the better performance of the bentonite grout, and the rather better quality of the backfill.

Raise boring

The Robbins 34R machine was set up as in the first test. The 250mm diameter pilot hole was the drilled smoothly to the breakthrough depth of 59m, at the extremely high penetration rate of 12m/hour. The rods were than extracted and the 710mm diameter downream head was fitted. Rig thrust pressure was maintained the same as for the pilot, but rotational speed decreased from 40rpm to 16rpm. A slight problem was encountered at 25m depth when the reamed debris bridged in the pilot hole below. This blockage was quickly cleared from the 850m level, and a video camera insalled there to allow the drillers, at the upper level, to verify the free fall of subsequent reamed debris. The penetration rate for the reaming averaged 4.2m/hour.

A video camera was then lowered down the completed raise. This showed that although some zones of coarse, bonded fill remained (similar to the first test), the overall condition of the fill was very stable and well cemented.

It was estimated that the average porosity of the treated fill was considerably less than 15%, as compared to the typical mine backfill value of over 35%. This improvement could be due to three factors:

1 The location of the raise in the fill cone, being in a zone of more fines, compared to the position of the first test in a segregated zone.
2 Improved grout mix design (more viscous, more stable).
3 The increased intensity of treatment afforded by the large number of grout holes in the second test.

Final remarks

The fill stabilisation at Kidd Creek Mine has proved to be an unusual but highly successful application of the MPSP system. The system is more commonly used to form cutoffs in difficult rock mass conditions. However, the special restrictions posed by working 800m underground in low headroom situations clearly did not compromise its effectiveness, even when used in this novel application of ground strengthening.

The key to the success of the system was the selection of a suitable drilling method. The double-head method proved perfect, given the hole geometry, the ground conditions, and the strictly limited drilling energy available. Experiments with modifications to cutting shoes and bits have demonstrated that the lost crown method is particularly effective in reducing the effort that has to be applied to advancing the critical outer casing. Benefits accrue in the form of improved linearity and reduced breakdown/blockage time, in addition to the obvious bonus of faster penetration.

The success of this programme has particular relevance to the deep hard rock mining industry but it should also be an encouragement to ground improvement engineers operating in other disciplines.

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