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DEVELOPMENT OF DESIGN SPECIFICATIONS FOR ROCK BOLTING FROM RESEARCH IN CANADIAN MINES

D. F. COATES AND T. S. COCHRANE

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Mines Branch Research Report R 224

DEVELOPMENT OF DESIGN SPECIFICATIONS FOR ROCK BOLTING FROM RESEARCH IN CANADIAN MINES

by
D. F. Coates* and T. S. Cochrane**

SUMMARY

Over the years a considerable amount of work has been done by the Mines Branch of the Government of Canada on the subject of rock bolting. Most of the work has been done in the mines; some studies have been conducted in the laboratory. A selection of some of the significant findings has been made for this report.

From the experience obtained in testing the anchorage capacity of different types of hardware in formations varying from strong sandstone to soft shales and also salt, it has been found that different anchorage shells with similar mechanical designs can have quite different anchorage capacities. It is only possible to determine the capacity of a particular anchorage system in a particular rock by conducting a series of pull tests. The one general conclusion that has been established is that in weak rocks the larger the bearing length of the shell the greater its capacity.

The testing of anchorages has also shown that like other structural components, the strength of an individual bolt will be dispersed about an average value. The degree of dispersion is important, as a higher frequency of failure than is acceptable can result if roofs that have a support system designed on the basis of average strength. For example, it was found in one series of tests that the coefficient of variation of yield loads was 27%, which means that only at 65% of the average yield load ($0.65\ P_y$) would 90% of the bolts have a yield load equal to or greater than this amount ($0.65\ P_y$).

In soft rocks a large amount of possible expansion is desirable to prevent wedges from being pulled through the shell. In other words, relatively wide-angle wedges are more appropriate than the small-angle wedges suitable for use in hard rocks. Tests showed that for many

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conventional shells 100% of the possible expansion was used in installing the bolt, leaving no margin for further deformation before the wedge would be pulled through the shell. In this regard, the minimum possible hole size will provide the best anchorage conditions. In extreme cases, of course, grouted bolts or other types of support may be necessary.

It was found difficult to obtain uniform installation tensions in all bolts even when hand tightening is used. The amount of torque dissipated during installation as a result of bending of the bearing plate is responsible for much of the variation and is affected by the rock conditions at the collar of the hole. Consequently, the bearing plate size and thickness are important.

Anchorage capacity can deteriorate with time. Aside from rock that creeps under stress or that suffers deterioration from ground water, the most significant cause of such deterioration is deflection of the roof accompanied by expansion in the rock. These deflections may increase with time under static geometrical conditions but are greatly affected by the mining of adjacent openings. Measuring programs have shown that it is possible to establish a deflection number that, if exceeded, heralds the onset of additional deflection together with significant deterioration of rock conditions and the associated bolt anchorages. For example, in mines in two different areas it was found that for entries with a 5-m breadth an expansion of 6 mm within the first 210 cm into the roof was the critical amount. It has also been found that the retorquing of bolts can be beneficial, increasing the working load and presumably inhibiting the development of critical deflections in the roof.

Laboratory model tests indicate that bolt spacing should be less than 5 times the typical joint spacing in the rock mass. In addition, it was seen in these tests that the spacing should not exceed the length of the bolts.

As a result of field research on rock bolting, tentative specifications have been formulated to provide guidance for design engineers using rock bolt support for temporary openings. These specifications require classification of the rock, selection of steel for the bolt, the use of the appropriate size of bearing plate, the calculation of bolt length and spacing, limitation of the magnitudes of installed tension and torque, and monitoring under many conditions. The tentative specification can be improved by analysing the observations and criticisms from design engineers who are dealing with a wide variety of conditions.

Direction des Mines

Rapport de Recherches R 224

DEVELOPPEMENT DE L'ETUDE DES SPECIFICATIONS DU SOUTENEMENT PAR BOULONS D'ANCRAGE DANS LA RECHERCHE MINIERE AU CANADA

par

D. F. Coates* et T. S. Cochrane**

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Au cours des années, des travaux d'envergure furent accomplis par la Direction des Mines du Gouvernement canadien dans le domaine du boulonnage des roches. La plupart des travaux furent exécutés en collaboration avec des mines et on poursuivit des études dans des laboratoires. Cette communication est un choix des constations d'importance que l'on a fait.

Nous nous sommes rendus compte, par l'expérience acquise en éprouvant la capacité d'ancrage de différents types de boulons, dans des formations, à partir de grès durs jusqu'aux schistes tendres, y compris le salpêtre, que différentes sortes de coquilles d'ancrage, de construction mécanique similaire, peuvent avoir des capacités d'ancrage assez marquées. C'est seulement par la réalisation d'une série d'essais en traction qu'il est possible d'établir la capacité d'ancrage d'un système particulier dans une roche particulière. La seule conclusion générale que l'on ait pû tirer, c'est qu'en roches tendres, plus grande est la longueur portante de la coquille, plus élevée est sa capacité d'ancrage.

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Les essais effectués sur des systèmes d'ancrage ont aussi démontré que la résistance d'un boulon individuel sera dispersée autour d'une valeur moyenne, de la même manière que le cas d'autres systèmes structuraux. Le degré de dispersion est important puisqu'il peut en résulter un taux de manquements plus grand que celui qui est accepté pour le cas de toits où la résistance moyenne conditionne l'étayage. Par exemple, nous avons trouvé, à la suite d'une série d'essais, que le coefficient de variation des charges-limite était de 27%, ce qui signifie que c'est seulement à 65% de la charge-limite moyenne (0.65 $\bar{P}_y$) que 90% des boulons atteindraient une charge-limite égale à ou plus élevée que cette valeur-ci (0.65 $\bar{P}_y$).

En roches tendres, il est désirable que l'expansion possible soit grande, ce qui empêcherait les coins de passer à travers la coquille lors du serrage de l'écrou. En d'autres mots, des coins à grands angles sont plus appropriés que ceux à petits angles, employés surtout en roches dures. Des essais ont démontré que pour plusieurs coquilles de types conventionnels on faisait usage à 100% de l'expansion disponible lors de l'installation du boulon, ne laissant aucune marge pour toute autre déformation supplémentaire avant que le coin, sous traction, ne passe à travers la coquille. A cet égard, un trou de dimension aussi restreinte que possible aussurera les meilleures conditions d'ancrage. Naturellement, pour les cas exceptionnels, il sera peut-être nécessaire d'employer des boulons cimentés ou tout autre mode de soutènement.

Nous nous sommes aperçus qu'il est difficile d'obtenir un serrage uniforme des boulons même lorsque celui-ci se fait manuellement. Le montant de torsion dispersé pendant l'installation, à la suite du fléchissement de la plaque portante est responsable pour la presque totalité de la variation et est affecté par les conditions de la roche à l'orifice du trou. En conséquence, la grandeur et l'épaisseur de la plaque portante sont de première importance.

La capacité d'ancrage peut diminuer avec le temps. A part la roche qui flue sous l'effet des contraintes ou qui s'altère en raison des eaux d'infiltration, la cause la plus significative d'une telle diminution est la déflexion du toit accompagnée de l'expansion de la roche. Ces déflexions qui peuvent s'accroître avec le temps dans des chantiers stationnaires, sont grandement affectées par les travaux d'exploitation adjacents. Des programmes de mesures ont démontré qu'il est possible d'établir un nombre de la déflexion lequel, lorsque dépassé, marque le début de déflexions additionnelles conjointement avec une détérioration prononcée des conditions de la roche et du boulonnage. Par exemple, dans des mines en deux endroits différents, nous avons trouvé que pour le cas d'entrées de 5 m de largeur, une expansion de 6 mm en-deçà des 200 premiers cm de pénétration du toit constituait la valeur critique. Nous avons aussi constaté que le resserrage des boulons peut avoir un effet bénéfique, augmentant ainsi la charge de travail et probablement empêchant le développement de déflexions critiques au toit.

Des essais sur modèles en laboratoire indiquent que l'espacement des boulons devrait être inférieur à 5 fois l'espacement typique des diaclases dans la masse rocheuse. En outre, ces essais ont montré que l'espacement ne devrait pas excéder la longueur des boulons.

A la suite de recherches sur le boulonnage exécutées en chantier, nous avons formulé provisoirement des spécifications qui serviraient de guides aux ingénieurs en dessins se servant de ce mode d'étaiement par boulonnage pour le cas d'ouvertures temporaires. Ces spécifications demandent la classification des roches, un choix de l'acier pour la fabrication des boulons, l'usage de plaques portantes de grandeurs appropriées, le calcul de la longueur et de l'espacement des boulons, une limite de la magnitude de la contrainte et de la torsion, et le rôle avertisseur sous des conditions importantes. Ces spécifications provisoires peuvent être précisées par l'analyse des observations et critiques énoncées par les ingénieurs en dessins qui sont occupés avec des multiples conditions.

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INTRODUCTION

Over the years a considerable amount of work has been done by the Mines Branch of the Government of Canada on the subject of rock bolting. Most of the work has been done in the mines; some studies have been conducted in the laboratory (1-3)*. A selection of some of the significant findings has been made for this research report.

This work has been synthesized into design rules that can be used, at least as a first approximation, to select a practical bolting system. With the trend, particularly in metal mining, towards reducing engineering staff and to decreasing the training period of engineers in underground operations, there is a conspicuous decrease in the number of engineers with firmly based judgement available for mine planning. This situation can be partly countereacted by formulating rules of practice for design (as has been done for other engineering work) that are applicable to the majority of the cases. The economic inefficiencies that may result from applying such rules to all cases can be considered as part of the price that must be paid for the modern developments in staffing.

In spite of the suggestions that are made herein for standard designs, it is recognized that many of the specifications are based on judgement. Furthermore, in view of the uncertainties that commonly are associated with a variable geological material, it is considered important to monitor the reaction of the ground around the underground excavations so that designs can be confirmed or modified. Finally, when standard procedures are adopted, practising engineers can then concentrate their ingenuity and criticism to evolve an improved methodology.

SOME RESULTS FROM STRATA MEASUREMENTS

INSTALLATION

It has been found that different anchorage shells with similar mechanical designs can have quite different anchorage capacities in weak rock, which can usually only be determined by pull-testing. At

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* These numbers refer to the sources of information listed in the Bibliography at the end of this paper.

the same time, it has been established that in weak rock the larger the bearing length of the shell the greater the capacity of that shell.

Series of pull tests were conducted in various mines. The concepts used in analysing the tests are illustrated in Figure 1, which describes a pull test where the anchorage capacity is greater than the strength of the steel. In this figure the point A represents the stage when, ideally, the pre-stress in the bolt is entirely transferred from the bearing plate to the jack; hence, the extrapolation of the curve BA back to the Y-axis provides some measure (although the true situation is somewhat more complicated) of the initial load at which the bolt has been installed. During the application of the load from points A to B, the steel is being stretched, the wedge in a shell-type anchor is being forced into the shell, and the shell is biting into the rock--all of which produces displacement at the end of the bolt. The slope of this line gives the anchorage modulus (displacementper unit load).

At the point B in Figure 1 the variation of displacement with bolt load increases. This point may result from a breakdown in the shell/rock contact or may be due to the yielding of the steel. Whatever the cause, the load at this point is called the yield load, P_y.

As the test is continued, the load reaches the point where displacement occurs with very little increment in load. Either the ultimate capacity of the shell/rock anchorage has been obtained or the ultimate strength of the steel has been reached. The load at this point is called the maximum load, P_m.

When the load is released at the point C, the displacement decreases to the point D, the recovery representing the elastic strain in the system. The plastic strain, both in the steel and in the shell/rock contact, is irrecoverable. On reloading the bolt back from D to its maximum load at E, the additional displacement should be primarily due to the elastic stretching of the bolt. For a good-quality anchorage the point E should be substantially coincident with the point C. However, it is possible for a high capacity to be obtained on the first cycle through some unusual circumstances that could break down and produce a much lower working capacity. An attempt to increase the load at point E should produce with very little effort the additional displacement to point F, confirming that the ultimate load had been reached at the point C.

Some typical curves are shown in Figure 2. Test No. 48 shows an initial resistance of about 5 tons that broke down on further displacement of the bolt. Eventually, in an attempt to cycle the load, the anchorage capacity reduced to less than 1 ton.

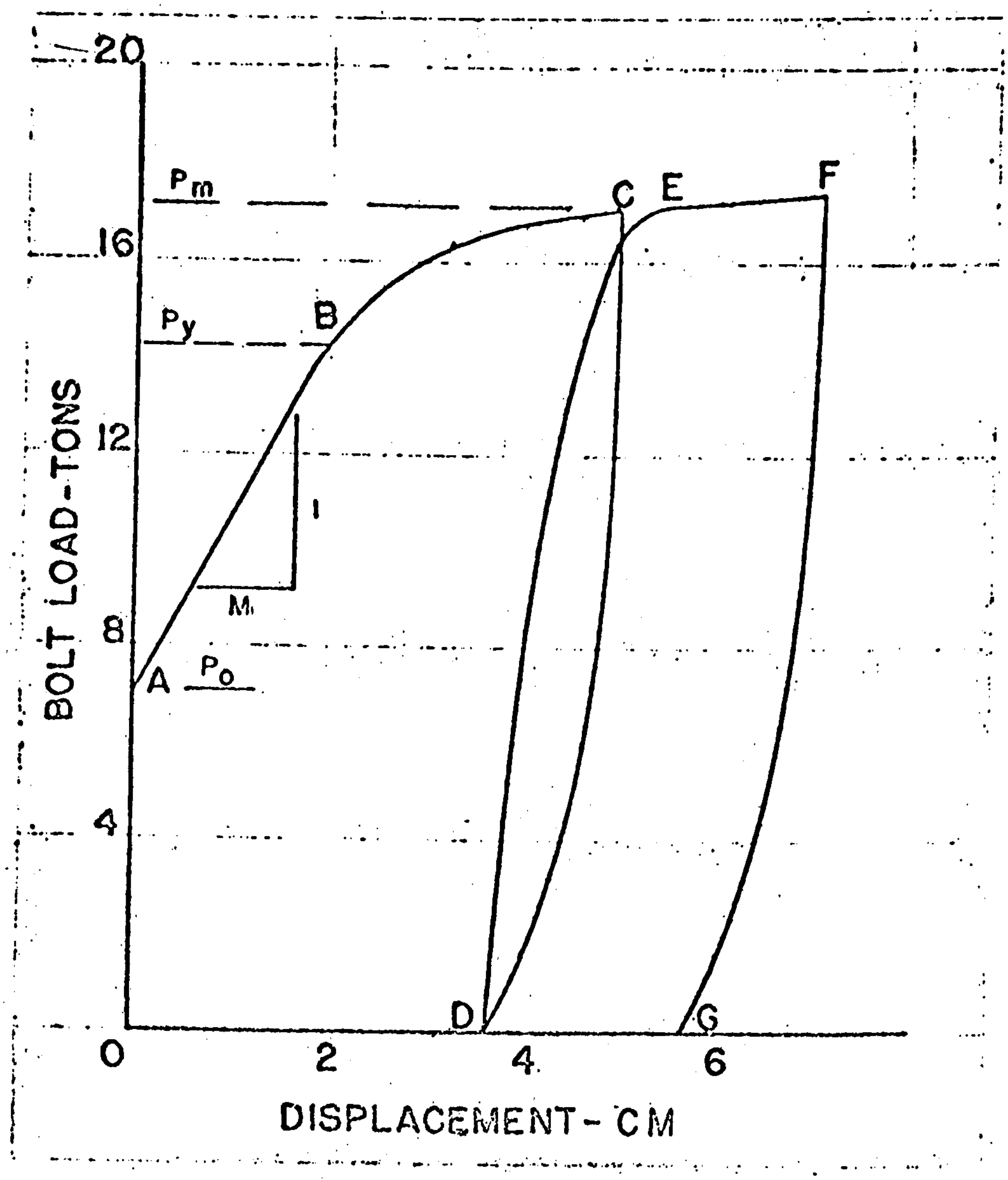


Figure 1. Typical curve for good anchorage.

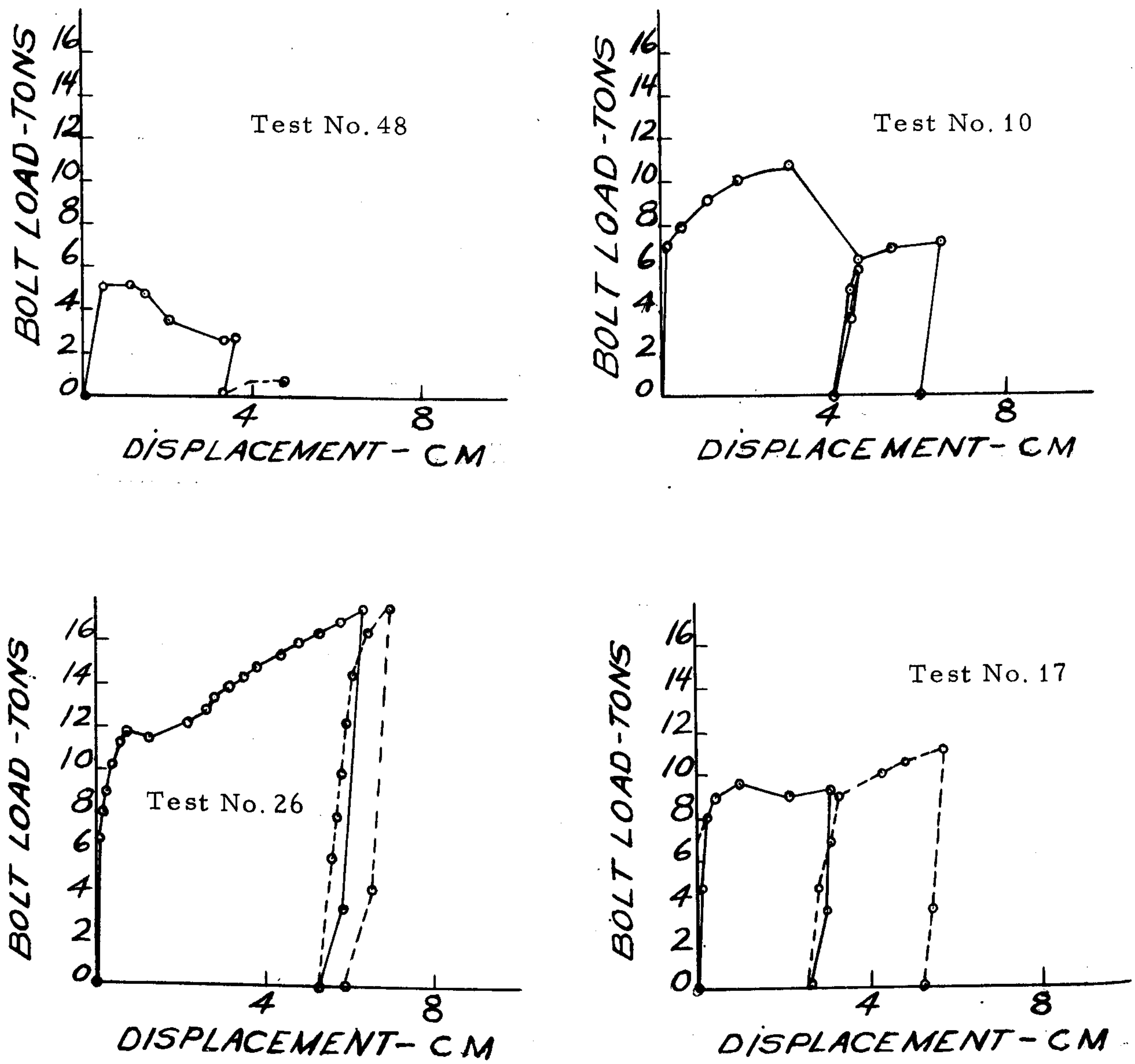


Figure 2. Samples of pull test curves.

Test No. 26 represents a case of good anchorage conditions. The yield load, P_y, was equal to the average yield load of the steel, indicating that the shell/rock capacity was probably greater than that of the steel. On cycling the load after the maximum load of over 17 tons had been reached, some displacement in excess of the elastic stretching of the bolt was obtained; however, this amount was not excessive, and the previous maximum anchorage capacity was obtained.

In Test No. 17 the yield load, P_y, was about 9 tons, indicating that the rock was governing the anchorage capacity. The maximum load, P_m, was little more than the yield load. However, on cycling the load, almost no extra displacement was required to reach the previously obtained maximum load, suggesting that the shell had been well set in the rock and that the system would sustain this capacity.

As an example of a typical anchorage testing program T1 shows the results obtained on one property. Shell types A, B, D and E are conventional shells, whereas C and F are king-size and hence more expensive. The bolts were generally 210 cm (7 ft) long, 19 mm (3/4 in.) in diameter, and installed with a torque of 31 m-kg (225 ft-lb).

These tests showed that shell B had a more satisfactory anchorage modulus, M, than shell C (one of the large, expensive shells), a somewhat lower yield load, P_y, but a substantially equal maximum load, P_m. In comparison with shell F (the other large, expensive shell), whereas the anchorage modulus of B was not as good and the maximum load was somewhat lower, the yield load was somewhat higher. Shell B was selected for production bolting; shell F was stocked for particularly poor roof conditions. Note that in the case of shell B the anchorage moduli indicate that rock does not have a high capacity (the elongation of the steel is only 18% of the total). Furthermore, the coefficient of variation of the dispersion of values of P_y being 27.0% meant that 90% of the bolts would have a yield load equal to or greater than only 5.0 tons, which is 65% of the average value (in the case of shell F, a 90% specification would produce a representative yield load of 58% of the average).

In soft rocks, other things being equal, a large amount of possible expansion is desirable to prevent wedges from being pulled through the shell. T2 shows some measurements that were taken after intallation of a series of test bolts in a second coal mine. After setting the shell in a conventional manner, the bolt was removed from the wedge. The distances from the hole collar to the end of the shell and to the end of the wedge were measured. The bolt was then reinstalled, and the pull test conducted. The superiority of shell K is established by examining the last column in the table showing the amount of expansion of the shell as a per cent of the maximum possible expansion before the wedge would pull through the shell. The key figure here is the maximum per cent that was obtained, which in

TABLE 1. Pull Test Results at One Coal Mine: Averages and Ranges

Shell Type	Anchorage Modulus, in./ton M	Initial Load, tons P_o	Yield Load, tons P_y	Maximum Load, tons P_m	No. of Tests
A-North, various locations					
A	0.0683 (0.006 -0.1426)	3.7 (3.0 - 4.6)	4.4 (3.3 - 5.8)	4.6 (3.3 - 6.3)	4
B	0.0840 (0.0253-0.438)	5.2 (0.56-10.0)	7.8 (0.56-10.0)	11.5 (0.56-18.4)	48
C	0.123 (0.050 -0.200)	6.3 (5.4 - 7.4)	9.1 (7.9 -11.0)	11.8 (9.69-15.25)	4
D	0.130 (0.0287-0.225)	4.8 (3.8 - 6.8)	7.1 (4.65-10.53)	8.6 (5.3 -12.4)	14
E	0.0930 (0.0512-0.200)	6.3 (4.8 - 7.4)	7.8 (6.3 -11.55)	12.4 (8.64-18.40)*	8
F	0.0546 (0.0308-0.1121)	6.1 (4.0 - 8.0)	7.3 (4.0 -11.55)	15.6 (12.63-18.72)	9
A-North, Test Entry No. 1					
B	0.0513 (0.0234-0.0715)	6.7 (5.5 - 7.3)	11.9 (9.0 -13.9)	12.5 (9.0 -13.9)	5
A-West					
B	0.0515 (0.030 -0.068)	5.4 (3.0 - 7.6)	13.4 (10.0 -18.5*)	18.1 (14.0 -23.8)	24
C	0.0504 (0.039 -0.067)	6.1 (5.2 - 7.0)	15.65 (14.3 -16.5*)	19.4 (18.8 -19.7)	4
D	0.0472 (0.048 -0.136)	6.5 (4.7 - 8.1)	9.2 (6.8 -12.75)	12.85 (8.16-16.9)	7
F	0.036 (0.032 -0.042)	7.0 (5.7 - 8.2)	12.5 (11.3 -14.3)	19.3 (17.0 -20.8)	4
No. 1 Seam					
A	0.129 (0.116 -0.143)	2.3 (0 - 4)	3.3 (0 - 5.8)	3.4 (0 - 5.3)	4
B	0.050 (0.038 -0.062)	6.45 (5.39- 3.8)	12.6 (11.55-13.47)	16.0 (13.15-17.88)	5
L	0.053 (0.045 -0.060)	3.7 (3.5 - 4.0)	4.3 (4.0 - 4.9)	10.2 (4.5 -12.6)	4
Balmer North, Site 1					
B	0.076 (0.057 -0.095)	5.2 (3.8 - 6.5)	9.5 (5.2 -13.8)	14.2 (14.2 -14.2)	2
K	0.061 (0.041 -0.080)	5.0 (4.8 - 5.7)	12.5 (11.0 -14.1)	14.7 (14.2 -15.3)	2
H	0.078 (0.077 -0.078)	5.3 (5.0 - 5.6)	14.5 (14.0 -14.9)	15.3 (15.3 -15.3)	2
Balmer North, Site 2					
B	0.031 (0.021 -0.040)	5.7 (5.5 - 6.0)	8+ (6.0 -11.5+)	8.7+ (7.0 -11.5+)	3
J	0.010 (0.007 -0.015)	- -	15	15	4

A = single wedge for 1½-in. hole, B = two-leaf bail for 1 1/4-in. hole, C = two-leaf bail for 1 5/8-in. hole, D = four-leaf bail for 1 1/4-in. hole, E = four-leaf prong for 1 1/4-in. hole, F = four-leaf prong for 1½-in. hole, H = four-leaf prong, J = 1 ft x 1-in. diameter glass resin capsule for 1½-in. hole, K = two-leaf bail for 1 3/8-in. hole, L = three-leaf prong for 1 3/8-in. hole.

*Probably the result of work-hardening due to the re-use of the bolt, and hence the average is not absolutely comparable with other shells.

TABLE 2

Expansion of Shells during Installation at a Second Coal Mine

Shell Type	Wedge Length cm	Wedge Travel cm	Expansion cm	Expansion/Maximum %	No. of Tests
B	8.1	3.6 (3.3-4.6)	0.9 (0.6-1.1)	89 (58-100)	9
G	4.8	2.0 (0.8-2.3)	0.6 (0.5-0.8)	71 (51- 91)	9
K	7.6	1.8 (1.0-2.8)	0.8 (0.7-1.0)	77 (64- 90)	9
L	3.0	3.6 (3.0-4.1)	0.5 (0.4-0.7)	76 (57-100)	12

B = two-leaf bail for 1 1/4-in. hole, G = two-leaf bail for $1\frac{1}{2}$-in. hole, K = two-leaf bail for 1 3/8-in. hole, L = three-leaf prong for 1 3/8-in. hole.

the case of shell K was 90% compared to the higher figures of the others, particularly shells B and L, which in some cases had achieved 100% of their possible expansions during installation. In addition, the ratio of expansion to wedge travel can be accepted as being a key parameter for the setting of shells in weak rock, which for shell K is greater than for the others. The results of removing the bolts from the wedges after the pull tests were completed, and measuring the movements of the shells and wedges from their position on installation, showed that relatively small movements, particularly at the shell/rock interface, occurred for shell K.

It cannot be over-emphasized that the minimum possible hole size with respect to the diameter of the anchor shell will provide the best anchorage conditions. In other words, the amount of expansion required to set the shell in the rock should be kept to a minimum, as otherwise the maximum possible expansion of the shell may not be sufficient.

In very weak rocks, mechanical anchorages may not be adequate. It has been found necessary, in such cases, to use either some type of grouted anchor or another type of support. Indeed, in some cases it was found that bolting was not feasible because of the immediate falls on exposure and the danger of falls during installation, to say nothing of irregular surfaces for collaring holes and for setting bearing plates.

It has been found difficult to obtain uniform installation tensions in all bolts, even when an uneconomic procedure like hand-tightening is used. Figure 3 gives the results of such a study. The variation in tensile load produced by torque, and conversely the amount of torque that is used in scoring the bearing plate, is undoubtedly affected by the rock conditions at the collar of the hole. Consequently, the bearing plate size and thickness are important, particularly where either anchorage capacity is marginal, hole collars are irregular, or the bearing capacity of the immediate roof is low. Without uniformity of installation load and anchorage modulus, the bolts will not necessarily share the rock loads equally, which should be reflected in the design of the system.

WORKING CONDITIONS

Anchorage capacity can deteriorate with time. The rock properties may change owing to ground water conditions or to the reaction to stress; however, the most probable cause of such deterioration is deflection of the roof accompanied by some expansion (e. g. , bed separation in stratified rock). This expansion may increase with the passage of time (or with the distance from the face), but it is greatly affected by the mining of adjacent openings, particularly when these intersect the first opening

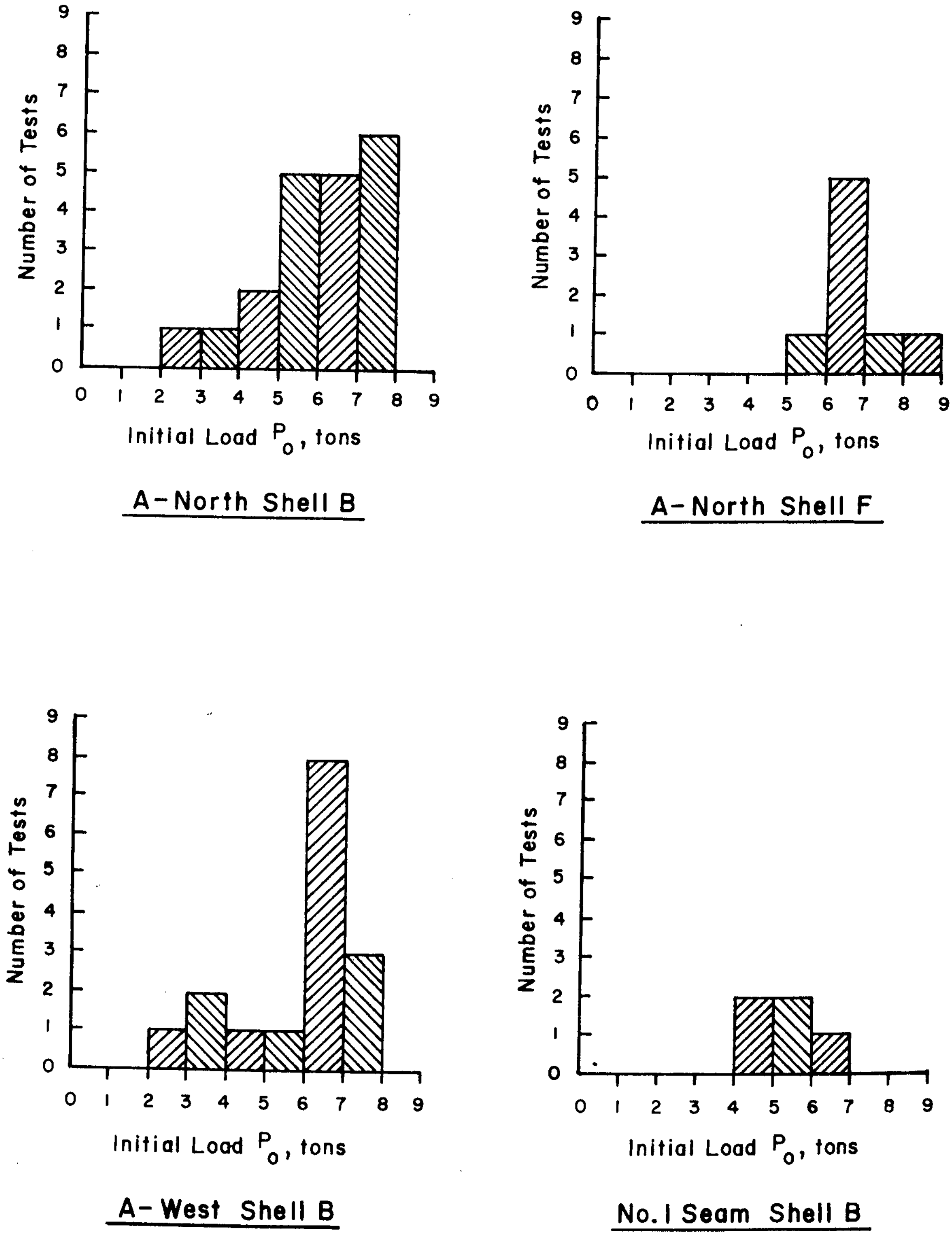


Figure 3. Histograms of installed load with 31 m-kg of torque in one coal mine.

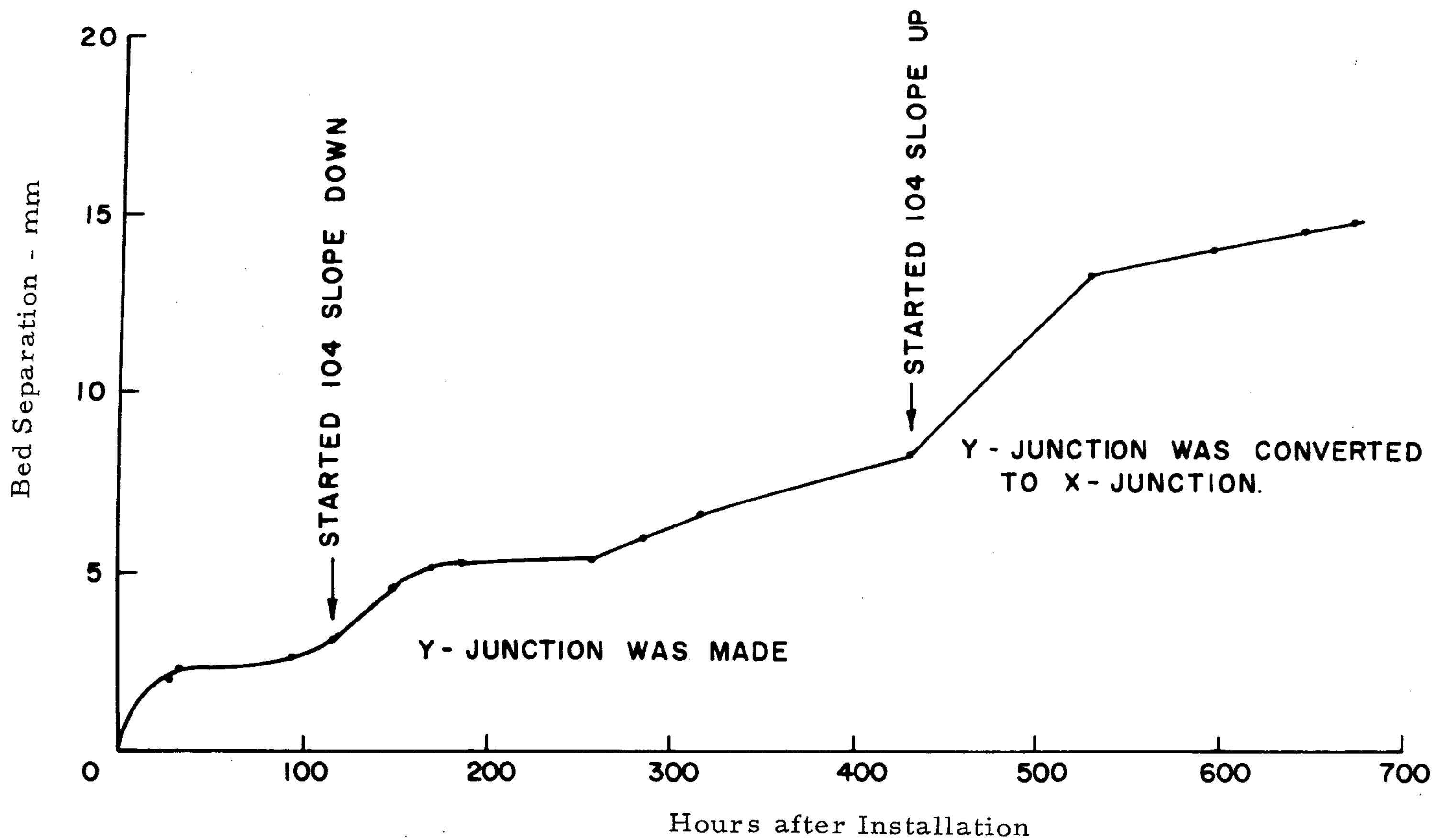


Figure 4. Effects on bed separation of driving a slope into one side of an entry and continuing on the other side.

(X-intersections being worse than Y-intersections, as shown by some typical measurements in Figure 4). The greatest effect results when the adjacent mining is a pillar-recovery operation. Depending on the cause of deterioration, in many circumstances it has been found advisable to reinforce the support by installing additional rock bolts before severe deflections of the ground are induced.

Measurements have shown that critical amounts of rock expansion can be established that herald the onset of additional expansion, followed by the deterioration of rock conditions and the bolt anchorages -- for example, in two mines, 6 mm (0.25 in.) of expansion over the first 210 cm (7 ft) into the roof for entries with a 5-m (16-ft) breadth was the critical amount. Such critical deflections can be easily monitored with a rock bolt hanging freely in a metal collar wrapped with an appropriate width of reflective tape so that on disappearance of the reflective tape into the hole it can be seen that the critical deflection has occurred.

In addition, whereas the absolute capacity of the anchorage, is important, in some cases it was found that the deflection that is permitted, or required, by this capacity was too great for maintaining the integrity of the rock mass; consequently, the importance of the deflection characteristics of the installed rock bolt should also be recognized.

In weak rock and/or with compressible pillars, many operating details become critical. For example, it was found in one mine that bolting within a $\frac{1}{2}$-hour after exposure of the roof was important as it inhibited damaging deflections. In other mines a longer time could be used. Also, in marginal ground it has been found that the load obtained on the bolt on installation is substantially the maximum load that the bolt will sustain.

It has also been found in many studies, as has been the experience of others, that a rough correlation exists between the amount of torque that is sustained in installing the bolt and the tension that is produced in the bolt. Retorquing after a period of time can be beneficial, because it can reset bolts that, for one reason or another, have become loosened at their anchorage and because it can also detect where anchorage has deteriorated so that the bolts can no longer sustain load. In this regard, whereas it has been found that bolting can be effective in salt mines, even though there is a relaxation of load with time, some asymptotic, finite value is approached; retorquing seems to decrease this relaxation and to increase the asymptotic value that the bolt load will approach as shown in Figure 5.

With regard to the deterioration of anchorage conditions and its detection, pull tests have shown, as mentioned above, that the yield and maximum loads obtained initially may not be sustained; after a second cycle of loading, the ultimate capacity of the bolt can be much lower. Also this

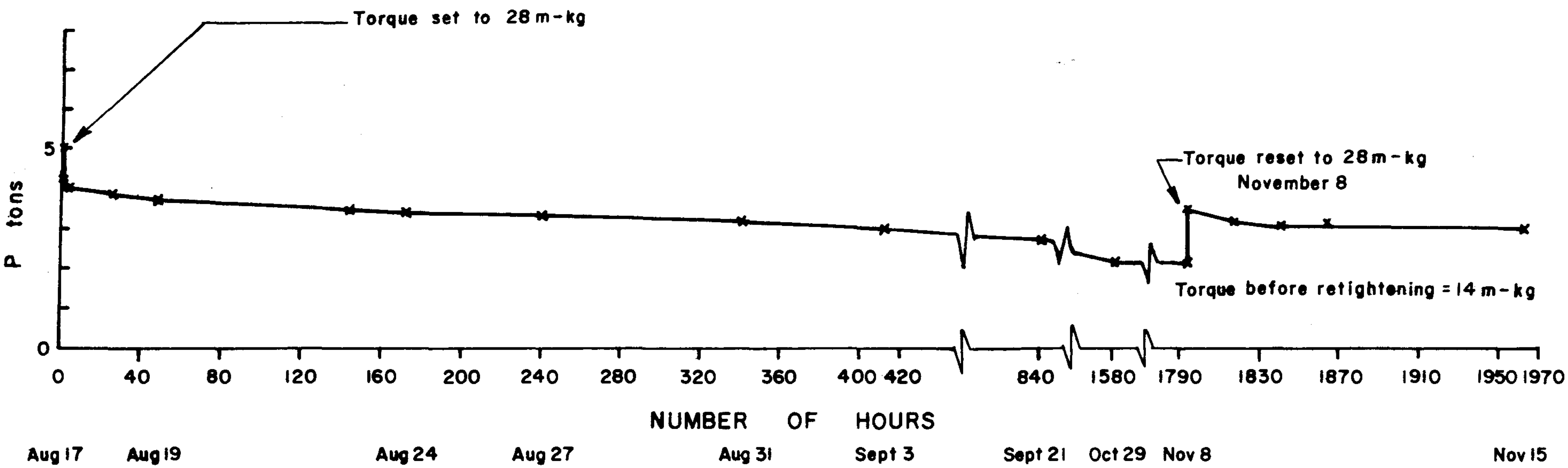


Figure 5. Variation of bolt load with time and the effects of retorquing in salt.

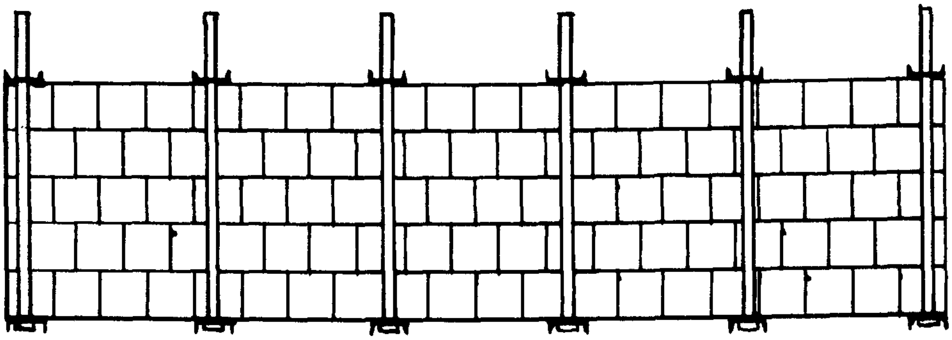
reaction can occur without the damaging effect of increased closure.

Several factors have been observed that are likely to affect the design of bolt systems. One of these factors that has been investigated is the nominal span of an opening. It is often smaller than the actual span to the points in the pillars where support is being provided to the roof or walls. This means that deflections will be greater than would be predicted using only the nominal breadth of opening.

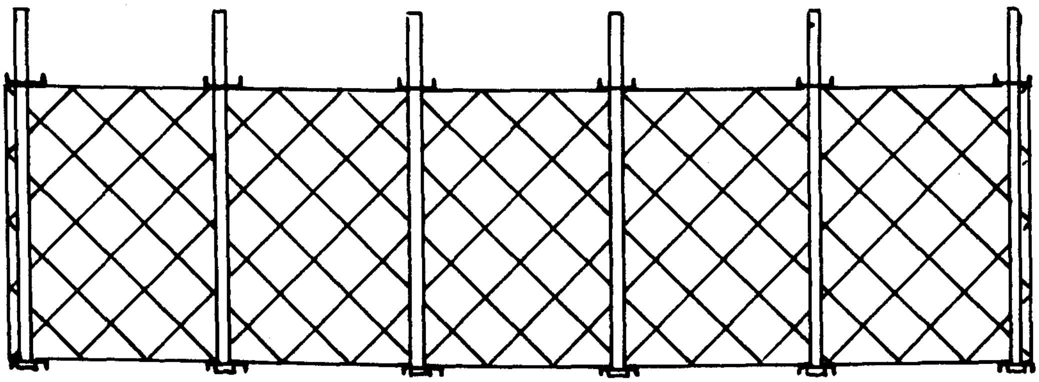
It seems clear from tests that, where area bolting is required, the spacing of the bolts should not exceed their length. Besides observations underground, some model tests were conducted on the influence of bolt spacing on block models. Beams with a span of 63 cm, constructed with 2.5-cm-square by 15-cm-long plaster blocks, were stabilized by vertical transverse bolts acting on the blocks through spring-loaded yokes as shown in Fig. 6. A concentrated load was applied at the centreline of the beam, and the resultant deflection was measured. The results of these tests showed that the load deflection relationship was linear up to some critical load, at which load individual blocks fell from the bottom of the beam between the bolts.

When the spacing of the bolts in the models exceeded their length, a stable beam could not be constructed. With bolt spacing up to as much as 5 times the size of the individual blocks, the beams could be made stable; however, when the blocks were made of hardwood and plexiglass, the beams were somewhat unstable at spacings greater than 3 times the block size. Beam deflections at equal load were found to be proportional to the spacing of the bolts and to vary inversely with the load on the bolts. No significant difference in behaviour was obtained when the blocks were arranged with vertical or 45° joints.

In view of the numerous variables that can be encountered in marginal conditions, monitoring was found to be useful for confirming or modifying the design of a system. This conclusion was reinforced by attempts to relate geological observations on rock structure, and miners' observations on ground conditions, with the ultimate conditions experienced, both of which were found to provide very poor bases for prediction. For monitoring purposes, it has been found that accurate bolt loads can be measured with vibrating wire dynamometers, less accurate measures of load can be obtained with the rubber pad dynamometers, and torque wrenches can be used to test for the present of load or no-load. Of the various techniques tried, it was found that the most useful instruments for monitoring are at the present time the rubber pad dynamometers, borehole extensometers, automatic closure rods, and torque wrenches.



(a). Arrangement used for the first series of tests.



(b). Arrangement used for the second series of tests.

Figure 6. Models of rock bolting.

TENTATIVE DESIGN SPECIFICATIONS

CONCEPTS

The following tentative specification has been drafted to provide guidance for design engineers. Some of the clauses are based on research, others are based on judgement.

Rather than attempt to provide a comprehensive design specification for all types of bolting, the common case of temporary roof bolting was selected (what is temporary being a matter of judgement). This means that the other cases --where bolting is to be used for permanent roof support, or in walls of shafts or stopes, or around drawpoints, or combined with other elements such as mesh and guniting--are not covered by this report. However, some guidance will be provided for these other cases by the concepts and quantitative relationships that are valid for temporary roof bolting.

One of the key phrases in the design specification is that bolting is to be used "where necessary" (see para. 1). Some areas may be bolted where area support is not required, in which case the observation that the clauses of the tentative specification are unduly conservative would not be a valid criticism of the specifications but of the judgement in deciding what areas required bolting.

Other criticisms might occur to the casual reader. Also, it might be thought that the specifications should deal more specifically with types of hardware; however, with continuous new developments, it is more important to specify performance (e.g. paras. 7 and 9). (It may seem that some phrases are repeated unnecessarily; however, in specification writing, clarity has a higher value than good prose.)

For specifying bolt material and tolerances, several standard specifications are already available for use in construction and manufacturing that can be usefully adapted until possibly more appropriate regulations can be provided.

In the following tentative specification there is a purpose for each clause, based on common or assumed conditions. If these conditions can be shown not to exist in a particular case, then the design can always be based, rather than on the specification, on the more than normal detailed information; similarly, a more detailed analysis of the structural action can be the basis for deviating from these clauses.

It might be questioned whether the design sequence is appropriate or logical. Actually, the sequence is not important, because the design is required to fulfil several criteria, which is common in structural engineering. The use of the traditional 'cut and try' procedure as opposed to solving an algebraic equation for a single answer is thus appropriate. Certainly spacing, s, based on joint spacing could be determined, and then the length, L, calculated. However, the reverse procedure, determining L first, has been suggested because a dimension such as span of the opening is more definite than the representative joint spacing in the rock and hence is a firmer starting point.

In spacing of bolts there will be a minimum practical spacing for each site; however, such a feature should not require specification, although possibly a figure of 2 ft might be used.

It is recognized that the relationship between torque on installation and tension in the bolt is very crude. Nevertheless, for the vast majority of bolts there will be a relationship between these two factors, as shown by numerous studies in the U.S.A., U.K., and Canada that have found surprisingly similar correlation constants. Therefore, torque can be used as a very crude measure of tension. At the very least, it can indicate the maximum possible tension that exists, which may be too low. (The concept is similar to the limited value accorded testing of the rock substance; it provides an upper limit to the possible strength of the rock mass.) In addition, some control is required on the applied torque to avoid damaging the steel.

In spite of the formulation of this design specification, it is important to retain the concept that our information on this subject is inadequate and will probably always be so in view of the fact that we are dealing with a variable, geologic material. Consequently, the procedure of "design as you go" or at least "confirm as you go" should be used. For this reason, the section on monitoring has been included.

It is conceivable that it could be found after installation that some of the parameters are different than assumed. For example, the representative joint spacing may be less than assumed in the design, which may require modification of the design. The tensions of the bolts may be less than specified and if failure does not occur, it might be concluded that the specifications are unnecessary. This is a bit like saying that a roof beam in a building was stressed inadvertently to more than the design stresses without failure and hence the specified permissible stresses were not really required. The correct conclusions in such a case might be either that the safety factor has been reduced or that area support in this opening was incorrectly assumed to be necessary.

Finally, it is recognised that the proposed tentative specification can be improved. It will be, therefore, important to analyse observations and criticisms from engineers dealing with a wide variety of conditions.

SCOPE

1. This specification describes the procedures to be followed in selecting roof bolts, in designing patterns, and in monitoring various aspects of the ground reaction. The purpose of the bolting is to provide, where necessary, support for areas of roofs, or backs, for the limited period of time common in construction before permanent linings are placed and common in mining stopes and drifts within mining blocks; consequently, it does not cover the use of bolts for such purposes as in drawpoints, walls of shafts, and permanent installations. It deals only with the conventional, end-anchored rock bolts installed in a selected site; special conditions, such as faults, require special consideration.

BOLT SELECTION

GENERAL

2. The bolt assemblage shall be explicitly selected for the rock, other environmental factors, and the installation method. The materials, fabrication, and delivery conditions shall be specified. The rock shall be classified.

ROCK CLASSIFICATION

3. Rock Substance. The rock substance is the predominant material in a formation and, as such, does not include the structural features making up the rock mass. The substance shall be classified with a simple field geological name where this can be easily determined (e.g. granite, gabbro, limestone, shale); otherwise, it shall be simply called 'rock'. It shall be classified either as 'strong' if the uniaxial compressive strength is greater than 700ksc(kg/cm^2) (10,000 psi) or as 'weak'. It shall be classified either as 'elastic' if on a laboratory specimen the permanent strain is less than 25% of the total strain and the creep rate is less than 2×10^{-6} cm/cm(in./in.) per hour, or otherwise as 'yielding'. The tests for determining these properties are contained in Appendix A. The first part of the classification is then:

Substance: Geological Name (if possible)
Strong or Weak
Elastic or Yielding

The properties of the rock substance provide an upper limit to the range of possible properties of the rock mass.

4. Rock Mass. The continuity of the formation shall be described as 'massive' when the distance between layers and joints is more than 2 m (6 ft); 'layered' when the spacing between layers is less than 2 m (6 ft) (layering in a mechanical sense exists where bonding between layers is much less than within any one layer); 'blocky' when the joint spacing is less than 2 m (6 ft) and greater than 30 cm (1 ft); and 'broken' when the joint spacing is less than 30 cm (1 ft). (Joints are planar discontinuities other than layers or faults where the bonding is much less than within the rock substance, i.e. the surfaces are easily separated when free from constraint.) The second part of the classification is then:

Rock Mass: Massive, Layered, Blocky, Broken.

In addition to classifying the rock, it may be necessary to obtain detailed engineering information on such additional structural features as faults, altered zones, the nature of joint infilling material, the permeability of the formation, and the nature of the ground water.

BOLTS

5. The steel to be used in bolts, their fabrication including permissible deviations from specified requirements and delivery conditions shall be specified. (Although it has been more common to purchase according to supplier's proprietary standards, one or more of the following Canadian Standards Association, 178 Rexdale Blvd, Toronto 603, Specifications may be suitable: G40.1 - 1966 - General Requirements for Delivery of Rolled Steel Plates, Shapes, Sheet Piling, and Bars for Structural Use; G40.4 - 1959 - Medium Structural Steel; G40.12 - 1964 - General Purpose Structural Steel; G40.14 - 1969 - High Strength Carbon Structural Steel.)

6. Capacity. The load capacity of the steel in the bolt, Q_s, shall be equal to the specified minimum stress at the yield point of the steel multiplied by the minimum net cross-sectional area of the bolt.

ANCHORAGE

7. General. Except where previous experience can be used to select appropriate anchorage types, pull tests shall be conducted to determine the anchorage capacity of the bolt/rock system.

8. Capacity. A minimum of ten pull tests as described in Appendix B shall be conducted to establish the capacity of the anchor in any

one rock classification. The criteria for describing the representative values for the bolt/rock system are that the yield points of 90% of the tests shall be greater than the representative yield load, Q_y, of the system and that the ultimate loads of 90% of the tests shall be greater than the representative ultimate load, Q_m, of the system. The representative load capacity of the anchorage, Q_r, shall be either Q_y or $0.7\,Q_m$ whichever is the lesser.

BEARING PLATES

9. Bearing plates shall conform to the test requirements contained in Appendix C. The bearing plates fulfilling this test requirement shall be for bolts in rock where 90% of the diameters of the collars of the holes, D, shall be equal to or less than 10 cm (4 in.) (i. e. assuming the ravelled hole diameter at the collar is greater than the drilled hole diameter).

10. Where the representative collar diameter, D, (90% of the collars are equal to or less than this diameter) is greater than 10 cm (4 in.), the side of a square bearing plate shall be (D + 5) cm or (D + 2) in.: bearing plates having other shapes shall have equivalent bending capacity. The thickness of the bearing plate, t, shall be related to the thickness of an acceptable washer for 10-cm (4 in.)-diameter collar holes, t_4, by the following formula:

$$(t/t_4) \geq (D/D_4)^{4/3}$$

where D is the width of the plate and D_4 is 10 cm (4 in.). In particularly good rock where the diameter of the collar of the hole is consistently less than 10 cm (4 in.), the formula may also be used to determine the appropriate reduced size of the plate.

DESIGN

GENERAL

11. If support is required for the roof, or back, and rock bolting is to be used, then the following design requirements shall be fulfilled unless detailed testing and analysis can show that such requirements are not necessary in specific locations.

12. The load capacity of the roof bolt, Q_a, shall be equal to either the load capacity of the steel, Q_s, or the load capacity of the anchorage, Q_r, whichever is less.

LENGTH OF BOLT

13. The minimum length of bolt, L, shall be either 1 m (4 ft) (to avoid fractured ground adjacent to the free surface) or more than the depth of the rock block containing the collar of the hole (to anchor beyond the block), whichever is greater. The maximum length of bolt (if not governed by the height of the opening) shall be determined from the following equation:

$$L \leq Q_a/(s^2 g)$$

where Q_a is the capacity of the roof bolt, s is the average spacing of the bolts, and g is the average rock density in the formation. Where it is not possible to install the bolt with a tensile load equal to or greater than $0.5\ Q_a$ as specified in para. 15, the maximum length of bolt shall be:

$$L \leq Q_a/(2s^2 g)$$

SPACING

14. The maximum bolt spacing shall be three times the representative spacing of the joints, layers, or fractures (whereby 90% of the actual spacing is greater than this figure) unless a membrane - e. g., lagging, wiremesh, or gunite -- is provided between the bolts, in which case the maximum spacing would depend on the type of membrane. The maximum spacing shall also be limited to either 0.9 L where L (the length of the bolt) is greater than 1/4 of the breadth or span of the opening, or 0.5 L where L is less than 1/4 of the span of the opening. The span of the opening at an intersection with another opening shall be the diagonal distance between surrounding abutments.

TENSION

15. The bolts shall be tensioned on installation to a load between $0.5Q_a$ and $0.8Q_a$ (to avoid some of the dynamic loading that could occur on detachment of blocks from the formation that lower pre-loading would permit, to reduce differential loading between adjacent bolts that lower and more variable pre-loading could produce, and to avoid yielding in the steel or rock as a result of combined torque and tension that higher pre-loading could cause). If the bolt cannot be installed with the tension equal to or greater than $0.5Q_a$, the design must be modified as specified in para. 13.

TORQUE

16. Where the installed tension is created by torquing action, the

maximum torque, T, shall be calculated according to the following formula:

$$T \leq 0.8\, r^3 \sigma_y$$

where r is the radius of the bolt at the minimum section and σ_y is the specified minimum stress at the yield point of the steel in tension.

MONITORING

GENERAL

17. If the rock is classified as yielding, or there is good reason for suspecting that the design bases may change with time (a few weeks or months), or the excavation of adjacent openings can be expected to affect the roof rock, monitoring shall be conducted to determine either that design conditions are being maintained or that the design should be changed to be appropriate to the new conditions.

TORQUE

18. A torque wrench shall be used to determine, on a sampling of bolts, if the minimum torque required to rotate the nut is less than that used in installing the bolt. If the torque is found on one bolt to be less, this condition will be considered as an indication of a reduction in tension on the bolt (hence para. 19 must be considered) and a possible reduction in anchorage capacity (hence para. 21 must be considered).

TENSION

19. Where there is a definite possibility of the tension in the bolts decreasing significantly with time, bolt loads shall be monitored either by dynamometers, by using a jack to determine the load, or by conducting complete pull tests (see Appendix B). If 10% of the bolts sampled are shown to have a tension less than $0.5Q_a$, either the tensions shall be increased on all deficient bolts or the number of bolts shall be increased to fulfil paras. 13 and 14.

SPACING

20. If it is found that either the spacing of joints has decreased or the effective span is greater than that assumed in para. 14, the number of bolts shall be increased to fulfil para. 14.

CAPACITY

21. If the rock is classified as yielding; if the torque required

for rotation, when tested after installation, is less than specified in para. 16; if the tension is less than specified in para. 15; if the joint spacing is less than originally assumed for the purpose of para. 14; if serious corrosion seems to have occurred; if the average strain in the roof rock is greater than the expected elastic strain; or if the effective span is shown to be greater than the span assumed in para. 14; then the bolt capacity, Q_a, shall be checked with pull tests, using a sampling frequency of at least 10%.

DEFORMATION

22. Where it is necessary either to determine the average strain in the roof to compare with the expected elastic strain, or to determine the effective span of the opening to compare with the nominal span, borehole extensometers shall be installed in the roof at the centreline of the opening and at the abutments with anchorage points into the roof a distance equal to approximately 0.3 times the span of the opening, so that any damaging inelastic deformation can be detected. Such stations shall be spaced along the opening, at distances of approximately 2 to 3 times the span of the opening, throughout the zone that is to be examined. In addition, such stations are to include closure measurements between the roof and floor adjacent to the abutments of the span.

ACKNOWLEDGEMENTS

The contributors to this work are not only those whose names appear in the bibliography of Mines Branch reports but also the staff members over the years who have supported these authors. The U.S. Bureau of Mines, particularly through its Roof Control Research Group, Health and Safety Research and Testing Center, Pittsburgh, has always been a helpful collaborator. In addition, many mining companies have provided helpful criticisms of the draft specifications.

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APPENDIX A

CLASSIFICATION UNIAXIAL COMPRESSION TEST

1. Prepare a suite of 10 samples. A standard length/diameter ratio is 2/1, but a ratio down to a minimum of 1/1 can be used. Specimens are to be roller lapped, if necessary, so that the maximum difference in diameter is less than 0.025 mm (0.001 in.). The ends of the samples are to be lapped on a wheel so that they are parallel within 0.025 mm (0.001 in.). After lapping, allow the samples to dry at room temperature for at least 24 hr. Measure the samples to 0.025 mm (0.001 in.) at 3 points for length and at 3 points for diameter. Weigh samples to the nearest 0.01 gram.

2. On one sample, measure longitudinal strain either with two strain gauges cemented at the mid-height of the sample and on opposite sides or with a compressometer that measures the change in length over a 1-in. gauge length. Apply the load at the rate of 35-70 ksc/sec (500 to 1000 psi/sec) until failure occurs. If an X-Y recorder is not used, record load and strain, or deformation, at load intervals of approximately 140-280 ksc (2000 to 4000 psi). Record the maximum load and the duration of the test. Describe qualitatively the type of failure as indicated by the noise produced, e.g. very violent, violent, or quiet. Describe the orientation of the fractures, e.g. top cone, bottom cone, longitudinal, diagonal, or irregular; describe the fragment size, e.g. powdered, highly fragmented, quarter-inch slivers. Where possible determine the fracture angle (measured from the horizontal).

3. Calculate the uniaxial compressive strength, Q_u, based on the original cross-sectional area. Calculate the modulus of deformation from the slope of the stress-strain curve at the stress level equal to 0.5 Q_u.

4. On a second sample, measuring strain as in item 2 and recording strain readings either continuously or at every increment of 1/5 of the load during the loading cycle, increase the load to approximately 0.5 Q_u. Maintain the load constant for either 1 hr or until the strain rate is less than 2 microstrain per 10 min whichever is the lesser.

5. Unload the sample as quickly as possible, and then maintain the sample at zero stress until the strain rate is less than 2 microstrain per 10 min or for a period of 1 hr, whichever is the lesser.

(Appendix A, concluded)

6. Re-apply the load to 0.5 Q_u, read the strain, then unload and read the strain.

7. Where the ultimate strain rate is less than 2 microstrain per 10 min, calculate the strain rate for 0.5 Q_u at 1 hr on the first loading cycle, and calculate the ratio of permanent strain after unloading to total strain at 0.5 Q_u for both loading cycles. If results are judged to be unrepresentative, test another sample.

8. Test the remaining 8 samples as in item 2 but without strain readings. For the suite of 10 samples, calculate mean values and standard deviations of strength.

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APPENDIX B

PULL TESTS FOR ANCHORAGE CAPACITY

1. After drilling the hole to the layer of rock to be tested, measure the inside diameter of the hole at the anchorage depth (to compare with the diameter of the drill bits, to provide guidance on the selection of anchor and hole diameters, and for explaining low test results).

2. Install the bolt in the manner used during operations and pre-load it to the load that will be used during operations, recording torque and/or tension together with the date and time, type of drilling (wet or dry), type of anchor and bolt, and location.

3. Install the loading jack and apply an initial load of 500-kg (1000-lb), recording the date and time of test.

4. Set the dial gauge (with divisions of at least 0.025 mm or 0.001 in.) on the pull rod of the jack.

5. Increase the load on the jack in 500-kg (1000-lb), increments, reading the movement of the pull rod for each increment.

6. Continue increasing the load until deformation of the pull rod occurs with little or not additional load; release the load down to the initial load of 500-kg (1000-lb) and read the dial gauge.

7. Apply in one increment the maximum load previously applied, and read the dial gauge.

8. Release the load to the initial load of 500-kg (1000-lb), and read the dial gauge.

9. (a) Determine the yield load, P_y, either from an obvious knee in the load-deformation curve, or from the intersection of the load-deformation curve and a line drawn from the origin at a slope of 12.5 mm/ton (0.05 in./ton), whichever is the lesser.

(b) Determine the ultime load of the anchorage, P_m, as either the load which causes continuous deformation with little additional load or the load which causes complete breakdown of the anchorage.

(Appendix B, concluded)

(c) The anchorage modulus, M, shall be calculated as the representative slope of the load-deformation curve in cm or in./ton between the load that causes measurable deformation and the yield load. Calculate M_s, the amount of stretch in the steel of the bolt per ton of load.

(d) The installed load, P_o, shall be estimated by extrapolating back from the yield load along the representative slope of the load-deformation curve, if this can be reasonably established, to the y-axis.

(e) The reverse deformation that occurs when the load is released initially and after cycling shall be expressed as a ratio of the total deformation at the maximum load and the increment of deformation occurring on cycling of load respectively (to provide information for appraising the behaviour of the anchorage system).

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APPENDIX C

BEARING PLATE TEST

(Draft specification of the American Mining Congress; from Mining Congress Journal, October 1965, Vol. 51, No. 10, p. 40, "Standards for Roof Bolt Plates and Drill Bits" by Frank L. Gaddy, and from information supplied by J. A. McCormick, Roof Control Research Group, Health and Safety Research and Testing Center, U.S. Bureau of Mines, Pittsburgh)

1. The bearing plate shall be placed on a steel platen which contains a 10-cm (4-in.) diameter hole, and shall then be loaded with a 44.5-mm (1 3/4-in.)-diameter plunger to an initial load of 2700 kg (6000 lb).

2. The load shall be increased in increments of approximately 500-kg (1000 lb) up to a maximum load of 7000-kg (15,000 lb), observing load and deflection of the plate assuming zero deflection at the initial load of 2700-kg or 6000-lb), unless an X-Y recorder is used, in which case the loading can be continuous.

3. The plate is considered acceptable if the maximum increase in deflection is 3.0 mm (0.12 in.).

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DFC:TSC:(PES)mms