PIT SLOPE MANUAL

chapter 6

MECHANICAL SUPPORT

This chapter has been prepared as part of the

PIT SLOPE PROJECT

of the

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THE PIT SLOPE MANUAL

The Pit Slope Manual consists of ten chapters, published separately. Most chapters have supplements, also published separately. The ten chapters are:

1. Summary
2. Structural Geology
3. Mechanical Properties
4. Groundwater
5. Design
6. Mechanical Support
7. Perimeter Blasting
8. Monitoring
9. Waste Embankments
10. Environmental Planning

The chapters and supplements can be obtained from the Publications Distribution Office, CANMET, Energy, Mines and Resources Canada, 555 Booth Street, Ottawa, Ontario, K1A 0G1, Canada.

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FOREWORD

Open pit mining accounts for some 70% of Canada's ore production. With the expansion of coal and tar sands operations, open pit mining will continue to increase in importance to the mineral industry. Recognizing this, CANMET embarked on a major project to produce the Pit Slope Manual, which is expected to bring substantial benefits in mining efficiency through improved slope design.

Strong interest in the project has been shown throughout its progress both in Canada and in other countries. Indeed, many of the results of the project are already being used in mine design. However, it is recognized that publication of the manual alone is not enough. Help is needed to assist engineers and planners to adopt the procedures described in the manual. This need for technology transfer will be met by a series of workshops for mine staff. These workshops will be held in various mining centres during the period 1977-81 following publication of the manual.

A noteworthy feature of the project has been its cooperative nature. Most organizations and individuals concerned with open pit planning in the country have made a contribution to the manual. It has been financed jointly by industry and the federal government.

Credit must be given to the core of staff who pursued with considerable personal devotion throughout the five-year period the objectives of the work from beginning to end. Their reward lies in knowing that they have completed a difficult job and, perhaps, in being named here: M. Gyenge, G. Herget, G. Larocque, R. Sage and M. Service.

D.F. Coates
Director-General
Canada Centre for Mineral and
Energy Technology

SUMMARY

Mechanical support stabilizes pit slopes by increasing rock strength. However, the increase achieved is relatively small. Support therefore is usually used only if a slope shows signs of instability during mining. Support in this case may increase strength enough to ensure long-term stability.

It may occasionally be cheaper to design a slope with support rather than excavate to a naturally stable slope. An example would be a clearly defined ore contact which might be a natural mining boundary except for potential sliding on bedding planes. However, in such a case the stability investigation and analysis must be very accurate. The degree of instability must be clearly known before support can be justified.

METHODS OF SUPPORT

Rock Anchors

Most open pit slides are caused by slip on planes of weakness such as joint sets and bedding planes. The resistance to sliding can be increased by compressive forces acting across these planes. This can be done by installing rock anchors through the planes of weakness.

A typical rock anchor consists of a steel cable inside a borehole. The cable is anchored by cement grout in the bottom of the hole. At the surface an anchorage is formed by a concrete bearing block and a metal bearing plate. A steel anchor block with wedges secures the cable after it is tensioned. The borehole is subsequently filled with grout to bond the cable permanently. Solid bars can also be used.

The tension in a rock anchor is usually from 30-300 tons (300-3000 kN), though anchors up to 2000 tons (20,000 kN) have been used.

The techniques and materials of rock anchors were developed by the prestressed concrete industry. Steel must be of very high strength and requires care in handling. It should not be kinked or permanently bent, nor pitted by rust - the pits can become points of stress concentration leading to failure when tensioned. Cutting must be by carborundum wheel; the heat from flame cutting may cause loss of strength.

The installation of a rock anchor requires care. Large forces are used in jacking and broken cables may fly. Precautions to protect personnel

are necessary.

Rock Anchor Corrosion

The high tensile stresses involved in rock anchors both aggravate the effects of rusting and introduce the brittle failure problems of stress corrosion. This is a poorly understood phenomenon that occurs only at relatively high stresses and results in internal cracking and sudden failure.

The best form of corrosion protection is to fully grout the anchor after tensioning. However, if the anchor load must be monitored grouting cannot be used and greasing or sheathing the anchor is necessary.

Shotcrete

Shotcrete is a surface layer of concrete applied by spraying a mixture of aggregate, sand, cement, water and a flash-set additive. The shotcrete effectively sets on impact. There is some loss of material through rebound from the face.

Shotcrete is used to retain loose surface rock and prevent progressive deterioration. It can provide shear resistance across joint boundaries.

Reinforced shotcrete has recently been developed. Chopped steel wire is added to the mix, increasing the tensile strength. Drawbacks are cost, and difficulty in handling.

A disadvantage in using shotcrete is the risk of sealing the slope face and allowing build-up of groundwater with a possible threat to stability. Where necessary, drain outlets must be installed. Another disadvantage is that shotcrete can only be applied at temperatures above freezing.

Buttresses

Buttresses stabilize by adding dead weight to counteract a tendency to slide. They have been used in Canada to stabilize pit slopes but are not usually a suitable support method.

DESIGN

Support design has three steps:

a. identifying instabilities and collecting relevant data on geology, mechanical properties and groundwater,
b. engineering analysis, and
c. economic analysis.

The analyses of the design chapter are used to determine the support force required to achieve the desired wall reliability. Once the required support force is known, economic appraisals are made. Both wall reliability and the cost of support depend on the layout of rock anchors, particularly on the angle of inclination. However, reliability is relatively insensitive to anchor inclination and the least cost layout can usually be chosen.

MONITORING

Load cells can be used to monitor anchor behaviour directly. The cell is placed between the anchor block and bearing plate and thus transmits the full anchor load to the rock surface.

Load cells can be hydraulic or mechanical. In the former, fluid pressure within the cell is measured to calculate the load. The latter has gauges to measure the strain in a steel cylinder previously calibrated to correlate strain and load.

A monitored rock anchor cannot be grouted. Corrosion protection must be provided by greasing and sheathing the cable.

A monitoring technique which is not recommended is the "lift-off" test, in which the anchor is rejacked to check the tension. The disturbance to the anchor is most undesirable.

COSTS

Estimating support cost is difficult because of lack of large-scale experience in mining. Experience in civil engineering is usually not applicable because of differing purposes and standards.

Based on field trials and experience, rock anchor support costs in 1974 dollars are in the order of \$2.50/ft^2 (27/m^2) per sq ft of slope face supported. Shotcrete costs are about \$10.60/ft^2 (\$6.50/m^2) and wire reinforced shotcrete costs about \$1.60/ft^2 (\$17/m^2).

ACKNOWLEDGEMENTS

Roy Sage was responsible for production of this chapter. Address enquiries to him at: 555 Booth Street, Ottawa, Ontario, K1A 0G1.

Roy Sage wrote the chapter proper and appendices B, C and E. Allan MacRae wrote the draft versions of appendices A and D. The criticisms of Don Coates and Gerhard Herget were particularly valuable. Field trials of support were organized for CANMET by John Smith, Dick Bray and Allan MacRae, in conjunction with Sherritt-Gordon Mines Limited and Hilton Mines Limited, and by Bruce Briggs and Rick Achter of INCO Limited (Manitoba Division).

The principal contractors have been:

John Smith Engineering Limited
MacRae Rock Mechanics
INCO Limited
Sherritt-Gordon Mines Limited
Gaspe Copper Mines Limited
Golder Associates
Seegmiller Associates
Conenco Limited

The Pit Slope Project is the result of five years' research and development cooperatively funded by the Canadian Mining Industry and the Government of Canada.

The Pit Slope Group has been led successively by D.F. Coates, M. Gyenge and R. Sage; their colleagues have been G. Herget, B. Hoare, G. Larocque, D. Murray and M. Service.

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INTRODUCTION

PURPOSE

1. The term "support" is used in this manual to describe mechanical methods of increasing the stability of rock slopes. These mechanical methods improve stability by augmenting rock strength as opposed, for example, to draining groundwater, which improves stability by removing a cause of reduced rock strength.

2. Selecting and designing support requires the modes of instability to be identified, the likelihood and extent of instability to be assessed quantitatively, both theoretical and practical knowledge of possible support methods, and equipment and facilities for fabricating, installing and monitoring. Adopting any support method requires both engineering and economic analyses to demonstrate feasibility and viability.

3. This chapter describes methods of establishing the feasibility and economic viability of support, and the necessary techniques for design and installation. The body of the chapter describes what to do and why it is done. How the work should be done and sources of additional information, are described in Appendices A to D. Case histories of support installations are contained in Appendix E.

ECONOMIC ADVANTAGES OF SUPPORT

4. Support is expensive. It can only be used where the benefits can be clearly identified and the expense justified. In practice, this usually requires (a) that the slope is critical to the operation, such as where a slide would close access, damage plant or cover ore, (b) that the slope is unstable or about to slide, and (c) that alternate methods of stabilization, such as excavating to a shallower angle or draining the area, are not feasible or are more expensive.

5. It may occasionally be economically sound to design a wall using support. An example would be a clearly defined ore contact with unfavourable bedding planes, ie, the contact would be a natural mining boundary except for possible sliding on the bedding planes (Fig 1). It may be much cheaper to achieve stability using support than to excavate to a naturally stable slope. However, emphasis in such a situation must be placed on the accuracy of the geotechnical investigation because the degree of instability must be clearly defined before support can be justified.

6. Support is usually considered only for stabilizing the final wall but could logically be used to permit steepening of interim walls as

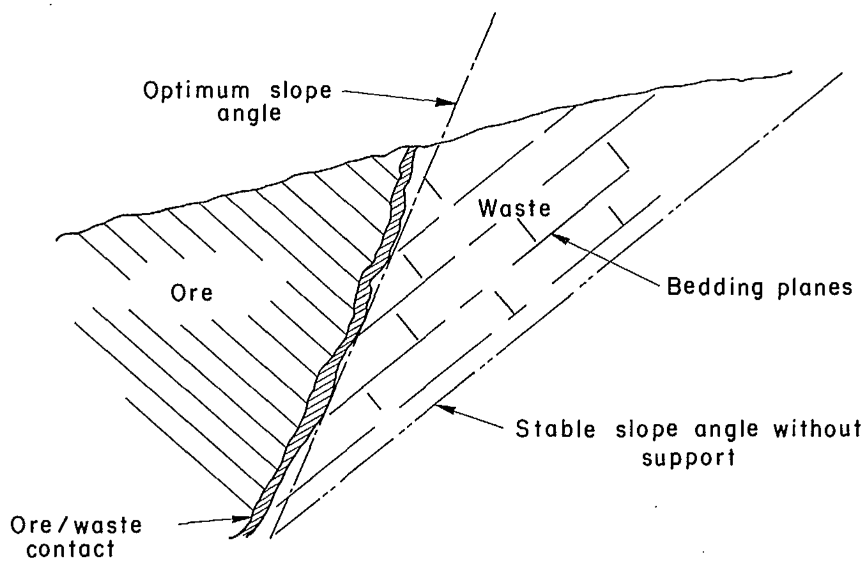


Fig 1 - Support may be appropriate if the ore contact is clearly defined but bedding planes are unfavourable.

well. If the interim walls are expected to stand for a sufficiently long period, the financial savings by delaying excavation may make the cost/benefit ratio of support advantageous.

PRINCIPLES OF SUPPORT

7. The most effective methods of rock slope support function by mobilizing strength inherent in the rock mass. The most significant factor in rock strength is the presence of discontinuities - joints, faults and, in sedimentary rocks, bedding planes. Instability occurs through movement along and by separation or opening of discontinuities, together with fracture of intact rock between discontinuities. Figure 2 shows a simple plane shear instability formed by a series of discontinuities. A sliding mechanism forms if fracturing of rock between the discontinuities occurs. Instability or movement will occur if the driving forces such as the weight of rock exceed the resisting forces.

8. Friction is always a function of the material properties of the sliding surface and of the normal force across the sliding surfaces. Figure 3 shows typical curves of resisting force vs displacement for rock discontinuities. Clearly, increasing normal stress increases the strength of discontinuities and therefore increases stability. Thus, an effective way of increasing rock strength is to increase normal stress on the discontinuities associated with instability.

9. This increase can be achieved by using rock anchors. These tensioned steel cables or bars are anchored so that they exert force across the discontinuity and so increase normal stress (Fig 4). The anchor force also has a component directly resisting sliding. The force that can realistically be exerted by anchors in most cases is small compared with the mass of rock involved. However, most instabilities of concern in mining are in limiting equilibrium, ie, are on the borderline between sliding and remaining stationary. A small increase in the resistance to sliding may often be sufficient to ensure stability indefinitely. Rock anchors can provide

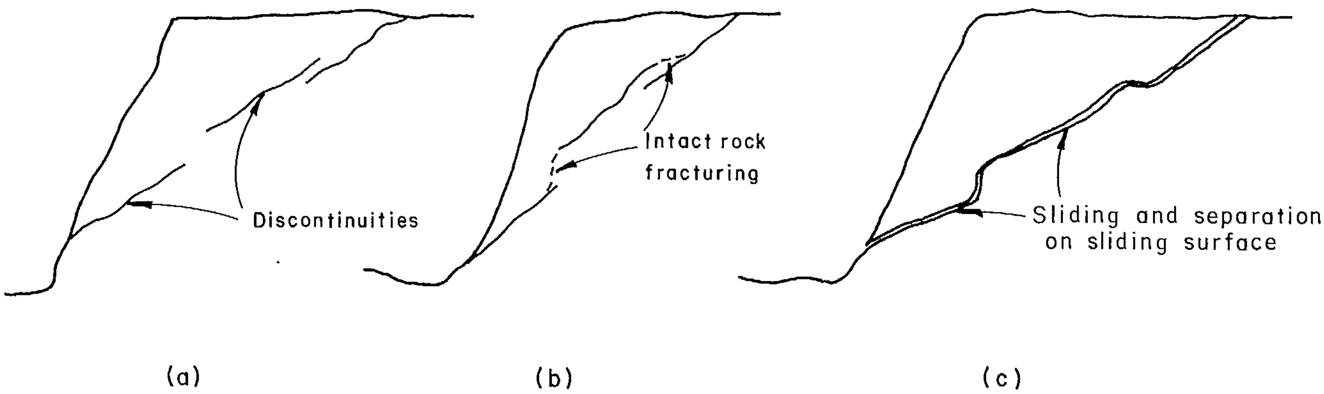


Fig 2 - A possible sliding mechanism: (a) discontinuities may become linked when intact rock (b) fractures leading to (c) a continuous surface of sliding.

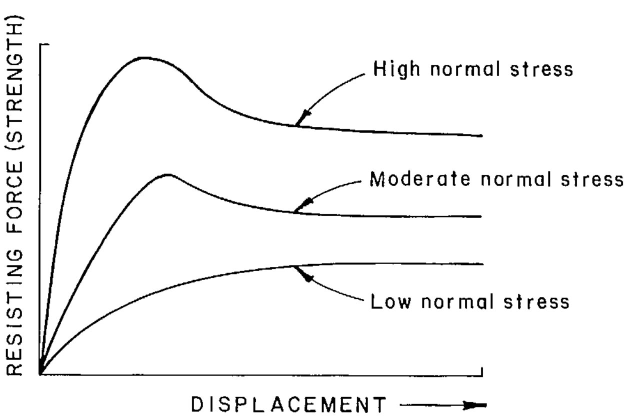


Fig 3 - Resistance to sliding is proportional to normal stress on the surface of sliding.

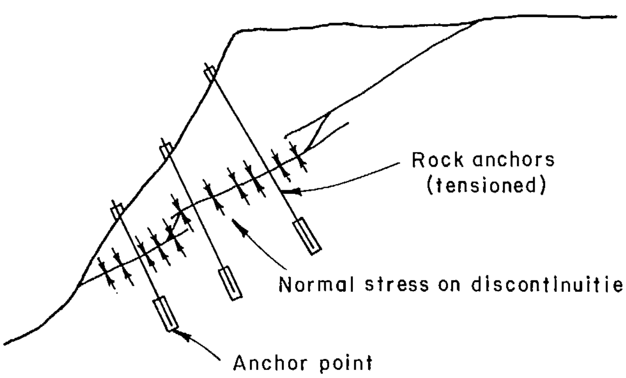


Fig 4 - Rock anchors increase normal stress on a surface of sliding, and so increase resistance to sliding.

this increase.

10. Other support methods include sprayed concrete and rock buttresses. Sprayed concrete is a surface treatment that can increase slope stability. Buttresses stabilize by providing massive dead weight restraint at the toe of a slope. These and other support techniques are described more fully below.

METHODS OF SUPPORT

ROCK ANCHORS

11. A rock anchor (Fig 5) consists of a bar or cable of high strength steel, tensioned inside a borehole to 60-70% of its breaking load. Tension in the anchor is transferred to the surrounding rock mass by anchorage points at the ends.

12. The length of a rock anchor is from 30 ft to 300 ft (10-100 m). The lower limit is dictated by end anchorage requirements, and by the method of tensioning. The upper limit is dictated by fabrication and installation considerations. The tension in a rock anchor is from 50 to 500 kips (220-2200 kN) and occasionally up to 1000 kips (4500 kN). The rock anchor has most application in stabilizing deep-seated instability modes in which sliding or separation on a discontinuity is an inherent characteristic.

13. The materials and technology of rock anchors are taken from prestressed concrete construction. Prestressing cables are designed especially for tensioning and give best results. In mining discarded hoist cable is occasionally used for rock anchors. This is not recommended. It may have good strength but often cannot be easily tensioned because the wire layers slip over each other. There may also be a fibre core which allows the cable to compress, affecting grip of the anchoring wedges. The cost of new prestressing cable is a small part of the total anchor cost and is fully justified by superior characteristics and guaranteed quality.

14. Support with rock anchors is achieved partly by direct resistance to sliding and partly through the increased normal stress across discontinuities, with a corresponding increase in friction resistance to sliding. Rock anchors are placed at an angle to the discontinuities that form the instability (Fig 6). A component of the anchor force then acts directly to oppose sliding. A full analysis of rock anchors includes calculating the direction of anchors that optimizes the combined frictional strength increase and direct support force.

15. The anchorage within the rock mass is formed by grouting the end of the anchor for a length of about 20 ft (6 m). The actual length of anchorage is determined from the grout/rock bond strength (Appendix B). A conventional cement grout is often used, though additives to speed setting time or produce grout expansion may be included. Mechanical anchorages such as expanding shells may be used for solid bar anchors of low

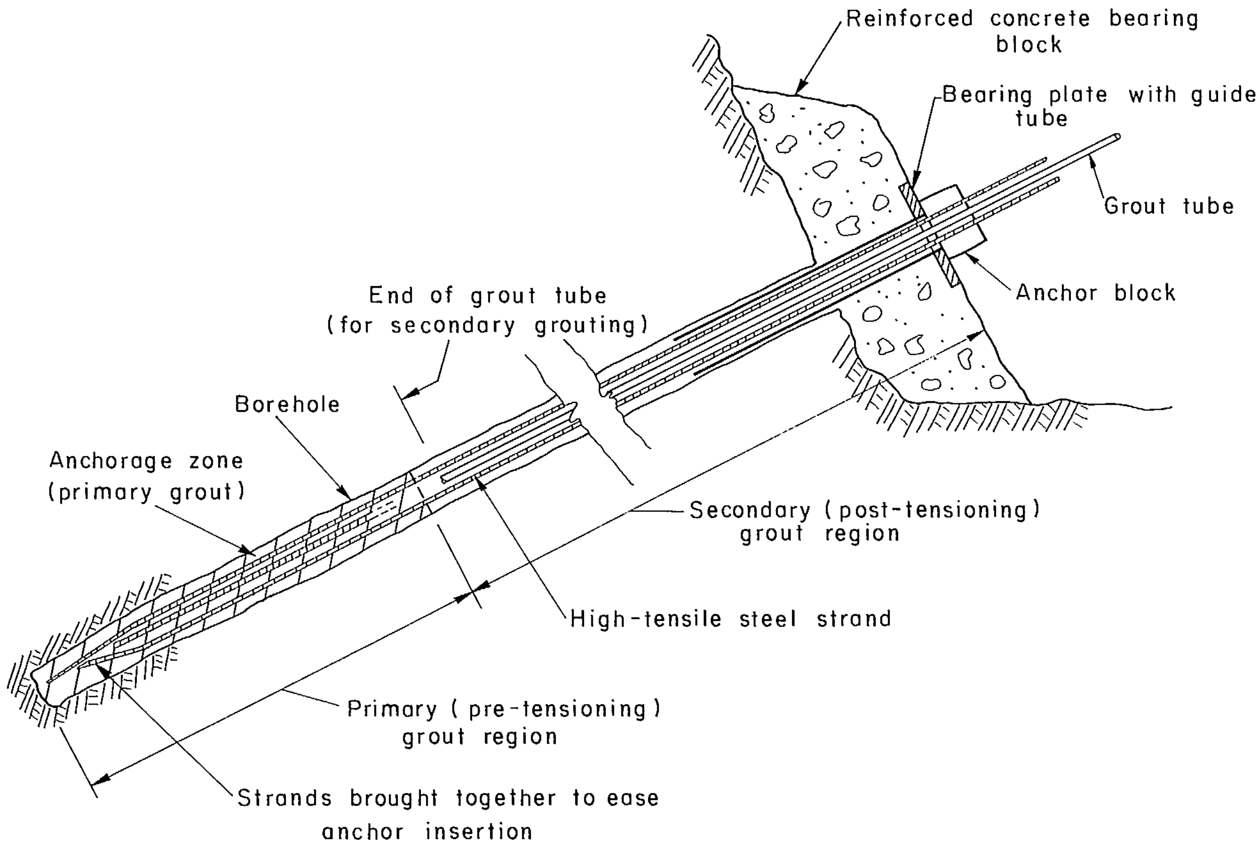


Fig 5 - Typical rock anchor installation.

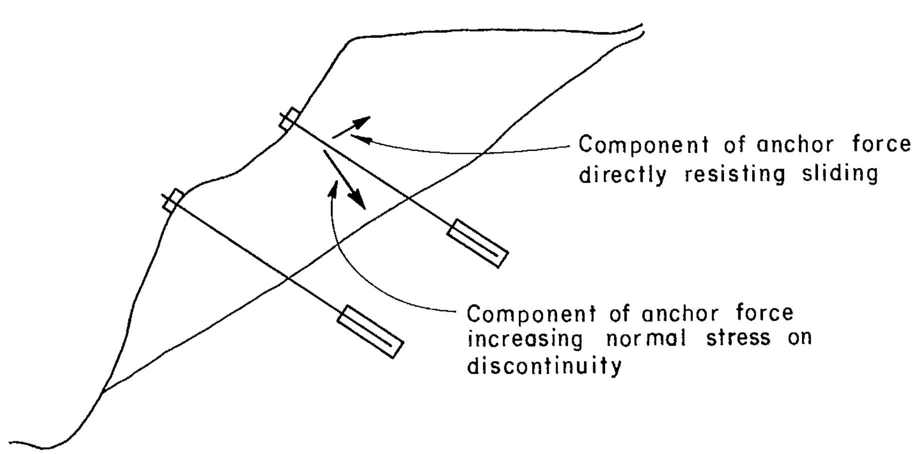


Fig 6 - Rock anchors may both directly resist sliding and increase friction resistance to sliding through higher normal stresses on a discontinuity.

capacity - less than 50 kips (220 kN) - but these are less satisfactory than grouted anchorages. They do have the advantage that installation is both simpler and possible in cold temperatures, when grout might require special treatment or be unusable.

16. The surface anchorage is formed by a bearing plate and a mechanical locking device that holds the anchor in tension (Fig 7). Tensioning is usually carried out by hydraulic jack after the grouted anchorage is set (Fig 8). Low load bar anchors are occasionally tensioned using a torque wrench (Fig 9).

17. The life of a rock anchor is governed by corrosion. An unprotected anchor may have a life as short as a few months and corrosion protection is essential. Two methods of protection are recommended. The simplest is filling the borehole with grout after the anchor has been tensioned. This has the additional advantage of bonding the full length of the anchor, thereby increasing security against possible future damage, eg due to blasting. The second, more expensive method is the sheathing of individual strands or bars in polyethylene tubes filled with grease. This is recommended if movement is likely following tensioning because a protective grout coating would crack and permit corrosion.

18. Corrosion in tensioned rock anchors is not limited to conventional effects such as rusting or attack by acid groundwater. Stress corrosion, which takes place place at relatively high stress, may occur even in a neutral environment. This problem manifests itself in the formation of brittle regions in the anchor, followed by sudden failure. The only safeguard is complete protection of the anchor, either by coating with a suitable grease and sheathing, or by grouting the full length of the anchor as soon as possible after tensioning. A full description of rock anchor corrosion and its treatment is given in Appendix B.

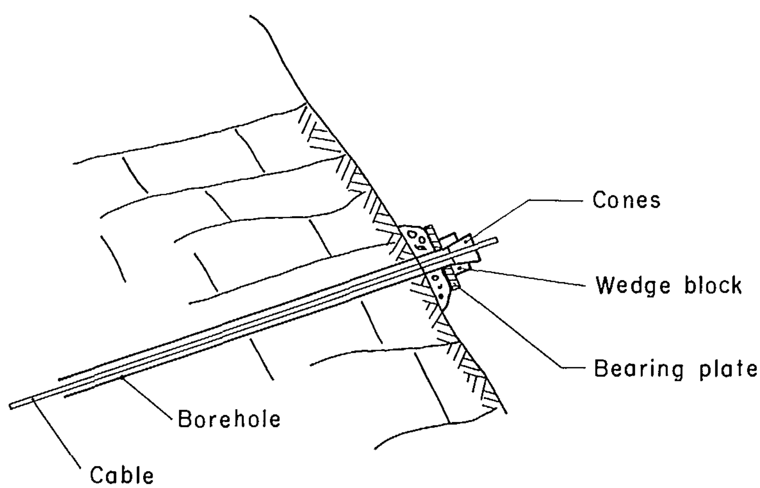


Fig 7 - Surface anchor detail for a rock anchor.

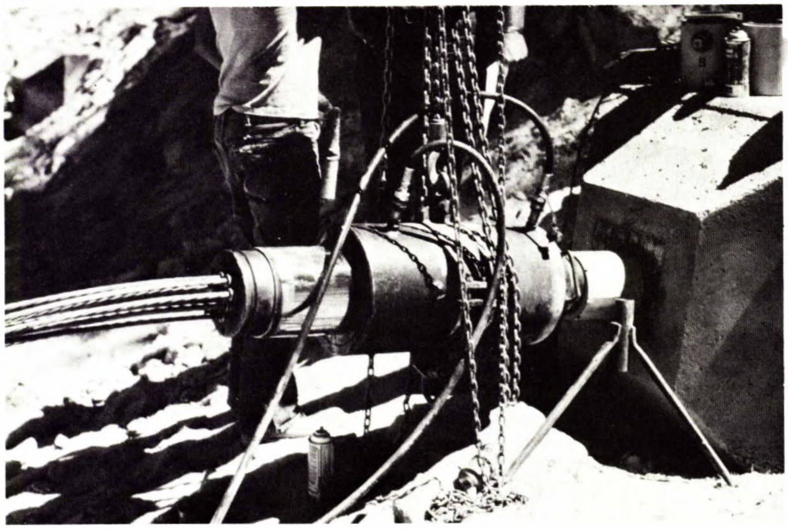


Fig 8 - Hydraulic jack for anchor tensioning.

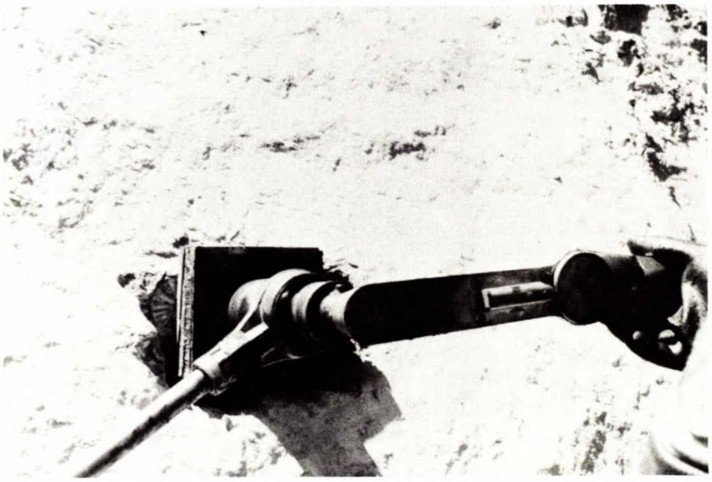


Fig 9 - Torque wrench for tensioning bar rock anchor.

Untensioned Rock Anchors

19. In some cases, adequate support may be provided by bars or cables fully grouted into the borehole but not tensioned. The support action requires expansion, or "dilation", of the rock mass being stabilized. Figure 10 shows the mechanism of movement that leads to dilation. Movement along the discontinuity is accompanied by a tendency to ride over the asperities with resulting separation on the discontinuity and expansion of the rock.

20. Figure 11 shows representative curves of dilation against slip on a discontinuity. Movement is quite small - typically, a maximum dilation of about of 0.01 in. (0.3 mm) will accompany a slip of the order of 0.1 in. (3 mm). At larger slips, the dilation may be correspondingly larger. The peak shear strength typically corresponds to peak dilation (Fig 3).

21. If an anchor - tensioned or untensioned - crosses the discontinuity and is firmly anchored on each side of it, dilation will induce tension in the anchor. This tension in turn increases the normal stress on the discontinuity and therefore increases friction resistance to slip.

22. The support value of the increased friction resistance depends entirely on the properties of the rock mass. If there is substantial dilation, there will be a reasonable increase in total resistance to sliding. If peak shear resistance precedes a moderate amount of dilation and there is a substantial drop in resistance after the peak, there will be little or no increase in total resistance (Fig 3). This is illustrated in Fig 12.

23. Figure 12(a) shows a substantial increase in total resistance to sliding. Peak dilation - and corresponding strength increase - coincides

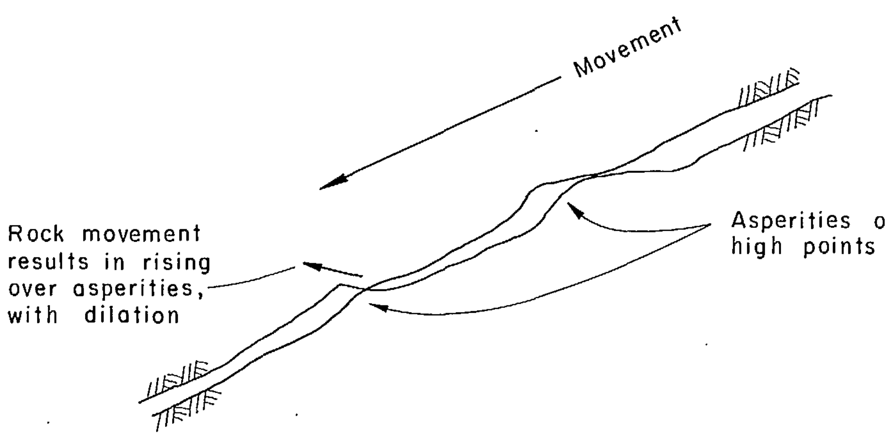


Fig 10 - Sliding in rock is usually accompanied by dilation, or expansion of the rock mass.

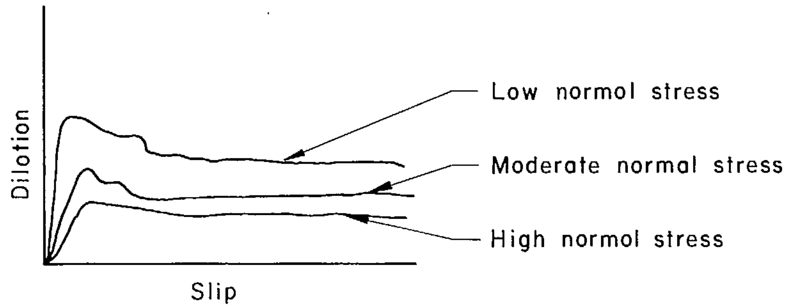


Fig 11 - Dilation tends to be inversely proportional to the normal stress on a surface or sliding.

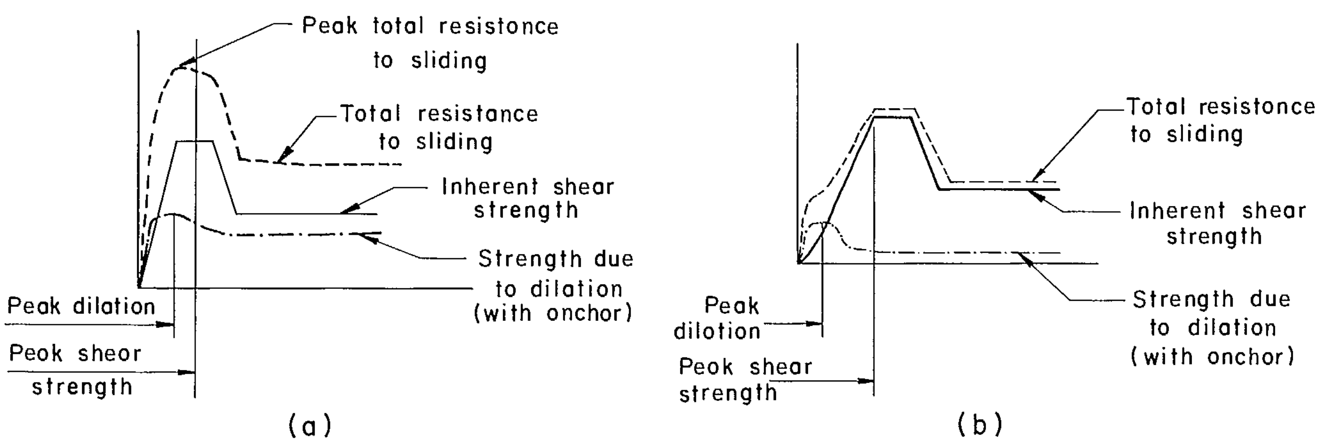


Fig 12 - Diagrammatic illustration of principle of support with untensioned anchors: in (a) peak inherent shear strength and peak strength due to dilation are additive, and untensioned rock anchors may give useful support; in (b) dilation and inherent shear strength do not coincide and untensioned anchors may be of less value.

with peak inherent strength and the two are additive. Figure 12(b) represents peak dilation preceding peak inherent strength; the increase in total sliding resistance is small.

24. Whether or not untensioned anchors are viable depends on the rock mass dilation characteristics in addition to any other engineering or economic factors. Clearly, if there is no dilation there is little benefit.

25. An inherent characteristic of untensioned anchors is that there must be rock movement and cracking of grout before anchor strength can be mobilized. This cracking may result in corrosion at the very point where anchor strength is most needed. Because grout would hold the anchor firmly in place above and below the zone, corrosion would be undetectable before sufficient support were lost for a slide to occur.

26. From the point of view of strength, the tensioned rock anchor is superior to the unten-

sioned. It invariably provides more support for a given weight of steel. However, installation of untensioned anchors is simpler, cheaper and quicker. Which type to use depends on the results of engineering and cost/benefit analyses.

SHOTCRETE

27. Shotcrete is a layer of concrete sprayed on a surface. The concrete mix is conventional in many respects, with aggregate size up to 0.75 in. (20 mm). Grading of the aggregate must be uniform, however, so that the dry cement-aggregate mix can be pumped through large diameter hose to the spraying nozzle. A flash-set additive is added to the mix at the pump and water is added at the nozzle. Passage through the nozzle and impact on the wall are sufficient to blend the water and dry mix and the concrete effectively sets on impact, producing a coating on the surface being sprayed.

28. Spraying requires a special pumping and mixing machine and access for the nozzle operator. Access is usually provided by a platform suspended by mobile crane (Fig 13). There is a loss of cement and aggregate in the spraying operation due to rebound. This depends largely on the skill of

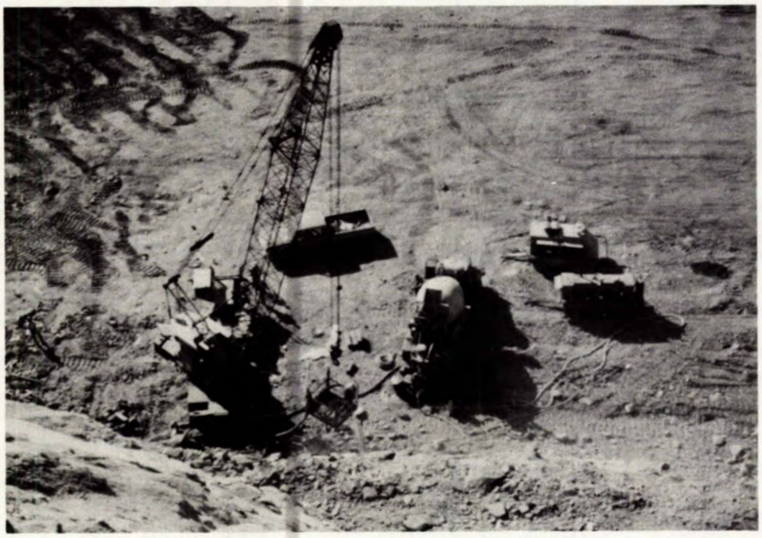


Fig 13 - Typical shotcreting operation in an open pit mine.

the operator but should be about 10%. The coating is built up to the required thickness in several passes. The uniformity of coating also depends largely on the operator's skill; typically a nominal 4 in. (100 mm) coating will be deposited with a variable thickness of 3-5 in. (75-125 mm).

29. Shotcrete is a surface treatment. It can support surface material, and may be used in conjunction with rock anchors. Shotcrete prevents weathering of the rock and progressive deterioration, thus maintaining long-term stability. Shotcrete will also provide strength around the boundaries of discontinuities that daylight on the face. Where these form part of a wedge or planar discontinuity the shotcrete will add to the resistance of the rock against sliding (Fig 14).

30. Shotcrete is sprayed onto the rock face at a high impact velocity. Initially, the large particles bounce off and a matrix of cement and fine particles is driven onto the rock surface.

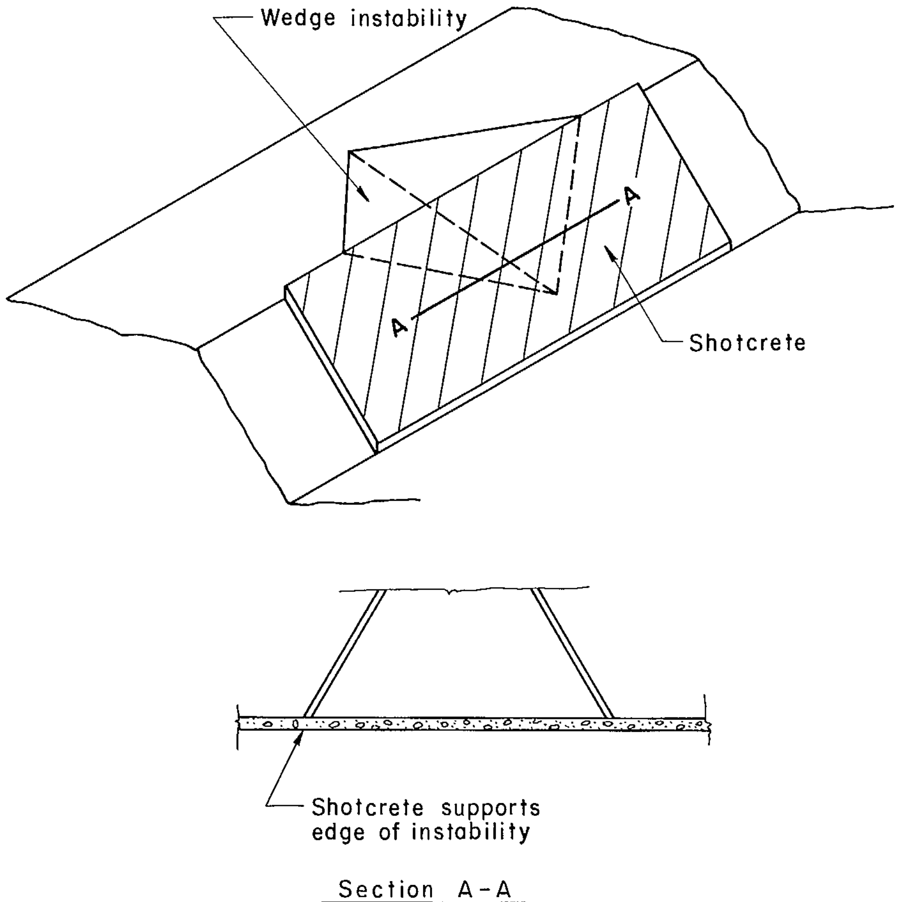


Fig 14 - A layer of shotcrete may provide support to a wedge instability.

Gradually larger particles start adhering and the layer is built up. This process is partly responsible for the high tensile and shear strengths usually found in the bond between shotcrete and rock - up to 1000 psi (7000 kPa) shear strength. Shear strengths of 850 psi (6000 kPa) and tensile strengths of 650 psi (4500 kPa) are common within the shotcrete itself.

31. Shotcrete, like conventional concrete, is a non-ductile, or brittle, material. Failure within the shotcrete occurs at points of high stress and then spreads in a progressive failure mode typical of brittle material. The ductility and strength of shotcrete can be increased by reinforcement. This can be added in two ways: by placing wire mesh on the slope before spraying; or, more efficiently and effectively, by adding chopped wire to the dry concrete mix. This wire reinforced shotcrete has good tensile strength - about 2000 psi (14,000 kPa) - and increases the support value of shotcrete. It has the drawbacks of requiring extra care in handling and applying, and greater cost.

32. An important consideration in using shotcrete is preventing water buildup behind the concrete layer. If there is buildup of water when the slope is sealed with shotcrete, the increased groundwater pressure may itself produce insta-

bility. Adequate drainage through the shotcrete must be provided. This can be done by casting short lengths of drain pipe through the shotcrete to tap possible water-bearing discontinuities.

33. Shotcrete can only be applied at temperatures well above freezing. The minimum rock face and air temperature for application is 5°C (45°F) and a further week of frost-free weather is required for full curing. The shotcrete should be kept moist during the curing period.

34. A detailed description of the methods and procedures for shotcreting is given in Appendix C.

BUTTRESSES

35. A buttress is a massive structure which provides support in two ways: by providing lateral restraint through deadweight, and by increasing the strength of material below the buttress by increasing the normal stress. The principle is illustrated in Fig 15. A buttress usually consists of a timber or concrete wall or crib to retain rock fill. An important requirement of such fill is that it should be free-draining. If a build-up of groundwater pressure occurs, this may counter the strengthening effect

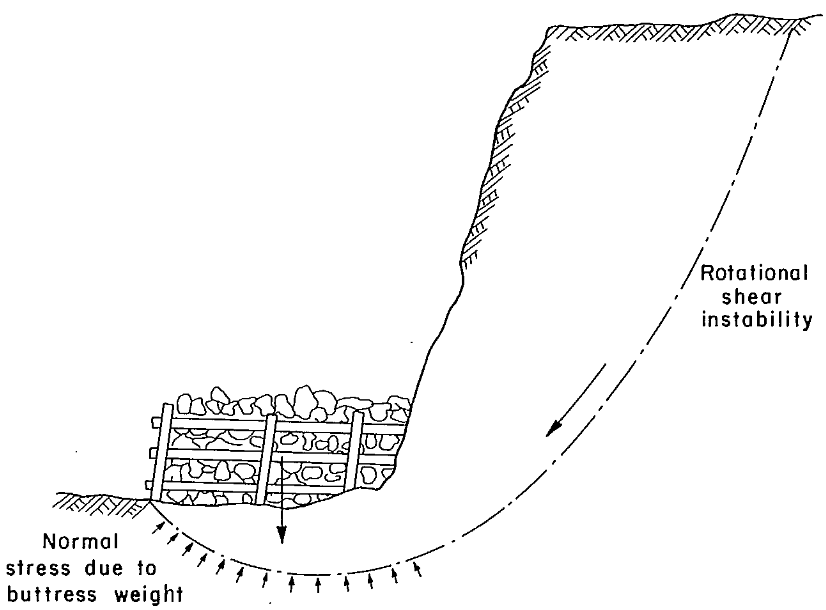


Fig 15 - A buttress or crib with deadload of rock fill provides counter weight to resist rotational shear and increases normal stress and therefore sliding resistance in toe region.

of the buttress. Figure 16 shows a buttress used to stabilize a slope in a Canadian iron mine.

36. Buttresses can stabilize slopes up to 100 ft (30 m) or more high, but are necessarily limited to providing restraint at the toe of a slope. The design and construction of buttresses is described in Supplement 6-1.

RETAINING WALLS

37. A retaining wall is a timber or reinforced concrete structure for retaining loose material or to provide direct support to a slope or embankment. These walls are illustrated in Fig 17. Timber walls are best suited for short-term requirements. Reinforced concrete is durable but

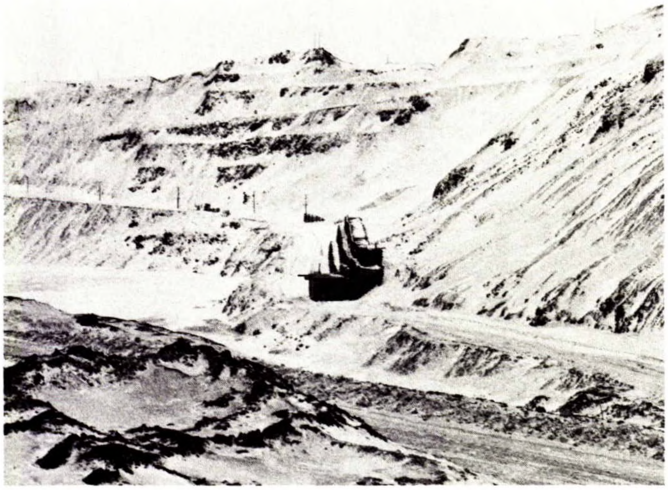


Fig 16 - A buttress used to stabilize a slope in a large Canadian iron ore mine.

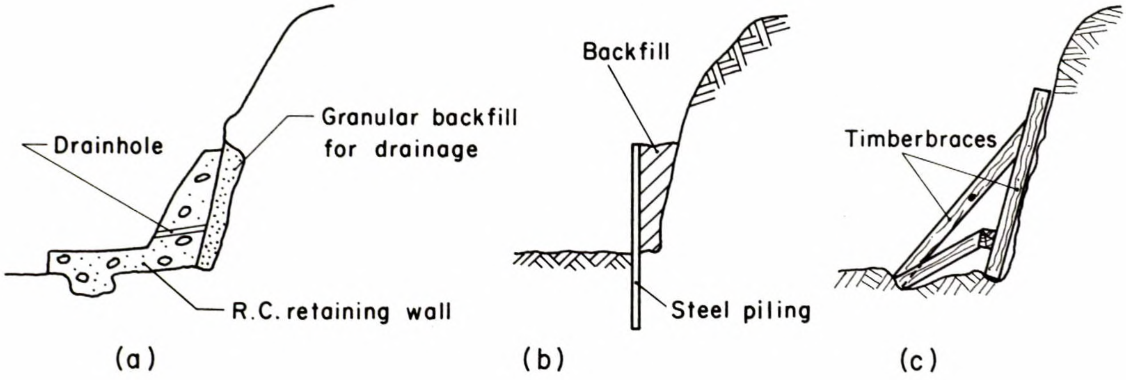


Fig 17 - Various types of retaining wall: (a) reinforced concrete; (b) steel piling with backfill; (c) timber braces.

requires more time and expense to install.

38. Retaining walls are relatively costly to design and construct. They are used only for special applications such as protection of plant or access, and in particular to protect or support haul roads threatened by embankment collapse or by falling material. Retaining walls are described in detail in Supplement 6-1.

MESH

39. Mesh is used to retain loose rock and as a curtain to control falling rock, ie, to prevent loose rock falling or bouncing out from the rock face with danger to men and equipment. Mesh is

Fig 18 - Mesh used as protection against loose rock. A concrete bearing block for a rock anchor is seen in the centre of picture.

held in place by bolts, by deadweight at the top of a bench or by a concrete groundbeam which is itself tied down with rock anchors. Mesh used in conjunction with rock anchors is shown in Fig 18.

40. The design of mesh, ie, selecting size and spacing of wires in the mesh, is largely empirical. Mesh is a relatively inexpensive, easily installed surface treatment. However, it does not provide positive slope support in the sense of increasing slope stability.

DEPTH OF SUPPORT ACTION

41. Support methods vary greatly in the depth to which they are effective within the rock mass. The deepest support is provided by rock anchors which can raise the shear strength of a discontinuity at depths as much as 300 ft. Shotcrete and mesh provide surface support only.

42. Rock anchors are usually the only practical support method for major instabilities. In some circumstances, mass support to the toe region of a potential slide using buttresses can be viable. Minor instabilities may be stabilized by rock anchors, if necessary in conjunction with surface treatment. Small-scale instability problems, such as loose rock, can often be overcome by surface treatment.

DESIGN OF SUPPORT

43. The design of a support installation requires:

a. identification of an actual or potential instability (including the mode or type of instability and relevant information about structural geology, groundwater and mechanical properties;
b. quantitative engineering analysis of slope reliability both with and without support;
c. economic appraisal of support.

44. Steps b and c may be carried out together; the economic appraisal - assuming support is technically feasible - is followed by the decision whether or not to install support.

45. The Design chapter describes the various instability modes and the data and techniques needed to analyze stability. The detailed analyses given in the Design chapter provide for the effects of support to be considered where relevant; similarly, the financial cost/benefit and risk analyses can include factors reflecting both the cost of support and the improved stability resulting from support.

46. This section briefly describes the various instability modes and the steps required in designing support systems. In some instances, simple analyses are presented, both to illustrate the principles and to provide a tool for a first estimate of support feasibility. Occasionally, these simple techniques may be adequate for final design. In others, a description is given of the steps required to utilize the methods of the Design chapter for analyzing support.

47. The analysis techniques of the Design Chapter are written in terms of probability of instability. For simplicity, the techniques given here are in terms of the more conventional, though less flexible, factor of safety (FS). The definition of FS is given with each technique.

SIMPLE PLANE SHEAR

48. Figure 19(a) shows the simple plane shear instability mode which arises when a discontinuity strikes roughly parallel to the slope at a shallower dip. The extent of the instability parallel to the slope is assumed to be large; in this case the analysis can be carried out in two dimensions only. The plane shear mode can be stabilized with rock anchors. If the critical discontinuity daylights at or near the toe of the slope, buttresses may be suitable for stabilization. The stabilizing effect of rock anchors

will be considered first.

49. Figure 19(b) shows the plane shear mode in the static dry case with rock anchors acting. The disturbing force is W, the weight of the rock. The resisting forces are S, the shear strength on the surface of sliding, and T, the forces due to rock anchors. Resolving parallel to the surface of sliding,

$$W \sin\alpha = S + T \cos(\alpha + \delta)$$

The factor of safety against sliding is defined as

$$FS = \frac{\text{resisting force}}{\text{disturbing force}} = \frac{S + T\cos(\alpha + \delta)}{W \sin\alpha} \qquad \text{eq 1}$$

If resistance to sliding is represented by Coulomb's law

$$s = c + \sigma \tan\phi$$

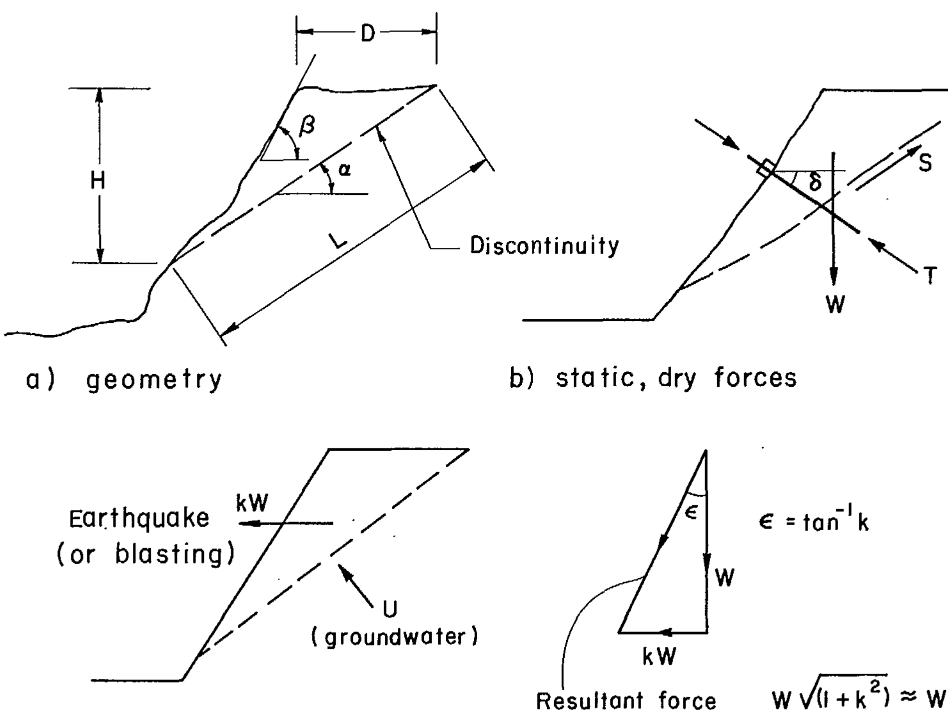
where s is shear strength per unit area,
c is cohesion strength per unit area
σ is normal stress
ϕ is angle of friction resistance (ie, $\tan\phi$ is the coefficient of friction)

and if the normal stress, σ, is assumed to be uniformly distributed over the sliding surface, which is a common assumption, eq 1 can be written as

$$FS = \frac{1}{W\sin\alpha}\,[cH\mathrm{cosec}\alpha + [W\cos\alpha + T\sin(\alpha + \delta)]\tan\phi + T\cos(\alpha + \delta)] \qquad \text{eq 2}$$

where $W = 0.5\gamma H^2 (\cot\alpha - \cot\beta)$ and γ is the average density of the slope material.

50. Provided $(\alpha + \delta)$ is always in the range 0° to 90°, FS increases with T (note that T is the



c) forces due to groundwater and earthquakes or blasting

Fig 19 - Simplified analysis of support of a plane shear instability.

total force due to anchors acting on the surface of sliding).

51. For a given T, the optimum direction of the anchors, which are assumed to be parallel, can be found by maximizing FS in eq 2. After differentiating with respect to δ, this gives a maximum FS when

$$\tan(\alpha + \delta) = \tan\phi$$

52. If groundwater is present or if seismic or blasting forces act, the analysis is more complex. Groundwater usually decreases resistance to sliding by transmitting part of the normal stress through water. The groundwater chapter contains a full description of this phenomenom under "Principle of Effective Stress". Seismic and blasting forces may both result in ground movement. Seismic disturbance, if it occurs, may be very serious. Blast effects are not likely to cause the long period waves that result from earthquakes but ground accelerations should be estimated and an equivalent force included in the analysis if necessary.

53. The forces involved in both groundwater and seismic or blasting forces are complex in magnitude and direction. The Groundwater and Blasting chapters describe these forces and their measurement; the analyses in the Design chapter have provision for including these forces. For the simplified analysis given here, it will be assumed that groundwater exerts a total force, U, uniformly distributed on the surface of sliding, and that seismic or blasting effects are equivalent to a horizontal force. For convenience, this force will be assumed to be given by kW, where W is the weight of the sliding mass. (Earthquake analysis often assumes a peak horizontal force of 0.1W). The horizontal force is included by inclining the weight vector, as shown in Fig 19(c). The inclined resultant force R is given by:

$$R = W\sqrt{(1 + k^2)}$$

but if k is less than 0.25, which is usually the case, the resultant can be approximated by W.

54. Eq 2 can now be rewritten to include both U and kW, as follows:

$$FS = \frac{1}{W\sin(\alpha + \varepsilon)} [cH\operatorname{cosec}\alpha + [W\cos(\alpha + \varepsilon) - U + T\sin(\alpha + \delta)]\tan\phi + T\cos(\alpha + \delta)] \quad \text{eq 3}$$

where $\tan\varepsilon = k$. The optimum inclination remains as for the dry case.

55. In eq 3 the effect of the support force, T, occurs twice. The term $T\sin(\alpha + \delta)$ represents the increase in normal stress and therefore in friction resistance to sliding. The term $T\cos(\alpha + \delta)$ represents the component of force directly resisting sliding.

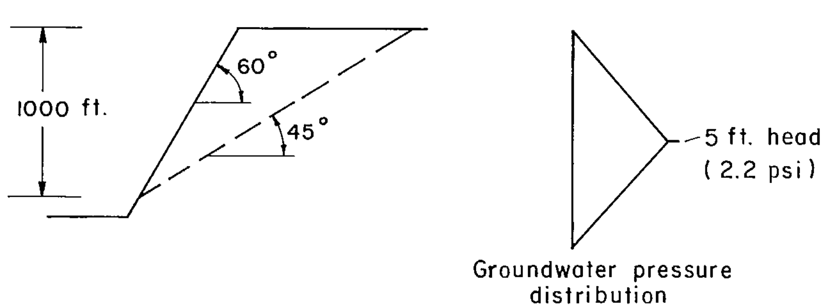


Fig 20 - Example for calculation of plane shear support.

56. The term $T\cos(\alpha + \delta)$ is sometimes written in the denominator of eq 3, giving

$$FS = \frac{1}{W\sin(\alpha+\varepsilon) - T\cos(\alpha+\delta)}\{cH\mathrm{cosec}\alpha + [W\cos(\alpha+\varepsilon) - U + T\sin(\alpha+\delta)]\tan\phi\} \qquad \text{eq 4}$$

Equations 3 and 4 give different values for the factor of safety, FS.

57. The reason for using eq 4 is that $T\cos(\alpha+\delta)$ can be considered as directly reducing the disturbing force, $W\sin\alpha$, resulting in a higher value of FS. Whichever equation is used is arbitrary. It is important, however, to be consistent and to appreciate the significance of the numbers obtained. The differing values of FS have no relevance to the actual degree of stability, which does not change. This reflects a drawback in the use of factors of safety, which is overcome by taking a reliability approach. FS from eq 3 will be used in the examples that follow.

Example

58. Figure 20 shows a hypothetical plane shear instability. Groundwater pressure is as shown and a lateral force of 0.1 W due to seismic effects is considered possible. Tests establish the Coulomb parameters c and ϕ as 5 psi (35 kPa) and 35°. The rock density is 170 pcf (2800 kg/m^3). The weight, W, is given by

$$W = 0.5 \times 170 \times 100^2(\cot 45 - \cot 60) = 359 \text{ kips/ft} \quad (5.22 \times 10^6 \text{ N/m})$$

FS for the dry, static case is

$$FS = \frac{1}{359\sin 45}[100 \times 5 \times 0.144 \,\mathrm{cosec}45 + 359 \cos 45 \tan 35] = 1.1$$

The groundwater force is given by

$$U = 2.2 \times 0.144 \times 0.5 \times 100 \,\mathrm{cosec}45 = 22.4 \text{ kips/ft} \quad (328 \text{ kN/m})$$

FS for the case with groundwater is

$$FS = \frac{1}{359\sin 45}[100 \times 5 \times 0.144 \,\mathrm{cosec}45 + (359\cos 45 - 22.4)\tan 35] = 1.04$$

FS for the case with seismic force and groundwater pressure acting is found from eq 3 with T = 0 and $\varepsilon = 6°$ ($\tan 6° \simeq 0.1$)

$$FS = \frac{1}{359\sin 51}[100 \times 5 \times 0.144 \,\mathrm{cosec}45 + (359\cos 51 - 22.4)\tan 35] = 0.88$$

59. The value of FS that would indicate acceptable stability is open to considerable discussion. In mining, provided personnel safety is ensured by proper monitoring, it might be reasonable to design to an FS of 1.05 to 1.1 under the worst possible conditions. In this case, that would be with groundwater and seismic effects. From eq 3, a support force of 29 kips/ft (420 kN/m) at 10° above the horizontal, ie, $\delta = -10°$, would give a worst case FS of 1.0. A support force of 52 kips/ft (720 kN/m) would give an FS of 1.1.

60. This support could be provided by three rows of anchors, each anchor of 200 kips (900 kN) capacity, at 20 ft (6 m) centres. It is assumed in this analysis that the stress due to the anchors acts uniformly on the surface of sliding. Anchors should be spaced so that this assumption is reasonably satisfied. One possible method of achieving this is to assume stresses due to an anchor arise within a 45° cone from the nearest anchor point.

61. The angle of 10° above horizontal found by minimizing the support force required for a given factor of safety may be impractical. However, in most cases the FS for a given anchor force is insensitive to the inclination of rock anchors; an inclination of 10° below horizontal in the example above results in virtually the same FS. For the support force of 52 kips/ft (720 kN/m), the FS would become 1.09 instead of 1.1.

62. A similar analysis can be made for untensioned rock anchors. In this case, however, an

estimate of dilation must be made and the consequent anchor force calculated. This requires an estimate of the length of anchor which would be tensioned as a result of dilation. For a fully grouted anchor, this length might be as little as 4-5 ft (1-2 m) though the true length cannot be determined (Fig 21).

63. The force available following dilation can be calculated if the tensioned length is known. The force, T, in a bolt or anchor is given by

$$T = \frac{EAe}{L}$$

where A is the cross-section area of the bolt or anchor
L is the tensioned length
E is Young's Modulus (for steel, 30×10^6 psi - 200×10^6 kPa)
e is the dilation

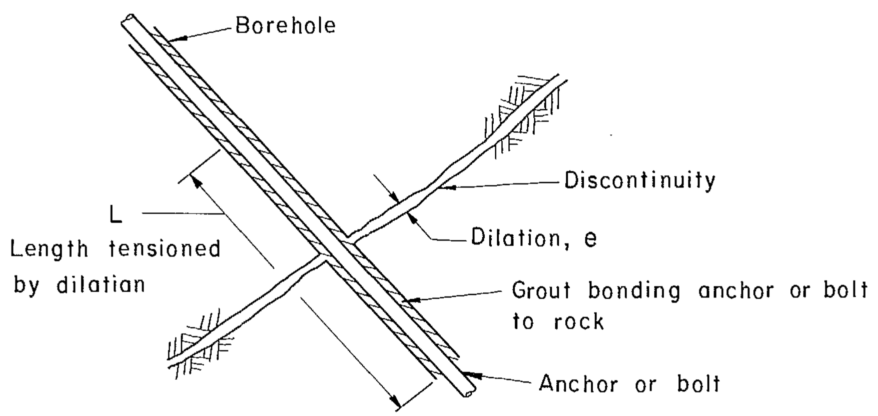
For a dilation of 0.1 in. (0.25 cm), and for a 1 in. (2.5 cm) diameter bar tensioned over 4 ft (1.25 m), this force is

$$T = \frac{30 \times 10^6 \times 1^2 \times \pi \times 0.1}{4 \times 4 \times 12} = 50 \text{ kips (220 kN)}$$

The drawback in this type of analysis is the impossibility of knowing e and L with any accuracy.

64. The use of a buttress to stabilize a simple plane shear instability is shown in Fig 22. The analysis of stability assumes the buttress has a dead weight, V, and a resulting resistance to horizontal sliding of μV, where μ is the coefficient of friction between the buttress and the ground. The above analysis used for rock anchors can be adapted conservatively for the buttress by ignoring T, the rock anchor load, in eq 3, and adding a resisting force parallel to the discontinuity due to the sliding resistance of the buttress. A conservative value for the additional resistance would be $\mu V\cos\alpha$ and eq 3 becomes

$$FS = \frac{1}{W\sin(\alpha + \varepsilon)} \{cH\mathrm{cosec}\alpha + \mu V\cos\alpha + [W\cos(\alpha + \varepsilon) - U]\tan\phi\}$$



The anchor or bolt becomes tensioned following dilation. The grout cracks at discontinuity, and transfers load along the borehole. Further grout cracking occurs until the average rock to grout bond is reduced sufficiently for equilibrium.

Fig 21 - The support mechanism of an untensioned rock anchor.

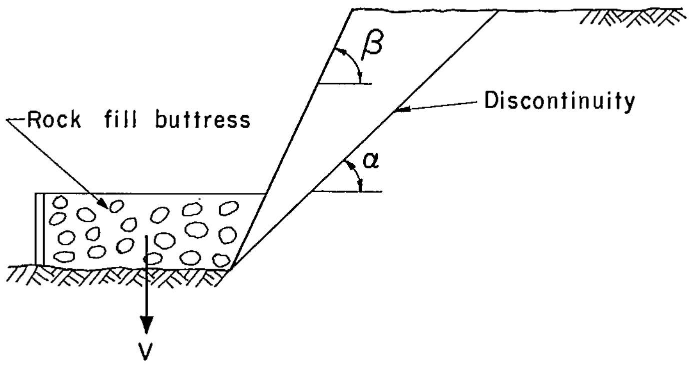


Fig 22 - Principles of buttress support to plane shear instability.

Example

65. The previous example (Fig 20) can be reanalyzed to determine the buttress support required to give an FS of 1.1 under the worst conditions. From previous results, with 0.1W horizontal acceleration force and water pressure as shown, the FS for V = 0 is 0.88. To raise the FS to 1.1 requires

$$\mu V\cos\alpha = (1.1 - 0.88)W\sin(\alpha + \varepsilon)$$

For this example $\alpha = 45°$, $\varepsilon = 6°$. Assuming μ = 0.7, corresponding to a friction angle of 35°, gives

$$V = \frac{(1.1 - 0.88) \times 359 \times \sin 51}{0.7\cos 35} = 107 \text{ kips/ft}$$
$$(1560 \text{ kN/m})$$

The required buttress weight would be 107 kips per ft (1560 kN/m) of wall. This is substantial; 60 ft (20 m) of rock fill 10 ft (3 m) deep would be required for the length of the instability.

66. The analysis of simple plane shear presented in the Design chapter is similar to the above analysis, but allows the variation of such parameters as joint inclination and water pressure to be treated statistically. The Design chapter procedure requires as input the geometry of the slope, location of the potential sliding plane, material parameters, water table location and the location and magnitude of support forces. If relevant, the variation of support force about a mean, both in magnitude and direction, can be input. The result of the analysis is the probability of sliding with the effects of support included.

3-D-WEDGE

67. Figure 23 shows a typical wedge instability. Sliding cannot occur unless the line of intersection of the discontinuities defining the wedge daylights. If this is so, the wedge is "kinematically admissible", and sliding will occur if the driving forces, due to weight, seismic effects, etc, are greater than the shear strength of the discontinuities. Sliding may occur on one or both discontinuities, depending on their orientation.

68. Figure 23 shows a clearly defined wedge. This is a simplification; in reality, the discontinuities occur in families and the particular combination that results in an actual sliding wedge cannot always be predicted. Support in such cases must therefore be provided uniformly along the slope face where slides are possible.

69. The nature of wedge instabilities, which

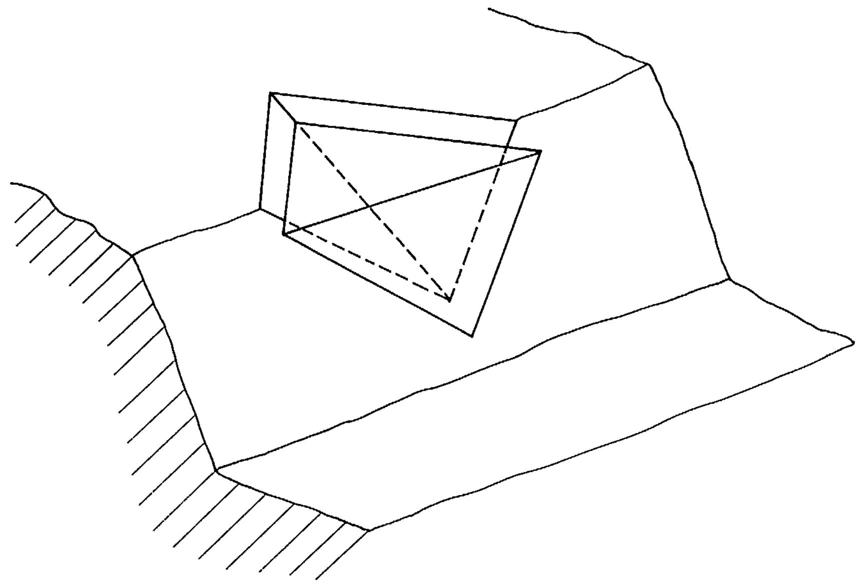


Fig 23 - The plane shear 3D wedge instability.

may occur well above the toe of the slope, means that only rock anchors or sprayed concrete, or a combination of these, provide practical support.

70. Rock anchors provide support to a wedge instability by increasing normal stress on the surface of sliding, and therefore increasing the shear strength, as described for simple plane shear, and also by directly resisting sliding.

71. The analysis of wedge sliding and support must consider the three-dimensional nature of the instability, and therefore is much more complex than the simple plane shear analysis. The Design chapter includes a computer program which can account for anchor support forces in a reliability analysis. Input for this analysis is the geometry of the wedge, ie, of the defining discontinuities, and of the slope, strength properties, water pressures expected on the wedge and the direction and inclination of support forces. A test for kinematic admissibility is included. The result of the analysis is the probability that a wedge slide will occur. Untensioned anchors require the same considerations as described under Plane Shear above. The force due to dilation must be estimated and included in the analysis.

72. Several techniques exist for hand analysis of the stability of a supported wedge. These usually require a combined graphical/analytical approach. The analysis for sliding on one discontinuity only is straightforward, but for sliding on both discontinuities the calculations are both complex and tedious. It is therefore usually best to use the computer methods of the Design chapter. However, an analysis technique for manual calculation is given with an example in Appendix A.

73. Analyses for support with anchors assume the support force is distributed uniformly over the surfaces of sliding. The layout of the anchors must therefore, as far as possible, produce a uniform distribution of support force.

74. Sprayed concrete supports wedge instabilities by providing a bonding layer over the discontinuities at the slope face (Fig 14). For the wedge to slide, the shotcrete would have to shear along the length of the discontinuities which

define the wedge at the slope face. The resisting force, T, is therefore given by

$$T = Lts_c$$

where L is the length of the discontinuities on the slope face,

t is the average thickness of the shotcrete layer

s_c is the average shear strength of the shotcrete.

The nature of this resisting force is such that it will always act opposite to the direction of sliding. This force can be included in the analyses of the Design chapter, or in the method of Appendix A. The thickness and shear strength of shotcrete must be measured on site; values for design purposes are given in Appendix C.

MULTI-BLOCK PLANE SHEAR

75. This instability mode is shown in Fig 24. It is similar to the simple plane shear mode and is analyzed in two dimensions, ie, the extent of the instability parallel to the slope is assumed to be large. The surface of sliding may not be straight, however, and the sliding mass is assumed to break into two or more blocks.

76. Support to this mode can be provided by rock anchors, which would usually be the most effective method, or by a buttress. The Design chapter describes the detailed analysis of this mode; support forces can be included. If assumptions are made about the forces on the blocks the simplified analysis described below can be used.

77. Figure 25 shows the forces acting on the two parts of a simple two block instability. The forces are statically determinate if points of action are disregarded, and if the line of action of P, the reaction between the blocks, is assumed. For this analysis it is assumed P acts parallel to the bottom part of the surface of sliding and that S, the shear force on the upper block, is at its maximum value ie, is equal to the shear strength.

78. Resolving for block 1 gives

$$P = W_1 \sin\alpha - S_1 \cos(\lambda - \alpha) + N_1 \sin(\lambda - \alpha) \qquad \text{eq 4}$$

$$W_1 \cos\alpha = S_1 \sin(\lambda - \alpha) + N_1 \cos(\lambda - \alpha) \qquad \text{eq 5}$$

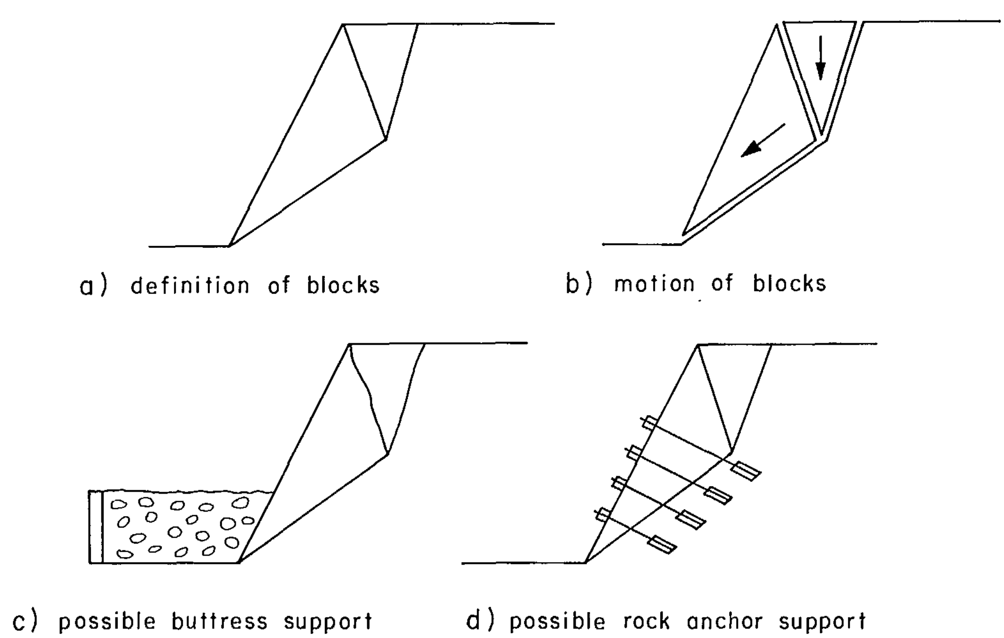


Fig 24 - Possible support for the two block plane shear instability.

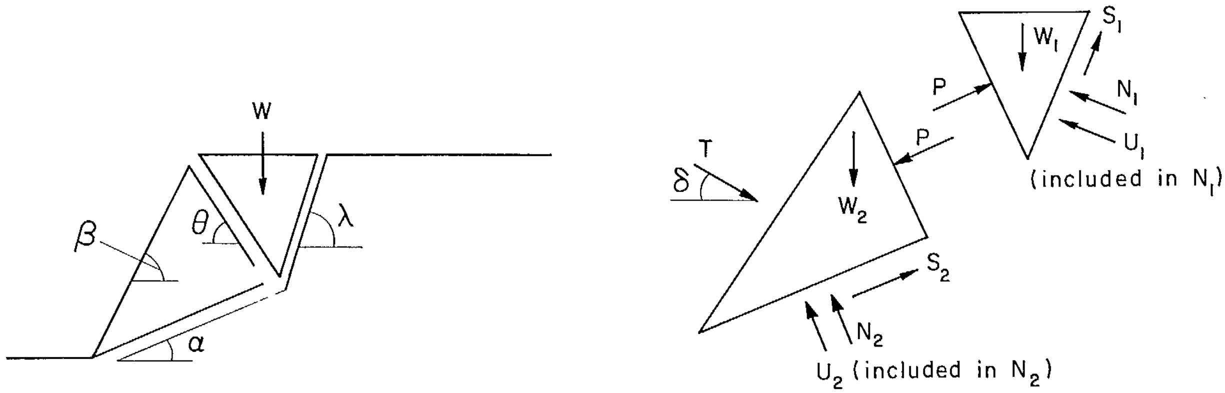


Fig 25 - Analysis of two-block instability.

Coulomb's law for shear strength gives

$$S_1 = c_1 + (N_1 - U_1) \tan\phi_1 \qquad \text{eq 6}$$

Equations 4, 5 and 6 can be solved to give P in terms of the weight, W, of block 1, water pressure, U, on block 1 and the geometry and shear strength parameters.

79. Resolving for block 2 gives

$$S_2 = P + W_2 \sin\alpha - T \cos(\alpha + \delta) \qquad \text{eq 7}$$

$$N_2 = W_2 \cos\alpha + T \sin(\alpha + \delta) \qquad \text{eq 8}$$

S_2 here is the sliding resistance required to maintain equilibrium of the lower block. The available strength, R, assuming Coulomb's law, is given by

$$R = C_2 + (N_2 - U_2) \tan\phi_2 \qquad \text{eq 9}$$

The factor of safety against sliding can therefore be written as

$$FS = R/S_2 \qquad \text{eq 10}$$

An analytical expression for FS can be obtained by solving eq 4 to 10, but it is simpler to solve for the various terms individually.

Example

80. Figure 26 shows a simple multi-block instability. For a slope material density of 170 pcf (2700 kg/m^3),

$$W_1 \simeq \frac{170 \times 58 \times 50}{2} = 247 \text{ kips/ft (3600 kN/m)}$$

$$W_2 \simeq \frac{170 \times 138 \times 57}{2} = 668 \text{ kips/ft (9500 kN/m)}$$

81. The shear strength parameters of cohesion and friction angle for planes 1 and 2 are respectively 5 psi (35 kPa), 35° and 7 psi (50 kPa), 20°

$$C_1 = 5 \times 144 \times 58 = 42 \text{ kips/ft (630 kN/m)}$$

$$\text{and} \quad C_2 = 7 \times 144 \times 138 = 139 \text{ kips/ft (2100 kN/m)}$$

82. Assuming for the sake of simplicity that no water pressure is acting on block 1, eq 4, 5 and 6 give:

$$P = 247\sin 20 - S_1\cos 40 + N_1\sin 40$$

$$247\cos 20 = S_1\sin 40 + N_1\cos 40$$

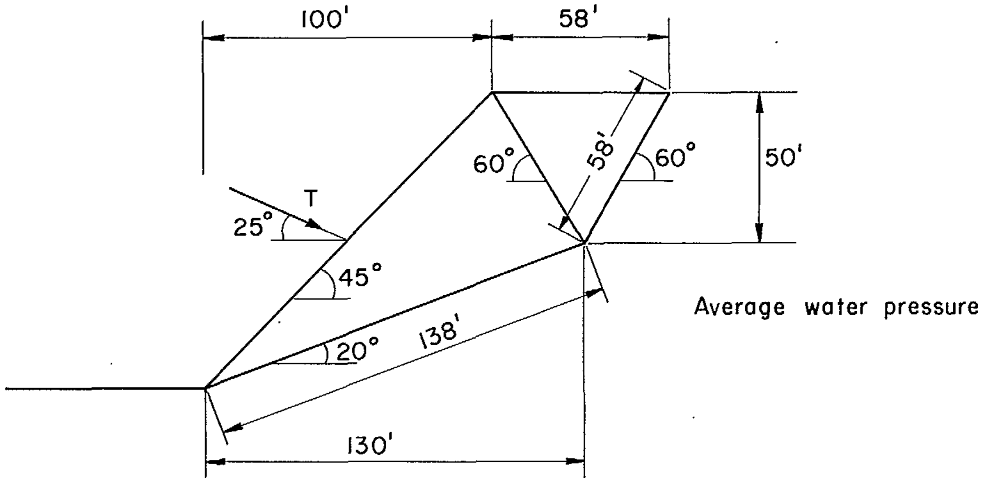


Fig 26 - Numerical example for analysis of two-block instability.

$S_1 = 42 + N_1\tan35$

Solving these gives P = 70 kips/ft (1020 kN/m)

83. Assume that an average water pressure of 12 psi (85 kPa) acts on the base of the lower block. For the case of no support (T = 0), the shear and normal forces are

$S_2 = 70 + 668\sin20 = 298$ kips/ft

$N_2 = 668\cos20 = 628$ kips/ft

$U_2 = 12 \times 144 \times 138 = 238$ kips/ft

$R = 139 + (628 - 238)\tan20 = 281$ kips/ft

The factor of safety is

$FS = \frac{281}{298} = 0.94$ indicating instability.

84. Suppose a support force of 30 kips/ft (440 kN/m) acts at 10° above horizontal.

$S_2 = 70 + 668\sin20 - 30\cos30 = 272$ kips/ft

$N_2 = 668\cos20 + 30\sin30 \quad = 643$ kips/ft

$R = 139 + (643 - 238)\tan20 \quad = 286$ kips/ft

and $FS = \frac{R}{S_2} = 1.05$

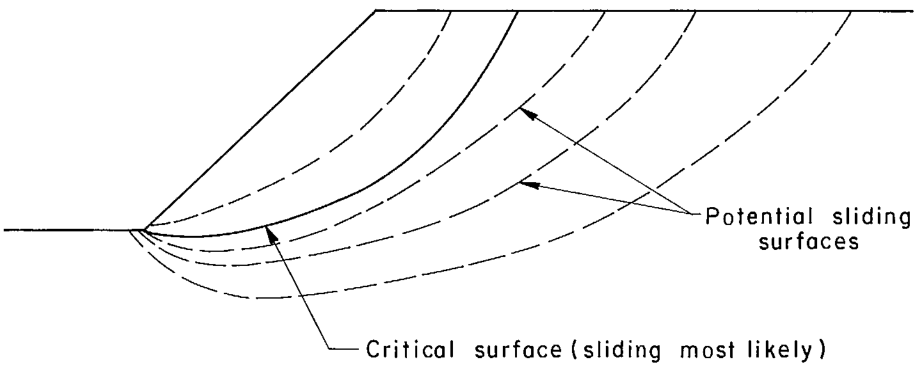
If this factor of safety is acceptable, four rows of 150 kip (660 kN) anchors at 20 ft (6 m) would provide

$4 \times 150/20 = 30$ kips/ft (440 kN/m)

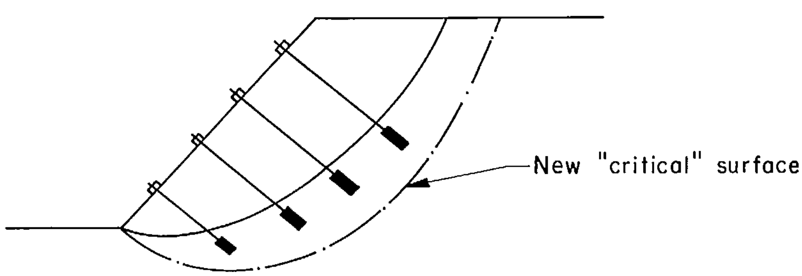
ROTATIONAL SHEAR

85. The rotational shear mode is not generally amenable to support. Rotational shear arises in homogeneous slopes of relatively soft, yielding material. Sliding occurs on a critical surface that is often approximately circular in section. The resistance to sliding on this surface invariably has a friction component, similar to the plane shear surfaces described above. In principle, rock anchors could be used to increase the normal stress on the critical surface and hence increase resistance against sliding.

86. However, the nature of rotational shear is such that there is an infinite number of possible sliding surfaces. The critical surface is the surface on which sliding is most likely to occur, but there will be a substantial number of surfaces on which sliding is only marginally less likely than on the critical surface. It is probable that the installation of anchors would merely result in a new "critical surface", as shown in Fig 27. The probability of sliding on this surface would be only marginally lower than on the original critical surface.



(a) ROTATIONAL SHEAR - NO SUPPORT



(b) ROTATIONAL SHEAR - SUPPORT

Fig 27 - Rotational shear has characteristic that several potential sliding surfaces are possible. If one such surface is supported, another may form. This means rotational shear is not amenable in general to support.

87. There are circumstances in which a preferential surface of sliding exists and forms part of a rotational shear surface. Such a surface is usually non-circular, as shown in Fig 28. In this case, anchors might well be used to increase resistance to sliding on the preferential surface. This is the only circumstance, however, in which support can be effectively used to stabilize rotational shear, and in general support is not recommended.

BLOCK FLOW

88. Where clearly defined planes of weakness do not exist or do not form a kinematically admis-

sible instability mode, breakdown of rock slopes may occur through crushing of the rock substance at the toe of the slope. This can lead to progressive breakdown and to eventual sliding with the slope degenerating into a blocky mass. Sometimes toe breakdown may lead to sliding on a discontinuity, as shown in Fig 29.

89. Conditions leading to the block flow mode of instability include relatively high stresses - approaching the rock substance strength - at the toe. These high stresses usually arise from existing horizontal stress (tectonic stress), exaggerated by stress concentrations at the toe.

90. This mode is not amenable to support.

SURFACE ROCK FALLS

91. Loose rock may be a controlling factor in slope design, both because the implied breakdown of the slope face may mean shallow bench angles and because extra-wide safety berms may be required to contain rock falls. Both these conditions will result in flatter overall slope angles.

92. The size of material involved in loose rock falls varies from large boulders to pebbles. The larger pieces may be stabilized individually by rock bolts, or held in place by mesh tied back to the rock face with bolts. One method of stabilizing all sizes of loose rock, including pebbles, is to spray the face with shotcrete. Shotcrete will bond strongly to any rock face, and develop enough shear and tensile strength to support large blocks. If necessary, shotcrete can be reinforced with cut wire included in the mix.

93. Calculating support provided by shotcrete, bolts or mesh may be possible, but the requirements of particular circumstances vary so much that no design rules can be given here.

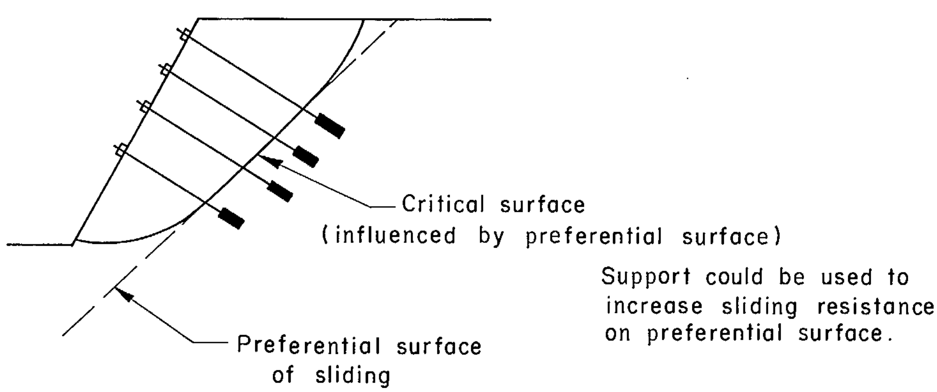


Fig 28 - Rotational shear with a preferential surface of sliding is amenable to support.

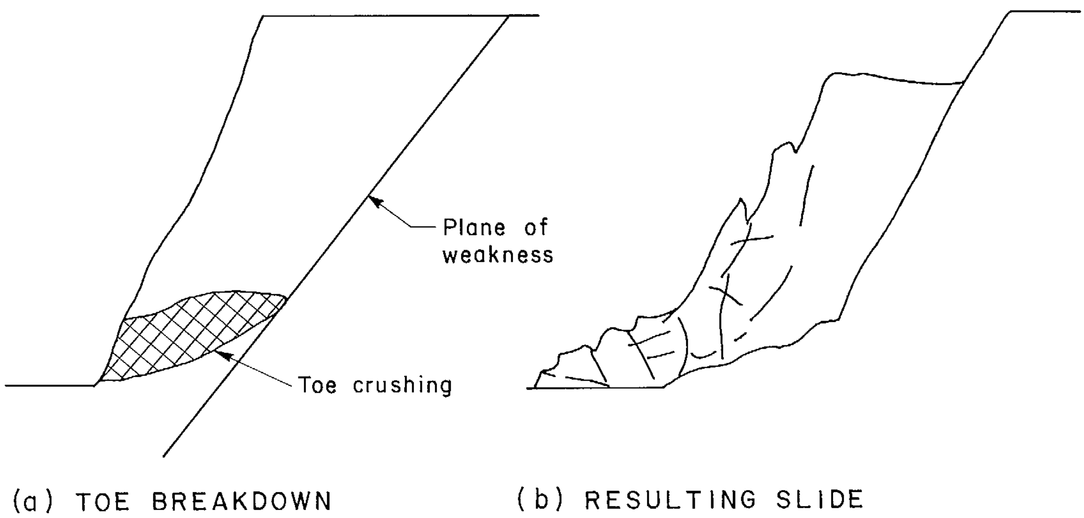


Fig 29 - Possible mechanism of block flow. This instability mode is not amenable to support.

DESIGN STAGES

MINE FEASIBILITY STAGE

94. In the early stages of mine planning, detailed consideration of support is usually premature. As possible instability modes of the proposed walls are identified, it is sufficient to note the support methods that are applicable. Then, as tables of slope angle versus reliability are prepared for use in the initial design, the influence of support can be considered.

MINE DESIGN STAGE

95. A decision is made at this stage whether or not to include support as an integral part of mine design. The steps required to make this decision include detailed investigation for design data - structural geology, groundwater and mechanical properties - identifying potential instability modes, estimating the improvement in stability that support may give, and estimating the costs of support. These data are input to the analyses described in the Design chapter and determine the influence and required magnitude of support.

96. The design calculations lead to an economic appraisal of support. If favourable, a tentative decision to include support in all or part of the walls can be made. The decision is tentative only because at this time an accurate engineering appraisal of the feasibility of support cannot usually be made. Additional investigations to augment the design data will be required. The extra information needed will necessitate additional drilling to amplify the structural geology and mechanical properties data, and tests to determine anchorage characteristics and likelihood of corrosion if rock anchors are used.

97. The additional design data are used to assess the feasibility of support. If the additional detailed engineering analyses are favourable, the tentative decision to use support as an integral part of mine design can become firm. However, as production begins, the support analyses must be re-appraised as further information on structure and rock properties becomes available.

98. The steps required in the mine design stage are shown in Fig 30. Appraising support can be considered an additional step parallel to the design process, providing input to design unless and until support is no longer considered.

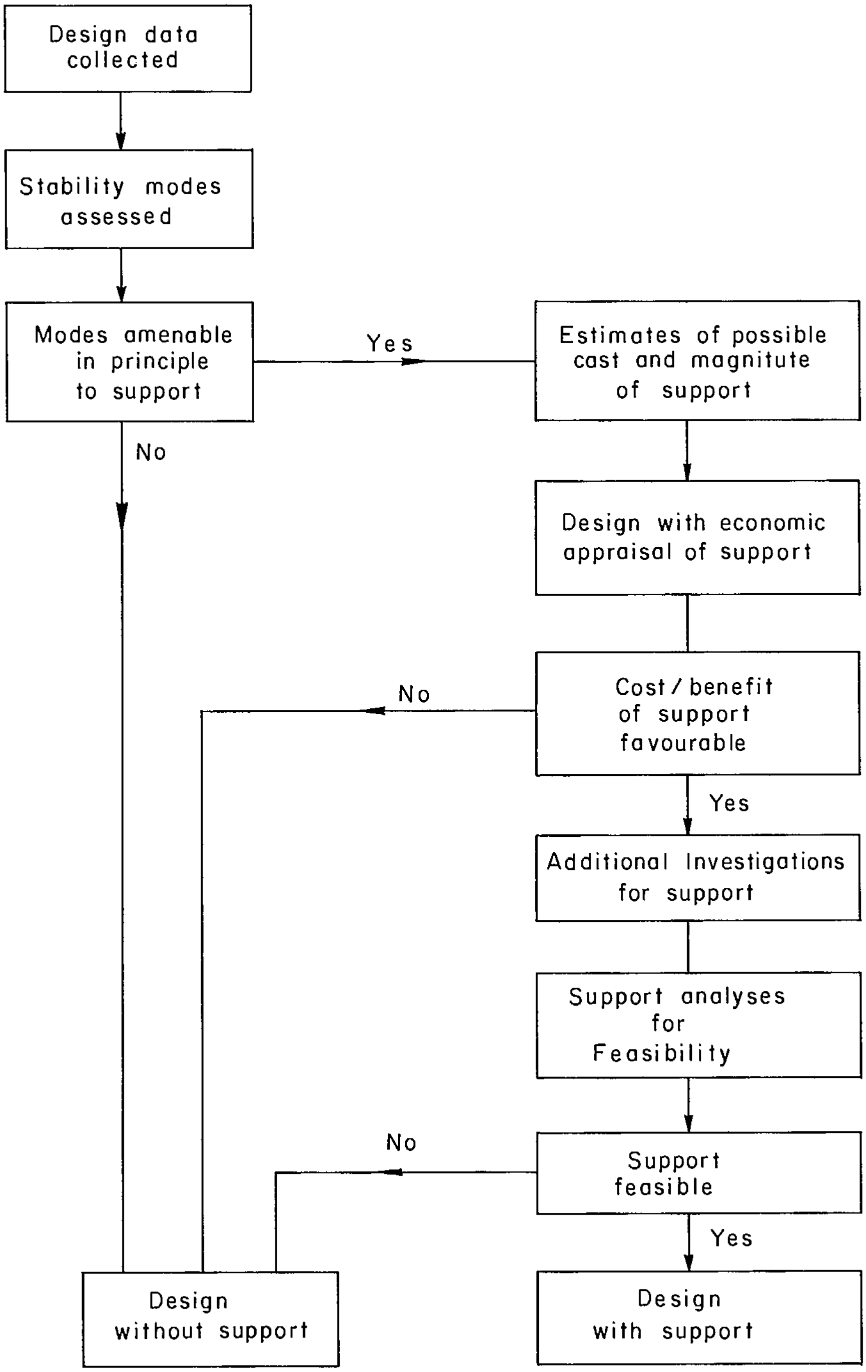


Fig 30 - Flow chart for support investigation at the mine design stage.

OPERATING STAGE

99. Most decisions to install support are made in the operating stage, for two reasons. First, the need for support is most often recognized when instability occurs during mining. Second, if support decisions are made in the mine design stage, the underlying assumptions must be verified during operations before a final commitment to support is made.

100. Support installation should begin as soon as possible, preferably as soon as any part of the appropriate slope has been exposed. This ensures good access and minimizes the slope "relaxation". As mining advances fresh design data are obtained, and the assumptions made in design must be checked as soon as possible. The original support calculations must be revised, if necessary, in the light of new data, and the relevance of support re-appraised. The steps to be followed are shown in Fig 31.

Report Specifications

101. The steps taken for the investigation, analysis and installation of support should be fully documented at all stages. A complete record of the data used, decisions made and procedures followed makes re-evaluation and similar work much easier in the future. The reports should be written as part of the support activity, when the important factors and data are fresh.

102. The following specifications form a check list of essential elements in a report.

a. A title page must include the name of the organization, the report title, the author, date, and possibly the number of the particular copy if close control of distribution is desired.

b. The summary must be a condensation of the important information in the report. It should not be merely an expanded title, listing what can be found in the report. An attempt should be made to write the summary in nonspecialist language to ensure effective communication.

c. A contents page must show page numbers of all headings and sub-headings; this supplements the summary for the busy person who only wishes to scan the report, as well as being useful to those who must refer to various sections repeatedly. An optional addition, which can be very useful, is a list of illustrations and tables.

d. Terms of reference or purpose and scope must start with a statement of the date and authorization of the work. It should be made clear to which stage of operations the report applies, eg, feasibility study, preliminary calculations to guide design analyses, or a full design and installation of support.

e. Data should describe the investigations for design data relevant to support and the values used for the analysis, with assumptions and their justification where relevant. A cross reference to the report of investigations for mine design may cover much of this. The instability modes anticipated and the chosen support methods should be described.

f. Analysis should describe the calculations made for support, including both preliminary estimates and full calculations using methods of the Design chapter. Even though bulky and tedious full details of calculations must be given, and all input data for, and relevant output from, computer programs listed. Future checking and re-appraisal of analyses will only be possible if all this material is available.

g. Installation should include detailed descriptions of the fabrication and installation of support, including safety precautions, equipment and manpower requirements and dates when each step was completed eg, dates of installation, tensioning, grouting, etc. Unusual conditions should be noted.

h. Monitoring should describe methods used to monitor the support system with results at the time of writing. Threshold values, ie, displacement or stress values expected to be significant should be stated and required action on recording these values described. For example, a rock anchor tensioned at 100 kips (450 kN) might be considered to have lost significant load if monitored at 70 kips (300 kN). Alternative coarses of action might then include: an attempt to retension; installation of additional anchors; or a review of the support

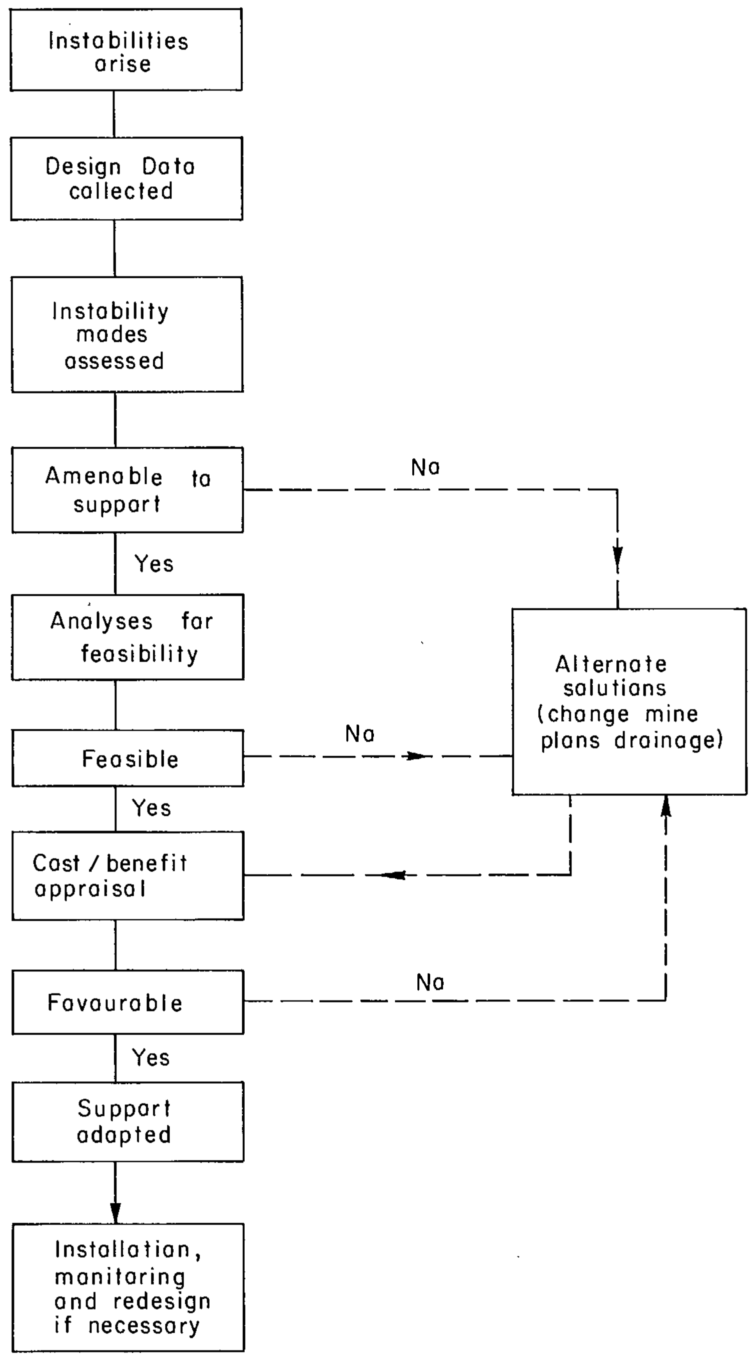


Fig 31 - Flow chart for support design at the operating stage.

system. It is also necessary to report regularly on the ongoing monitoring activity.

i. Costs should detail the cost, manpower and equipment required for the whole support activity, from investigation, ie, beyond that undertaken for mine design, through analysis and installation.

j. Conclusions and recommendations should describe the results of the support installation and comment on the significance of actual conditions encountered, as opposed to the design assumptions. Recommendations for further work, either for the current or a future support system, should be given.

k. Acknowledgements provide information on the assistance given by individuals and organizations to the author of the report and to those responsible for the design.

l. A Glossary or definition of terms should usually be included to ensure that ambiguities are eliminated from the text, eg, distinguishing, if appropriate, between the use of the terms benches and berms. Abbreviations and symbols, no matter how seemingly common, should also be defined in this section.

m. A List of References of reports and possibly outside publications should be included to permit tracing of the data used.

INSTALLATION

103. Detailed procedures for installing support are described in appendices to this chapter. This section gives the important considerations and general procedures required.

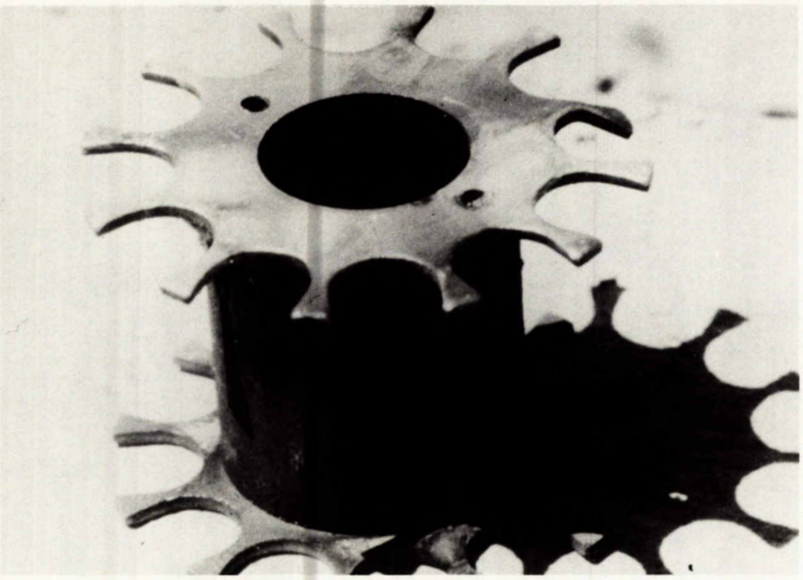
ROCK ANCHORS

104. The installation of rock anchors requires several distinct phases, some of which can take place concurrently. These are fabrication or assembly of the cable or bar, hole drilling, insertion and initial grouting to develop the bottom anchorage, preparation of the surface anchorage and tensioning. A delay must occur between initial grouting and tensioning to allow the bottom anchorage to attain strength. The tensioning phase is usually followed immediately by secondary grouting of the whole anchor, except where monitoring is required (see para 122 et seq).

105. Fabrication of a rock anchor is straightforward. A clean, dry working area long enough to lay out the full length of the anchor is required. Ideally, the area should be near the slope to minimize transportation. Spacers are used to hold the strands in a cable anchor as shown in Fig 32. Solid anchors are assembled with couplers supplied by the manufacturers if more than one length of bar is required. Assembly of solid anchors can be done on insertion, with sections being added as the anchor is pushed into the hole.

106. Some precautions are required. Damaged or rusted cable must be discarded as resulting weakness may cause breakage on tensioning. The bottom anchorage length must be clean and free from grease so that the grout will bond. Flame cutting must not be used as heat may destroy the high strength properties of the steel. Threaded lengths of solid anchors must be handled with care to avoid thread damage and subsequent weakness.

107. Hole drilling is best done with percussion equipment, both because costs are lower than for diamond drill holes and because the rougher hole side aids grout bond. Holes should be as straight as possible. Drilling fluids must not contaminate the hole; this requires that air, water or possibly water with detergent be used for flushing and cleaning. If drilling fluid is lost or weak zones are encountered, it is probable that subsequent grout for anchoring would be lost. It may be possible to overcome this by grouting the hole and redrilling. If not, a new, adjacent hole must be drilled.



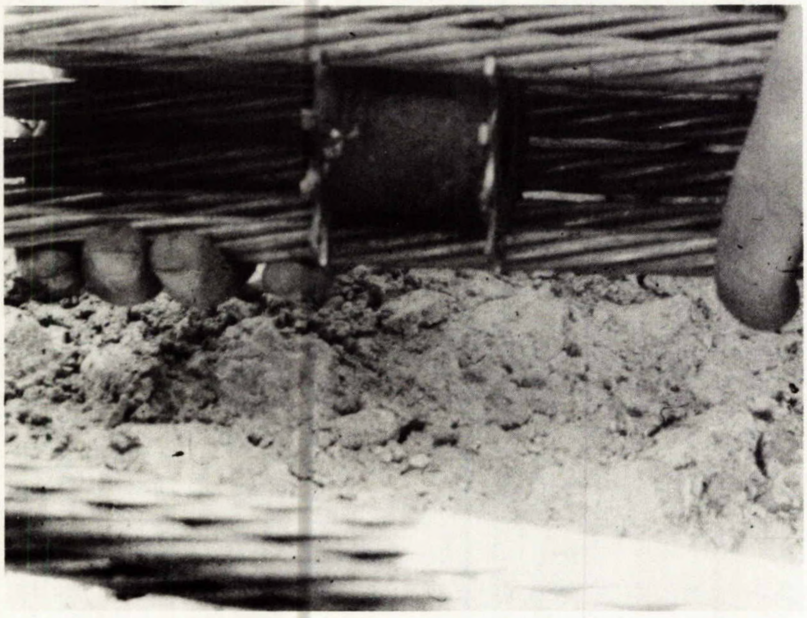


Fig 32 - Top: a spacer used in constructing a multi-strand rock anchor. Bottom: strands being assembled around a spacer.

108. The surface anchor block is cast from reinforced concrete. A guide tube projecting from the drill hole ensures alignment; a bearing plate may be welded onto the guide tube. The surface anchor block (Fig 5) ensures good transfer of the anchor tension to the slope without local overstress of the rock, and avoids eccentric tensioning of the anchor, which may cause anchor breakage. Figure 33 shows a rock anchor and jack positioned on a concrete bearing block; the anchor capacity is about 500 kips (1000 kN).

109. If the surface rock substance is strong, say with a uniaxial compressive strength above 20,000 psi (140 MPa) and the structure is tight, it may be possible to ream the hole collar to provide a seating for the anchor block. This would eliminate the need for a concrete block. Figure 34 shows a reamed anchor seat; the anchor in this photograph is 50 kip (220 kN) capacity.

Fig 33 - A large capacity rock anchor being tensioned.

110. Inserting the anchor through the block is usually best done with a mechanical tugger or a drill feed. Grouting of the bottom anchorage, using a pump and a tube attached to the anchor, is done as soon as possible after insertion.

111. The time between grouting and tensioning is usually two weeks, though quickset additives may reduce this. Tensioning is done with hydraulic jacks. Cables are locked with wedge-shaped grips; bars are usually held with a locknut. Safety precautions are necessary during tensioning to avoid injury if an anchor breaks and releases jacks, sections of cables or wedges.

112. Secondary grouting for corrosion protection is carried out as soon as possible after tensioning, unless the anchor is being monitored by a load cell. In that case, fabrication would include applying a protective coat to the anchor to prevent corrosion.

SHOTCRETE

113. Application of shotcrete requires scaling of the face to be sprayed, to remove as much loose material as possible. Supplies of aggregate and cement must be stockpiled close to the face, unless a batching truck - a mobile storage unit capable of measuring supplies of aggregate and/or cement - is used.

114. If mesh is to be used to reinforce the shotcrete, this must be laid and fastened close to the face with bolts. A requirement for using mesh

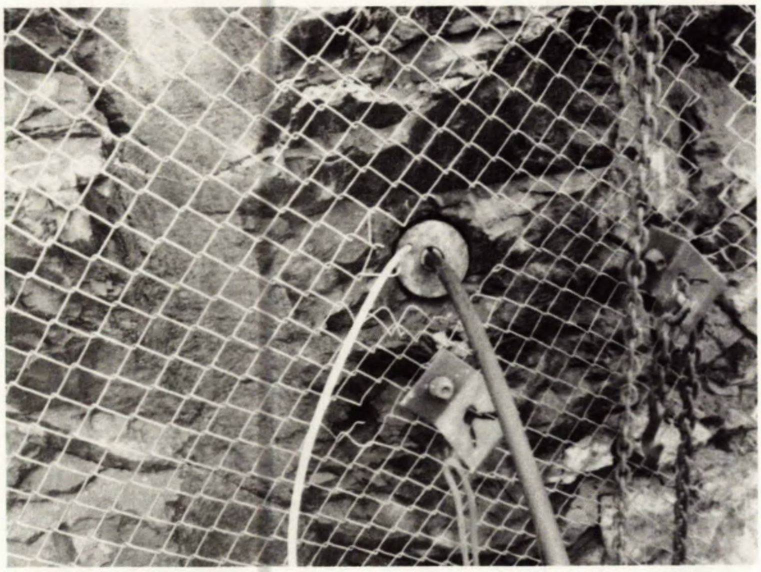


Fig 34 - Example of a rock anchor bearing directly on the reamed rock surface.

is that it must be kept within several inches of the slope face.

115. The mixing plant, with air to power the dry mix pump, water and flash-set additive, must be close enough to the face for spraying. For a large face, this may mean that equipment is required to move the plant rapidly to minimize down time. A mobile platform or a cage suspended from a mobile crane is required for spraying.

116. The thickness of concrete is judged by the operator, but thickness gauges - projecting rods of appropriate length - can be installed for guidance. Boxes of shotcrete for subsequent testing should be sprayed at the same time as the face.

117. Chopped wire for reinforced shotcrete is added at the mixer. A longer mixing time is required to ensure uniform distribution of the wire. Protective clothing and goggles are essential in handling the wire and working with the reinforced shotcrete.

MESH

118. The easiest way to lay mesh is to prepare sufficient lengths to cover one bench height with an additional length for anchoring on the berm above the bench. These lengths are then rolled, transported to the berm and connected to supporting beams of timber or steel. The mesh is then pushed over the bench, unrolling as it falls.

119. Small rock bolts can be used to tie the mesh to the face if necessary. However, for control of loose rock, mesh may be more effective if it hangs as a loose curtain, keeping falling rock close to the face.

BUTTRESSES AND RETAINING WALLS

120. Rock buttresses require the construction of a timber or concrete wall to retain the rock fill. Standard civil engineering construction procedures apply; the wall must be keyed to the ground or otherwise prevented from moving laterally and designed against overturning.

121. An important point, often over-looked, is that the fill must be designed so that water can easily flow through. If an overburden slope is supported, a further requirement is that fine material from the slope be prevented from washing into the fill, and changing its seepage properties. A filter can be incorporated in the buttress to prevent this.

MONITORING

122. Supported slopes are usually of critical importance to mining operations. Because of this, monitoring is of particular importance, both to confirm that the support system is functioning as designed and to reveal problems that may arise as early as possible. Early detection of instability, in supported as well as non-supported slopes, often permits remedial action to be taken.

123. Supported slopes are monitored by measuring the displacement of targets on the slope face, by measuring groundwater levels behind the slope, and, where relevant, by measuring support forces - ie, the force in rock anchors. Displacement and groundwater monitoring are described fully in the Monitoring and Groundwater chapters, and will not be covered here.

124. Monitoring of rock anchor or bolt forces requires a load cell attached to the head. The load on the anchor or bolt is transferred through the load cell to the rock face, as shown in Fig 35. The load in the cell is usually determined by measuring the strain change from the unloaded position and then reading the load from a previously calibrated chart. An alternative is a hydraulic cell, where load is transmitted through a fluid and the load is determined by measuring fluid pressure. A typical commercially available load cell is shown in Fig 36.

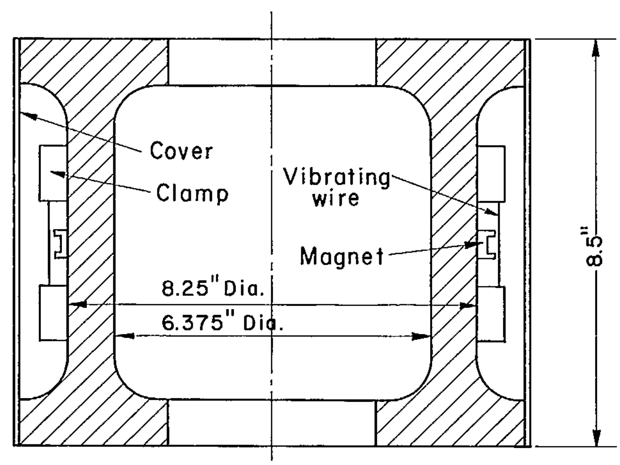


Fig 35 - Layout of 250 ton (2500 kN) load cell: strain in the steel cylinder through which the rock anchor passes is measured by vibrating wire gauges. Electrical read-out connections are not shown.

125. The significant characteristic of a rock anchor with a load cell is that the anchor must be left ungrouted except for the end anchorage. If

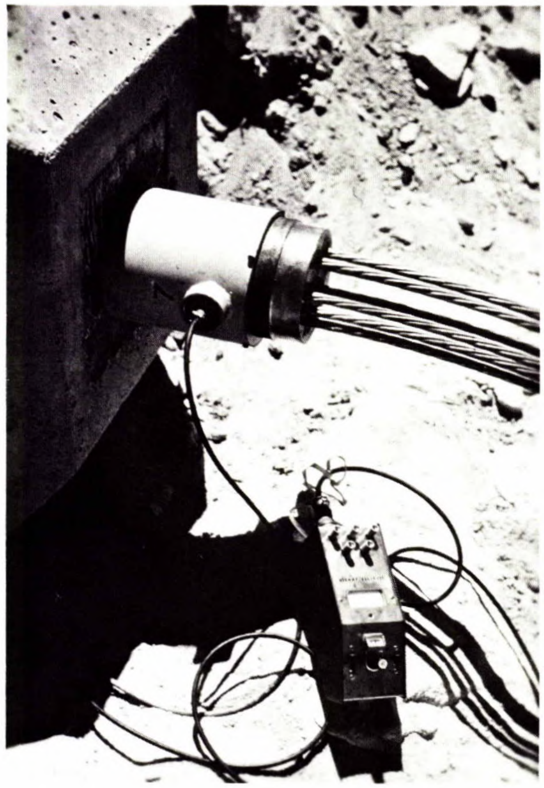


Fig 36 - A typical commercially available load cell.

the anchor is fully grouted, the load cell would only record changes in the top few feet of the anchor, which would not yield significant information.

126. Corrosion protection for an anchor with a load cell is therefore very important. The anchor must be protected for its required life and be unrestrained within the borehole. Such protection is best afforded by greasing and sheathing the anchor. The grease must be inert. Some bitumen preparations and petroleum-based greases contain sulphates, and may induce stress corrosion in the anchor. Paraffin-based greases appear to be the best for corrosion protection.

127. The location of monitored anchors must be considered from two aspects - the most critical for the support system and convenience of access for reading. Read-out units must be connected to the load cell, either to read hydraulic pressure or, more commonly, electric potential or resistance. Electric readouts may be remote, but access anchor for servicing may still be required.

128. The critical anchor locations for monitoring depend on the characteristics of the particular support system. As a rule, the longest anchors should be monitored, because they are more likely to intersect potential slip surfaces. Because the cost of a load cell increases with the maximum force to be measured, it may be economically sound to install long, low capacity anchors especially for monitoring. Thus an installation using anchors of 500 kip (2200 kN) capacity might be designed with additional 200 kip (900 kN) anchors for monitoring as well as for support. As many anchors as possible within cost limitations should be monitored.

129. A monitoring technique which is not recommended is the periodic re-jacking of an anchor head, to check the load at which the head lifts off. Apart from the equipment and access required, the disturbance to the anchor is inadvisable.

130. As with all monitoring systems, the results should be anticipated and action planned if deviations are recorded. There should be threshold levels to indicate for example, the need for further support, for re-analysis, or perhaps for increased frequency of monitoring. The values set depend on the characteristics of a particular support system.

131. The expected load behaviour of a rock anchor is as follows:

a. Immediately after the jack is released and the anchor tension is transferred to the surface anchorage, a drop in load of about 5% occurs. This is due to movement during load transfer, such as seating of anchor wedges.

b. In the following 5-10 days, a gradual further loss of load occurs. This is due to creep or relaxation in the anchor. The additional drop may be 5-10% of the initial load.

c. In the remaining life of the anchor, a further drop of up to 5% of the initial load may occur due to long-term creep in the steel.

132. Deviations from the above usually indicate problems. A drop in anchor load indicates either anchor failure or rock contraction, the latter being unlikely. An increase in anchor load indicates rock expansion which could mean the onset of instability. Note that a given movement in the rock mass will cause a greater load change in a short than in a long anchor.

133. There is no benefit in using load cells on untensioned anchors because these must be fully grouted to be effective. There also is no practical method to monitor forces in shotcrete.

COSTS

134. Estimating support cost is difficult because there is little large-scale experience in mining to draw on. Experience in civil engineering is usually not applicable because of differing purposes and standards.

135. The following examples provide a guide to the cost of additional activities beyond normal mining investigation and monitoring required for support. Three cases are considered: bench scale, moderate slope and large slopes. Cost details in 1974 dollars are given in Table 1.

BENCH SUPPORT

136. A bench 40 ft (12 m) high by 100 ft (30 m) long is to be supported with 20 solid bar rock anchors of 40 kips (180 kN) capacity, averaging 40 ft (12 m) long. Four anchors are monitored. Two diamond drill holes 50 ft (15 m) long are required for investigation and to provide test samples. Support cost is $1.75/ft^2$ ($19/m^2$).

MODERATE SLOPE

137. A slope 100 ft (30 m) high by 320 ft (100 m) long is supported by 64 cable rock anchors, each of 300 kips (1300 kN) working load and an average length of 100 ft (30 m). Ten anchors are monitored. Six diamond drill holes 150 ft (50 m) long are required for investigation. Five man days are spent in structural mapping. Shear box tests are carried out on core from the drilling program and on samples collected from the slope. Piezometers are installed in two of the diamond drill holes behind the slope crest. Support cost is $3.25/ft^2$ ($35/m^2$).

LARGE SLOPE

138. A slope 500 ft (150 m) high by 1000 ft (300 m) long is supported by 300 cable rock anchors, each of 450 kips (2000 kN) capacity and average length of 150 ft (50 m). Thirty anchors are monitored. Ten diamond drill holes 300 ft (100 m) long are required for investigation. Twenty man days are spent on structural mapping and 20 man days in logging core, collecting samples for testing and supervising field work. Four of the drill holes are used for piezometers. A laser distance-measuring instrument is used for monitoring. Support cost is $1.60/ft^2$ ($17/m^2$).

139. No allowance is included in these costs for ongoing monitoring effort or report writing. These activities are essential but should add only moderately to the total outlay. These rough

Table 1: Support costs (1974)

		Bench	Moderate slope	Large slope
Investigation and analysis				
Supervision at $150/day		150	750	2250
Diamond drilling at $10/ft ($33/m)		1000	9000	30,000
Structural mapping at $150/day		300	750	3000
Sampling and testing		200	2000	7750
Piezometers		-	1000	3000
Analysis at $150/day		150	2250	6000
Installation and monitoring				
Percussion holes	at $2/ft ($7/m)	1600	-	-
	at $3/ft ($10/m)	-	19,200	-
	at $4/ft ($13/m)	-	-	180,000
Rock anchors	at $3/ft ($10/m)	2400	-	-
(materials	at $10/ft ($33/m)	-	64,000	-
and labour)	at $12/ft ($40/m)	-	-	540,000
Load cells and readout		1200	5000	15,000
Laser unit and targets		-	-	14,000
TOTAL		7000	103,950	801,000

estimates indicate that rock anchor support costs are in the order of $1.50 to $2.50 per square foot of slope face ($16-$27/m^2), and that 10% to 20% of the total cost is required for analysis and design.

SHOTCRETE COSTS

140. The costs given below for shotcreting are based on actual material cost plus an hourly rate for equipment. Coverage per cubic yard with a 4 in. nominal thickness and 10% rebound loss is 70 ft^2/yd^3 (10 m^2/m^3). At an application rate of 15 cubic yards per hour (12 m^3/h) cost is $0.60/ft^2 ($6.50/m^2).

141. The cost of wire reinforced shotcrete is about $100 per cubic yard ($130/m^3) for material; equipment costs are similar but production per hour is lower at about 12 cubic yards per hour (10 m^3/h). The appropriate support cost is therefore approximately $1.65/ft^2 ($17/m^2).

	1974 dollars
Shotcrete material - aggregate and additive per cu yard (m^3)	$30 ($39)
Equipment, per hour	
Shotcrete machine	10
Crane	30
Concrete trucks, 2	50
Loader	20
Compressor	20
Labour per hour (6 men)	60
TOTAL	$190

142. No costs for other support methods (buttresses etc) will be presented because their use is limited to special circumstances that cannot be treated in a general way. Fuller cost information for rock anchors and shotcrete is given in the appendices.

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APPENDIX A

SIMPLIFIED ANALYSIS FOR SUPPORT OF WEDGE INSTABILITY

1. This appendix presents a simplified method for analyzing support of a wedge instability, suitable for manual calculations. The factor of safety approach is used.

2. Fig A-1 shows a wedge defined by two discontinuities, planes A and B, the slope face and a top surface which is assumed horizontal for the sake of simplicity. The forces acting on the wedge are weight, W, water pressures, U_a and U_b, and a support force, T.

3. The first step in the analysis is to establish that sliding is possible. The condition for this is that OP, the line of intersection of planes A and B, shall day-light, ie, exit on the face. This is most easily established by constructing a stereo projection of the instability, as shown in Fig A-2.

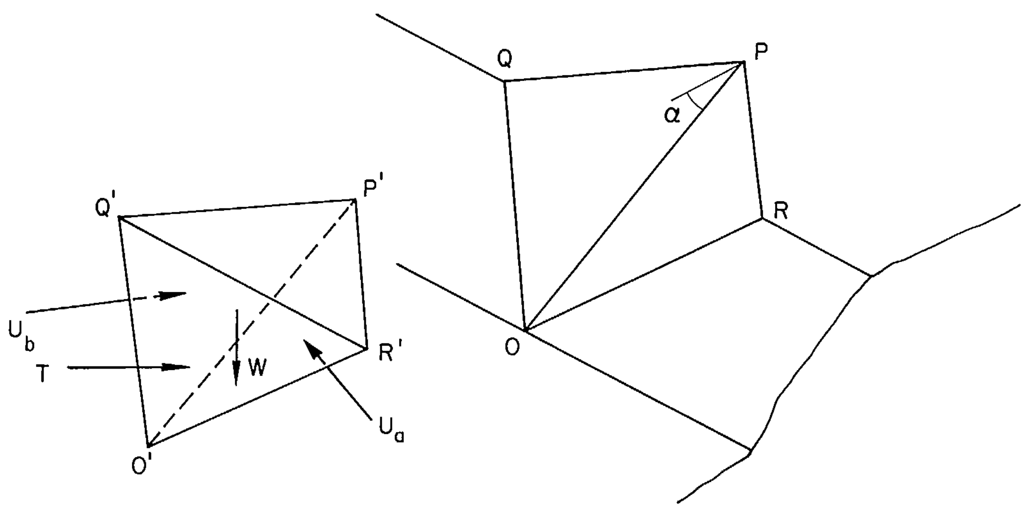


Fig A-1 - A wedge instability.

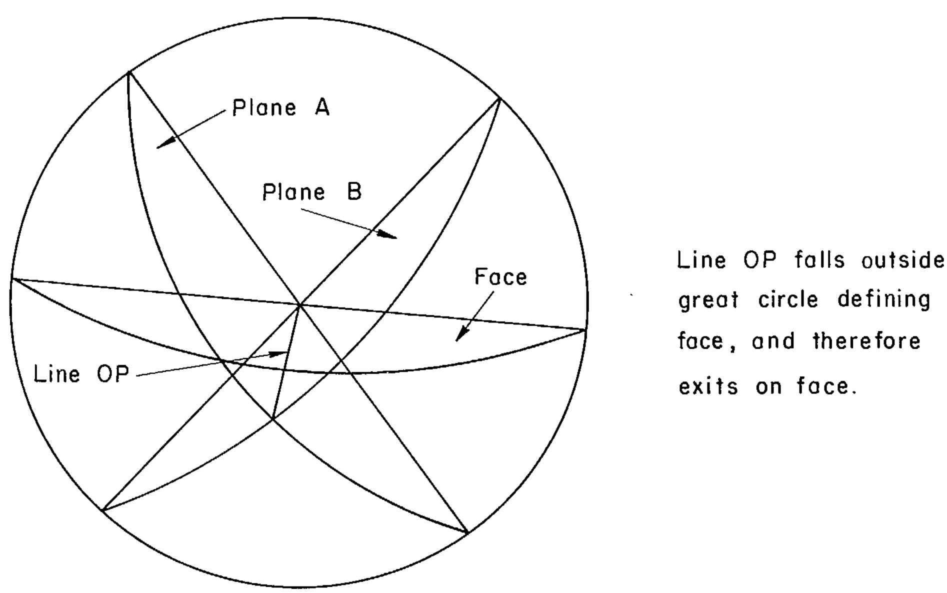


Fig A-2 - Stereo plot to determine kinematic admissibility.

4. A factor of safety for the wedge can be written by resolving forces parallel to OP (A-1).

$$FS = \frac{1}{W\sin\alpha}[N_a\tan\phi_a + c_aA_a + N_b\tan\phi_b + c_bA_b + T_o] \quad \text{eq A-1}$$

where W is the weight of the wedge

N_a, N_b are the net forces normal to planes A and B

c_a, c_b are cohesion for planes A and B

ϕ_a, ϕ_b are angles of friction for planes A and B

T_o is the component of support parallel to OP

α is the inclination of OP below the horizontal

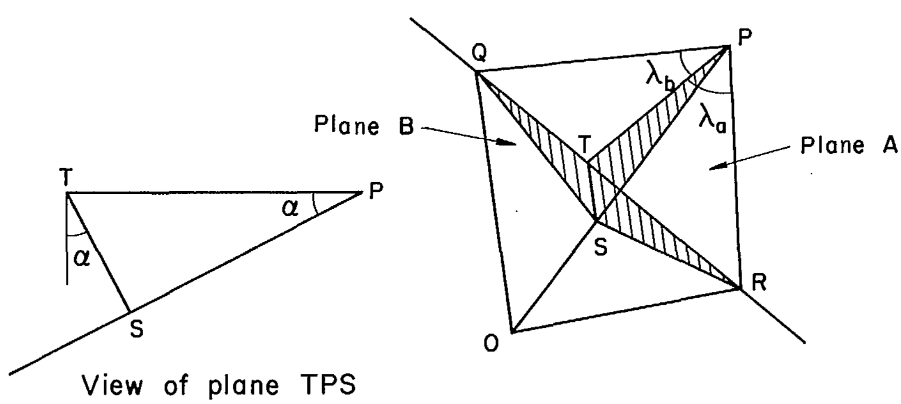
A_a, A_b are the areas of planes A and B.

N_a and N_b can be determined by resolving forces in a plane normal to OP, as shown in Fig A-3. T_a and T_b in Fig A-3 are the components of the support force, T, normal to the planes A and B.

5. The general solution of eq A-1 involves tedious but straightforward mathematics. A full analytical method - with some approximate assumptions - is given by Hoek and Bray (A-2). For the simplified method given here, it is assumed that plane TPS is normal to plane QRS (Fig A-3), ie, sliding is at right angles to the strike of the slope face. In this case, calculating the wedge weight and the areas of planes is simplified. The calculation procedures are illustrated by the following example.

6. A wedge is defined by the following planes:

plane	dip	dip direction
slope face	70°	90°
A	57°	120°
B	60°	50°



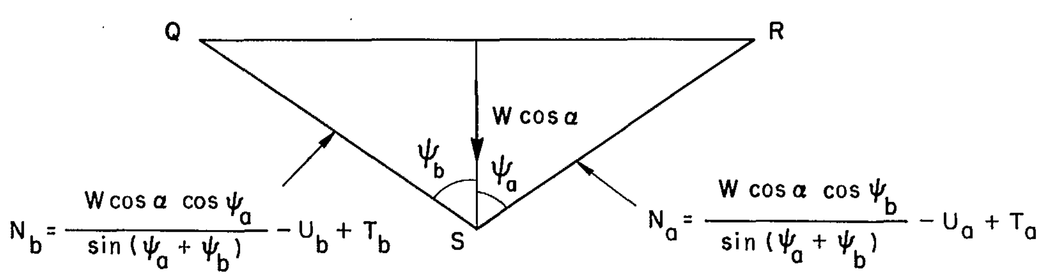


Fig A-3 - Definition of wedge geometry and forces for analysis.

The crest surface is horizontal.
Rock density is 170 pcf (2700 kg/m³)
Strength properties are:

c_a	c_b	ϕ_a	ϕ_b
8 psi (56 kPa)	5 psi (35 kPa)	42°	40°

The height of the wedge is 80 ft (25 m).

Step 1

7. The planes A, B and the slope face are plotted on a lower hemisphere equatorial stereo net, as shown in Fig A-4. The kinematic admissibility of the wedge is checked (point X falls further from the centre of the net than the great circle defining the face). The plane QRS is plotted; this is normal to OP (Fig A-3) and therefore OP is a pole to the plane QRS. The angles ψ_a and ψ_b (Fig A-3) can be read from the stereo projection, as shown in Fig A-4. They are the angles between plane TPS and planes A and B respectively, measured in plane QRS. For this example,

$$\psi_a = 65°, \ \psi_b = 56°$$

The dip, α, of line OP is measured on the stereo net; it corresponds to point X. For this example,

$$\alpha = 53°$$

The angle λ_a and λ_b are measured on the stereo net; they are the angles between plane TPS and planes A and B respectively, measured in the horizontal plane. For this example,

$$\lambda_a = 60°, \ \lambda_b = 50°$$

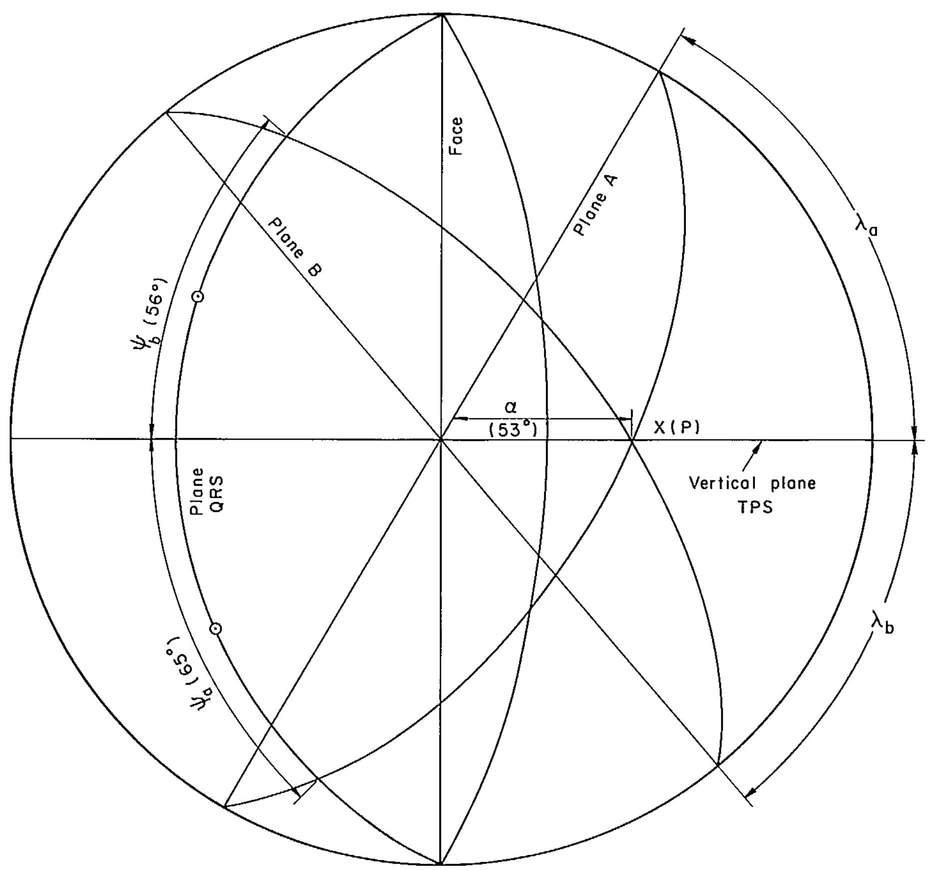


Fig A-4 - Determination of angles defining wedge geometry by stereo plot.

8. For the special case of plane TPS normal to plane QRS (as is true in the example given) the measured values can be checked by the formulas:

$$\cos\delta_a = \cos\alpha\sin\psi_a$$

$$\tan\lambda_a = \sin\alpha\tan\psi_a$$

where δ_a is the dip of plane A. (Similar equations hold for plane A).

Thus, in the example, for plane A

cos57 = 0.54 , cos53 sin65 = 0.55

tan60 = 1.73 , sin53 tan65 = 1.71

and for plane B

cos60 = 0.5 , cos53 sin56 = 0.5

tan50 = 1.19 , sin53 tan56 = 1.18

Step 2

9. The weight of the wedge and the surface areas of planes A and B are determined. These can be found geometrically or, for the special case of plane TPS normal to plane QRS, they can be calculated from the following formulas:

$$W = \frac{\gamma H^3}{6}(\cot\alpha - \cot i)^2(\tan\psi_a + \tan\psi_b)\sin\alpha$$

$$A_a = \frac{H^2}{2}(\cot\alpha - \cot i)\sec\psi_a$$

where i is the dip of the slope face;

for the example given,

$$W = \frac{1}{6}.170.80^3(\cot 53 - \cot 70)^2(\tan 65$$

$$+ \tan 56)\sin 53 = 6.38 \times 10^6 \text{ lbs } (2.9 \times 10^6 \text{ kg})$$

$$A_a = \frac{1}{2}.80^2.(\cot 53 - \cot 70)\sec 65 = 2950 \text{ ft}^2(275 \text{ m}^2)$$

$$A_b = \frac{1}{2}.80^2.(\cot 53 - \cot 70)\sec 56 = 2230 \text{ ft}^2(207 \text{ m}^2)$$

Step 3

10. The values of N_a and N_b for T = 0 are determined. This requires a calculation of water forces. The distribution of water pressure on the wedge surfaces is difficult to determine. One possible case of water pressure arises if the pressure is zero (ie, atmospheric) on the crest and face and planes A and B are completely saturated. The pressure could be assumed to be a maximum at the mid-point of the line of intersection (OP in Fig A-1), with a pressure head of H/2. If the pressure varies linearly, the force exerted on each side of the wedge is

$$U = \frac{AH}{6} \times \text{density of water}$$

where A is the area of the side.

11. For the example given, the forces due to water in the worst case would be

$$U_a = \frac{2950 \times 80 \times 62.4}{6} = 2.45 \times 10^6 \text{ lbs } (10.9 \text{ MN})$$

$$U_b = \frac{2230 \times 80 \times 62.4}{6} = 1.86 \times 10^6 \text{ lbs } (8.3 \text{ MN})$$

N_a and N_b are determined as shown in Fig A-3.

$$N_a = \frac{6.38 \times 10^6 \cos 53 \cos 56}{\sin(56 + 65)} - 2.45 \times 10^6$$

$$= 5.5 \times 10^4 \text{ lbs } (245 \text{ kN})$$

$$N_b = \frac{6.38 \times 10^6 \cos 53 \cos 65}{\sin(56 + 65)} - 1.86 \times 10^6$$

$$= 3.3 \times 10^4 \text{ lbs } (147 \text{ kN})$$

Step 4

12. The factor of safety without support is determined from eq A-1.

$$FS = [5.5 \times 10^4 \tan 42 + 8 \times 144 \times 2950 + 3.3 \times 10^4 \tan 40 + 5 \times 144 \times 2230] / [6.38 \times 10^6 \sin 53] = 1.0$$

indicating the wedge would just slide under the worst groundwater conditions.

Step 5

13. The support force, T, is resolved into components parallel to OP and normal to planes A and B. For simplicity, assume the force, T, is applied horizontally at right angles to the slope face (Fig A-5). The component parallel to OP is

$$T_o = T\cos\alpha$$

The component of T normal to OP is $T\sin\alpha$, and re-

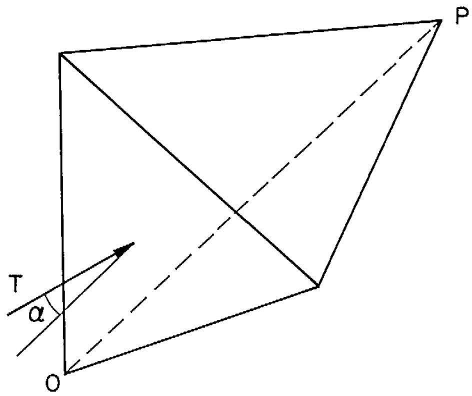


Fig A-5 - Application of support to stabilize wedge.

solving normal to planes A and B gives

$$T_a = \frac{T\sin\alpha \cos\psi_b}{\sin(\psi_a + \psi_b)}$$

$$T_b = \frac{T\sin\alpha \cos\psi_a}{\sin(\psi_a + \psi_b)}$$

The magnitude of support required to increase the worst case factor of safety to say, 1.1 can be determined from eq A-1. T_a and T_b must be included in the terms for N_a and N_b (Fig A-3).

$$1.1 = [(5.5 \times 10^4 + T_a)\tan 42 + 8 \times 144 \times 2950$$

$$+ (3.3 \times 10^4 + T_b)\tan 40 + 5 \times 144 \times 2230 + T_o]$$

$$/[6.38 \times 10^6 \sin 53]$$

Solving for T gives

$$T = 374{,}000 \text{ lbs } (1660 \text{ kN})$$

This support force could be provided by ten 40 kip (180 kN) rock anchors.

14. The simplified method described above has several limitations. A uniform pressure distribution on the wedge sides is assumed. The direction of sliding is assumed to be normal to the slope face. The water pressure distribution is the worst possible. Allowance for these limitations may be made in practice. For example, the weight and side areas of the wedge can be determined more accurately and a reduced water force used. Despite these limitations, the simplified analysis will permit an initial appraisal of wedge stability, and guide later, more detailed analyses.

REFERENCES

A-1 Jaeger, J.C. "Friction of rocks and stability of rock slopes"; Geotechnique 21, no. 2, pp 97-134; 1971.

APPENDIX B

ROCK ANCHOR FABRICATION AND INSTALLATION

1. This appendix describes how to determine the size of an anchor and gives procedures for fabrication and installation. It is assumed that the slope designer has specified the required working loads, lengths and locations. The descriptions given draw partly on a previous report by Coates and Sage (B-1).

STEEL

2. Rock anchors must be made from high-strength steel - typical ultimate strength is about 200,000 psi (1400 MPa) for steel cables or wires and about 150,000 psi (1000 MPa) for solid bars. The reason is that steel will creep in tension. In a rock anchor that is given an initial fixed extension, a gradual reduction of anchor load occurs in the weeks and months after installation. This loss of load is roughly the same for all types of steel. For mild steel, which has a comparatively low strength, the loss amounts to nearly all the load that can initially be applied to the steel, thereby making mild steel useless for all pre-stressing work, including rock anchors. For high-strength steel, which may be tensioned to eight times the load of mild steel, this loss becomes only a small proportion of the total load.

3. Prestressing cables, in addition to being manufactured from high strength steel, are designed especially for ease of tensioning and anchoring. A typical "strand" is shown in Fig B-1. It consists of six wires around a central slightly larger wire. When the strand is gripped, the outer wires clamp tightly against the middle wire, thus locking the entire strand. A cable rock anchor consists of one or more strands tied together - a large anchor might have 20 strands.

4. Solid bar rock anchors are available up to about 2 in. in diameter. They are manufactured in lengths up to about 30 ft. Lengths can be joined with couplers to form longer anchors; the couplers are so designed that the coupled joint is as strong as the remainder of the anchor.

ANCHOR SIZE

5. There are several sizes of cable anchors available. Generally, the cable with the fewest strands will be cheapest and simplest to install; for example, a cable with four 0.5 in. (1.25 cm)

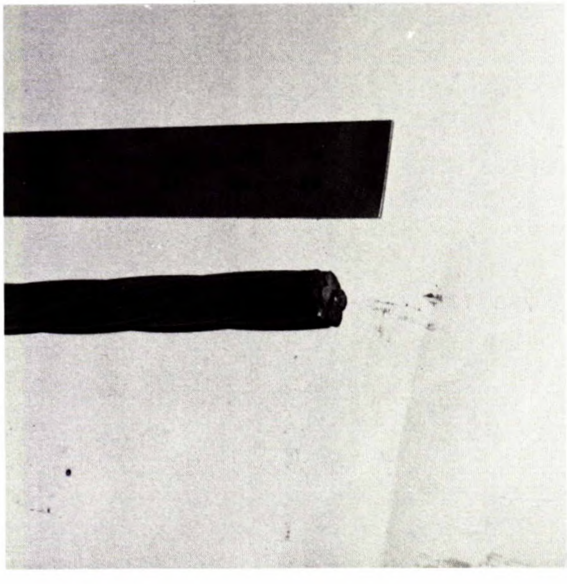


Fig B-1 - Typical pre-stressing strand.

strands has about the same capacity as one with three 0.6 in. (1.5 cm) strands, so the 3-strand cable is preferable in this case.

6. The load capacity depends on the number, size and strength of the strands in the cable. The working load in a strand is typically 60% of its breaking load and the number of strands required is found by dividing the required cable capacity by this working load. As an example, suppose a cable with a capacity of 250,000 lb (1100 kN) is required. If a 0.6 in. (1.5 cm) strand has a breaking load of 54,000 lb (240 kN), the cable will need:

$$\frac{250{,}000}{0.6 \times 54{,}000} = 7.6 \text{ or } 8 \text{ strands}$$

7. The factor 0.6 in the above calculation is the 60% of ultimate load recommended for design purposes. This figure is arrived at in the following way. At loads above 90% of the breaking point, a strand begins to stretch considerably. Consequently, 80% of the breaking load is usually specified as the maximum allowable load at any time. The process of anchoring the strand at the surface after jacking results in some loss of extension and hence of load. If jacked to 80% of ultimate strength, a strand will typically have a load of 70% of ultimate strength after anchoring. The drop from 70% to 60% is due to creep of the steel. Table B-1 shows capacities and sizes of some typical cable anchors.

8. There is usually a choice of anchor head and tensioning method. The strands can either be tensioned all together, or separately.

9. Separate tensioning is preferable since the jack required is smaller, more portable and can be pumped by hand. Tensioning all the strands together, however, is faster. It also has advantages when short - less than 30 ft (10 m) - cables are used because the loss of load on anchoring, which may be critical in a short cable, can sometimes be reduced. Figure B-2 shows typical anchor heads required for the two methods of tensioning. Figure B-3 shows the two types of jack required.

GROUTED ANCHORAGE LENGTH

10. The length of grouted anchorage depends on the anchor load, strength of grout, surface area of strands, area of grout/rock interface and the strength of the surrounding rock. However, in practice the governing factor is always the area and shear strength of the grout/rock interface.

11. This strength should be determined by laboratory tests or by a field pull test. In the former, a grout plug is poured into a hole through a typical rock sample (Fig B-4). The force to push the set plug out is measured.

12. The pull test is preferable to the laboratory test because it simulates field conditions. A short length of anchor is grouted into a borehole, and pulled to failure after the grout has set. The load at failure can be used to determine the grout/rock bond strength. This should be reduced by a factor of safety of, say, 1.5 for actual grout length calculations. The grout length required is calculated as shown in the following example.

13. A rock anchor of 180 kips (800 kN) working load is to be grouted in a 4 in. (10 cm) hole. Grout/rock bond strength in shear is found from tests to be 900 psi (6200 kPa). The length is calculated using a grout/rock bond strength of 900/1.5 = 600 psi (4100 kPa). The maximum anchor load expected occurs during tensioning and is 33% higher than the working load - ie, 80% of ultimate strength during jacking and a working load of 60% of ultimate strength, as described above. The length required is:

$$\frac{180 \times 1.33 \times 10^3}{600 \times \pi \times 4} = 32 \text{ in. (80 cm)}$$

14. In practice, the grout length should be at least 10% of the anchor length, with a minimum of 10 ft (3 m), to allow for inaccuracies in hole length.

ANCHOR ASSEMBLY

15. Once the required anchor length, including anchorage and an additional 5 ft (2 m) for jacking, is known, assembly can begin.

Table B-1: Typical sizes and capacities of cable rock anchors

No. of strands	Strand size (inches)	Ultimate strength of cable (kips)	Maximum allowable load while jacking (kips)	Design load (kips)	Overall diam. of cable (inches)
1	0.5	41.3	33.0	24.8	0.5
	0.6	54.0	43.2	32.4	0.6
2	0.5	82.6	66.1	49.6	1.0
	0.6	108.0	86.4	64.8	1.25
3	0.5	124.0	99.2	74.4	1.5
	0.6	162.0	.130.0	97.2	1.75
4	0.5	165.0	132.0	99.0	1.75
	0.6	216.0	173.0	130.0	2.25
5	0.5	248.0	198.0	149.0	2.25
	0.6	324.0	259.0	194.0	2.5

16. Solid rock anchors are assembled as they are inserted into the anchor hole. The first length is inserted until only the threaded section is exposed and a second length added with a coupler. Additional sections are similarly joined.

17. The grouted anchorage can be made more effective by a nut and washer (Fig B-5). Nuts and couplers must be carefully assembled to avoid damaging the threads and so weakening the anchor. Plastic tubes for grout and air-bleeding must be taped to the bar. Tubes should have a minimum inside diameter of 0.5 in. (1.25 cm).

18. Multi-strand cable anchors must be assembled before insertion. Strand is usually available in rolls. The required lengths are cut by carborundum wheel - flame cutting should not be used because heat may damage the strand - and laid out in a clean, dry area (Fig B-6). The strands are taped around spacers (Fig B-7) which separate the strands so that each can be bonded. The spacers also prevent strands from tangling, which may complicate tensioning. Strands should not be bent sharply and any that are kinked should be rejected.

19. Prestressing strand encased in plastic grease-filled tubes is commercially available. This type may facilitate fabrication of anchors if this form of corrosion protection is desired.

20. A nose cone of shaped metal can be added to help cable insertion; alternatively, the strands can be staggered slightly and bound together to create a tapered end (Fig B-8). Plastic tubes for grout and air-bleeding are required. These can conveniently be run through holes in the spacers.

ANCHOR HOLE

21. Percussion drilling for rock anchors is recommended, both for economy and to improve bond between grout and rock. The anchor hole should be 1 in. (2.5 cm) larger in diameter than the assembled cable. Approximate cable sizes are given in Table B-1, but these should be checked on site. The actual length of hole drilled must be recorded.

22. If bad ground or marked loss of water is

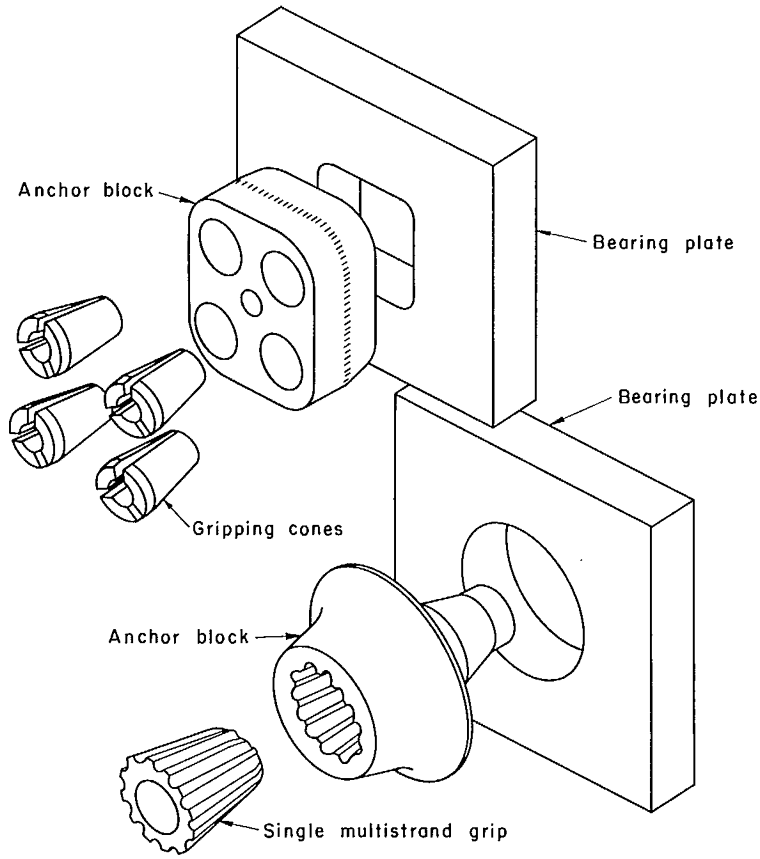


Fig B-2 - Typical surface anchor blocks.

encountered, the hole will probably be unsatisfactory - it may cave before the anchor can be inserted or grout may be lost. It may be possible to grout and redrill, but it is better to drill a fresh hole nearby if possible. Holes should be tested by filling with water and noting the rate of loss of water. A loss rate under gravity of more than 0.03 gallons per square foot of hole wall would suggest grout would be lost.

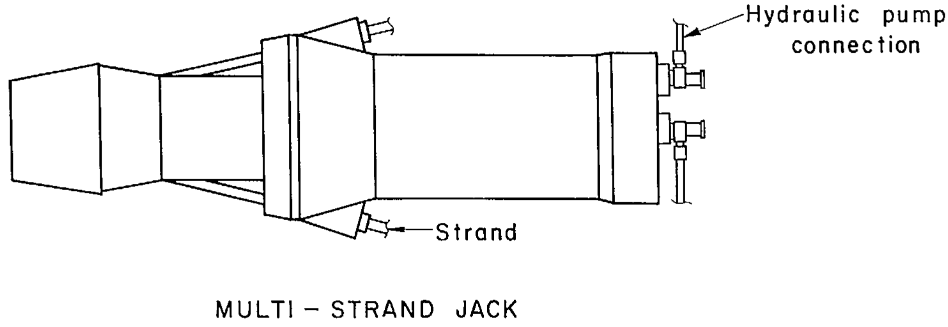
23. Holes should be as straight as possible. A curved hole may make anchor insertion difficult and can cause problems in tensioning the anchor because of friction on the borehole sides. Between completion of the hole and insertion of the anchor, the collar should be plugged with rags or other suitable material to prevent entry of extraneous objects.

ANCHOR INSERTION

24. Insertion is usually done manually, which is easier with solid anchors than with flexible cables. Long cables are most difficult to insert and may require the help of many workers. Mechanical assistance with a drill feed or block and tackle may be feasible. The true length of the borehole should be marked on the anchor so that the extent of insertion is known. It is possible that loose material in the borehole will prevent some penetration, but the loss of a little depth may not be serious. The consequences of the loss of more than about 5% of the intended length may require reappraisal of the design.

SURFACE ANCHORAGE

25. For low-capacity rock anchors of less than



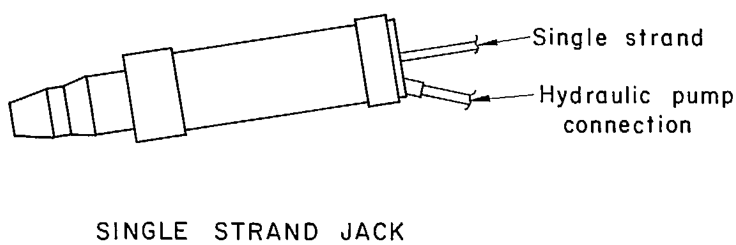


Fig B-3 - Typical jacks for tensioning anchors. The upper jack tensions a number of strands at the same time; the lower, only one strand at a time.

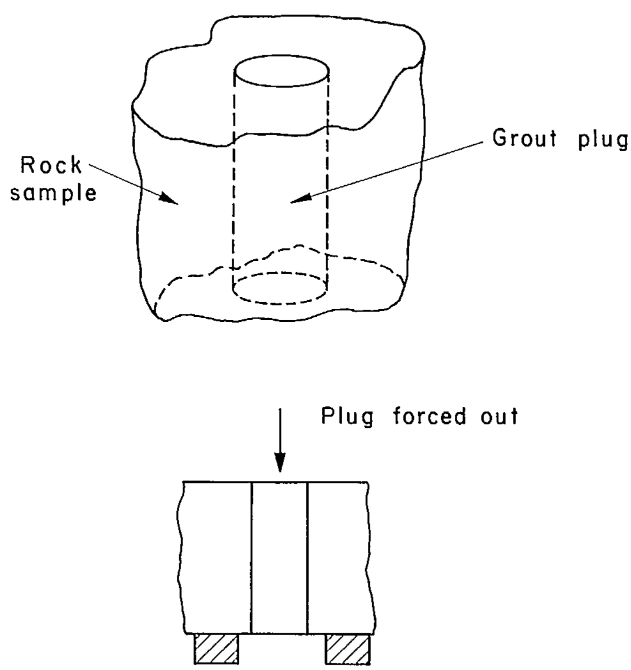


Fig B-4 - Layout of test to determine grout/rock bond.

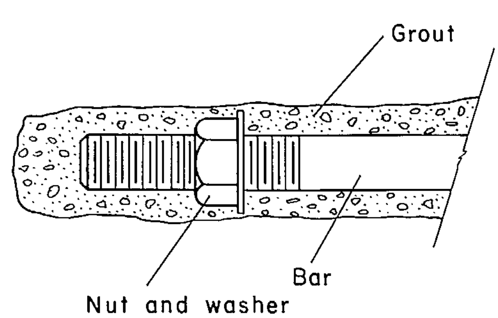


Fig B-5 - A nut and washer facilitates forming end anchorage for solid bar.

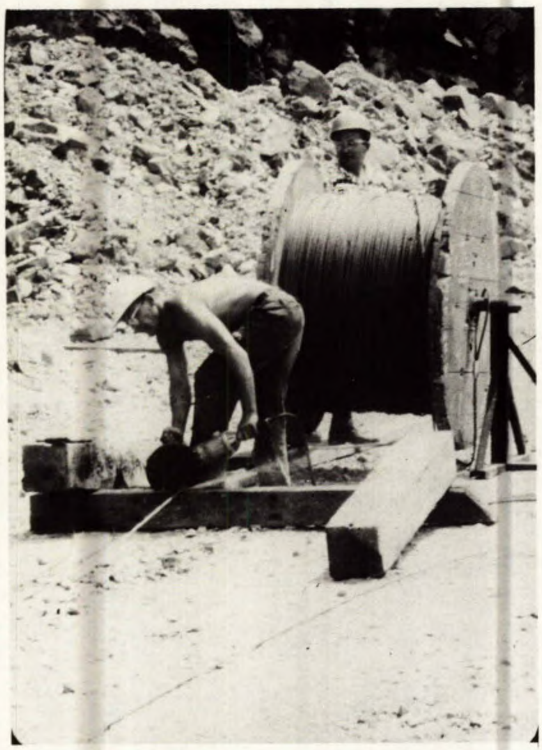


Fig B-6 - Cutting pre-stressing strand prior to laying out an anchor. Cutting is done with a carborundum wheel.

100 kips (450 kN) and if the surface rock is strong and any joints are tight, the rock can be reamed to provide seating for the anchor block (Fig B-9). This is done before the anchor is inserted.

26. In most cases, however, a reinforced concrete bearing block is required. This block spreads the load from the anchor head uniformly over a relatively large rock area. A typical block is shown in Fig B-10.

27. The bearing area on the rock must be large enough to ensure the rock is not overstressed. The maximum stress can be found from uniaxial tests on rock samples. However, a relatively modest size block will usually result in a satisfactory low stress. For example, a 2 ft by 2 ft (60 cm x 60 cm) bearing area for a 500 kip (2200 kN) anchor gives a net stress of less than 1000 psi (7000 kPa), which is within the bearing strength of most rock types.

28. The 45° sides of the block mean that the concrete is in compression and therefore the reinforcement required is nominal. Reinforcing bar of 0.75 in. (20 mm) diameter arranged as shown in Fig B-11 is satisfactory. The bearing area beneath the anchor block must be large enough to ensure that the average stress is within the concrete compressive strength. Typical strength is about 4000 psi (28,000 kPa); a 400 kip (1800 kN) anchor would require a bearing area about 10 in. by 10 in. (25 cm x 25 cm), so that the average stress is

$$\frac{400{,}000}{10 \times 10} = 4000 \text{ psi (2800 kPa)}$$

A steel plate welded to a guide tube, as shown in Fig B-12, is the simplest way of ensuring an adequate bearing area.

29. Concrete surface anchor blocks should be cast after the anchor is installed. They are not cast before in case drilling problems or difficulty in inserting the anchor mean the hole must be abandoned.

30. Anchor tensioning must not be attempted before the concrete bearing block has reached the required strength. This may be 28 days at temperatures above 10°C for ordinary portland cement. If desired, a rapid-hardening cement can be used for the bearing block.

GROUTED ANCHORAGE

31. Anchor grout is a neat cement/water mix, sometimes with an expansion agent. Ordinary portland cement is recommended but if a rapid set time is desired, a high-early-strength portland cement can be used. Quick set additives such as calcium chloride or high-alumina cements are not recommended as their chemical constituents may cause stress corrosion.

32. The recommended grout mix is 4 to 4.5 gallons (Imperial) of water per 100 lb (18-20 l per 45 Kg) of cement. Expansion agents should be added according to the manufacturer's direction.

33. Forming the grouted anchorage is often called the primary grouting stage. A grout pump

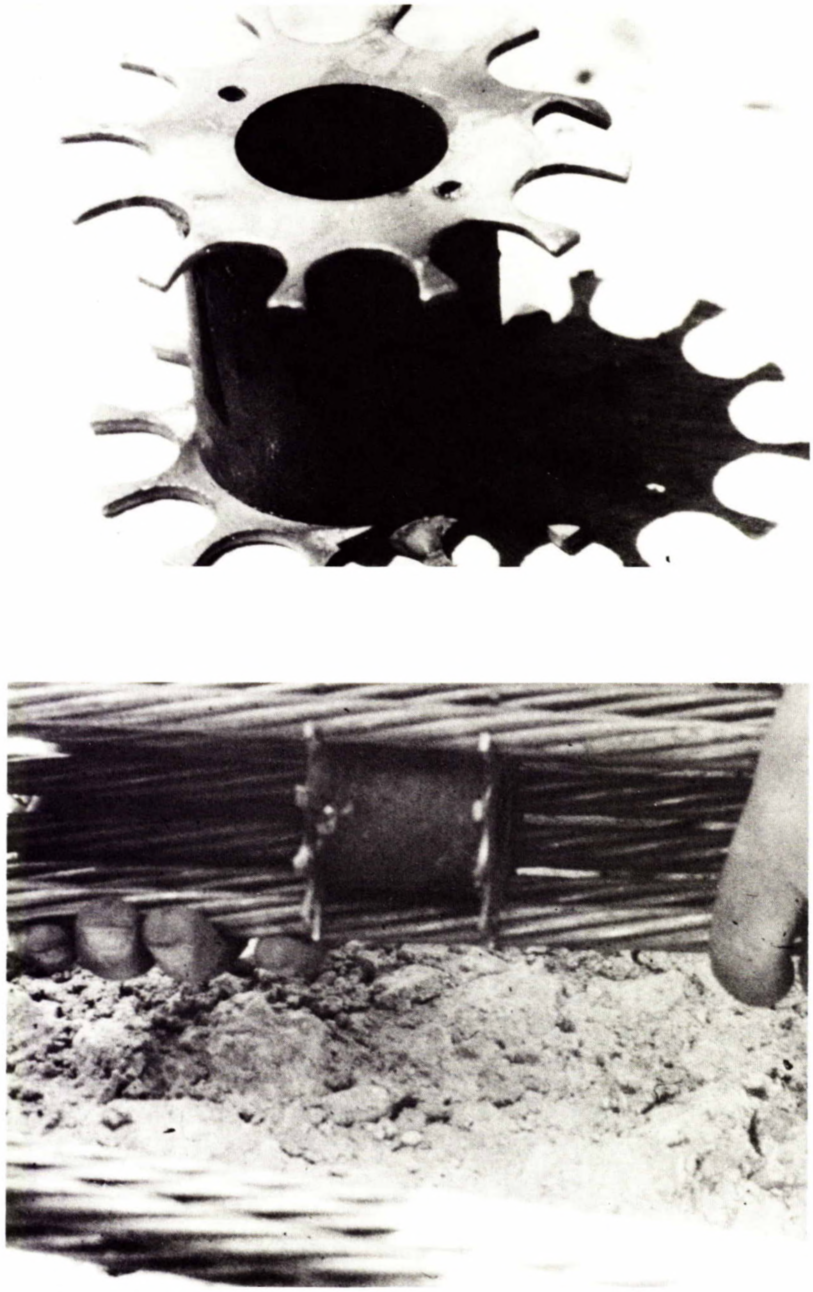


Fig B-7 - Spacers for assembling rock anchors. A spacer, shown in the upper photograph, keeps strands separate as shown in the lower photograph. Wire or steel strapping holds the strands firmly on the spacer.

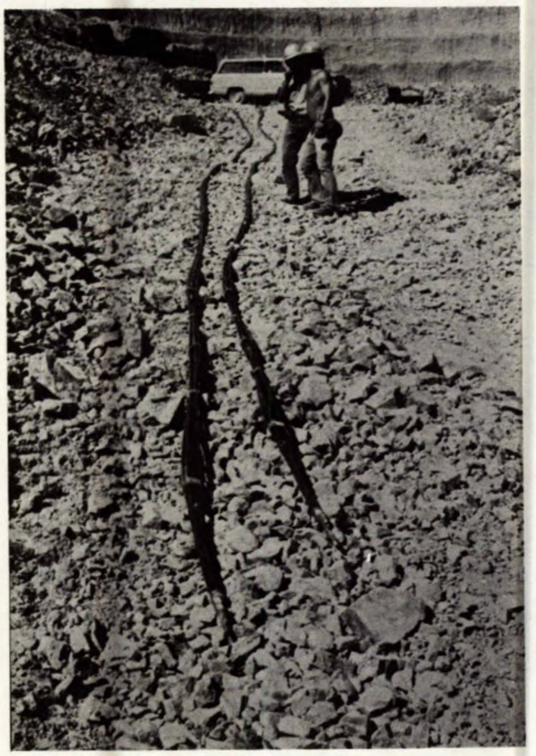


Fig B-8 - Inserting strands is easier if tapered at the end as shown at left, or if a nose-cone is fitted as shown at right.

(Fig B-13) is required; these can be bought or rented commercially. The volume of grout required is calculated from the length of primary grouting, the diameter of the hole and the cross sectional area of the anchor. However, the length should be checked by sounding with a rod after grouting in case of loss through joints.

34. Anchor tensioning must not be attempted before the grout has reached the desired strength. Test cylinders for strength determination should be cast at the time of primary grouting.

ANCHOR TENSIONING

Extension

35. An important part of the design and installation of a rock anchor is calculating the expected extension. New solid bars and high-strength steel strands are certified as to load and extension properties of the steel. Strand is usually supplied in reels from which the lengths required for a cable anchor are cut; each reel has its own certificate.

36. Figure B-14 shows a typical load/extension curve. The load applied in jacking the anchor to 80% of the ultimate strength is marked on the figure; it lies on the straight line section of the curve. From the chart, the extension for any load can be determined. For the particular chart shown, a load of 30,000 lb (130 kN) would cause an elongation of 0.008 in. (0.2 mm) in a length of 1 in. (2.5 cm). Therefore, a 25 ft (8 m) length of strand would stretch by: 25 x 12 x 0.008 = 2.4 in. (6 cm). In practice, a cable in a hole will elongate less than the theoretical amount owing to rock friction.

37. The expected loss in anchor load during installation can be calculated. A typical anchoring operation may result in the strand pulling into the hole (draw-in) by 0.25 in. (6 mm). For

Fig B-9 - The rock at the surface of the anchor is sufficiently competent for the block to bear directly on the rock. The anchor hole has been over-reamed to allow this.

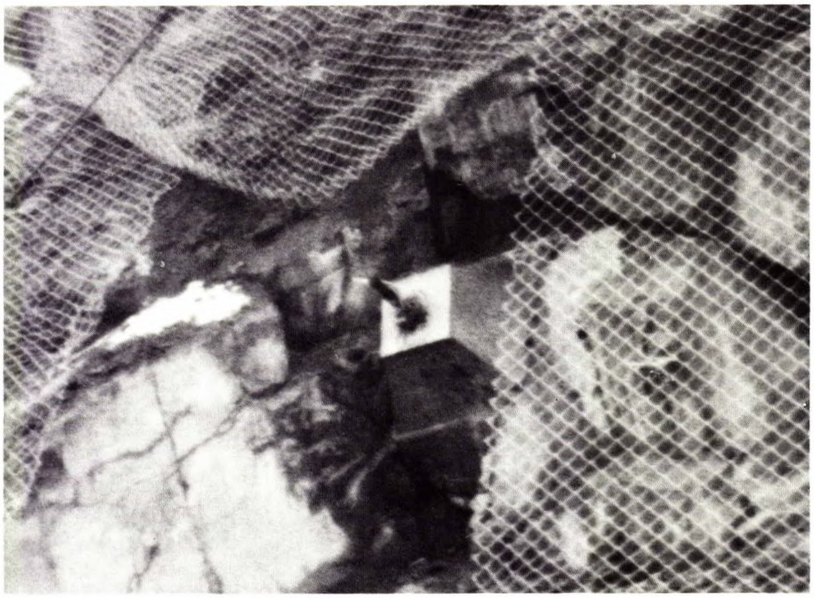


Fig B-10 - A reinforced concrete bearing block.

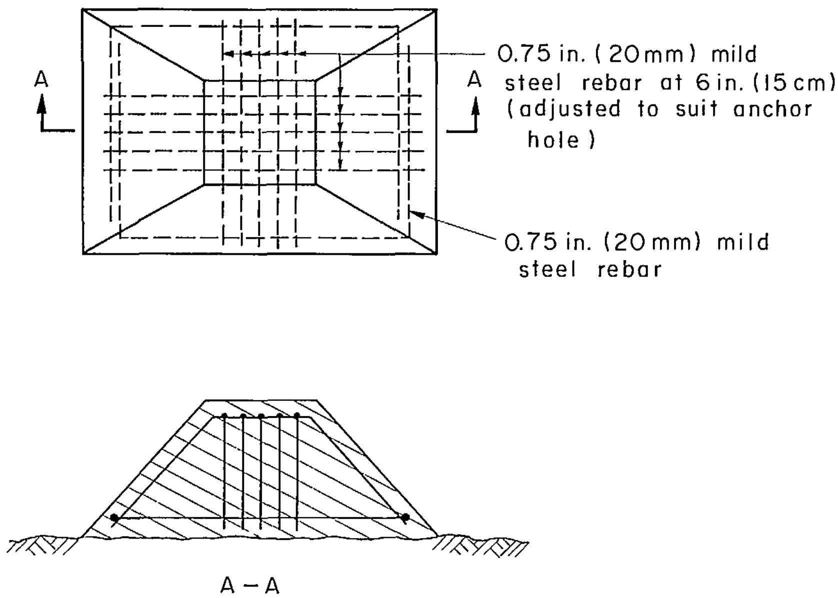


Fig B-11 - Layout of reinforcement for a concrete bearing block.

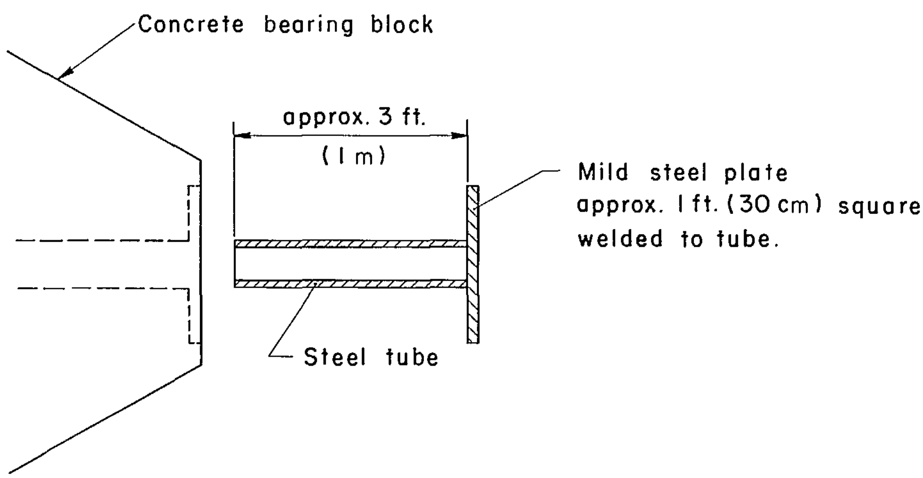


Fig B-12 - A mild steel guide tube and bearing plate.

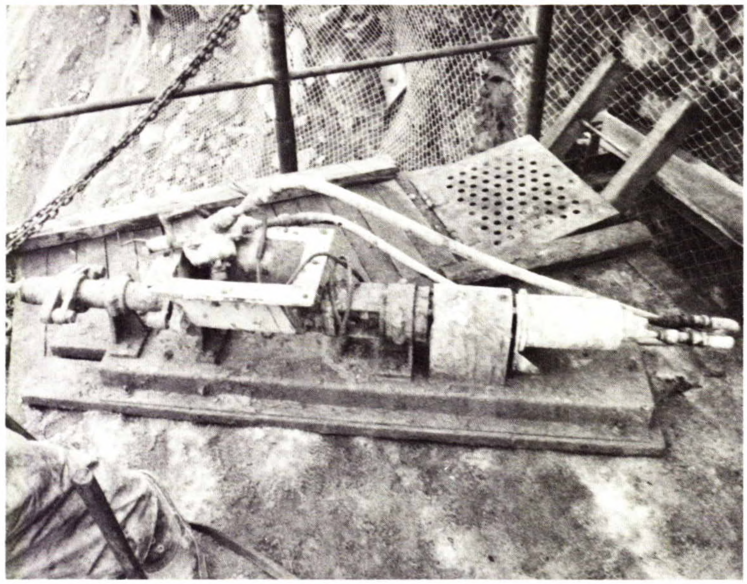


Fig B-13 - A typical grout pump.

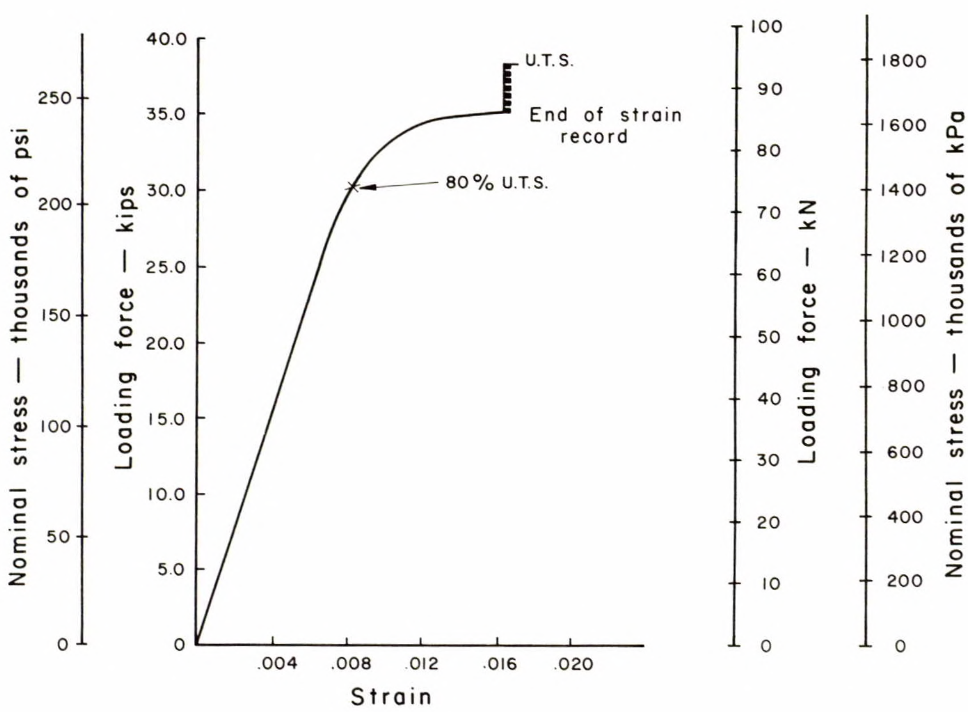


Fig B-14 - A typical stress-strain curve for a nominal half-inch high-tensile steel strand.

the strand considered above, a shortening of 0.25 in. (6 mm) in 2.4 in. (6 cm) would correspond to a loss of load of 30,000 x 0.25/2.4 = 3100 lb (14 kN). This is 10% of the original load. The actual strand load after tensioning would be 26,900 lb (120 kN). The shorter the anchor and the bigger the draw-in required for anchoring, the larger is the loss of load on anchoring. The amount of draw-in depends on the type of wedges and anchor block used. The actual draw-in should be measured and checked against the assumed amount.

38. A cable anchor should be made if possible with strand from the same reel because strand from different reels may have different properties.

Jacks

39. Figure B-3 shows two typical jacks for tensioning rock anchors. Both are hydraulic - the smaller can be hand pumped but a mechanical pump, which is essential for the large jack, is more convenient. Jacks should be equipped with a calibrated pressure gauge so that load can be read directly. The gauges should be checked at least daily when the jacks are in use; there may be a connection for a second, check gauge for this purpose. Figure B-15 shows a multi-strand jack in actual use. A spare jack should be on hand in case of jack or gauge failure.

Tensioning

40. The end of the cable or solid bar to be gripped by either the jack or the anchor wedges must be clean and free from grease. The anchor block and wedges must also be clean; they should be free from damage and sharp edges.

41. Once the anchor block and wedges are assembled, the jack is fitted onto the anchor and the jack wedges, or jacking nut for a solid anchor, positioned. The jack is then pumped until the gauge just begins to register load, to take up slack in the anchor. If necessary, the jack is released to restore full travel to the ram and then repumped to tighten the anchor.

42. The extension meter is next set to zero, and tensioning begins. It is recommended to pump to full load in about five increments, measuring the extension at the end of each; the draw-in on anchoring should also be measured.

SECONDARY GROUTING

43. Holes for anchors which are not to be

Fig B-15 - A pre-stressing jack tensions a large capacity rock anchor.

monitored should be filled with grout as soon as possible after tensioning. The mix is the same as for primary grouting. The secondary grout both ensures the anchor is completely bonded to the surrounding rock and provides corrosion protection.

44. After secondary grouting, the excess anchor can be cut away if desired. This cutting should also be done with a carborundum wheel rather than by flame cutting which can weaken the surface anchorage.

MONITORING

45. Anchors that are to be monitored with load cells cannot be secondary grouted, unless the cable or bar is sheathed so that it can move within the surrounding grout. The load cell is fitted between the anchor block and the bearing plate before tensioning the anchor. It should be checked during tensioning to see that the loads indicated by the jack and the cell are sensibly the same. Figure B-16 shows a load cell and jack in position; Fig B-17 shows a load cell and readout unit.

SAFETY PRECAUTIONS

46. The energy stored in a typical 100 ft (30 m) strand, tensioned to 40,000 lb (180 kN) is about the same as the energy of a small car moving at 20 mph (32 kph). If the anchorage slips or a strand breaks, some of this energy will be converted into motion of the strand or grips.

47. No one at any time must be in line with the anchor during tensioning. If possible, only personnel required for the tensioning operation should be within 20 ft (6 m), of the anchor. They must also keep to one side of the anchor line at all times.

48. If it is not possible to keep the area clear, then a timber or sandbag barricade should be erected for personnel protection. Alternatively, tensioning should be postponed until the area is clear. Only after the anchor is completely tensioned and the jack has been removed can the anchor be regarded as safe.

49. The jack, hoses, and fittings should be checked daily before use. To determine if the equipment is satisfactory, the jack should develop the maximum pressure of the system, ie, up to the setting of the relief valve, and maintain this pressure for at least two minutes. Even with this precaution, danger to personnel from the failing of a hose or fitting should be anticipated and minimized.

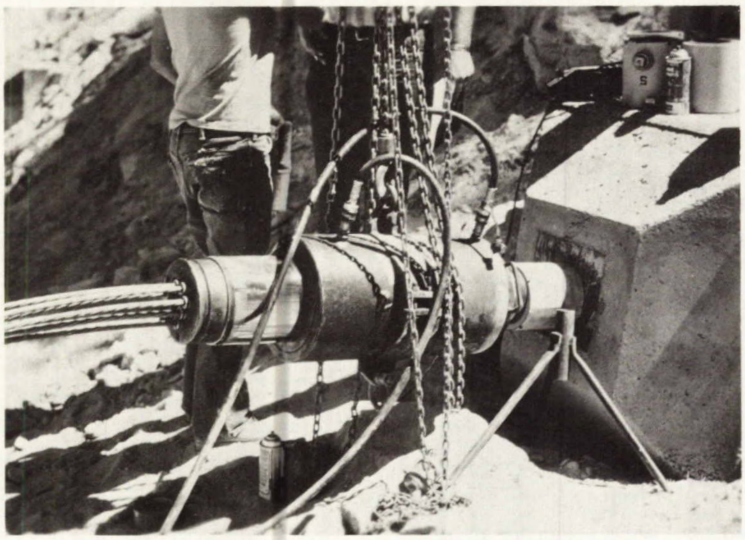


Fig B-16 - A pre-stressing jack with a load cell between the jack and the bearing plate. A second load cell can be seen on top of the concrete bearing block.

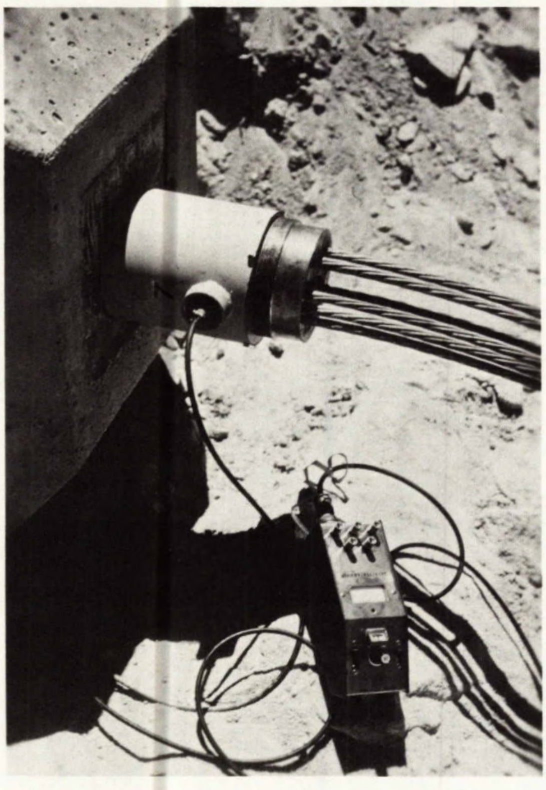


Fig B-17 - A load cell and read-out unit in place.

PROBLEMS AND SOLUTIONS

INCOMPLETE ANCHOR INSERTION

50. If no amount of force will complete penetration of the anchor into the borehole, check the amount of hole left unfilled (the hole length should have been marked on the cable before inserting). The most likely reason for lack of penetration is debris in the hole. A loss of about 5% of the planned hole length is usually acceptable; a sensible design policy is to allow this much anyway. If the loss of penetration is more than this, one solution is to withdraw the cable and attempt to re-drill the end of the borehole; another solution is to accept the reduced length and compensate with design changes in the system, eg, changing the spacing and/or length of adjacent anchors. It may be worthwhile to partly withdraw the anchor and then attempt to insert again.

UNEXPECTED EXTENSIONS ON JACKING

51. If the anchor extensions are not within 10% of the expected figure, check the jack calibration. This is usually done by replacing the jack gauge with a test gauge. If the jack cannot be checked, use the spare jack and check the load at which the grips release. This should correspond to the load indicated by the previous jack. The design calculations should also be checked.

52. If the jack is working and the calculations are correct, then unexpected extensions indicate that the anchor is not functioning correctly. Too much extension means the grouted anchorage has slipped. Little can be done about this. It is worthwhile leaving the anchor for a time such as a week, and then checking the load by jacking the anchor. If the load has been retained, the anchor is satisfactory and can be left indefinitely. If the load has dropped, an attempt can be made to re-tension the anchor. However, success is unlikely at this stage.

53. Sudden changes in the rate of increase, or a decrease, in jack pressure may indicate failure of the anchorage or of a wire in a strand. It may prove impossible to reach the required jack load without excessive extensions. This means that the bottom anchorage is slipping. It is best to lock the anchor by releasing the jack pressure very slowly and attempt to re-tension a few days later. Conclusive evidence that slippage is chronic means that a pre-stressed anchor cannot be obtained in this hole. It is probably worthwhile to fill the hole with grout to obtain some benefit from the

remaining tension in the anchor plus the dowel support of the grout column. However, reappraisal of the system is required.

54. If the extensions are less than expected, assuming the jack is working properly and the design calculations are correct, the cause will probably be friction on the anchor. This means that part of the anchor load is being transferred to the rock by friction above the bottom anchorage and as a result less of the anchor is being stretched. One solution may be to release the entire load and to re-tension. Alternatively, or if the anchor cannot be released, the anchor can be left for some days and then re-tensioned. The friction may dissipate in the meantime and full extensions can then be realized on re-jacking. A third possibility is to increase the initial jacking loads to overcome the friction losses; care must be taken in this case that the loads are within the strength of the anchor. Fourthly, a high-frequency vibrator, used with great caution on the collar end of the cable, might dissipate the friction.

RELEASING THE ANCHOR LOAD

55. Releasing the load on a cable anchor can be difficult and hazardous. It is possible only if the jack design allows removal of the anchor cones while the anchor is under tension. This usually requires a chair to be placed over the anchor head and the jack pumped up to take the load in the anchor. If the grips are properly locked, they will be difficult or even impossible to free. The jack can be pumped up to a load higher than the initial installation load, but care must be taken not to exceed the anchor strength. Often when the grips do release, they do so suddenly, and the resulting dynamic impulse in the anchor may cause damage to the anchorage. If pumping up to about 90% of ultimate load and releasing several times does not free the grips, the operation should be abandoned.

56. Releasing the load on a solid bar anchored by a nut is more straightforward and should not require replacement of the nut.

57. Before attempting the anchor release, the jack should be fully pumped out to give enough travel. On re-tensioning a cable anchor, new grips should be used and the load carefully controlled because the capacity of the strands may have been reduced by the damage caused by the original grips.

STRAND OR GRIP FAILURE

58. If one strand of a cable anchor fails, it is not possible to replace it. To compensate after a strand failure when the strands are tensioned separately, the remaining strands may be overtensioned to about 90% of ultimate load before anchoring. However, if the broken strand is one of the last strands tensioned, the reduced anchor capacity must be accepted and the effect of this on the overall support scheme considered. If the strands are tensioned together and one strand fails, again the jacking load on the remaining strands may be increased to compensate.

59. Grip failure may occur either on or after anchoring. The strand or bar can slip through the grips, refusing to anchor. If the grips completely refuse to hold the anchor, fresh grips should be fitted and tensioning again attempted. Failure after anchoring will usually be sudden, and in the case of a cable anchor, the strands involved may disappear into the borehole and so be lost. The worst problem will arise when the strand or anchor is partly gripped at some reduced load. If possible, the anchor should be released and re-tensioned using fresh grips. If this is impossible, the anchor must be left. In the case of a single strand with a reduced load, tensioning of the anchor should continue with the sound strands, taking special care to avoid personnel being in line with the suspect strand until secondary grouting has been completed.

60. If an anchor is faulty or cannot maintain the required load, it may be necessary to install another anchor nearby.

REFERENCES

B-1. Coates, D.F. and Sage, R. "Rock anchors in mining"; CANMET (Canada Centre for Mineral and Energy Technology, formerly Mines Branch), Technical Bulletin TB 181; 47 p; November 1973.

APPENDIX C

SHOTCRETE APPLICATION

1. This appendix describes the techniques, equipment and cost of applying shotcrete to open pit slopes. The descriptions draw on experience in field trials in two Canadian open pits. These trials were part of the development work for the Pit Slope Manual.

DRY AND WET MIX SHOTCRETE

2. Shotcrete is usually applied "dry" - that is, the sand, stone and cement are mixed and pumped by air to the shotcrete nozzle where water is added. In this process the water content is controlled by the spraying operator and adjustments can be made to suit the immediate conditions. For example, if there are wet areas on the face, the water content can be reduced accordingly. An experienced operator is required.

3. Wet mix shotcrete has water added to the sand, stone and cement as they are mixed. The shotcrete is pumped by piston to the nozzle, where a flash-set additive is added. Less skill is required of the operator, but the water content is not easily varied to suit face conditions. Wet mix shotcrete has the advantage of less dust at the nozzle.

4. The shotcrete trials for this manual involved dry mix only, and the rest of this appendix deals only with dry mix shotcrete. However, wet mix shotcrete is advocated by some authorities (C-1).

MATERIAL

5. Shotcrete requires a uniform gradation of sand and gravel for optimum performance. The recommended gradation is shown in Fig C-1 (see also Ref C-1). The coarser mix - minus 0.75 in. (2 cm) - results in higher shotcrete strength, but the finer mix feeds better through the shotcrete machine and losses through rebound from the slope face are reduced. The aggregate should be washed. Sand should have a water content between 2% and 5%. Very dry sand results in excessive dust. Wet sand reacts with cement in dry mix and reduces the "pot life" of the mix and the final strength.

6. Figure C-2 shows the aggregate grading curve for a shotcrete trial in 1973. The screened glacial till was used for both sand and stone. Although grading varies from that recommended, the

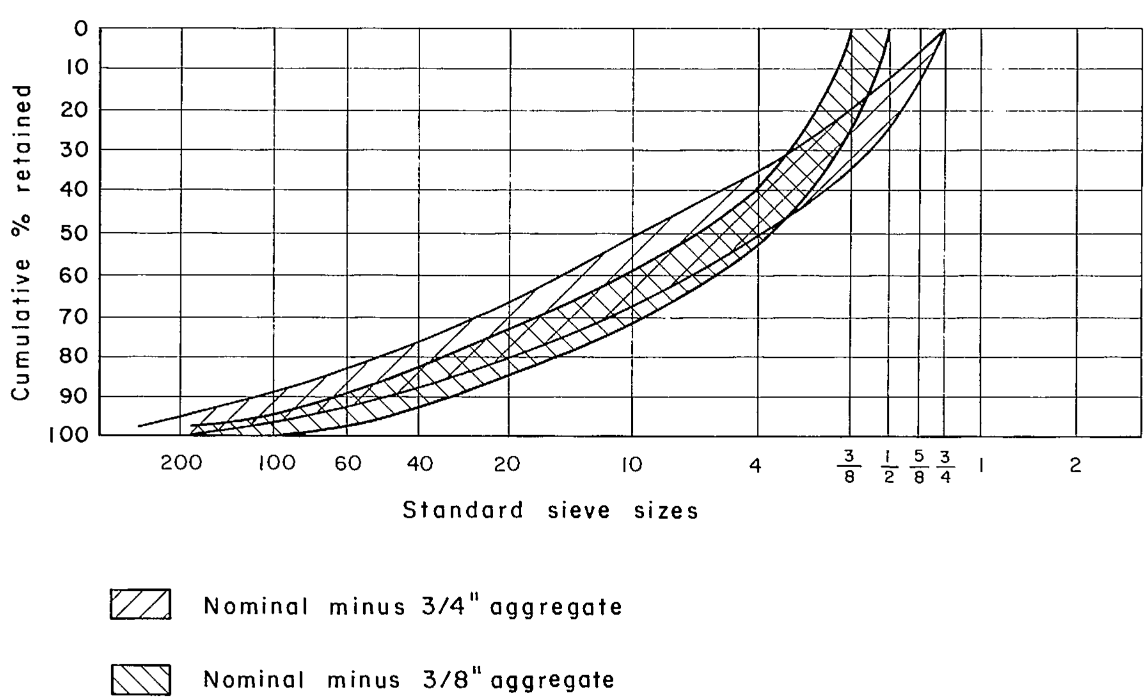


Fig C-1 - Recommended grading curve for shotcrete.

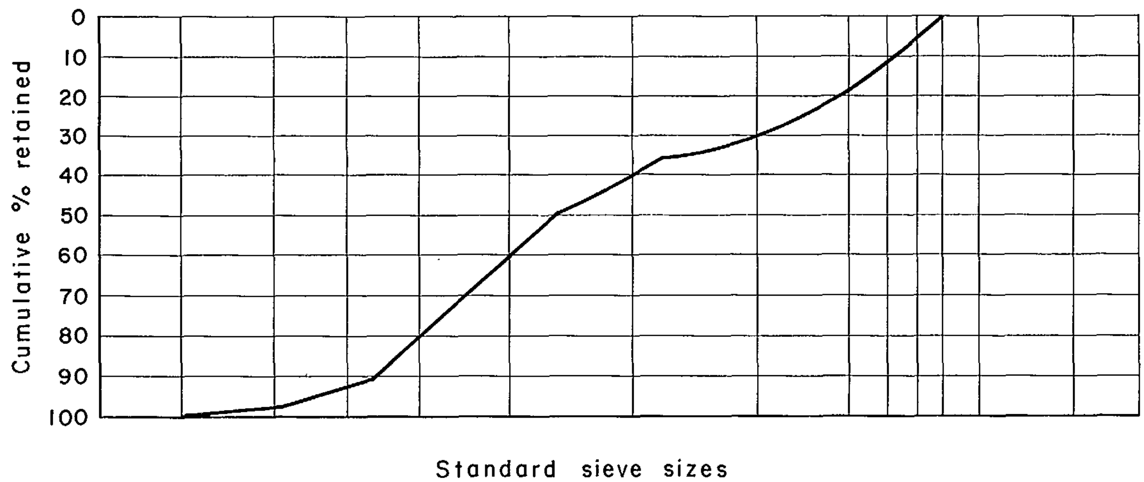


Fig C-2 - Grading curve for shotcrete at a trial.

shotcrete strength attained was around 5000 psi (35,000 kPa). The mix was satisfactory for spraying.

7. Type 1 normal portland cement is commonly used. The volume required is 650-750 lb per cu yd (390 - 450 kg/m^3) of shotcrete; this should result in a strength of about 4000-5000 psi (28,000 - 35,000 kPa) at 28 days.

8. Water should be clean and free of dissolved impurities that might affect shotcrete strength.

9. Several commercial flash-set additives are available, either in powder form for adding at the mixing stage or as liquids which are added at the nozzle. The field trials described here used Monoset powder at about 12 lb per cu yd (7 kg/m^3) or Barra Gunit 2 at about 5 lb per cu yd (3 kg/m^3). Calcium chloride can be used as an accelerator but may cause a reduction in final strength; it should not exceed 2% of cement weight. The additive should be tested for compatibility with the cement, according to manufacture's instruction. The Vicat penetrating needle test (ASTM C-191-70) is commonly used.

10. Wire fibre reinforcement, if used, should be about 1 in. (2.5 cm) long and 0.01 to 0.02 in. (0.25 - 0.5 mm) thick. Between 200 and 400 lb per cu yd (120 - 240 kg/m^3) are added at the dry mixing stage.

EQUIPMENT

11. Several commercial shotcreting machines are available. A Meyco GM57 (Fig C-3), an Aliva (Fig C-4) and an Icoma (Fig C-5) were used in the field trials for the manual. The Aliva is a small machine, delivering less than 4 cu yd per hour (3 m^3/h). The Icoma is rated at 12 to 16 cu yds per hour (9 - 12 m^3/h), but in trials the machine delivered only about 6 yd^3/h (5 m^3/h). The Meyco machine delivered about 12 yds^3/h (9 m^3/h after production techniques were established.

12. A concrete mixing plant is required together with loaders. Alternatively, ready-mix trucks can be used. Good results were achieved in one trial with a mobile batching truck (Fig C-6). It has separate bins for cement, sand and stone and delivers the correct proportions of ready-mixed material directly to the shotcrete machine. This has the advantage over conventional concrete trucks that cement is not added to the usually damp sand until the last minute. There is thus no danger of the shotcrete mix spoiling. However, stockpiles of cement, sand and stone and a method for loading the batching truck are still required.

13. Access for spraying is best provided by a mobile platform, or a platform suspended from a crane. A crane may also be needed to move the concrete machine.

Fig C-3 - The Meyco GM 57 Shotcrete Machine.

Fig C-4 - The Aliva Shotcrete Machine.

14. An air compressor for pumping the shotcrete and a water pump are needed. The air compressor should have a large capacity - 100 cfm per cu yd of shotcrete per hour (4 m^3/min per m^3/h) at 100 psi (700 kPa).

15. Figure C-7 shows the complete shotcreting set-up at one trial, using a batching truck and a mobile crane. The drill rig provided compressed air.

SLOPE PREPARATION

16. The face to be sprayed should be scaled to remove loose rock for protection of the spraying crew (Fig C-8). The slope should be sprayed with water just before shotcreting because a damp face is required for best results.

SHOTCRETING

17. The dry mix of cement, stone and sand should be proportioned by weight according to the mix design. A mix of about five minutes in a stationary mixer or concrete truck is sufficient; because of bulking of the dry materials, mixer capacity will be reduced to 75% of nominal.

18. The mix should be used within 90 minutes. This avoids loss of strength through premature interaction of cement and free water in the sand.

19. Additive should be accurately measured or preferably metered at the mixer or nozzle, as appropriate.

20. Water is added at the nozzle. Control of water content requires an experienced operator. However, the final water/cement ratio should be in the range of 0.35 - 0.50 by weight. If the moisture content of the dry material - principally of the sand - is measured, the required water flow at the nozzle can be determined.

21. The average hose length that can conveniently be used for shotcreting is about 100 - 150 ft (30 - 50 m). This length determines the area of face that can be sprayed without moving the machine. Longer hoses can be used but

Fig C-5 - The Icoma Shotcrete Machine being fed by a cement mobile batching truck.

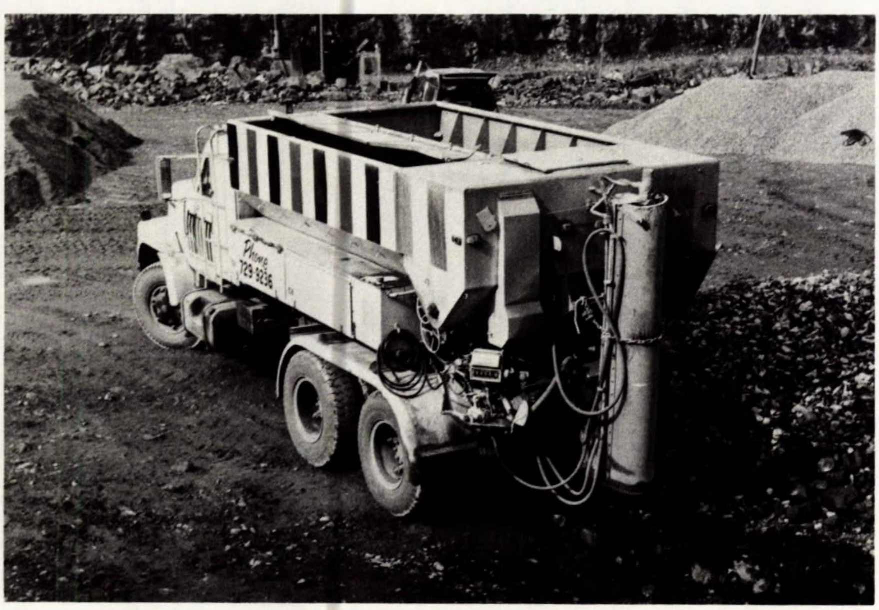


Fig C-6 - A mobile cement batching truck.

Fig C-7 - Top: General view of a spraying operation at an open pit mine. The drill rig in the centre is being used to supply compressed air; the shotcreting machine is behind the rig. Bottom: Close-up of the spraying platform.

correspondingly higher compressor capacity is required.

22. The spray nozzle should be held 3-5 ft (1 - 2 m) from the face and as perpendicular to it as possible, to minimize rebound. Rebound losses with a skilled operator should not be more than about 15%.

23. The thickness of shotcrete can be gauged by the operator, but measuring pins attached to the face are better. The volume of shotcrete required can be estimated from the area to be sprayed and the required thickness. However, up to 75% more than this nominal volume may be required to compensate for rebound and irregularities in the surface.

24. Figure C-9 shows shotcreting in progress. Figure C-10 shows the junction between sprayed and unsprayed parts of a bench.

REINFORCED SHOTCRETE

25. Wire reinforcement is added at the mixer. The wire often "clumps" into bundles of wires which may be broken up by mounting a vibrating screen over the hopper. Alternatively, a longer mixing time - up to 10 minutes - can be used.

26. The wire is very hazardous; all personnel should wear protective gloves and goggles.

27. The reinforced shotcrete mix flows less easily through the shotcreting machine. Volume rates may be 15% below those for unreinforced shotcrete.

28. Reinforced shotcrete has very little slump. Operators may add too much water in an effort to attain the slump expected of unreinforced shotcrete.

DRAINAGE

29. Shotcrete may seal the slope face and

Fig C-8 - Scaling a slope prior to shotcreting.

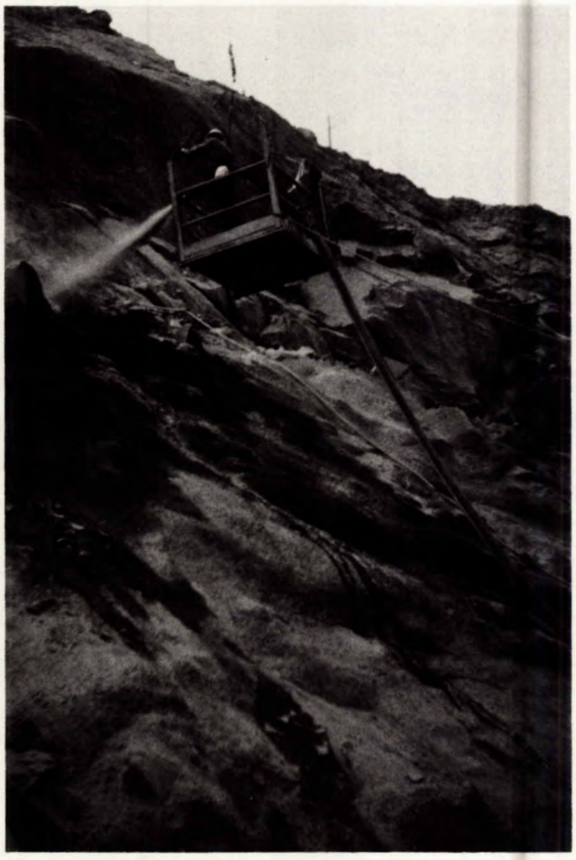


Fig C-9 - Shotcreting in progress.

Fig C-10 - A view of shotcreting showing the contact between sprayed and unsprayed sections.

allow detrimental groundwater pressures to build up. Drain pipes must be installed to prevent this. The pipes should be wedged into the face before shotcreting. The open end should be covered to prevent its being blocked by shotcrete. Figure C-11 shows flexible hose used for this purpose.

CURING

30. Shotcrete, like any concrete, should be kept damp for 2-4 days after spraying to develop full strength. This is best done by spraying the finished face with a curing compound, which effectively seals the face and allows the shotcrete to keep damp. Subsequent coats of shotcrete should not be sprayed over the curing compound, however, because it will not bond correctly. Sacking, spread over the face and kept wet, is an alternative curing method.

TEMPERATURE

31. Rock face temperature must be above 5°C when shotcrete is applied, and should not fall below 0°C the first week after spraying.

TESTING

32. If possible, the proposed shotcrete mix should be tested before full-scale spraying begins. A test panel should be shot into a wooden box, and test cylinders cut from the panel. A section of shotcrete can be sprayed onto rock and cores cut to test the rock/concrete bond (Fig C-12).

33. Cores can also be cut from the final sprayed slope. Alternatively, a machine specifically for testing shotcrete in situ is available commercially (Fig C-13). With this, a set of pins is cast into the shotcrete. After the shotcrete is at strength, the pins are pulled out by a hollow core jack. This produces a conical failure in tension and shear (Fig C-14).

34. Strengths produced in the shotcrete trials were approximately as follows:

Test	Strength, psi (kPa) Shotcrete	Reinforced shotcrete
Pullout (shear)	1000(7000)	1800(12,500)
Beam bending (tensile)	400(3000)	900(6000)
Compression	5000(35,000)	6000(42,000)

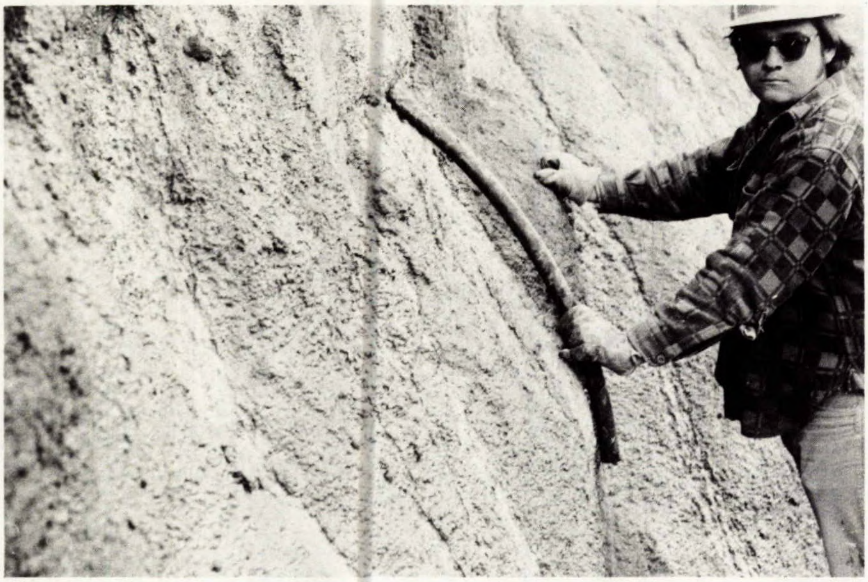


Fig C-11 - Provision must be made for draining shotcrete. Here a drainage tube has been wedged in a crack in the concrete face before spraying.

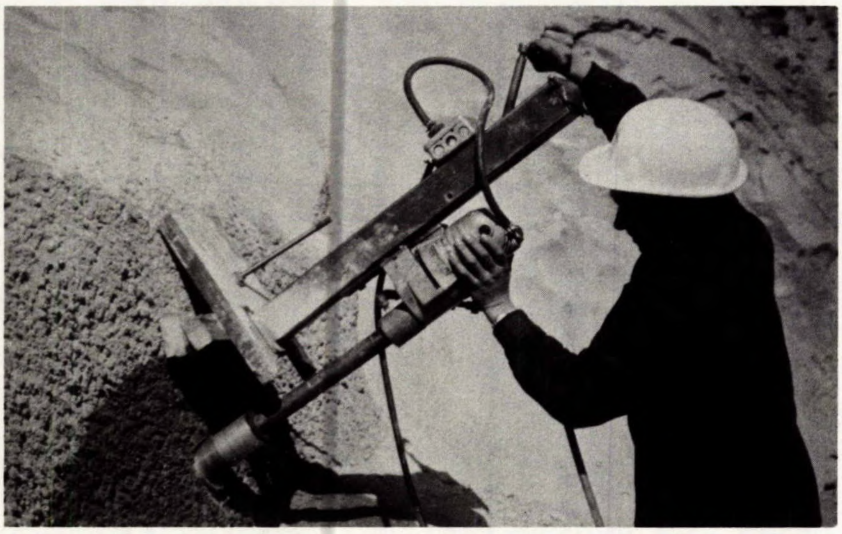
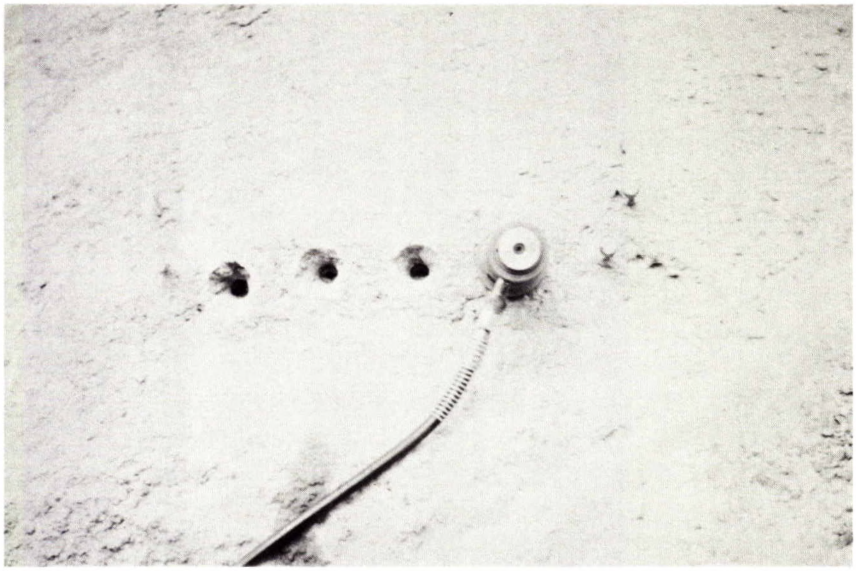


Fig C-12 - Sampling shotcrete with a portable drill. The sample obtained is used to test the bond between shotcrete and rock.



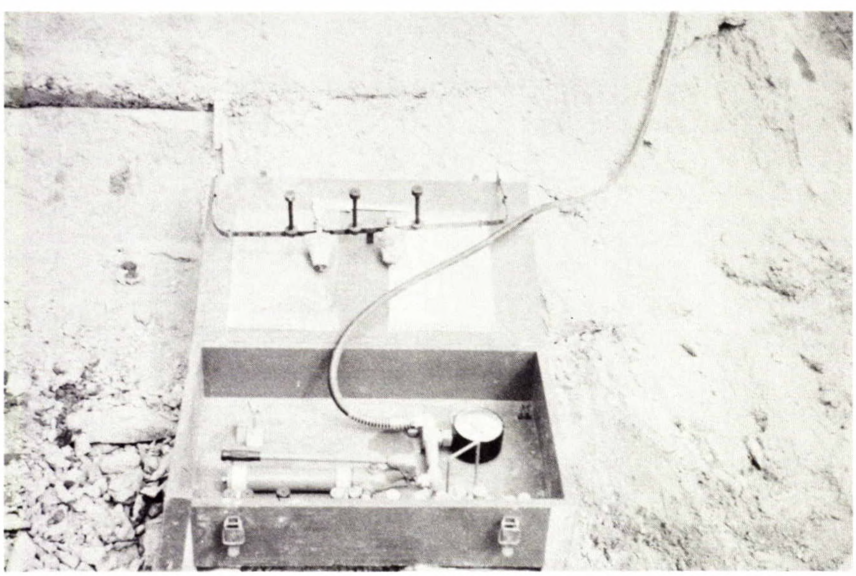
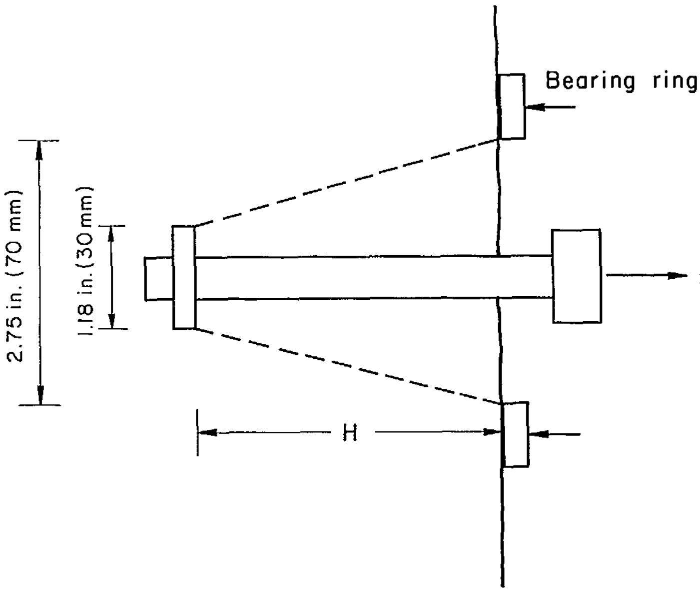


Fig C-13 - Top: View of shotcrete pull-out testing jack. Bottom: Pump for shotcrete pull-out testing jack, showing samples.



$$\text{Failure Area} = 50\pi\,(H^2 + 400)^{1/2}$$

$$\text{Failure Stress} = \text{Failure Load } P/\text{Area}$$

Fig C-14 - Principle of shotcrete pull-out test.

35. Figure C-15 shows a slope before and after shotcreting.

COSTS

36. Detailed cost records were kept for the various field trials. The cost of shotcrete placed was very high - in the order of $150 per cu yd ($200/m³) - because of the low volume and intensity of supervision. In normal production, however, the unit cost should be substantially lower. The following estimates are based on field experience. They assume a relatively modest shotcreting program of 1000 cubic yards (750 m³) per year, with a shotcrete machine purchased outright and other equipment rented by the hour.

Unit Costs (1974)

Shotcrete machine — $10,000 (written off over 5 years)

Spares — $ 1000 per year

Equipment rental

- 2 concrete trucks, $25/hour each
- 2 600 cfm (3 m³/min) compressors, $10/hour each
- 1 mobile platform or crane with operator, $30/hour

Total rental charges $100 per hour

Labour

2 shotcrete operators	at $10/hour	
4 labourers	at $ 5/hour	
1 foreman	at $10/hour	
	Total	$ 50 per hour

Materials		Cost per cubic yard: Shotcrete	Reinforced shotcrete
Stone	$11.50/cu yd	$ 7.85	$ 6.35
Sand	$ 3.50/cu yd	1.75	1.95
Cement	$60.00/ton	22.50	22.50
Additive	$ 0.50/lb	2.50	2.50
Wire fibre	$ 0.34/lb	-	85.00
		$34.60 ($45/m³)	$118.30 ($155/m³)

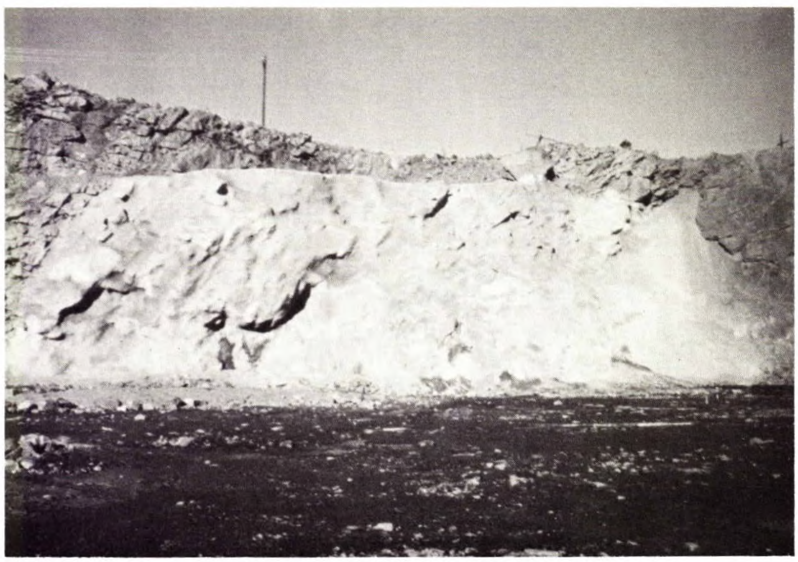


Fig C-15 - Top: A slope before spraying. Bottom: Centre of the same slope after shotcreting. Location of the sprayed section can be judged from the pole supporting a power line behind the slope.

Assuming 10 cubic yards are placed per operating hour, cost is:

		per cubic yard
Shotcrete machine		
$\frac{10{,}000}{5\text{x}1000} + \frac{1000}{1000}$	=	\$ 2
Other equipment		
100 ÷ 10	=	\$ 10
Labour		
$\frac{50}{10}$	=	\$ 5
Material - Shotcrete		\$ 35
- Reinforced shotcrete		\$120

Cost is \$52 per cu yd (\$68/m^3) for shotcrete
\$137 per cu yd (\$180/m^3) for reinforced shotcrete

For a nominal 4 in. thickness, coverage is approximately 50 ft /yd. Cost is therefore
\$0.70 per sq ft (\$7.50/m^2) of shotcrete
\$2.40 per sq ft (\$26/m^2) for reinforced concrete.

REFERENCES

C-1 Steenson, H.N. "Using accelerator-wet process"; Proc. Eng. Foundation Conf. ASCE Berwick Academy, Maine; 1974.

APPENDIX D

CANADIAN SUPPLIERS AND COST (1975)

OF SUPPORT MATERIALS

1. This appendix briefly describes and provides some cost and specification information on support material available in Canada or from convenient sources in other countries. The cost information is current at mid 1975.

2. The information serves as a guide to the availability of material. Some companies may not supply the essential hardware needed for anchor tensioning - eg, jacks, anchor blocks and grips - on the grounds that the work requires trained operators.

CABLE ROCK ANCHORS

3. Prestressing strand is produced in Canada by Stelco in Hamilton and by Wire Rope Industries in Vancouver. It consists of six individual strands twisted around a central seventh, and its design and quality is specified by ASTM Standard A416 and CSA Code A135. The most common Stelco sizes are 0.5 in. (13 mm) and 0.62 in. (16 mm). Grade is 270 ksi (1860 MPa) ultimate tensile strength. Wire Rope Industries produces mainly 0.5 in. (13 mm) in grades 250 and 270 ksi (1725 and 1860 MPa) and 0.6 in. (15 mm) in grade 270 ksi (1860 MPa). Price is roughly 31¢ per lb (70¢/kg) and 0.5 in. (13 mm) strand weighs about 525 lb per 1000 ft (0.78 kg/m).

4. Prestressing wire to ASTM A421 and CSA Code A135 specifications is produced by Stelco in Hamilton. The most common diameter is 0.276 in. (7 mm). Price is roughly 27¢ per pound (60¢/kg) and weight is 203 lb per 1000 ft (0.3 kg/m). Ultimate strength is 235 ksi (1620 MPa).

5. Wire rock anchors are similar to strand cable anchors, except that the solid wire permits positive termination by "buttonheading" the end of the wire.

6. This requires greater care in assembly since all wires must be cut to exactly the same length to have similar stresses imposed on each. It is also necessary to tension all wires simultaneously. Shims are placed under the bearing plate to maintain tension.

7. Several companies supply complete anchors, including the necessary anchor blocks and grips. Their systems vary to some extent; some may provide greased, sheathed strands for corrosion protection. All suppliers of strand anchors have single strand jacking systems. Some also have multistrand jacks. The various suppliers are described below.

VSL

8. Available from:

VSL Canada Ltd.,
318 Arvin Ave.,
Stoney Creek, Ont.

9. VSL uses 0.50 in. (13 mm) and 0.62 in. (16 mm) diameter strand of 270 ksi (1860 MPa) grade. Multistrand stressing is available for up to 31 0.50 in. (13 mm) cables or 24 0.62 in. (16 mm) cables. A single sheath encloses all cables over the stressing length. The cables are individually terminated with compression fittings at the downhole end.

CCL

10. Available from:

CCL Systems of Canada Ltd.,
1800 Britannia Road East,
Mississauga, Ontario.

11. CCL advertises strand in three diameters - 0.5 in., 0.6 in., and 0.7 in. (13 mm, 16 mm and 18 mm), in three grades - Standard, 240 ksi (1650 MPa), Super, 270 ksi (1860 MPa) and Dyform, 310 ksi (2140 MPa). Multistrand stressing is available for up to 19 0.7 in. (18 mm) cables. The design load for this assembly is 970 kips (4300 kN) using Dyform cable. The strands are terminated with compression fittings at the downhole end.

12. CCL employs individually sheathed strands which give way to a common sheathing near the mouth of the hole. Standard one-stage grouting and stressing is followed by a second grouting inside the common sheath.

SPT

13. Available from:

Prestress Pioneers Ltd.,
191 Nugget Avenue,
Agincourt, Ontario.

ASD Enterprises Inc.,
1545 Louvain West,
Montreal, Que.

Wymac Steel Ltd.,
Box 67489,
Station "0",
Vancouver, B.C.

14. SPT uses strand of 0.50 in. (13 mm) and 0.62 in. (16 mm) diameter and 270 ksi (1860 MPa) ultimate strength. Up to 37 strands may be tensioned simultaneously for a maximum design strength of 920 kips (4100 kN) at 60% ultimate tensile strength for the 0.50 in. (13 mm) size. A single sleeve is used to encase all cables.

15. SPT has produced the following rough cost estimates for complete installation less drilling:

Up to 100 kips (450 kN) working load:	\$4.00 - \$7.00/ft (\$13 - \$23/m)
100 to 300 kips (450 to 1350 kN) working load:	\$5.00 - \$9.00/ft (\$17 - \$30/m)
300 to 500 kips (1350 to 2250 kN) working load:	\$7.00 - \$12.00/ft (\$23 - \$40/m)

Freysinnet

16. Available from:

Conenco International Ltd.,
39 Esna Park Drive,
Don Mills, Ontario.

Potenco Inc.,
10100 Parkway Blvd.,
Montreal, Que.

Con-Force Products Ltd.,
P.O. Box 398,
Calgary, Alta.

17. Freyssinet produces anchors with 1 to 37 strands, either 0.5 in. (13 mm) or 0.6 in. (15 mm) in diameter. The 37 - 0.6 in. (15 mm) size has a design load of 1330 kips (5900 kN). Individual sheaths are used for each strand.

Stronghold

18. Available from:

Multi Structure Tension Ltee.,
214 - Route 138,
St. Augustin de Quebec,
P.Q.

19. Stronghold produces rock anchors with up to 19 0.5 in. (13 mm) diameter strands. At 60% ultimate tensile strength, the design load is 470 kips (2100 kN) for the 19 strand configuration. A single sheath is used for all strands.

20. Stronghold also produces wire anchor in 0.276 in. (7 mm) diameter for up to 54 wires (470 kips working load). Buttonheading is used only at the downhole end, with split wedge grips to maintain tension at the bearing plate.

BBR

21. Available from:

Canadian BBR Ltd.,
P.O. Box 37,
Agincourt, Ontario.

22. BBR produces mainly wire anchors using 0.276 in. (7 mm) wires of 235 or 255 ksi (1620 or 1760 MPa) grade. Standard sizes exist up to 85 wires, or 770 kips (3400 kN) working load at 60% ultimate tensile strength. In addition to the single common sleeve used to decouple the stressing length, an optional sleeve may be added to provide double corrosion protection for the anchorage length as well. This necessitates a three-stage grouting process. First, the entire hole is grouted outside the sleeves. Second, the anchorage length is grouted, allowed to set, and the tendon tensioned. Third, the tendon length is grouted. Elimination of the optional sleeve allows the first and second steps to be combined.

23. BBR produces a single strand post tensioning system for low stress applications. Strands of 0.5 in. (13 mm) and 0.62 in. (16 mm) are used.

24. Cost estimates for wire anchors, less drilling, are as follows:

	2-stage grouting	3-stage grouting
200 kips	$8.00/ft	$10.00/ft
(900 kN)	($26/m)	($35/m)
400 kips	$10.00-$12.00/ft	$12.50-$14.50/ft
(1800 kN)	($33 - $40/m)	($41 - $48/m)
600 kips	$12.50-$14.00/ft	$15.00-$17.00/ft
(2700 kN)	($41 - $46/m)	($50 - $56/m)

Prescon

25. Prescon is a CCL licensee of BBR, and their rock anchors are identical.

SOLID BAR ROCK ANCHORS

26. Solid bar rock anchors employ lower-strength steel than either strand or wire types, and their discrete lengths necessitate coupling for long anchors. The bars may be terminated by a threaded nut, split wedge, or a special serrated Howlett grip. Threaded and Howlett couplers are available. Single bar anchors usually employ a nut and tack-welded washer threaded onto the down-hole end to increase bond to the grout.

Dywidag

27. Available from:

Dywidag Canada Ltd.,
1111 Finch Ave. W., Suite 450,
Downsview, Ontario.

28. Dywidag produces a continuously threaded prestressing rod in 0.625 in. diameter in coils (230 ksi ultimate), and in 1 in., 1.25 in., and 1.375 in. diameters in 60 ft long rods (150 ksi ultimate). The 1.375 in. rod has a design loading of 141 kips. Threaded nuts and couplings are used.

29. Rock anchors are provided either with smooth sheathing over the stressing portion only (for a single stage grouted, unbonded system), or with a continuous, convoluted tubing running the full length of the bar. The convoluted tubing is filled with grout before placing in the hole and is decoupled along the stressing portion by a second, smooth-walled tube. The latter system is used for greater corrosion protection.

30. A bundle anchor consisting of solid bars is now being developed, which will have a design load up to 570 kips.

Stress Steel

31. Stress Steel bars are supplied by

Conenco International,
39 Esna Park Drive,
Don Mills, Ontario.

32. Stress Steel bars are available in 0.125 in. (3 mm) increments from 0.50 in. to 1.375 in. (13 to 35 mm) diameter. Two grades are available - 145 and 160 ksi (1000 MPa and 1100 MPa) ultimate tensile strength. For a 1.375 in., 160 ksi (35 mm, 1100 MPa) bar, the working load (60% UTS) is 154 kips (685 kN). Larger bars are available on special order. Bars are available with a variety of decoupling sheaths and galvanized and other corrosion resistant coatings.

33. Cost of the 160 ksi (11 MPa) steel bar is between $680.00 and $750.00 per ton ($680 - 750/tonne).

34. A 0.6 in. (15 mm) diameter threaded bar using an epoxy grout is available for mining applications.

ROCK BOLTS

35. Rock bolts are essentially solid bar anchors that use a mechanical, rather than a grouted end anchorage.

36. There are three Canadian manufacturers of large diameter - > 1 in. (25 mm) - rock bolts. Their products are described below, and listed in Table D-1.

Williams

37. Available from:

Williams Form Hardware and Rock Bolt Ltd.,
P.O. Box 5,
Ingersol, Ontario.

38. Williams manufactures rock bolts in sizes up to 2 in. (50 mm) diameter in both solid bar (smooth shaft) and hollow bar groutable (deformed shaft) configurations.

Table D-1: Large diameter rock bolt specifications

Manufacturer and type	Nominal diameter inches	mm	Hole diameters inches	mm	Recommended design load kips	kN
Williams Solid Bar	1.0	25	1.675	43	40.	178
			1.75	44		
			1.875	48		
			2.75	70		
			2.25	57		
	1.125	29	2.0	51	48.	214
			2.25	57		
	1.25	32	2.0	51	60.	267
			2.25	57		
Williams Hollow Bar	1.0	25	1.675	43	25.	111
			1.75	44		
			1.875	48		
			2.0	51		
			2.25	57		
	1.375	35	2.0	51	50.	222
			2.25	57		
	2.0	51	3.0	76	60.	445
			3.50	89		
Stelco Plain Shaft	1.0	25	1.75	44	24.	107
	1.125	29	2.0	51	30.	134
	1.25	32	2.0	51	39.	174
	1.375	35	2.50	64	46.	205
Stelco Deformed Shaft	1.0	25	1.75	44	24.	107
			2.0	51		
	1.125	29	2.0	51	30.	134
	1.25	32	2.0	51	39.	174
	1.375	35	2.50	64	46.	205
Lektrodes Plain Shaft	1.0	25	1.675	43	24.	107
			1.75	44		
			2.0	51		
	1.125	29	2.0	51	31.5	140
	1.25	32	2.0	51	40.	178
Lektrodes Solid Deformed	1.0	25	1.675	43	24.	107
			1.75	44		
			2.0	51		
	1.125	29	2.0	51	31.5	140
	1.25	32	2.0	51	40.	178
Lektrodes Hollow Deformed	1.0	25	2.0	51	30.	134
	1.125	29	2.0	51	50.	223
	1.25	32	2.0	51	60.	267

39. Design loading is recommended to be 67% of elastic limit. Anchors are the two-leaf, bail-less, torque set type with smooth (non-serrated) outer surfaces. A variety of shell lengths and configurations are available in each hole to suit different rock strengths and working conditions.

Stelco

40. Available from:

Steel Company of Canada Ltd.,
525 Dominion St.,
Montreal, Quebec.

41. Stelco produces solid bar bolts only, with smooth or deformed shafts. Maximum shaft diameter is 1.375 in. (35 mm) to fit a 2.5 in. (65 mm)

hole.

42. Design loading is recommended to be 67% of minimum yield. The anchor is the two-leaf bail type with serrated outer surfaces. A single anchor design is produced for each hole diameter. The anchor is seated by tensioning the bolt.

43. Higher strength grades of steel are available on special order.

Lektrodes

44. Available from:

Lektrodes Limited - Division of
G & H Steel,
56 Six Point Road,
Toronto, Ontario.

45. Ledtrodes produces solid bar bolts (plain and deformed shaft) and hollow deformed shaft bolts in diameters up to 1.25 in. (32 mm). Recommended design loading is 67% of minimum yield.

46. The shell is a four-leaf bail-less type having serrated outer surfaces. Initial seating is provided by torquing the shaft; subsequent seating is provided by tensioning the shaft. Lektrodes also produces bolts in low-temperature alloys on special order.

Small diameter rock bolts

47. The following companies manufacture rock bolts of 1 in. (25 mm) diameter or less.

MBE Limited,
845 Logan Ave.,
Winnipeg, Manitoba.

Bethlehem Steel Export Ltd.,
Dominion Square Bldg.,
Montreal, Quebec.

Western Canada Steel,
Vancouver, B.C.

WIRE FOR REINFORCED SHOTCRETE

48. Shotcrete reinforced with wire fibre is relatively new. The fibres are from 0.005 in. to 0.02 in. (0.1 - 0.5 mm) in diameter, and from 0.25 to 1.5 in. (5 - 40 mm) in length. Between 100 and 200 lb of fibre are added per cubic yard of shotcrete (60 to 120 kg/m^3).

49. Fibre production in Canada should begin in mid 1976 at Stelco's Hamilton works. One American source is National Standard Company of Niles, Michigan.

National Standard Wire Fibre

50. Three different fibre cross-sections are available - round, flat and Duoform, which incorporates both round and flat sections in a single strand.

51. Round wire is available in lengths of 0.25 in. to 1.50 in. (5 - 40 mm) and diameters from 0.004 in. to 0.017 in. (0.1 - 0.4 mm).

52. Flat fibres range in length from 0.25 in. to 1.50 in. (5 - 40 mm) and typical sizes are 0.005 in. by 0.015 in. (0.1 mm by 0.4 mm), and 0.010 in. by 0.020 in (0.2 mm by 0.5 mm). The flat shape provides a greater surface area for bonding per unit cross-sectional area of wire.

53. Duoform fibres are available in lengths from 0.375 in. to 1.50 in. (10 to 40 mm). A typical size is 0.010 in. (0.2 mm) round, alternating with 0.005 in. by 0.015 in. (0.2 mm by 0.5 mm) flat sections.

54. Costs range from $23.00 to $33.00 per 100 lb of fibre (50¢ - 75¢/kg).

55. Available material includes high and low tensile carbon steel and several grades of stainless steel.

Stelco Wire Fibres

56. Available information on the anticipated Stelco product is limited. Dimensions will be approximately 0.016 in. (0.4 mm) in diameter (or flat equivalent) by about 1 in. (25 mm) long. Anticipated price is $25.00 to $27.00 per 100 lb of fibre (55¢ - 60¢/kg).

WIRE MESH

57. Wire mesh for control of loose face rock is available in chain-link form from several manufacturers, and as gabion-type screening from one more. The gabion wire has a triple twist where two wires cross, which has two important advantages over the chain-link type. The gabion

wire screen maintains its full width better than chain link, which tends to become narrower when draped vertically. Raveling is arrested by the triple twist when a single strand is broken, whereas the chain-link tends to "run".

Stelco

58. Available from:

Steel Company of Canada Ltd.,
100 King St. W.,
Hamilton, Ontario.

59. Stelco chain-link fence is produced in three gauges: 6 gauge or 0.192 in. (4.9 mm) diameter, costing approximately 45¢/sq ft ($4.90/m^2), 9 gauge or 0.144 in. (3.7 mm) diameter, costing approximately 26¢/sq ft ($2.80/m^2), and 11 gauge or 0.116 in. (2.9 mm) diameter, costing approximately 20¢/sq ft ($2.20/m^2). The standard mesh is 2 in. (50 mm) square although different sizes are available on special order. Maximum roll width is 12 ft (3.5 m), and 50 ft (15 m) long rolls are available. Wire is galvanized to 1.6 oz of coating per sq ft of wire surface area (500 g/m^2).

Van-Can

60. Available from:

Van-Can Industries Limited,
5780 Production Way,
Langley, B.C.

61. Van-Can produces the same gauges of chain-link screen as Stelco and in addition, a 13 gauge or .092 in. (2.3 mm) diameter which is considered too light for use on rock slopes. Approximate prices for 2 in. (50 mm) openings are: 11 gauge, 14¢/sq ft ($1.50/m^2); 9 gauge, 21¢/sq ft ($2.30/m^2); 6 gauge, 42¢/sq ft ($4.60/m^2). Other opening sizes are 1.5 in., 1.75 in. and 3 in. (40, 45 and 75 mm). Roll length is 50 ft (15 m) standard with longer rolls available. Standard galvanizing is 1.2 oz per sq ft of wire area (375 g/m^2).

Frost

62. Available from:

Frost Steel and Wire Co. Ltd.,
P.O. Box 55,
Station B,
Hamilton, Ontario.

63. Frost produces chain-link fence in the following sizes and gauges: 2 in. (50 mm) openings in 11 gauge, approximately 16¢ per sq ft ($41 m^2); 2 in. (50 mm) openings in 9 gauge, approximately 25¢ per sq ft ($6.50/m^2); 1.50 in. (40 mm) openings in 9 gauge, approximately 30¢ per sq ft ($7.60/m^2); 2 in. (50 mm) openings in 6 gauge, approximately 38¢ per sq ft ($9.60/m^2). Maximum roll width is 12 ft (3.5 m), and lengths greater than the standard 50 ft (15 m) are available. Galvanizing is 1.6 oz per sq ft of wire surface area (500 g/m^2).

Maccaferri

64. Available from:

Maccaferri Gabions of Canada Ltd.,
797 Don Mills Road,
Don Mills, Ontario.

65. Maccaferri gabion wire screen has hexagonal openings of approximately 3 in. by 4 in. (75 by 100 mm), and wires are triple twisted where they cross. Wire is 11 gauge and the screen costs about 17¢ per sq ft ($4.30/m^2). Maximum roll width is 13.0 ft (4 metres) and length is limited only by handling considerations. Zinc galvanizing is applied at 0.85 oz per sq ft of wire surface area (250 g/m^2).

APPENDIX E

CASE HISTORIES OF ROCK ANCHOR SUPPORT

1. Although rock anchors have been used in civil engineering for more than forty years, their use for slope stabilization in open pit mining is relatively rare. This appendix describes existing experience in open pit mining, and also presents brief details of pertinent experience in underground mining and in civil engineering.

HILTON MINE

2. In 1969, the Mining Research Laboratories in Ottawa undertook a rock anchor installation at the Hilton iron mine. The objective was to obtain experience and first-hand cost information on installing rock anchors in a mining environment. Four rock anchors were installed in lengths varying from 13 to 175 ft (4 - 53 m). Three of the anchors were fabricated from twelve 0.5 in. (1.25 cm) diameter strands, giving a working load of 340 kips (1500 kN). The fourth anchor was a solid bar, 1.375 in. (3.5 cm) in diameter, with a working load of 170 kips (760 kN).

3. The use of mesh as auxiliary reinforcement was investigated in this trial. Horizontal beams tied down by the rock anchors held welded steel mesh in place over the surface of the bench where the installation was made. The mesh was wired into a panel large enough to cover the test area, and then rolled for transportation to the top of the bench. There it was fastened to the rock with rock bolts, rolled across the berm and the free end dropped down the bench face.

4. The anchor holes were diamond drilled. This was done both for core recovery, and also to provide smooth walls for subsequent viewing with a borehole television camera. Two hole sizes were tried: HX of 3.89 in. (10 cm) diameter, and NX of 3.5 in. (9 cm) diameter. The holes were all drilled at 10° below horizontal.

5. Except for the longest anchor - 205 ft (63 m) - no difficulty was experienced during the installation. The diamond-drill hole was believed to aid installation but otherwise had no advantages for rock anchors. It was felt a slightly larger percussion-drilled hole would in fact be preferable.

6. Grout for anchoring was pumped down plastic pipe attached to the anchor. In each case a 20 ft (6 m) anchorage length was formed. High early strength cement with an expansion additive, Sika Intraplast, was used. However, despite these features, the grout was allowed to set for 28 days before the cables were tensioned.

7. All the anchors were tensioned using hydraulic jacks and were fitted with load cells. The behaviour of the entire support system was monitored for nine months. The anchors behaved satisfactorily throughout this period, despite the lack of specific corrosion protection. At the end of nine months, the anchors were secondary-grouted, preventing further monitoring.

8. All the work was carried out by regular mine staff under the supervision of a pre-stressing contractor. The trial established the feasibility of installing rock anchors with the equipment and personnel normally available on a mine site, with the exception of such specialized hardware as jacks for tensioning. The cost of the project, including mesh and beams, worked out to approximately $2000 per anchor. Full details of the project can be found in reference E-1.

TWIN BUTTES

9. In 1972, the Anamax mining company installed 40 rock anchors in a trial support project on an unstable mine slope. Most of the anchors installed had working capacities of 150 or 200 kips (670 or 890 kN). However, four 300 kip (1335 kN) and one 400 kip (1780 kN) anchors were installed. The cables were all manufactured from 0.5 in.(1.25 cm) diameter strand - the largest having 16 strands.

10. The stabilized zone was approximately 200 ft long by 200 ft high, (60 m x 60 m) and had an extensive alluvium zone above the rock face. Altogether some 200,000 tons (180,000 tonnes) of rock and 200,000 tons (180,000 tonnes) of alluvium were stabilized.

11. Eleven of the rock anchors were fitted with load cells for monitoring. These anchors were protected from corrosion by greasing the tension length of each strand and encasing the strands individually in 0.5 in. (125 cm) diameter PVC tube.

12. Holes for the anchors were per-

cussion-drilled. Anchors were installed with the aid of a mechanical tugger, utilizing a snatch block attached to the hole collar by a rock bolt. A cable running around the snatch block pulled the anchor into the hole.

13. PVC tubing formed a grouted bottom anchorage. After tensioning, all anchors were secondary grouted; the monitored anchors were free to move inside their greased sheathing.

14. The entire operation took three and one-half months. The average cost for rock anchors, including all fabrication, installation and the necessary supervision, ranged from $6/ft to $10/ft ($70/m to $33/m), depending on anchor size. An additional cost of approximately $5/ft ($16/m) was required for the monitored anchors for greasing and sheathing and the load cell. The anchors ranged from 100 to 150 ft (30-45 m) in length, with grouted anchorage zones between 35 ft (10 m) and 45 ft (13 m) ft.

15. An interesting aspect of this particular installation is that part-way through the work, heavy rain resulted in some displacement of the rock being stabilized. It was decided to continue the installation although five holes drilled before displacement began became distorted and had to be redrilled. Movement of the slope decreased as the rock anchor installation was completed. It thus appeared that, not only did the anchors stabilize the slope, but they also prevented an active slide from developing further (E-2).

NACIMIENTO MINE

16. The largest installation of rock anchor support in an open pit mine to date has been carried out by the Earth Resources Company in Cuba, New Mexico. In this project, 360 anchors - mostly of 400 kip (1800 kN) or greater capacity - were used to stabilize approximately three million tons (2.7 million tonnes) of rock.

17. The Nacimiento copper mine has a history of instability on the east wall of the pit. After a major slide in 1973, management realized that measures to stabilize the wall would be necessary if mining were to continue. Basically, there was a choice between cutting back the slope to a stable angle, or stabilizing with rock anchors. Economic appraisal led management to choose the latter.

18. The instability mode of the east wall is a typical plane shear. A preferential sliding surface is formed by a contact between a sandstone and a siltstone formation. This contact dips at roughly 30° and strikes approximately parallel to the east wall.

19. Preliminary engineering investigations indicated that rock anchor support would be viable, although quite massive support would be necessary. Because of the urgency - further slides would have cut off all supplies to the mill - a support scheme was adopted on the basis of the preliminary investigation. In effect, the final design of the support was done while rock anchors were being installed. The final support scheme used 100 anchors of 470 kip (2100 kN) capacity, 196 anchors of 400 kip (1800 kN) capacity and 65 anchors of 200 kip (900 kN) capacity. The smaller capacity anchors were used expressly for monitoring and were fitted with load cells. The smaller size load cell required resulted in savings.

20. The average length of the anchors was 125 ft (40 m), the extremes being 60 ft (18 m) and 260 ft (80 m). The average anchoring length was 40 ft (12 m) for the large anchors and 30 ft (9 m) for the monitored anchors.

21. This important support installation unfortunately encountered severe problems due to stress corrosion in the anchors. Corrosion protection was to be provided by secondary grouting of the rock anchors. However, it was specified that a delay should take place between tensioning and secondary grouting, with the intention that it would be possible to return to the anchors some time after tensioning and re-jack to obtain the maximum possible working load. This delay in secondary grouting was compounded by a slow-down in mining production which arose from a depressed copper market at the time the support was being installed. This resulted in de-emphasizing urgency of support and contributed to a delay in providing corrosion protection.

22. Stress corrosion manifested itself in the obvious failure of individual rock anchors. The strands projecting from the surface anchorage were

seen to become loose. With some anchors, the concrete surface bearing block became completely free. When these problems became evident, a further delay in providing corrosion protection took place as management wished, quite correctly, to determine the cause of the problem before taking remedial measures.

23. The net result of anchor failure due to stress corrosion was that in April, 1975, a month after the mine closed because of the depressed copper market, about a third of the anchored slope became unstable. Subsequently, the remaining anchors were flushed with a lime solution to inhibit further stress corrosion, and all accessible anchors were secondary grouted.

24. Failure of the anchors and the subsequent slide represented a major set-back to the support scheme. However, viewed in another light, the incident did illustrate the value of support. It appears that a major instability was effectively stabilized with rock anchors and then part of the support was inadvertently removed because of stress corrosion. As the support was removed, the slope became unstable and part of it slid. When precautions were taken to prevent further loss of support through stress corrosion, the rest of the supported wall remained stable. Full details of the work can be found in reference E-3.

PIPE MINE

25. INCO Ltd mine nickel from the Pipe open pit mine near Thompson, Manitoba. Production started in 1970, and by 1975 the pit had reached 400 ft (120 m) of a projected final depth of 720 ft (220 m). The pit is designed with an overall slope of 45°, with 40 ft (12 m) benches. The slope angle in some areas is 50°.

26. The east wall rock formations are mainly well foliated, steeply dipping quartzite. This rock has a platey structure and tends to break into thin slabs along vertical foliations. North of the quartzite is a highly jointed and soft peridotite. Geological exploration indicates that the east wall below the current mining level will be comprised mainly of the weak peridotite. Sloughing is presently causing problems in maintaining berms in the east wall, particularly in the peridotite.

27. The ultimate pit plan calls for one access ramp, running along the east wall approximately at the current mining level. This ramp presents two concerns for present and future mining. One is the question of traffic safety on the ramp because of material sloughing from above. The second is the problem of long-term stability of the ramp, in view of the expected weakness of the material to be mined below. Loss of this ramp in future mining would probably mean closure of the pit. Figure E-1 shows the south and east walls of the pit. The ramp can be seen diverted from its ultimate route along the east wall.

28. Investigations were undertaken to determine the best long-term solution to this problem. One possibility was to mine back the wall with small diameter drills to establish a more competent final face. However, the expense and, more important, the time required to do this would have seriously affected production. It was felt necessary to provide immediate protection to the haulage ramp along the east wall, and also to provide some overall support mechanism which would reduce the danger of a major collapse of the wall.

29. In 1975, with support of the Mining Research Laboratories, the company began a program of research and development aimed at investigating mechanical support of the east wall. A pilot proposal for supporting a section of the east wall above the existing ramp was adopted. This called for approximately 40 100 kip (450 kN) rock anchors, 150 ft (45 m) long to be installed over three benches of the wall for a 500 ft (150 m) length.

30. Preliminary investigation was undertaken to determine the feasibility of installing anchors. One problem was access. The condition of the three benches to be anchored was such that access would have to be from the ramp below. This meant that some form of drilling platform would be required, together with ready access for both equipment and men. The solution adopted was a platform bolted to the face with ladders for access. In addition, a large mobile platform (Fig E-2) was rented for the duration of the project. The mobile platform was used for installating the

Fig E-1 - General view of east wall of Pipe mine.

Fig E-2 - The Condor mobile platform used for access to the east wall.

drilling platforms and for rapid movement of men and equipment.

31. Preliminary tests were also made on the discarded hoist cable which management was proposing to use for the support scheme. There were two reasons for using of this cable. First, INCO had considerable experience with hoist cable for rock anchors as support for underground mining operations; supplies of cable, wedges, grips and jacks were thus readily available. The second reason was that management felt it would not be possible to obtain conventional pre-stressing material and supplies before the trial was due to begin.

32. Special tests were made on the hoist cable to assess its suitability for surface anchoring. These showed that its strength would be adequate but that anchoring the locked coil cable would be difficult. In particular, the cable has a tendency to unravel and break at the point where it is gripped by the surface anchorage (Fig E-3 and E-4). This anchoring difficulty would apply both when tensioning and when finally locking off the cable. The solution adopted was to use a double jack arrangement with special blocks behind the second jack to hold the cable during

Fig E-3 - Test set-up for determining strength of hoist cable.

Fig E-4 - Detail showing failure of hoist cable in test.

tensioning (Fig E-5). The smaller jack was required to drive the anchor wedges firmly into the conical anchor block, to ensure proper grip when the jack was released.

33. The actual support installation was begun in April, 1975. The first step was to scale the wall thoroughly using a slusher (Fig E-6). In addition to scaling, a wire mesh screen was installed on the least safe sections of the wall to provide additional protection (Fig E-7). Work then began on the drill stagings (Fig E-8).

34. Hole drilling from the suspended platforms proved satisfactory. The platforms were also used for inserting the anchor cable and for grouting and tensioning. A mobile crane took the weight of the cable while it was being inserted in the hole.

35. The bottom 25 ft of each hole was filled with grout to form the primary anchorage. A grout pump is shown in Fig E-9. The grout mix was 3 gal (14 l) of water to 80 lb (35 kg) of portland type II cement.

36. The grout was left to set for ten days. During this time, the surface anchorage was formed. If the rock at the collar was reasonably competent and free of joints, the anchor hole was over-reamed to 6 in. (15 cm) diameter at the

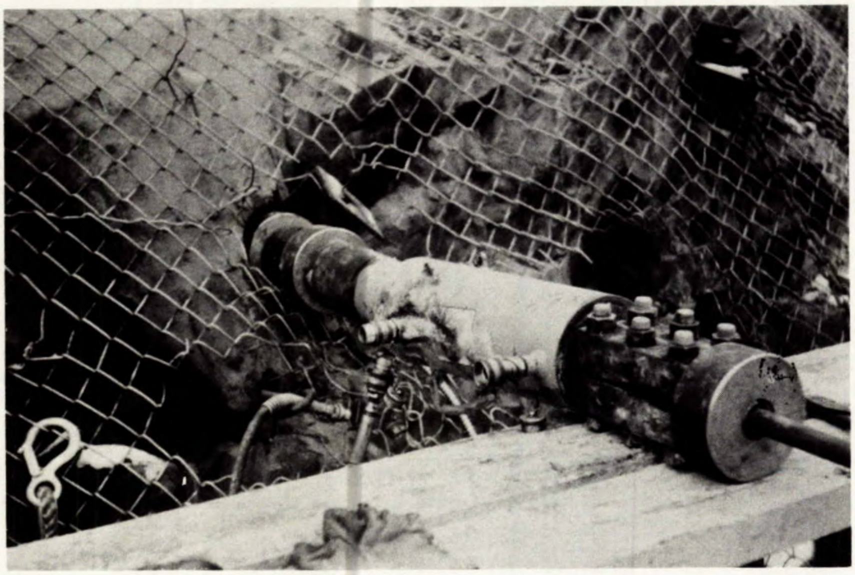


Fig E-5 - Arrangement of jacks for tensioning anchors.

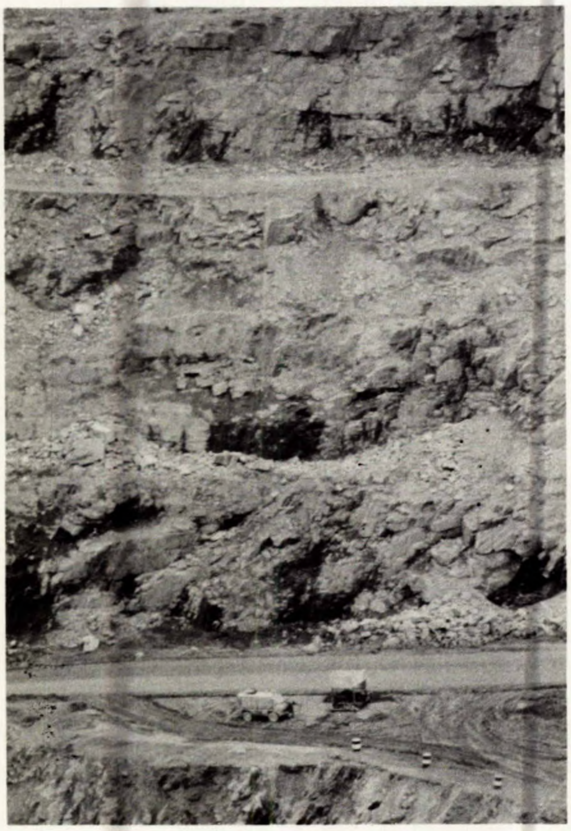


Fig E-6 - Scaling the east wall with a slusher.

collar. This was done by the drilling contractor before inserting the anchor. In other locations, a reinforced concrete bearing block was constructed. Figure E-10 shows details of a surface anchorage.

37. Although 40 anchors were planned, delays at different stages of the project meant that, by the end of the summer season, only 22 anchors had been installed. The onset of freezing weather prevented further work. Seven of the anchors were fitted with load cells and in addition, five wire extensometers were installed to monitor rock movement (Fig E-11).

38. The cost of the support project was high. Much of the expense was in attaining access to the benches, and in slushing and screening the face for safety. In addition to the extensometers, a laser distance-measuring instrument and targets were used. In all, the total cost was $270,000. However, considering only the expenses likely to be incurred in a normal support operation - that is, with good access and therefore no requirement for such expensive equipment as the mobile platform - the cost per rock anchor would be about $2000.

Fig E-7 - Wire mesh used for control of loose rock during anchor installation.

Fig E-8 - Completed platforms for drilling.

39. After one winter and a spring break-up, the anchors appeared to be functioning satisfactorily. Management expected to evaluate the support more fully in the months ahead and to decide in due course whether full-scale support of the east wall below the ramp is justified.

CIVIL ENGINEERING EXPERIENCE

40. Rock anchors are widely used in civil engineering construction, primarily for maintaining the sides of foundation excavations without restricting access. However, they have also been used for remedial work, which has more relevance to mining.

41. The stabilization of a rock slide at Windy Point, Australia, is a good example. In this slide, approximately 200,000 tons (205,000 tonnes) of rock was moving up to about 1 in. (2.5 cm) per day. The rock consisted of jointed sandstone, sliding on silty clay beds dipping at about 27°. The slide was successfully stabilized by horizontal drain holes to relieve groundwater pressure and 45 rock anchors with an average working load of 375 kips (1700 kN). Details are given in

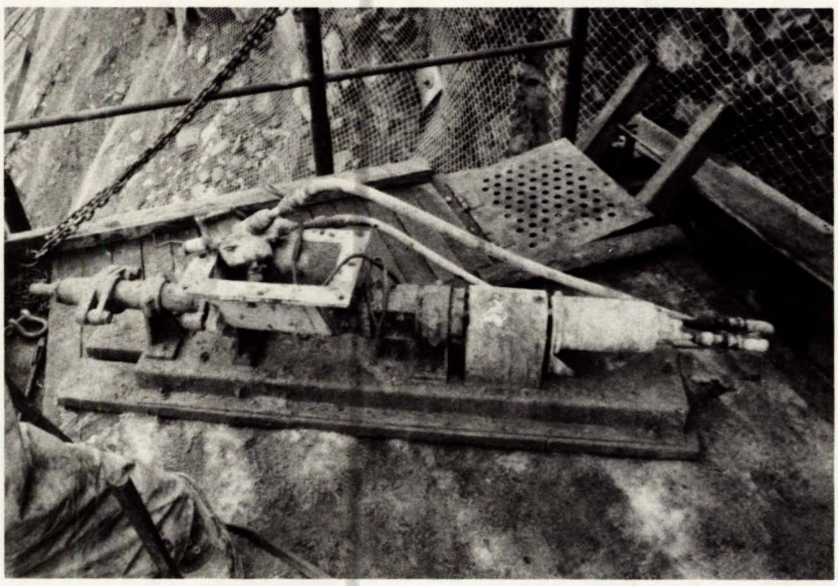


Fig E-9 - A grout pump.

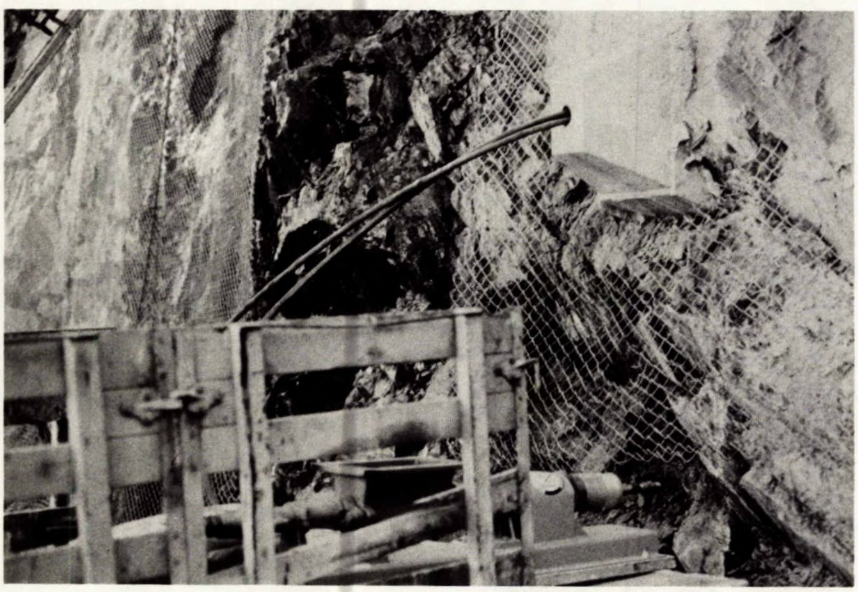


Fig E-10 - A completed surface anchorage showing the concrete bearing block. A grout tube can be seen emerging next to the anchor cable.

Fig E-11 - Taking measurements of a borehole extensometer.

reference E-4.

42. Several slides above an access road to a dam in Tasmania (E-5) were stabilized by rock anchors up to 50 ft (15 m) long and 250 kips (1100 kN) working load. The support design was based on measurements of one slide which actually took place.

UNDERGROUND MINING EXPERIENCE

43. Several underground mines regularly use rock anchors to support stope backs.

44. At the GECO mine in Ontario, many hundreds of rock anchors have been used in the last ten to fifteen years to prevent sloughing of stope backs. The anchors are made from discarded hoist cable, 1.25 in. (3.2 cm) in diameter. The mine has developed an interesting technique of inserting these cables in overhead holes, using a very stiff grout. A length of grout is deposited in the upper end of the hole with a special extruder, and the cable is pushed into the grout. It is found that the grout stays in position while it sets, and the cable can subsequently be tensioned. After tensioning, the cable is fully grouted. These cables are around 40 ft (12 m) in length and are tensioned to about 80 kips (360 kN) (E-6 and E-7).

45. Support has been provided by overhead rock anchors to a large underground gallery in a salt mine at Goderich, Ontario. The anchors were placed at approximately 45° above horizontal; a seal at the collar allowed the hole to be filled with grout.

46. One hundred and eighty-five cables were installed; despite the highly corrosive environment, they have behaved satisfactorily since installation. Full details are given in reference E-8.

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GLOSSARY

AGGREGATE

The sand (fine material) and stone (coarse material) used in making concrete.

ANCHOR BLOCK

A steel block at the surface, or head, of a rock anchor. The block clamps the rock anchor under tension, and so transfers the tension in the rock anchor into the surrounding rock, usually through a bearing block.

ASPERITY

A high point on a discontinuity in rock. Asperities form the actual contact points along a discontinuity.

BUTTRESS

A structure of massive dead weight used to support the toe of a slope.

DILATION

The tendency of a rock mass to increase in volume, or expand, during movement associated with instability.

GROUT

A mixture of cement and water or of cement, sand and water. May contain chemical additives to speed setting time or result in expansion on setting.

LOAD CELL

A load measuring device, usually a hollow steel cylinder, placed between a surface anchor block of a rock anchor and the bearing plate. Load in the rock anchor is thus transferred through the cell, and can be measured.

PRE-STRESSING CABLE

Steel cable capable of maintaining permanently a very high tensile force. Developed for the pre-stressed concrete construction industry.

PRIMARY GROUTING

The pumping of grout into the bottom of a bore-hole containing a rock anchor, to form the bottom anchorage of the anchor.

REINFORCED SHOTCRETE

Shotcrete reinforced with steel; usually refers to shotcrete containing chopped wire.

RETAINING WALL

A structural wall used to retain material or resist movement.

ROCK ANCHOR

A tensioned steel cable or bar anchored permanently in a borehole. The tension in the anchor is resisted by compression in the surrounding rock.

SECONDARY GROUTING

The filling of a borehole containing a rock anchor with grout after anchor tensioning.

SPRAYED CONCRETE

A layer of concrete sprayed onto a slope surface. Additives produce an effective set on impact. Also known as shotcrete.

STRESS CORROSION

A poorly understood process in which hydrogen forms within steel, producing a weakness which may result in a sudden brittle-type failure. Only occurs at relatively high tensile stresses, and in particular chemical environments.

SYMBOLS

A	- cross-section area of a rock bolt or rock anchor
c	- cohesion
E	- Young's modulus
e	- dilation, or movement normal to a discontinuity.
FS	- factor of safety
H	- slope height
k	- static earthquake coefficient, representing the fraction of gravity acting horizontally equivalent to an earthquake force
L	- the tensioned length of a rock bolt or rock anchor
N	- normal reaction in a two block sliding analysis
P	- inter block reaction in a two block sliding analysis
R	- resultant of weight and earthquake force, or available strength in a two block sliding analysis
S	- shear strength on a surface of sliding, or shear force in a two block sliding analysis
s_c	- average shear strength of shotcrete
T	- support force from rock bolts, rock anchors or shotcrete
t	- average thickness of shotcrete
U	- force exerted by groundwater
V	- dead weight of a buttress
W	- weight of potential sliding material in a slope
α	- inclination of sliding surface from horizontal
β	- inclination of slope face from horizontal
δ	- inclination of rock bolt or rock anchor support force from horizontal
ε	- inclination from the vertical of the resultant of weight and natural static equivalent earthquake force
λ	- inclination of upper sliding surface from horizontal in a two block sliding analysis
μ	- coefficient of friction against sliding of a buttress
σ	- normal stress
ϕ	- angle of internal friction