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	Engineering and Design ROCK REINFORCEMENT	
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DEPARTMENT OF THE ARMY
Office of the Chief of Engineers
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Engineer Manual
No. 1110-2-2907

15 February 1980

Engineering and Design
ROCK REINFORCEMENT

1. Purpose. The purpose of this manual is to outline techniques and procedures of rock reinforcement for underground and surface structures in civil èngineering works. Design procedures and examples of successful installations are presented for guidance in the design and construction of rock reinforcement systems.

2. Applicability. This manual applies to all field operating agencies having responsibility for the design of Civil Works projects.

3. General. The design of reinforced rock structures follows the same basic steps used in the design of other structures. The primary emphasis should be to guard against the most probable modes of deformation that may lead to collapse. Design procedures outlined in this manual are not restricted to the reinforcement elements only but also considers the overall design of the rock structure.

FOR THE CHIEF OF ENGINEERS:

for William E Lang ... cr

FORREST T. GAY III
Colonel, Corps of Engineers
Executive Director, Engineer Staff

Initial Edition

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CHAPTER 1
INTRODUCTION

1-1. Purpose. The purpose of this manual is to outline techniques and procedures of rock reinforcement for underground and surface structures in civil engineering works. Design procedures and examples of successful installations are presented for guidance in the design and construction of rock reinforcement systems.

1-2. Applicability. This manual provides guidance to all elements of the Corps of Engineers responsible for the design and construction of rock reinforcement systems.

1-3. References.

a. Works Cited. Standard references pertaining to this manual are listed in Appendix F. Superior numbers are used in the text to identify similarly numbered items in Appendix F.

b. Bibliography. Additional sources of information for supplementary reading are listed in Appendix G.

1-4. Terminology. The following definitions are presented as terminology that is essential to the use of this manual.

a. Rock Reinforcement. The placement of rock bolts, untensioned rock dowels, prestressed rock anchors, or wire tendons in a rock mass to reinforce and mobilize the rock's natural competency to support itself.

b. Rock Support. The placement of supports such as wood sets, steel sets, or reinforced concrete linings to provide resistance to inward movement of rock toward the excavation.

c. Rock Bolt. A tensioned reinforcement element consisting of a rod, a mechanical or grouted anchorage, and a plate and nut for tensioning or for retaining tension applied by direct pull or by torquing.

d. Prestressed Rock Anchor or Tendon. A tensioned reinforcing element, generally of higher capacity than a rock bolt, consisting of a high strength steel tendon (made up of one or more wires, strands or bars) fitted with a stressing anchorage at one end and a means permitting force transfer to the grout and rock at the other end.

e. Rock Dowel. An untensioned reinforcement element consisting of a rod embedded in a mortar or grout filled hole.

f. Element. General term for rock bolts, tendons, and rock anchors.

g. Pattern Reinforcement. The installation of reinforcement elements in a regular pattern over the excavation surface.

h. Spot Reinforcement. The installation of reinforcement elements in localized areas of potential instability or weakness as determined during excavation. This spot reinforcement may be in addition to pattern bolting or structural support.

1-5. Concept of Rock Reinforcement.

a. There are numerous variations of the concept of rock reinforcement. Each variation is usually derived from a particular theory that is used to calculate required reinforcement. The central concept found in all variations is that of rock mass strengthening. In other words, reinforcement is used to enhance the ability of the rock to be self-supporting. Rock masses are quite strong if progressive failure along planes of low strength is prevented. It is the purpose of reinforcement to prevent this failure, thereby allowing the rock to support itself with its inherent strength.

b. Rock in situ may be thought of as a complex structure of discrete blocks or fragments with near perfect interlocking of these blocks and fragments (EM 1110-1-1801[1]). In most civil engineering applications the material strength of the intact rock between discontinuities is high relative to the expected stresses. For this reason, deformation of the rock is generally controlled by the discontinuities. These discontinuities may be joints, bedding plane joints, foliation surfaces, shear zones, or faults.

c. Progressive deformation and relaxation may result in the collapse of a portion of the rock structure when shear stresses along discontinuities are only a fraction of the in situ rock mass shear strength. In jointed rock masses, numerous factors determine the nature and extent of the rock mass deformation. These include the following:

(1) The strength, deformability, orientation, and frequency of discontinuities.

(2) The size, shape, and orientation of the excavation with respect to the discontinuities.

(3) Method of excavation.

(4) The state of stress in the rock mass surrounding the excavation.

(5) Strength of the intact rock.

d. Rock reinforcement prevents or limits the deformation and dilation of the rock that may lead to collapse. The strength of the rock is maintained by the reinforcement. A more specific explanation of rock reinforcement is that it provides tensile, shear, and/or frictional strength across discontinuities. In this respect it is similar to diagonal tension reinforcement in reinforced concrete structures. The primary reason for the success of rock reinforcement is the immediate restraint which reduces rock deformation thus greatly enhancing the possibility of early stabilization following excavation. The shear strength of the discontinuities will always be less after slippage or separation has taken place. For this reason the reinforcement should be installed as soon as possible after the excavation is made. As is the case in the design of any structure, the usual parameters are determined not only by available design procedures but also by experience and appropriate empirical rules. For rock reinforcement these parameters include element spacing, size, prestress forces, and lengths.

e. The advancing state-of-the-art of rock reinforcement has now reached the point that it is always considered as an alternative or partial alternative to direct structural support of rock excavations. In its various forms, reinforcement is in common use on projects with open cuts, portals, tunnels, shafts, and large chambers as well as for stabilizing existing slopes and strengthening weak foundation rock such as the passive wedge areas immediately downstream from the toes of concrete dams. The savings that may be realized by using rock reinforcement rather than internal structural support for underground openings make the consideration of reinforcement a necessity in the design for rock stabilization of these types of excavations.

f. To provide positive, corrosion resistant reinforcement, all reinforcing elements must be permanently bonded to the rock by surrounding the elements with grout, mortar, or resins.

CHAPTER 2
DESIGN OF ROCK REINFORCEMENT

2-1. General.

a. The design of reinforced rock structures follows the same basic steps used in the design of other structures. The differences, for example, between reinforced concrete design and reinforced rock design are in emphasis rather than basic design philosophy. The structural engineer in approaching the design of structures such as buildings and bridges uses conventional methods of structure analysis and provisions of design codes to produce a design which will perform as anticipated when it is loaded after completion. In these cases the modes of deformation and collapse of these structural configurations are well known. However, the methods of analysis and the provisions of design codes are not nearly so explicit when consideration is given to the behavior of the composite structure of steel, concrete, and rock, which is the actual case of a bridge or building and its foundations.

b. The discontinuous nature of rock masses permits many possible modes of deformation. Also, it should always be kept in mind that excavations in rock are made in a material that is always under in situ stress and strain and which generally is in stable equilibrium before the excavation is made. This is the opposite of most civil engineering structures where the structural materials are not fully loaded until the structure is completed and in service. The complexity of rock structures in a discontinuous rock mass has become apparent from experience with analysis techniques.

c. In the design of rock reinforcement, the primary emphasis should be to guard against the most probable modes of deformation that may lead to collapse. The information necessary to the design is not available in the early design stages but must be gathered from the time of preliminary geological investigations through the exploration, design, and construction stages of a project. The designer of rock reinforcement systems must place primary emphasis on modes of deformation rather than concentrating on calculations of stresses, strains, and load factors. Suitable construction procedures must also be considered as part of the design process and appropriate provisions made in the specifications to ensure that design requirements will be met. Also, the specification provisions must provide the contractual framework for modification to the basic design of the rock reinforcement as construction proceeds. It is important that the contractor is aware that such modifications will be made and this should be noted in the specifications.

d. The procedure to be followed in designing a rock reinforcement system should not be restricted to the reinforcement elements only but must also consider and be integrated with the overall design of the rock structure. In the following sections consideration is given first, to the several stages of design; second, to the basic characteristics of a sound design; third, to empirical guidelines that arise from past experience on other projects; and fourth, to analytical techniques that may be used to assist the designer.

2-2. Design Procedure.

a. General. It is beyond the scope of this manual to outline the considerations that lead to the need for an underground opening or an open cut of a given size and geometry in a given rock mass. The design procedure should, of course, be applied to each alternate configuration considered for a given project. It is assumed that the designer is confronted with the problem of designing a reinforced rock structure so that it maintains its stability under the service conditions to which it will be subjected.

b. Stages of Design.

(1) Preliminary design and estimate stage. The first design efforts should be directed to determining approximately the type and amount of reinforcement that might be required for a given project. At this point in the design the most useful information will be experience from similar jobs. Because the exploration and testing programs would not yet have provided the detailed information necessary for detailed analysis and design, the design engineer should become familiar with techniques of stabilization that have been successful. This familiarization should include a general knowledge of rock mechanics and rock stabilization which can be gained from text books, technical papers, and lectures. It should also include a review of the plans, specifications, and field experience for jobs with conditions similar to those expected on the project under consideration. Alternate types of reinforcement and schemes of excavation and reinforcement should be carefully outlined in preparation for final design.

(2) Final design stage.

(a) As geologic and rock engineering information becomes available and as the plan of the project is finalized, detailed design of the reinforcement system may be pursued. This detailed design has as its end product a set of plans and specifications which will indicate to the contractors what reinforcement the designer considers will be necessary to stabilize the rock structures. The design should include not only the number, length, size, and orientation of reinforcement elements but also excavation-reinforcement sequence and detailed installation requirements. Analyses of possible modes of deformation are made to the extent justified by the details known about the rock. Detailed study should be made of recent projects to ensure that better methods are not being overlooked. A series of laboratory and field tests should be performed to verify acceptability and practicality of all specified hardware and procedures. The specifications should also allow some flexibility in rock reinforcement requirements so that unanticipated geological conditions can be dealt with as economically as possible.

(b) A primary key to the success of a rock reinforcement system is the preparation of adequate provisions in the specifications. The specifications must serve not only to guide the contractor's work and quality control requirements but must also provide a means for informing both the contractor and the inspectors as to what the rock reinforcement requirements are for each project. Examples of quality control requirements included in specifications for a particular job are given by Smart and Friestad.[47] On some projects detailed study will have pinpointed zones requiring reinforcement in addition to the basic pattern. Such reinforcement should be designed and shown on the plans. No matter how detailed the geologic investigations may be, there will always be local conditions that cannot be foreseen and consequently will require additional reinforcement. The specifications should contain provisions for dealing with such conditions and paying for any additional reinforcement required. Guidance criteria to aid in deciding when to add the reinforcement should also be included.

(c) Instrumentation is a basic tool for monitoring rock behavior during construction and indicating variations from design assumptions. It should be planned and designed along with the basic excavation and reinforcement. Also, the specifications should indicate any interference with construction which the instrumentation program might cause. Contractor assistance with the instrumentation should be a pay item in the contract. The most meaningful and useful measurements in the past have been those recording rock deformation and movement. Extensometers, rock bolt deformeters, and survey reference points on the rock surfaces are the most common methods of monitoring rock mass deformations.

(3) Design modifications during construction. Requirements for rock reinforcement are not complete until the excavation is completed and all rock structures are stable. If maximum benefit is to accrue from flexibility in the specifications, then continuous checks on design assumptions should be made as construction proceeds. Signs of instability may call for further analysis and redesign based on modes of deformation not considered in the initial design and which can be

ascertained only through visual observation and measurements as the work proceeds. It is at this stage, that many analytical techniques may prove to be most useful. Modifications in the basic design that are made during construction may be of minor importance from the standpoint of construction cost but can be of major importance from the standpoint of overall stability.

c. Basic Characteristics of Sound Designs.

(1) Checklists. Checklists for the design of rock reinforcement systems are given below. The first includes desirable component activities of the design procedure, while the second gives desirable characteristics of the reinforcement system produced by the design.

(2) Design activities. The following activities should be an integral part of any design.

(a) Detailed geologic investigations. One of the designer's responsibilities is to request and obtain geologic data that are necessary for an adequate design. The collection of this data must begin very early in the design and gaps in the data must be allowed for by providing flexibility in the design procedure.

(b) Coordination between geologists and engineers. The design engineer must constantly review geologic data as they become available to check and modify assumptions made about the rock mass that were made when the preliminary designs were initiated (EM 1110-1-1801[1] and EM 1110-1-1806[2]). Also the geologists should be aware of design requirements so that they can supply geological data and interpretations which will be of maximum usefulness during both design and construction. In some cases the same geologists will be on the site during construction, which can be of considerable benefit to the designer and the resident engineer.

(c) Field and laboratory tests of reinforcement. If tests on the specified rock reinforcement installations are not performed prior to construction, problems may arise during the initial stages of excavation as the contractor applies the designer's specified installation procedures. This early period of construction is often very critical, for example, portal excavation at the beginning of a tunnel project. Construction problems with reinforcement installation at such critical times must be avoided. Consequently, the design engineers should develop a field testing program that includes drilling holes, element installation, and element testing. All details of such investigations should be recorded. Detailed procedures critical to the successful installation should be given special attention in the specifications. Such testing enables the designer to select the methods of rock reinforcement best suited to the site and to eliminate less favorable methods. It also provides bases for assessing unusual conditions that may arise during construction. Laboratory tests of hardware should be made a part of the design tests. Again it is emphasized that all details of tests should be recorded.

(d) Detailed study of case histories. The study of previous designs provides basic guidance on what has been found to be good and bad practice. However, studies of similar jobs cannot be limited only to bolting patterns, bolt lengths, and types. The geology and problems encountered during construction must be understood if the pitfalls at other projects are to be avoided.

(e) Analyses of probable mechanisms of deformation. The preconstruction design is not complete unless likely modes of rock deformation, including effects of hydrostatic pressure, have been investigated at least to the degree possible with available geologic data. Design details such as extra reinforcement at tunnel portals and intersections and in zones of highly fractured rock illustrate that such data have been considered so far as is practicable.

(3) Desirable characteristics of reinforcement systems. The plans and specifications should be checked to ascertain that the following desirable characteristics of reinforcement have been achieved to the maximum practical extent:

(a) Early installation of reinforcement. The behavior of rock masses under stresses induced by the excavation is one of strain-weakening in most cases where stability is in question. For this reason strains of an inelastic nature (permanent deformations) should be arrested as soon as practical following excavation in the case of an active construction project. In the case of natural slopes or existing structures, detection of such permanent deformations is basic to designing remedial measures to improve stability. The practicality of installing tensioned bolts immediately behind the working face in tunnels and recessed bolts and anchors through unexcavated rock has been proven. These practices should be followed in all cases where reinforcement is the primary means of rock stabilization.

(b) Ductility of the reinforcement elements. Ductility is critical to the successful use of rock reinforcement. Invariably there will be zones of rock that deform or yield with changing stress conditions. The reinforcement must be sufficiently ductile to accept reasonable deformations without failure. Common points of failure are in the anchorage and through the root of cut threads. Ideally, maximum use should be made of the ductility of the bar material itself. This means that the anchorage and bearing plate, washers, nut, and thread assemblies should have strengths greater than the yield point of the bolt shank. This may not always be achievable for the anchorage. However, full length bonding of the element to the rock by grouting will produce the ideal situation where failure at one point in the whole element assembly does not necessarily destroy its usefulness.

(c) Tensioning of bolts at the time of installation. The tension in a bolt at the time of installation combines with other factors including time of installation, time of grouting, and strength of the weakest part of a bolt to determine the effectiveness of the bolt assembly. If it is assumed that there are no parts weaker than the yield load of the bar or that the bar is bonded the full length of the drill hole, then the theoretically desirable tension in the bolt is the yield of the bar. This would achieve a maximum compressive stress being applied to the rock while still leaving all the post yield ductility of the steel available to accept rock deformation with constant or slightly increasing loads. However, if there are parts of this bolt assembly weaker than the yield strength of the bar and the bolt is not fully bonded to the rock, then the design is dependent on the reinforcement loads remaining in the elastic range. This is not sound rock reinforcement design. Also, the deformation necessary to increase the load in a 20-foot-long ungrouted rock bolt assembly from a working load of, say, three-quarters yield to full yield would be approximately 0.1 inch. If the deformation is concentrated between two rock blocks and the bolt is fully grouted much less deformation will bring the bolt assembly to yield. Under these conditions a rock reinforcement system should not be designed on the assumption that the elements will behave elastically. Experience has shown that specification of two-thirds to three-quarters of the yield load of the bolt assembly is a practical range for initial tension. This will provide a margin in the elastic range of the bolts to cope with variations in bolt installation and also provide a basis for realistically appraising the measurements from monitoring devices such as deformeters. These comments should not be construed to discount the use of untensioned anchors as rock reinforcement. Prereinforcement with recessed untensioned anchors may often prove to be the technically and economically desirable method of reinforcement. Untensioned anchors develop working loads as initial rock movements take place during excavation. The need for early installation and full length bonding of untensioned anchors cannot be overemphasized.

(d) Stable anchorage. The load and deformation conditions at the anchorage of a tensioned element are quite severe. This is particularly true of mechanical anchorages. High local stresses at the contact between anchorage and rock are conducive to both creep under sustained load and slip or partial failure under dynamic loading. Mechanical anchorages are more prone to relaxation than are grouted type anchorages. However, if the bond length is too short in grouted anchorages then slip may occur. After a bolt assembly is grouted full length, the likelihood of anchorage slip is very greatly reduced if not eliminated.

(e) Early full length bonding of elements to the rock. If the element is not fully grouted, there is always the possibility of loss of tension through anchorage or plate failure. Consequently, full length bonding of the element to the rock at the earliest practicable time provides assurance of the effectiveness of reinforcement during the critical period of nearby excavation. Damage to grout surrounding the reinforcement element from blasting is usually minimal, and the effectiveness of fully grouted reinforcement during this critical period in preventing rock movement is of primary consideration.

(f) Surface treatment. It is seldom possible to install reinforcement through each rock block exposed by the excavation, particularly if the rock is closely jointed. For this reason supplemental surface treatment is required to restrain the rock surface and prevent raveling that could lead to local fallout and possibly general fallout. This is particularly true in crown areas of underground excavations. Surface treatment includes the provision of chain link or welded wire fabric, strapping, and shotcrete. This treatment not only contributes to the structural effectiveness of a reinforcement system but also provides safer working conditions, particularly from rock fall. Early installation of surface support such as chainlink fabric may result in damage to the fabric by flyrock. However, this damage is more than offset by the advantages from its use.

(g) Quality control provisions in the specifications. Even though the designer may have provided appropriate methods of reinforcement and validated his specified procedures by field testing, the contractor's performance of specified installation must be checked. The specifications should require a test program prior to production installation of reinforcement that will verify that proper techniques are being used by the contractor's work force to install the reinforcement. Quantitative indicators of satisfactory installation should be included in the specification for the benefit of inspectors and the contractor. Pull testing of bolts and full flow return of grout as an indication of complete grouting of a bolt are examples of such indicators.

d. Empirical Guidelines for Sound Designs. A summary of many important rock reinforcement case histories is included in chapter 7. The final design of these projects provides the basis for the development of empirical rules that may be used as a guide for minimum reinforcement to be included in preliminary designs. Detailed analyses taking account of geological information derived from diamond drill cores, exploratory tunnels, and surface mapping, as well as results from laboratory and in situ testing to determine rock behavior characteristics, usually indicate the need for more reinforcement than called for by such rules. On the basis of data given in chapter 7, several empirical rules are presented in tables 2-1 and 2-2. It must be reemphasized that these rules give a preliminary configuration for rock reinforcements which must be checked, analyzed, and, as necessary, modified to meet the requirements of a specific rock reinforcement design.

e. Analytical Techniques for Rock Reinforcement Design.

(1) The analytical methods used for assessing the stability of rock structures are direct developments from structural analysis and applied mechanics. Their complexity ranges from the simple case of a block sliding on a surface of known frictional resistance to highly complex finite element solutions that include the effects of slippage along discontinuities and fracture of rock blocks. The analysis of the stability of a rock structure requires the behavior of the structure to be stated in terms of its geometry, the load deformation characteristics of its materials, the virgin in situ stresses, the geological characteristics of the rock, and the conditions induced by the excavation. Such statements may range from a simple case, such as a rock block on the surface under gravity load, to the highly indeterminate conditions associated with intersections of underground openings. The usefulness of an analysis is determined not by the arithmetical accuracy of the calculations but by the accuracy of the input data mentioned above. Various mathematical models that have been used to analyze reinforced rock structures and their applications are discussed below.

(2) Elastic analyses.

(a) Stress concentrations around openings. Solutions for calculating stress conditions near single and multiple openings in stressed elastic media are available for several simple shapes. These include circular or elliptical shapes; and square, rectangular, and triangular shapes with rounded corners as presented by Jaeger and Cook[34] and Obert and Duvall.[38] If the virgin in situ state of stress prior to excavation is known, then the theoretical stresses near the cavern walls can be calculated. These are generally the most important as failure begins at the new surface of an excavation. Comparisons of stresses and the rock strength parameters give a quick indication of areas where stability problems may exist.

(b) Finite element solutions. Elastic finite element analyses have been used to study the stress patterns around single and multiple openings of complicated geometry and in media of varying elastic

Table 2-1. Minimum Length and Maximum Spacing for Rock Reinforcement

Parameter	Empirical Rules	Notes
Minimum Length	Greatest of:	
	a. Two times the bolt spacing	
	b. Three times the width of critical and potentially unstable rock blocks*	
	c. For elements above the springline:	
	1. Spans less than 20 ft - 1/2 span	
	2. Spans from 60 ft to 100 ft - 1/4 span	
	3. Spans 20 ft to 60 ft - interpolate between 10-ft and 15-ft lengths, respectively.	
	d. For elements below the springline:	
	1. For openings less than 60 ft high - use lengths as determined in c. above	
	2. For openings greater than 60 ft high - 1/5 the height	
Maximum Spacing	Least of:	
	a. 1/2 the bolt length	
	b. 1-1/2 the width of critical and potentially unstable rock blocks*	
	c. 6 ft	Greater spacing than 6 ft would make attachment of surface treatment such as chain link fabric difficult
Minimum Spacing	3 to 4 ft	

* Where the joint spacing is close and the span is relatively large, the superposition of two bolting patterns may be appropriate; e.g., long heavy bolts on wide centers to support the span and shorter and thinner bolts on closer centers to stabilize the surface against ravelling due to close jointing as outlined by Reed.[44]

Table 2-2. Minimum Average Confining Pressure for Rock Reinforcement

Parameter	Empirical Rules	Notes
Minimum Average Confining Pressure at Yield Point of Elements	Greatest of:	This assumes the elements will behave in a ductile manner.
	I. Above Springline --	
	a. Pressure equal to a vertical rock load of 0.20 times the opening width	a. For example if the unit weight of the rock is 144 pcf and the opening span is 75 ft the internal confining pressure is 15 psi.
	b. 6 psi	b. For the maximum spacing of 6 ft this requires a yield strength of approximately 32,000 lb.
	II. Below Springline --	
	a. Pressure equal to a vertical rock load of 0.1 times the opening height	a. For example if the unit weight of the rock is 160 pcf and the cavity height is 144 ft the required confining pressure is 16 psi.
	b. 6 psi	b. See note b. under I above.
	III. At Intersections	
	a. 2 times the confining pressure as determined above	a. This reinforcement should be installed from the first opening excavated prior to forming the intersection. Stress concentrations are generally higher at intersections, and rock blocks are free to move toward both openings.

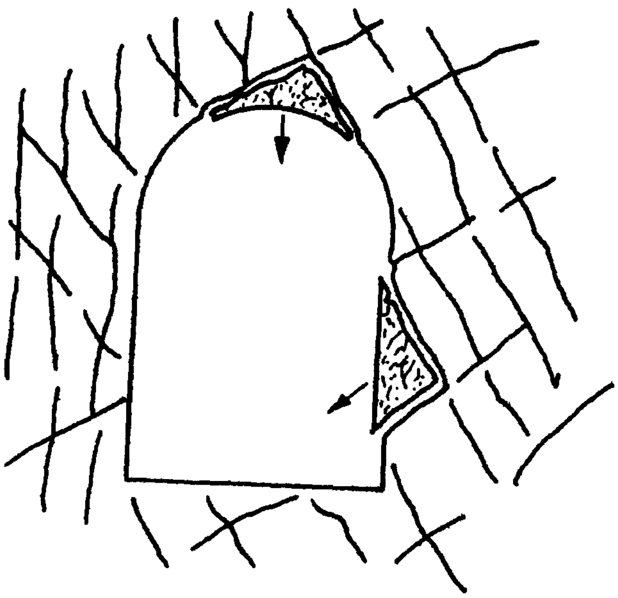
properties. From such analyses areas of high compressive stresses as well as tensile stresses can be delineated and the reinforcement planned accordingly. The Churchill Falls project is a recent example of such analysis.[27] Where the stresses resulting from rock bolting have been included in the analysis, they appear to have only a small effect on the overall stress pattern around openings. In many cases finite element analyses without including the bolting forces are sufficient to outline potential problem areas.

(c) Photoelastic methods. Photoelastic studies have provided much the same information as elastic finite element studies. Photoelastic methods predate finite element work and have been used primarily for homogeneous isotropic materials. However, limited studies of discontinuous and layered media have been made. It may be pointed out that the results of stress analyses either by photoelasticity or numerical methods combined with studies of case histories provide a good source of qualitative and often quantitative information for new designs.

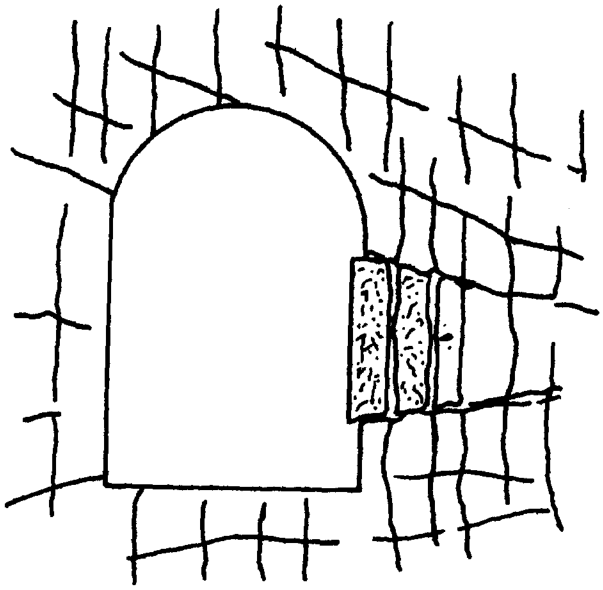
(3) Limit analyses. The most valuable analyses used in the design of rock reinforcement are those that consider possible modes of deformation and methods for arresting such deformation prior to collapse of a given rock structure. Such analyses are studies of failure mechanisms. In this respect they are similar to limit (or plastic) design of steel and reinforced concrete structures. This approach assumes that "yield" can occur at certain points without total collapse of the structure. This approach to rock structure stability realistically accounts for the behavior of the rock and the reinforcement. In rock structures, just as in steel or concrete structures, it is not always possible to keep all stresses at all times less than the intrinsic "strengths" of the materials. However, it is a matter of experience that many excavations where the rock around the opening is highly fractured (that is, it has "failed") are stable and have not collapsed. In such cases it is essential to know that overall deformations of the excavation are "stable." This often requires measures to be taken to improve the rock mass behavior by means such as grouting and rock reinforcement. The following methods are useful tools available for analysis of rock structures:

(a) Rock block stability. In any excavation, the force of gravity cannot be ignored when considering the forces which act on excavation surfaces. Specifically, gravity is a direct contributor to stability or instability immediately around the surface of an excavation, where relocation and permanent deformation has already taken place.

1. As illustrated in figure 2-1, slippage along joints could cause individual rock blocks to become separated from the main rock mass.[35]



(a) Fallout of blocks isolated from rock mass due to failure along joints.



(b) Progressive partial failure of joint blocks adjacent to excavation surface.

Figure 2-1. Gravity effects on jointed rock stability.

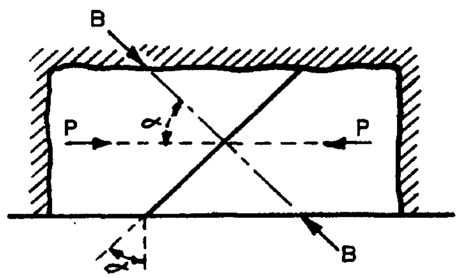
Factors which would induce such conditions are (1) the irregularities of joints are nominal, (2) the resistance force against sliding along joints is low, (3) the angle the joints make with the surface of the excavation is small, and (4) the force of gravity tends to induce motion of the block. Blocks in the roof may be entirely free to fall, but blocks in the wall would have to slide or rotate along the joints at its sides and base before fallout could occur. It is obvious that similar conditions in the floor would not cause concern, but in zones of high stresses or swelling ground the floor can heave into the opening.

2. Calculations to determine the effect of rock reinforcement on the movement of simple systems of rigid rock blocks are not difficult. Several simplified cases are shown in figure 2-2.[37] The expressions shown for each case indicate the required bolting force to maintain stability assuming cohesion along the joint is zero and that slippage along the joint is physically possible. The force of gravity in figure 2-2(a) and (b) is ignored. It is important to note that the forces P are not necessarily the result of elastic behavior in the rock mass, since small deformations across the discontinuities may reduce P to considerably less than what would be assumed from elastic analysis. The analyses of discrete blocks that may be formed by persistent discontinuities should always be analyzed even if definitive tests of in situ rock properties and positive verification of the existence of these discontinuities has not been made. Additional reinforcement to stabilize such blocks is usually required beyond that needed for general overall pattern reinforcement.

3. Similar analytical models to those in figure 2-2 may be postulated to take into account failure by rotation of rock blocks. Rotation as indicated in figure 2-3 almost always plays an important part in deformation and failure mechanisms in rock structures.

4. Sliding rock block models are the most practical method of analysis of rock slopes. Methods for analyzing such models are presented by Hendron, Cording, and Aiyer.[14] The analysis of rock slopes includes consideration of all probable sliding blocks and the possible directions of sliding. As is illustrated in the above reference, though the geometry and statics may be quite complicated in such analyses, the basic approach is simply one of rigid blocks sliding on failure planes of known resistance.

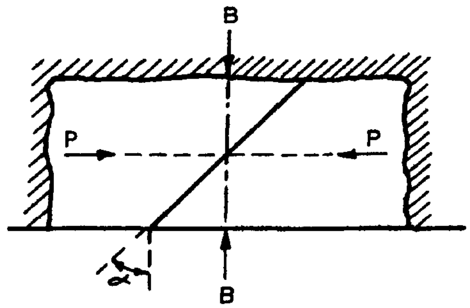
(b) Rock beam or slab concept. In order to gain a better understanding of rock behavior the simple case of flexure in a beam or slab can be considered. Where excavations are made in stratified rock, this concept is directly applicable. In a fixed end beam or slab or substantially uniform material, such as some rock or concrete where the



For stability:

$$\frac{B}{P} > \sin\alpha(\cot\phi - \cot\alpha)$$

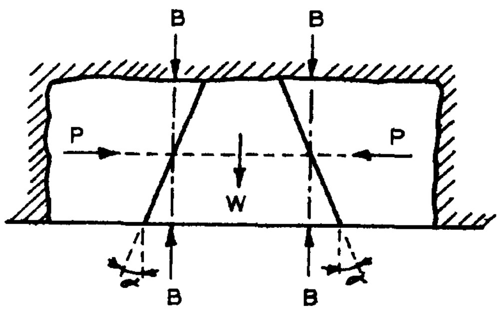
(a) Single joint with bolt normal to joint.



For stability:

$$\tan(\alpha - \phi) < \frac{B}{P} < \tan(\alpha + \phi)$$

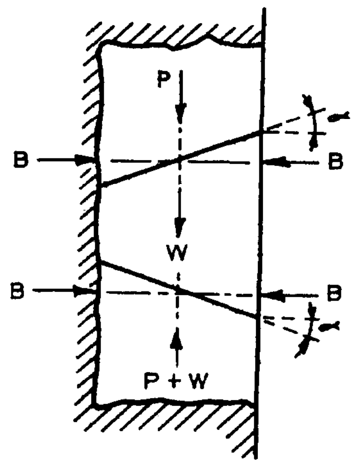
(b) Single joint with bolt normal to end load.



For stability:

$$\tan(\alpha - \phi) < \frac{B - 1/2W}{P} < \tan(\alpha + \phi)$$

(c) Block in horizontal surface.



For stability:

$$\tan(\alpha - \phi) < \frac{B}{P + 1/2W} < \tan(\alpha + \phi)$$

(d) Block in vertical surface.

B = Force exerted by bolt
P = Direct force on joint
tan ϕ = Coefficient of joint friction
W = Weight of block

Figure 2-2. Simple rock bolt models.

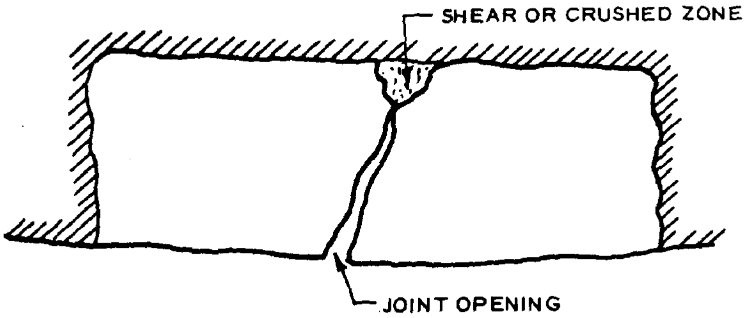


Figure 2-3. Failure by rotation.

tensile strength is less than the compressive strength, the mode of deformation and failure under increasing load will be as sketched in figure 2-4. Cracks will appear first at the ends, A and B, where the flexural tensile stresses are highest and then at the bottom in the center, C. This type of behavior is particularly apparent in materials which not only are stratified but also have joints, shears, or planes of weakness transverse to the axis of the beam. With increasing deformation there will be a tendency for one or more cracks near the center of the span to become "preferred." This leads to a condition as shown in the idealized sketch in figure 2-5. The beam, with increasing

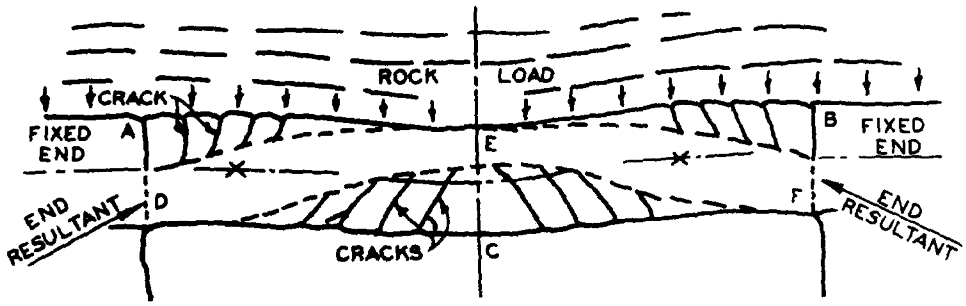


Figure 2-4. Mode of failure of uniform material beam.

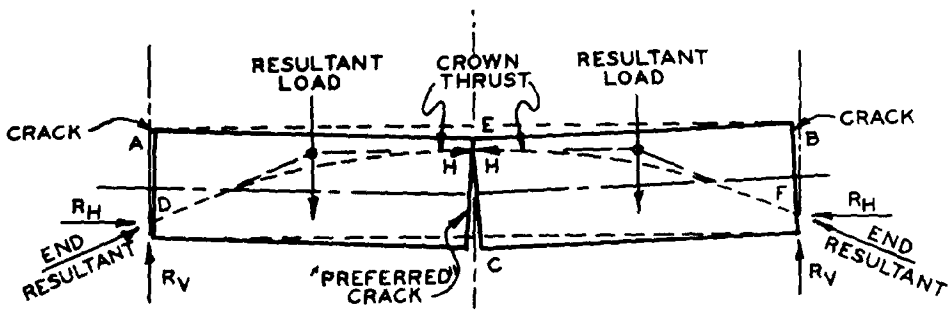


Figure 2-5. Idealized sketch of beam behavior.

deflection, is rotating about a bearing area at the ends, D and F, with a "hinge" at E, thus virtually forming a three hinged arch. With relatively rigid abutments this action leads to large horizontal reactions R_H at D and F, and a horizontal thrust at E. Failure will occur by crushing and shearing of the rock at D, F, and E, which will give increased deflections and ultimately lead to collapse. In practice the abutments at D and F are not rigid and do deform. This has the effect of increasing the central deflection and also of giving a larger bearing area at D than at the center top at E. Wright and Mirza[55] have investigated the stress distribution about such cracked beams photoelastically, and have determined that the bearing area at E is only about 18 percent of the depth of the beam. It is obvious that if the depth of such a beam is small relative to the span then the "arch" action described cannot be effective and collapse will take place with very small deflections. The design of reinforcement to inhibit this type of failure is discussed in the next section.

(c) Rock beam reinforcement. The earlier applications of rock reinforcement were mainly in mining work in sedimentary strata and gave rise to the concept that rock bolts created a beam or slab by clamping together a number of thin or incompetent horizontal strata. Rock reinforcement creates a structural member in any jointed rock mass if a systematic pattern of bolts is used (figure 2-6). The bolts, if tensioned, create a zone of uniform compression somewhat shorter in thickness than the length of the bolts. This zone is confined and acts effectively in stabilizing the rock excavations. Where untensioned grouted rebar is used instead of tensioned rock bolts a somewhat similar condition also develops after limited deformation has taken place. Such reinforcement of a beam or slab roof is sketched in figure 2-7a. The use of steel strapping rockbolt ties or steel channels under the bearing plates of the bolts (figure 2-7b) leads to the concept of a composite beam or slab with the steel channel acting as the tension

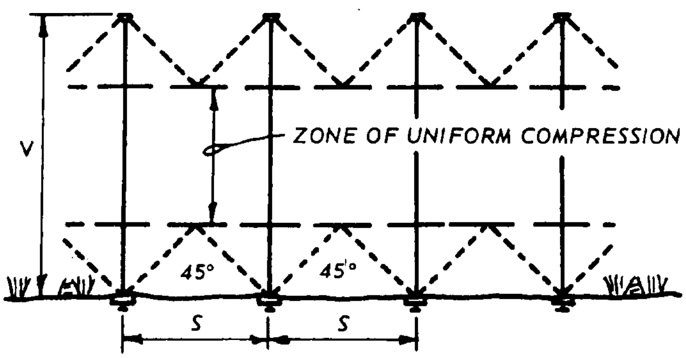


Figure 2-6. Structural member concept.

component. Undoubtedly the beam or slab tends to act at least partially as fixed ended, and angling the bolts near the supports as shown in figure 2-7c will increase their effectiveness. The construction stability that may be made by surface treatment such as steel straps, wire mesh, or a combination of shotcrete and wire mesh, if it is considered as tensile reinforcement at the bottom of the beam, can be assessed by using the archway reinforced concrete beam theory, Lang.[36]

1. In addition to "knitting" together the jointed rock layer between the ends of the bolts and increasing the basic shear strength of the rock in this layer, the rock bolts also act as shear or diagonal tension reinforcement for this layer considered as a beam or slab. Where steel channels or ties are used with angle bolts (figure 2-7c) the action is analogous to post-tensioning in reinforced concrete practice.

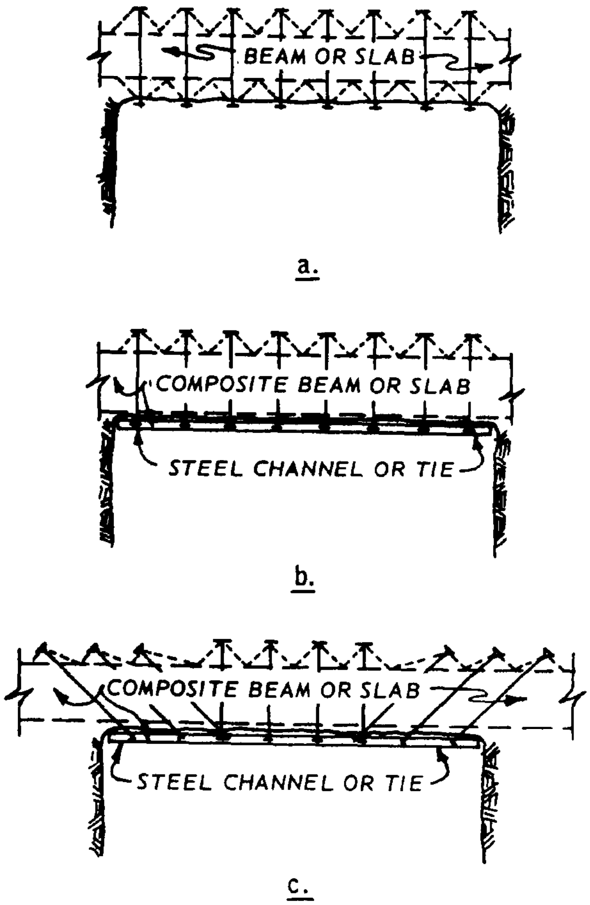


Figure 2-7. Beam or slab concept.

2. The length of the rock bolts is related not only to the geological features of the rock near the surface but also to the span of the opening. The structural member created by the bolts near the surface should be relatively deep compared to the span. It is also related to the spacing chosen for the bolt pattern. Due consideration must be given the type and condition of the rock that is being reinforced.

3. The analysis concept indicated above need not be limited to the crown of a rectangular underground chamber. Similar beam or slab action could be developed horizontally on the face of an open excavation or the walls of an underground excavation. Local instability resulting from rock block rotation as shown in figure 2-3 lends itself to similar analysis. In checking any existing or contemplated rock reinforcement pattern it should be kept in mind that beam action is not likely to occur alone. The rock beam may also be loaded axially as a column.

Axial loading will influence the formation of diagonal tension cracks and introduce the possibility of buckling.

(d) Arch reinforcement. In tunnels or curved roof excavations, rock reinforcement stabilizes the roof by creating a structural arch within the rock between the ends of the bolts. Typical examples are shown in figure 2-8. The effect on the arch member of varying the length and spacing of the bolts is also illustrated. In cases where

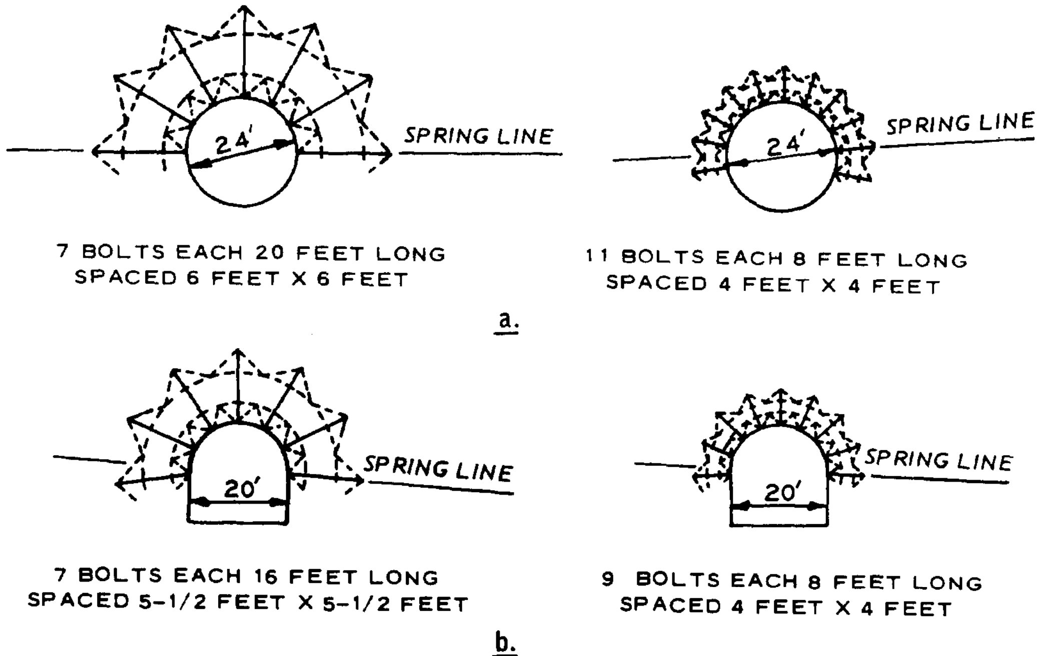


Figure 2.8. Arch concept of rock reinforcement.

the occurrence of persistent well defined joints requires the use of relatively long bolts, it may be feasible to use a smaller number of these and provide shorter supplementary rock bolts between the longer bolts, as shown in figure 2-9. This creates a more heavily reinforced zone near the surface and is effective in stabilizing closely fractured rock.[36] Such shorter bolts can also be used to "split" a regular pattern of primary bolts where monitoring has shown extra reinforcement to be necessary.

1. As in the case of the reinforced rock beam the thickness of the arch should be much larger relative to the span than is considered normal in reinforced concrete or masonry arches. In most cases a static analysis of the "effective arch" inside the reinforced area of the rock will show whether relatively high stresses or possible flexure of the

ROCK REINFORCEMENT

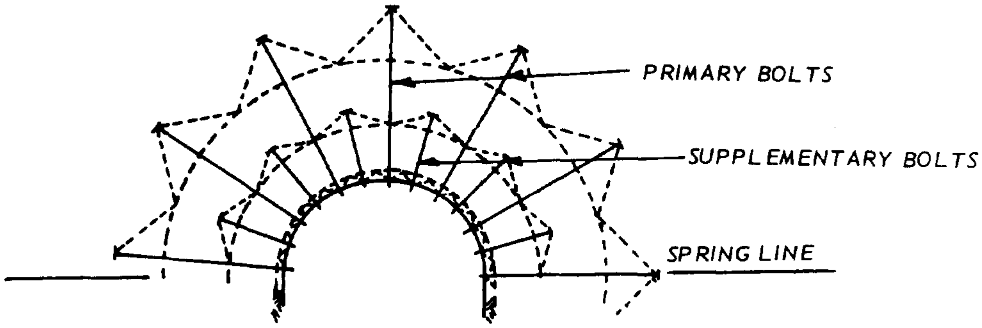


Figure 2-9. Supplementary rock reinforcement.

arch are possible. If the rock is stratified or has a system of joints cutting the effective arch, then shear on such planes of weakness should be checked.

2. With an arch type of roof in an excavation special attention should be given to the abutment or haunch areas. In many cases it will be found that longer bolts are required in this area than may be needed in the crown. The results of an elastic analysis would indicate the likelihood of high stress concentrations or tension zones in these areas as well as the walls and other parts of the excavation. Special analysis of reinforcement requirements in such areas may be required.

(e) Elastoplastic deformation analyses. Rock in situ before it is disturbed by the excavation of a tunnel or other opening is in equilibrium with the virgin in situ stresses. Following excavations stress concentrations are induced around the opening and at the new surface the principal stresses perpendicular to the surface are zero and the stresses tangential to the surface are the maximum principal stresses, with magnitudes which depend on the geometry of the opening and of the virgin in situ stress field. If the stresses tangential to the surface exceed the unconfined compressive strength of the rock, then even in an intact rock, failure will occur. If the rock is jointed or has other planes of weakness intersecting the new surface, then failure will occur and migrate from the surface into the rock mass. Initially, elastic deformation will occur, followed by permanent or plastic deformation. Using the Coulomb criterion for failure, it is possible, if the cohesion, angle of internal friction, and other rock parameters are known, to calculate the thickness of the plastic deformation zone where Coulomb criteria hold as well as the location of the boundary between this zone and the elastic deformation zone where elastic conditions prevail.

1. The theory has been given by Jaeger and Cook[34] and available closed-form solutions reviewed by Hendron and Aiyer.[11] Design approaches have also been investigated by Goodman and Dubois.[10] Existing solutions are for circular tunnels only, under conditions of hydrostatic virgin in situ stresses. However, such analyses for circular tunnel behavior are quite valuable in the design of any tunnel or other excavation with an arch-shaped roof and have been used as a basis for rock reinforcement design by Talobre[48] and others. These analyses allow calculation of the pressure needed on the surface of the excavation to stabilize the relaxed or plastic deformation zone around the excavation and prevent continuing migration of this zone away from the excavation. Theoretically, such a stabilizing pressure can be supplied by a system of rock reinforcement and the bolts anchored beyond the plastic deformation zone.

2. Shorter bolts that are not anchored beyond the plastic deformation can also improve conditions in the rock near the opening thereby achieving additional stability of the plastic deformation zone. The reinforced material is in triaxial compression rather than unconfined compression which occurs at the surface and hence is basically stronger. Also the reinforcement system, although not applying sufficient pressure to prevent the plastic deformation zone from forming, does prevent ravelling and fallout from the surface and thus inhibits "stoping" action that otherwise would take place. Consequently, surface treatment becomes of extreme importance. It may be noted that gravity effects on relaxed rock in the roof of excavations have also been approximated in these analyses.

(f) Finite element analysis. The finite element methods of analysis as well as the elastic analyses mentioned earlier, can be applied to simulate jointed rock consisting essentially of discrete blocks. In the more sophisticated models, predominant joint sets or other possible planes of weakness can be approximated and account taken of failure along discontinuities (joints, etc.) as well as the elastic behavior of the individual rock blocks and different physical properties for various elements. However, the detailed delineation of rock properties throughout a large area and the very large computer capacity required to cope with all these items in an underground complex limits the usefulness of such analyses as design tools. Emphasis in the past has been on models of an entire rock mass in a slope or around an opening using two-dimensional models. Three-dimensional models have been limited to simple axisymmetric cases. However, there appears to be some promise of using three-dimensional finite element analysis to examine local conditions in projects under construction where initial behavior of the rock is known and can be used as a check on the adequacy of the program to predict further behavior. At present, these methods of analysis are useful supplementary design tools where the magnitude of the job warrants the expense and time involved. Examples of recent work in finite element analysis are presented by Goodman[9] and Heuze and Goodman.[13,32]

(g) Interactive computer graphics. The use of interactive graphics for the input and output of geometrical data can be applied to support systems for rock slopes and tunnels. A computer program can model the behavior of assemblages of rock blocks and visually display this behavior on the screen of a Cathode Ray Tube (CRT). There are no restrictions on block shapes and no limits to the magnitude of displacement and rotations that are allowed. The user specifies the rock geometry by drawing lines on the CRT. This information is passed to a minicomputer which interprets each closed area as a discrete block and allows the blocks to move relative to one another under the action of gravity and user specified forces. Joint surface properties (maximum of ten values) may be specified and individual blocks may be excavated, fixed in place, or released as the program runs. The main assumption built into the program is that all deformations occur at the block surfaces. Contact forces can be displayed both in numbers and vectors. As the user may create vector forces to act upon or stabilize any particular block, the size, length, and direction of load (e.g. rock bolts) necessary to stabilize a rock slope or tunnel roof may be determined. The user may experiment with various patterns of rock bolts to determine the most effective distribution to stabilize a particular rock structure. The Distinct Element Method utilizing interactive graphics is useful for modeling numerically those rock systems for which the underlying mechanisms are not known. This system may be treated as a physical model having the additional advantage of being able to vary any parameter on demand. Methods and examples of computing tunnel supports are given by Cundall.[8]

(4) Physical modeling and pilot projects. As an aid to the designer, physical models of the rock structure to be reinforced may be tested under laboratory conditions. Quite simple models can often give a key to potential modes of behavior and failure. Qualitative simulation of rock reinforcement can also be introduced and provide a guide to the need for reinforcement in critical areas. Quantitatively, their usefulness is limited not only by the geologic information available but also by the difficulties of proper scale modeling of the rock properties. On large projects where exploratory tunnels are constructed in the project area, scale models of the large excavations can sometimes be made and serve to test both excavation and reinforcement procedures. Such tests, which give confidence to the designers and the contractors that the design is both technically sound and practical, have been reported by Endersbee and Hofto[30] for the Poatina hydroelectric project in

Tasmania, Australia. Physical model tests are also useful for checking the validity of mathematical model results or for providing a better understanding of the deformation behavior of the reinforcement, the rock, and the discontinuities in the rock. Such tests are reported on by Bureau, Goodman and Heuze.[12]

CHAPTER 3
TENSIONED ROCK BOLTS

3-1. General.

a. The use of tensioned reinforcement elements is included in most rock reinforcement systems. The desired result of a tensioned rock bolt installation is a permanently tensioned reinforcement element with positive bond to the rock. Basic areas of concern in rock bolt installation are:

(1) Obtaining anchorage.

(2) Tensioning to the desired prestress, thus placing the rock around the bolt in compression.

(3) Locking the prestress in the bolt.

(4) Protecting against loss of anchorage and corrosion and utilizing the shear strength of the bolt.

b. Each area of concern is critical to a good permanent reinforcement system. A number of hardware types, techniques, and bonding materials have been used to achieve the desired installation. Some methods are more common and therefore of more interest than others. The methods and hardware types in common use are described in this section. The effectiveness of these methods in achieving a good final installation is discussed.

c. Selection of specific rock bolt hardware, grouting material, and installation method is often the result of personal experience of the design engineer as well as the result of cost studies. Almost any well planned and tested procedure will produce the desired installation. However, even the most highly proven techniques may fail to give satisfactory results if careful attention to detail is not practiced during installation.

3-2. Anchorage Methods. Adequate anchorage is critical to the proper performance of the reinforcement system. It is most critical between the time of initial tensioning and the time of full length grouting. During this period any creep or slippage of the anchor negates a portion of the reinforcement potential of the bolt. Total failure of the anchorage may have costly if not disastrous consequences. Until a bolt has been grouted full length and the grout has set up, the anchorage determines the percentage of total bolt strength that is available to reinforce the rock against discontinuous movements. It is therefore desirable to achieve anchorage strength in excess of the ultimate strength of the bolt. Anchorage strength may be limited by the anchor itself or the rock type and quality. There are two types of anchorage in general use. These are mechanical anchorages and grouted end anchorages. The mechanical types make use of an expanding element that is forced against the walls of the borehole to deform the rock and to provide frictional resistance to pullout. Grouted end anchorages rely on a bonding medium between a portion of the reinforcing element and the rock to develop the desired anchorage strength. Combinations of grouted end and mechanical anchorages are also possible but are not generally used because installation techniques are more complicated and time consuming. However, slot and wedge bolts have been installed in downholes following the placement of grout in the bottom of the hole. Regardless of the anchorage type used, subsequent full length grouting after tensioning improves the reinforcement capability of the element and ensures permanence. In the following paragraphs, the various anchorage types are discussed.

a. Slot and Wedge Anchorage. With the slot and wedge type, anchorage is obtained by inserting the wedge into the slotted end of the bolt and expanding the slot by driving the wedge against the end of the drill hole. Bolts with wedge-type anchorage hold best in hard, sound rock. To assure good anchorage, the hole length must be accurately drilled to within 3 inches and heavy driving equipment is needed. The use of slot and wedge bolts was once very common, but with development of expansion shell anchorage their use has declined rapidly and they are now seldom used in civil engineering projects. A slot and wedge anchorage is illustrated in figure 3-1. This type of bolt can be rapidly fabricated on the job and used in an emergency.

b. Expansion Anchorages.

(1) The expansion-type anchorage device obtains its anchorage by the action of a wedge or cone moving against a shell (or fingers) and expanding the shell against the sides of the hole. Application of torque to the bolt moves the threaded wedge or cone forcing the shell against the rock. With headed bolts the anchor expansion and tensioning can be accomplished in one operation, provided enough thread remains after the initial anchor expansion is completed to bring the bolt under full design tension. Most types of expansion anchorages are patented and have variations in diameter, length, and serrations.

(2) Many expansion units are made of a cylindrical shell into which a tapered plug is drawn, as shown in figure 3-2. A support nut or upset ears on the rod are required for installation. This type of expansion unit usually has four faces and is used in soft rock. At the present time, these anchors are made and successfully used only for rock

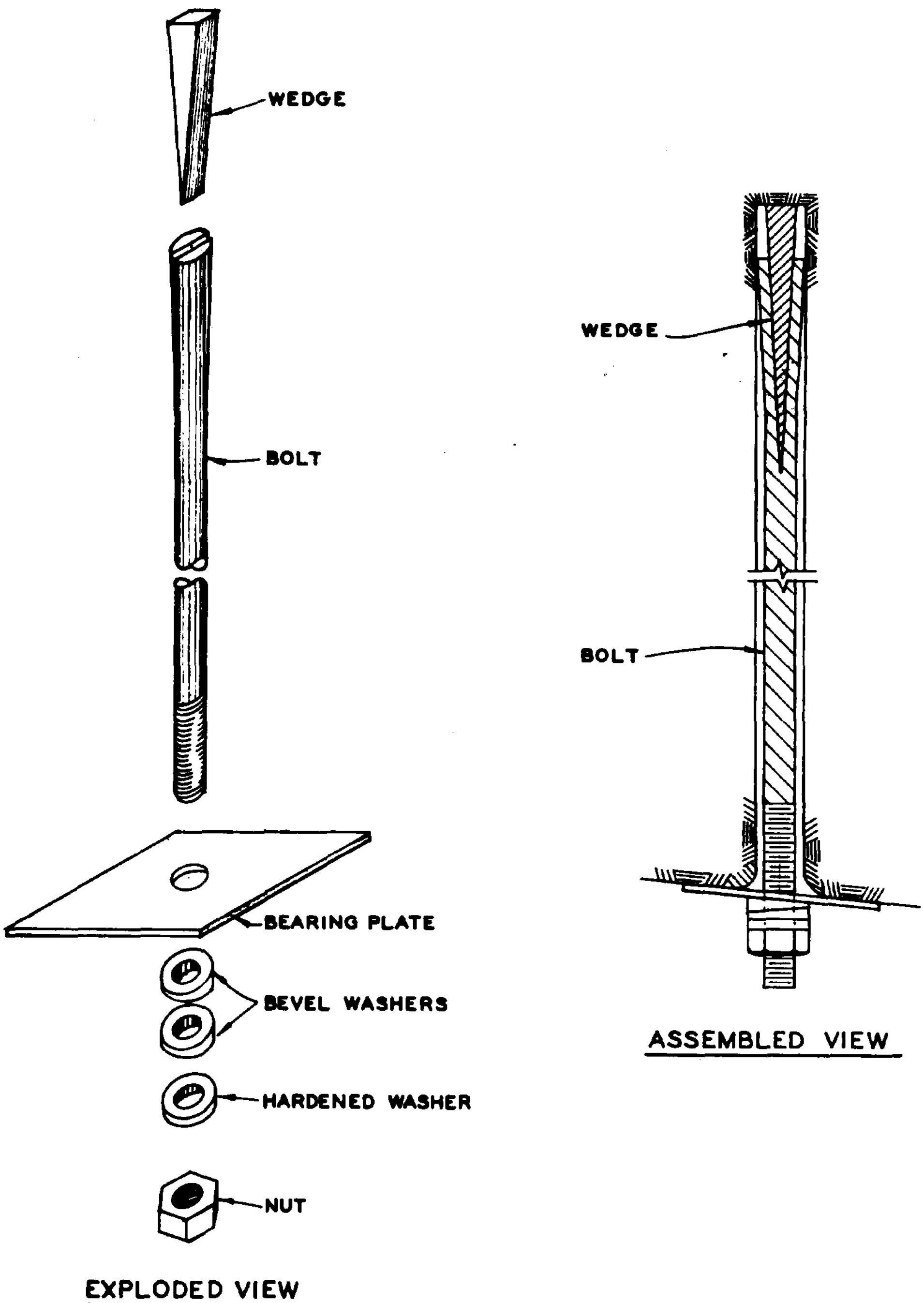


Figure 3-1. Slot and wedge rock bolt.

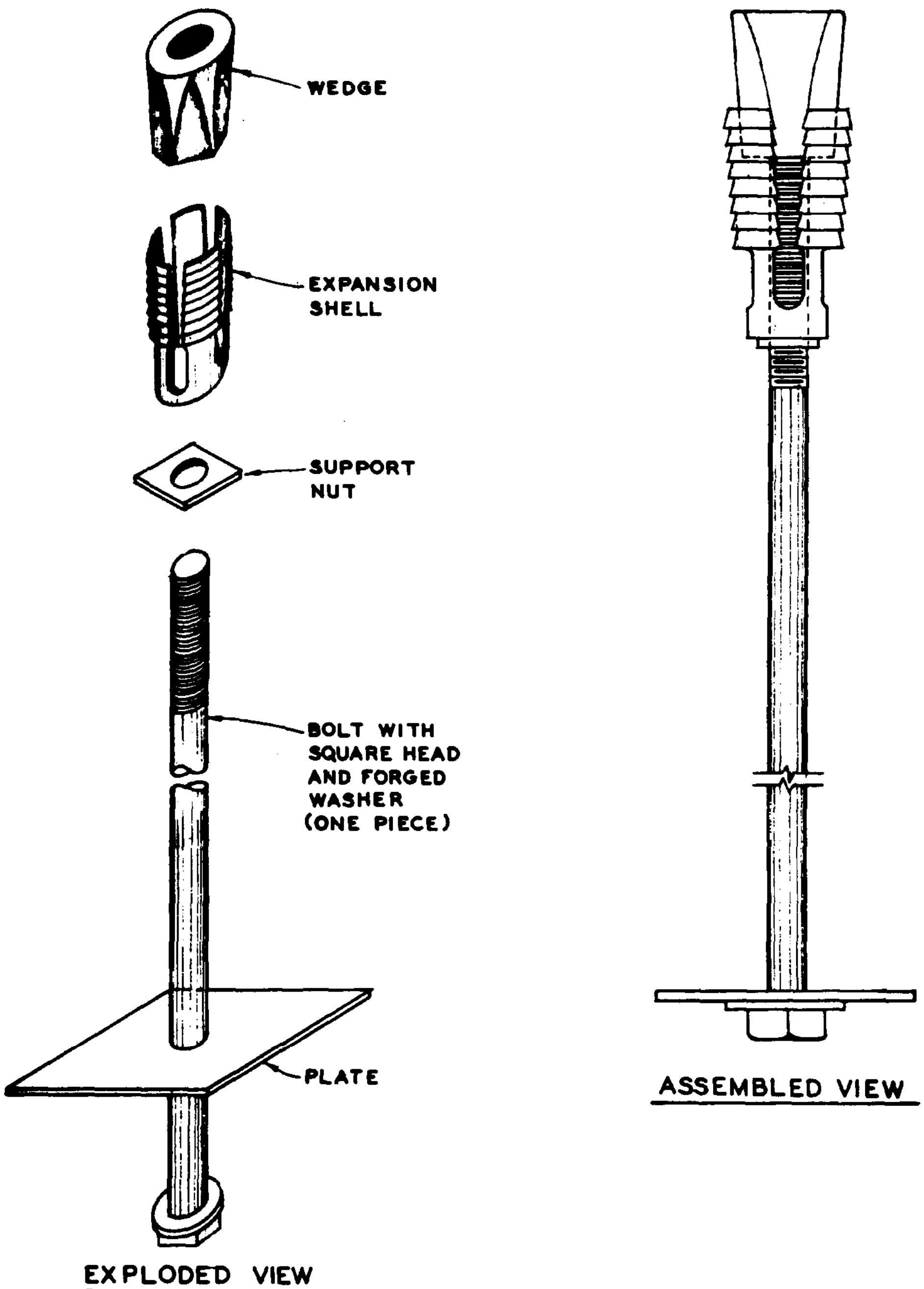


Figure 3-2. Regular expansion anchorage--headed bolt.

bolt diameters of 5/8 inch or 3/4 inch and are used primarily in mines rather than in civil engineering works.

(3) Bail type expansion units are made with a bail or strap between the leaves of the shell for support during installation (figures 3-3 and 3-4). During insertion of the bolt and torquing to set the anchor, the bail keeps the leaves at the same position on each side of the rod as the wedge is moved. Bail types can also be expanded directly (without torquing the bolt) if the bolt is tensioned by direct pull. These expansion units have two or four faces and are used in hard rock. These units, at the present time, are successfully used on 5/8-inch to 1-1/2-inch-diameter rock bolts.

(4) Another expansion shell type unit has a cylinder that is slotted on one side and expands as the cone-shaped wedge is moved toward a thrust collar (figure 3-5). These units have been successfully used on 5/8-inch to 2-inch-diameter rock bolts. By changing the length of the cylinder and plug these bolts can be used in moderately soft to hard rock.

(5) Expansion type anchorages can now be obtained which have a tandem or twin anchor system. Both anchors are set by the same operation. The use of this type of anchor has been limited to a few specific cases.

(6) For use primarily in softer rocks, a special one-piece expansion shell is available which requires a specially reamed conical cavity at the anchor end of the drill hole. A steel bar threads directly into the bottom of the shell. The upper portion of the shell is made of spring steel split into eight "leaves" held in a collapsed position by a special fitting. Once installed in the hole, the "leaves" are released to expand into the cavity by impacting the special fitting against the back of the hole. The rock compressive strength is thus utilized to develop anchorage rather than the friction force developed between the shell and the rock as is the case with other expansion shells.

(7) Table 3-1 lists sizes of commercially available mechanical anchorages for rock bolts.

c. Grouted End Anchorage.

(1) With a grouted end anchorage, the length of element embedment varies with the type and condition of the rock and the bonding medium used. Portland cement, gypsum, and chemical grouts or mortars have been used successfully. The required embedment length in a given rock must

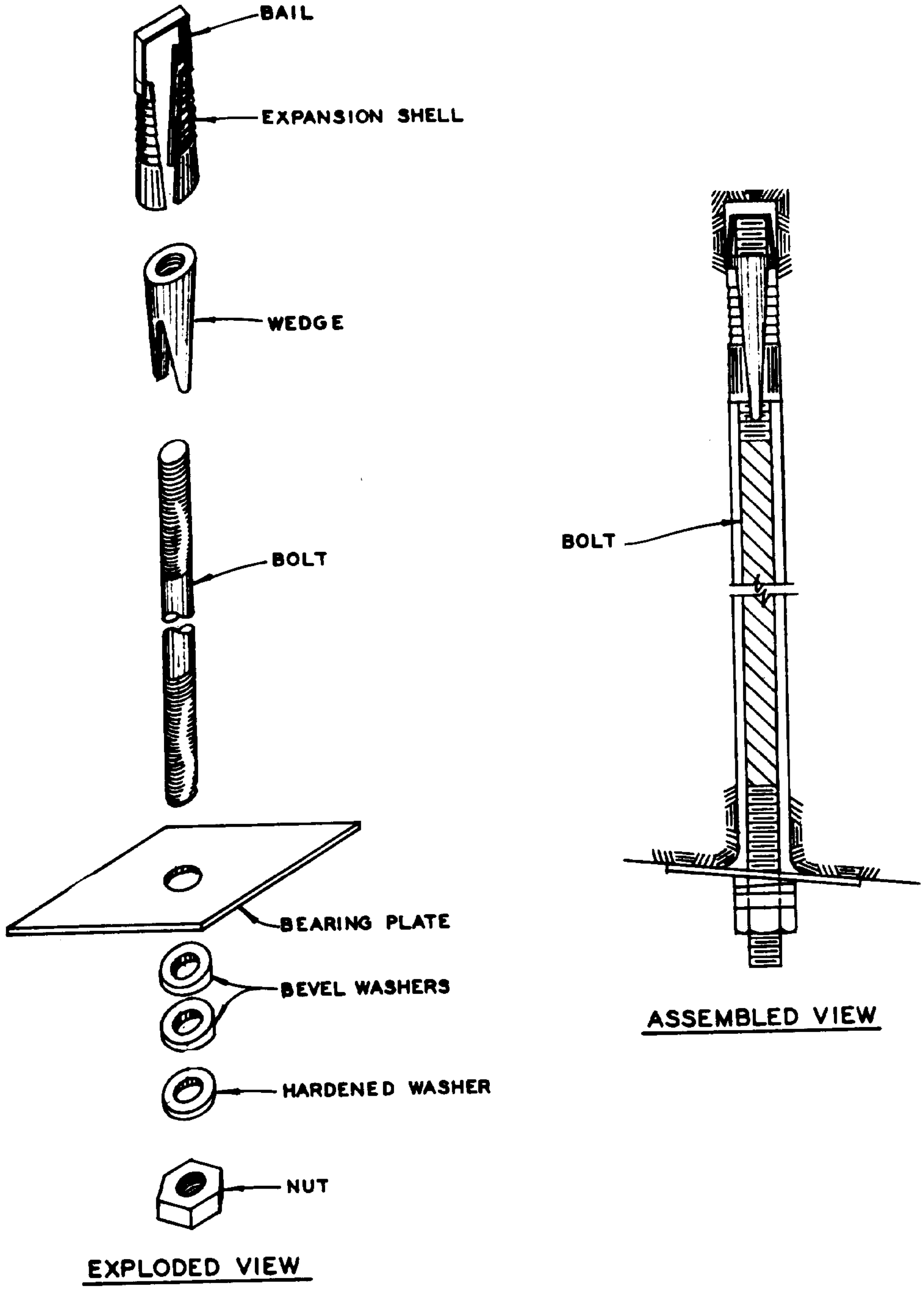


Figure 3-3. Bail expansion anchorage--solid bolt.

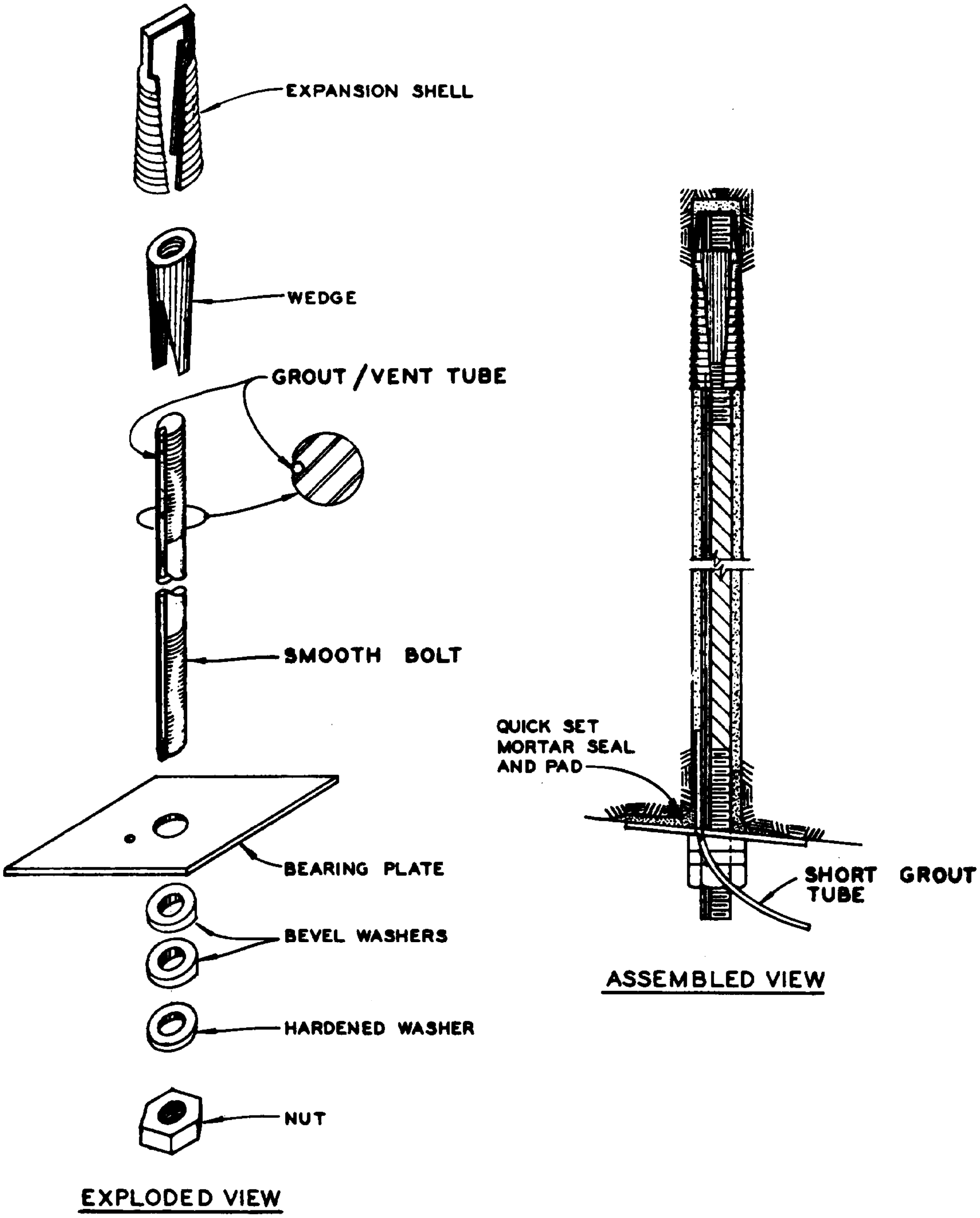


Figure 3-4. Groutable smooth bar rock bolt with integral grout tube.

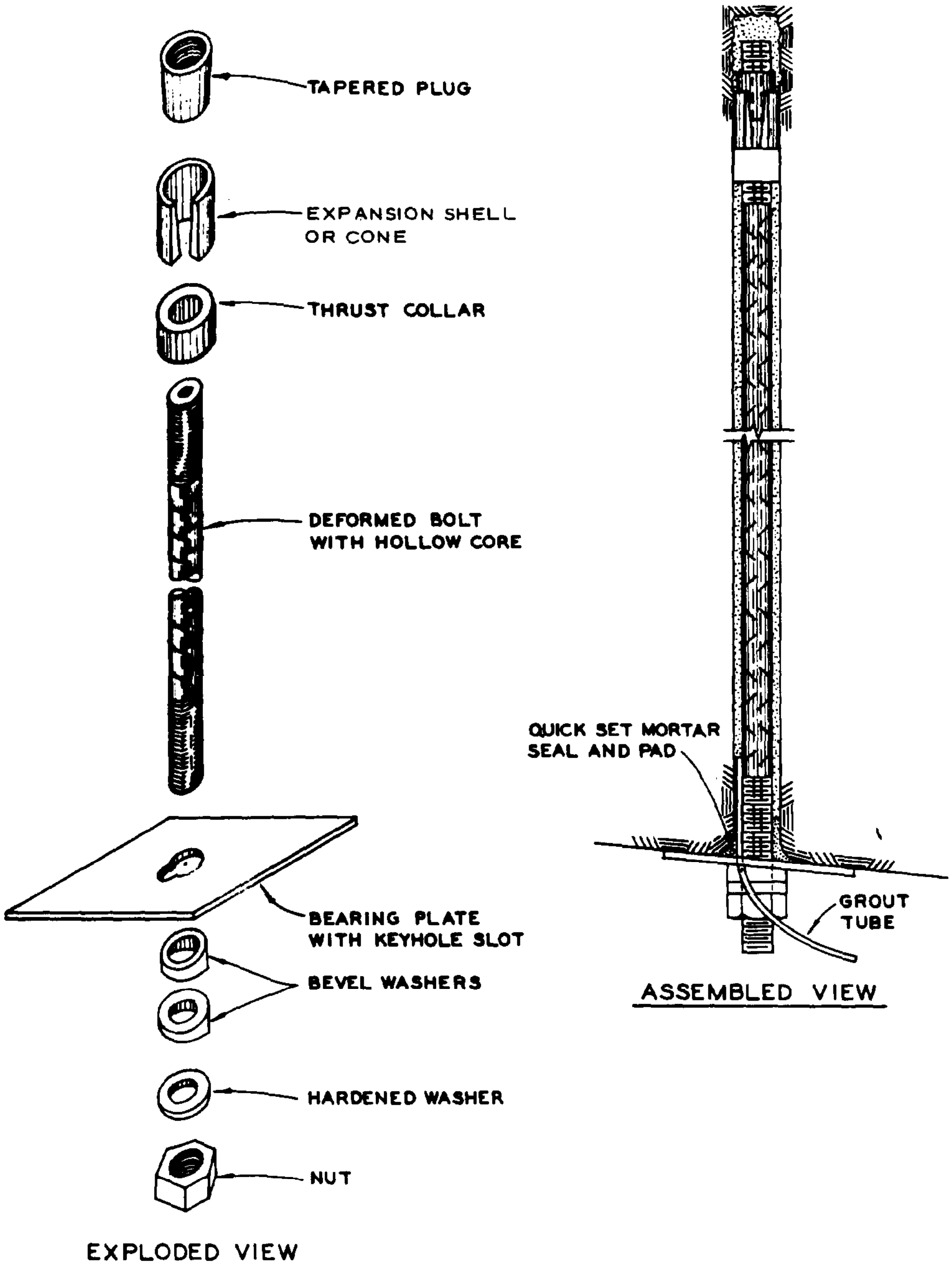


Figure 3-5. Hollow groutable deformed bar rock bolt.

Table 3-1. Examples of a Number of Commercially Available Rock Bolts with Mechanical Anchorage

	Bolt Sizes Available			
	Smooth Bar	Threaded Bolts with Nuts		
Type of Anchorage	Headed Bolts, in.	Solid Bar, in.	Groutable	Remarks
Expansion shell	--	1/2 - 2 in 1/8 increments	No. 8, No. 11, No. 16 (hollow, deformed)	Specializes in rock bolts for use in civil engineering works
Expansion shell, "Pattin"	5/8 - 1	5/8 - 1	--	
Slot and wedge	--	1	--	
Expansion shell	5/8 - 1	--	--	
Slot and wedge	--	1-2-1/2		
Expansion shell	5/8 and 3/4	--	--	
Slot and wedge	--	1	--	
Expansion shell	5/8 and 3/4	1	No. 8 (hollow, deformed)	
Slot and wedge	--	1 - 2-1/4	--	
Expansion shell	--	--	1-in. (smooth bar)	Manufactured with grout/vent tube installed in groove along bolt
Expansion shell for 5/8-in.- and 3/4-in.-diam bolts	--	--	--	Supplies shells only
"Cone" expansion shell	--	--	--	For use in soft rock with 3/4-in.- and 1-in.-diam bolts

be determined by conducting pull tests on the proposed installation. A grouted end anchorage is suitable for use in almost any rock type and holds well even in badly fractured rock. However, with portland cement and accelerators or with some of the chemical grouts, a waiting period of four to eight hours is required before sufficient strength is developed to allow tensioning of the element. Polyester grouts are available to develop sufficient strength within 5 minutes to 30 minutes. With gypsum grout, a waiting period of approximately 30 minutes is required.

(2) The ultimate strength of commonly used reinforcing elements installed in down-holes can be developed by simply embedding the lower end of the element in grout placed by gravity flow at the bottom of the hole. For up-holes, special techniques and aids have been developed to keep the grout at the upper end of the hole. Several processes (some patented) using perforated sleeves, prepackaged resin cartridges, grout transfer tubes, and pressure grouting with pumps are being used successfully. These are now also used in many down-hole installations.

(3) One type of grouted end anchorage utilizing grout tubes through which grout is pumped is shown in figure 3-6. Liquid cementations (portland cement or gypsum) and chemical grouts have been injected under pressure by this method. Where open joints or fractures exist near the anchor area, grout will tend to fill the fractures to help consolidate and strengthen the rock. In up-holes, grout is injected through the shorter tube with the longer tube serving as the air exhaust vent. In down-holes, the function of the tubes is reversed. A return of grout through the air exhaust tube indicates that the anchorage area is completely filled.

(4) Another grouted anchorage system (figure 3-7) uses perforated half-sleeves to retain mortar at the desired location. The half-sleeves are packed with mortar, tied together, and the bar inserted through the sleeve to extrude the mortar through the perforations and completely fill the anchorage area.

(5) The most recent developments in grouted anchorages have been in connection with the formulation and packaging of polyester resin grouts which develop ultimate element strengths within minutes of installation, figures 3-8 and 3-9. If properly installed, these systems incorporate most of the characteristics considered desirable for rock reinforcement. Somewhat longer length bars are employed with this system to obtain anchorage. However, they are particularly useful in weak rocks, or highly fractured zones.

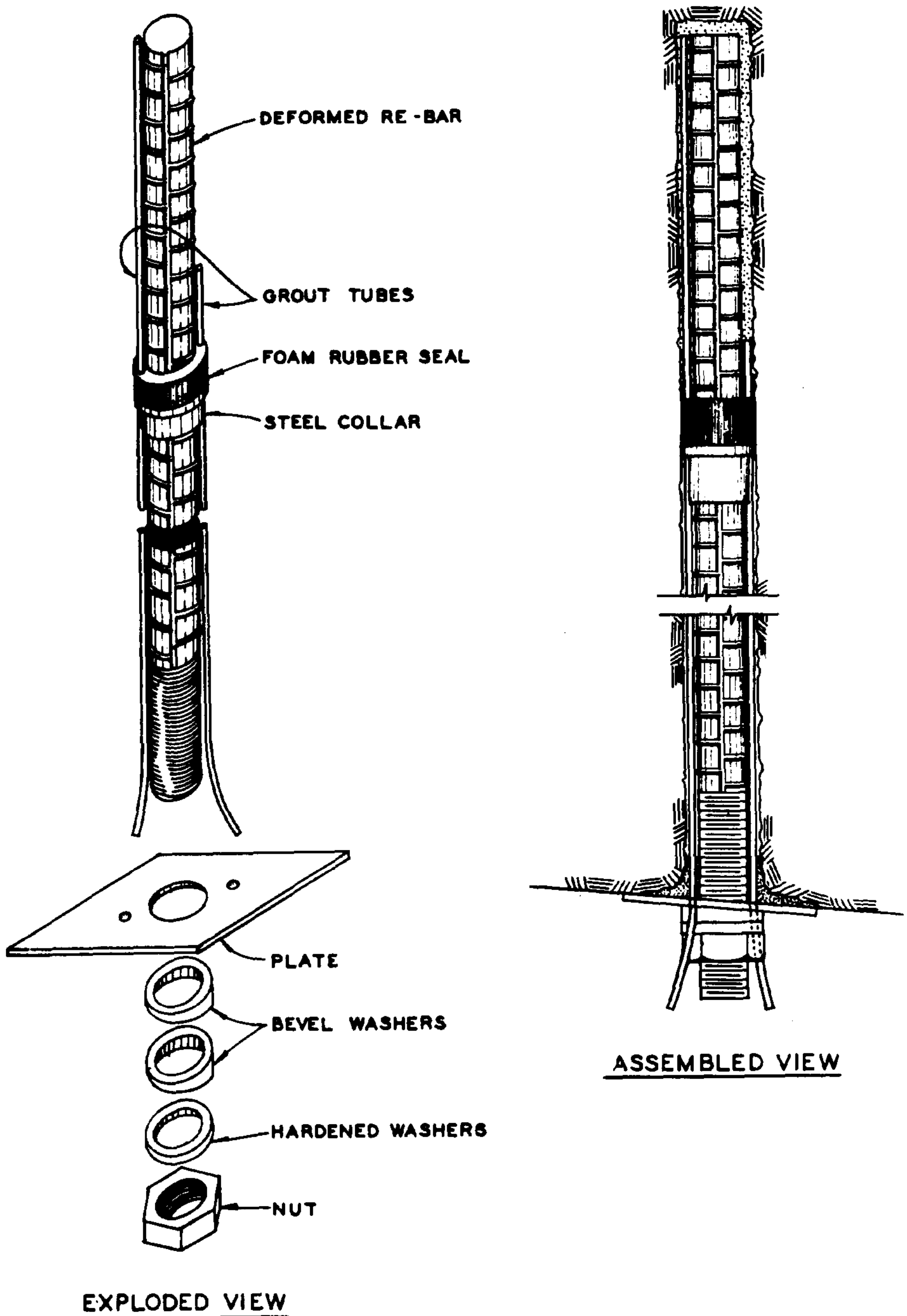


Figure 3-6. Grouted end anchorage, pumpable type.

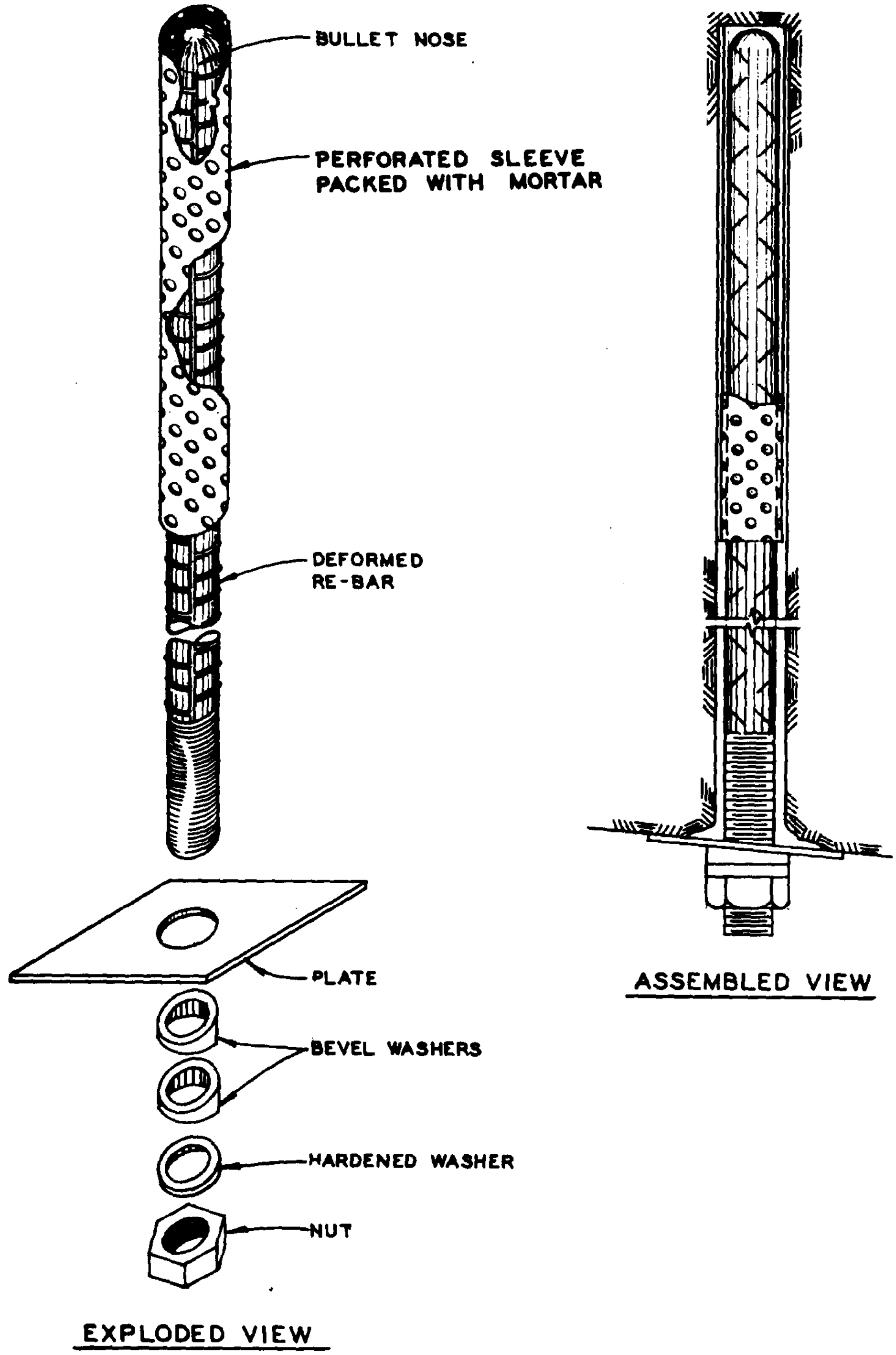


Figure 3-7. Grouted end anchorage, perforated sleeve and mortar type.

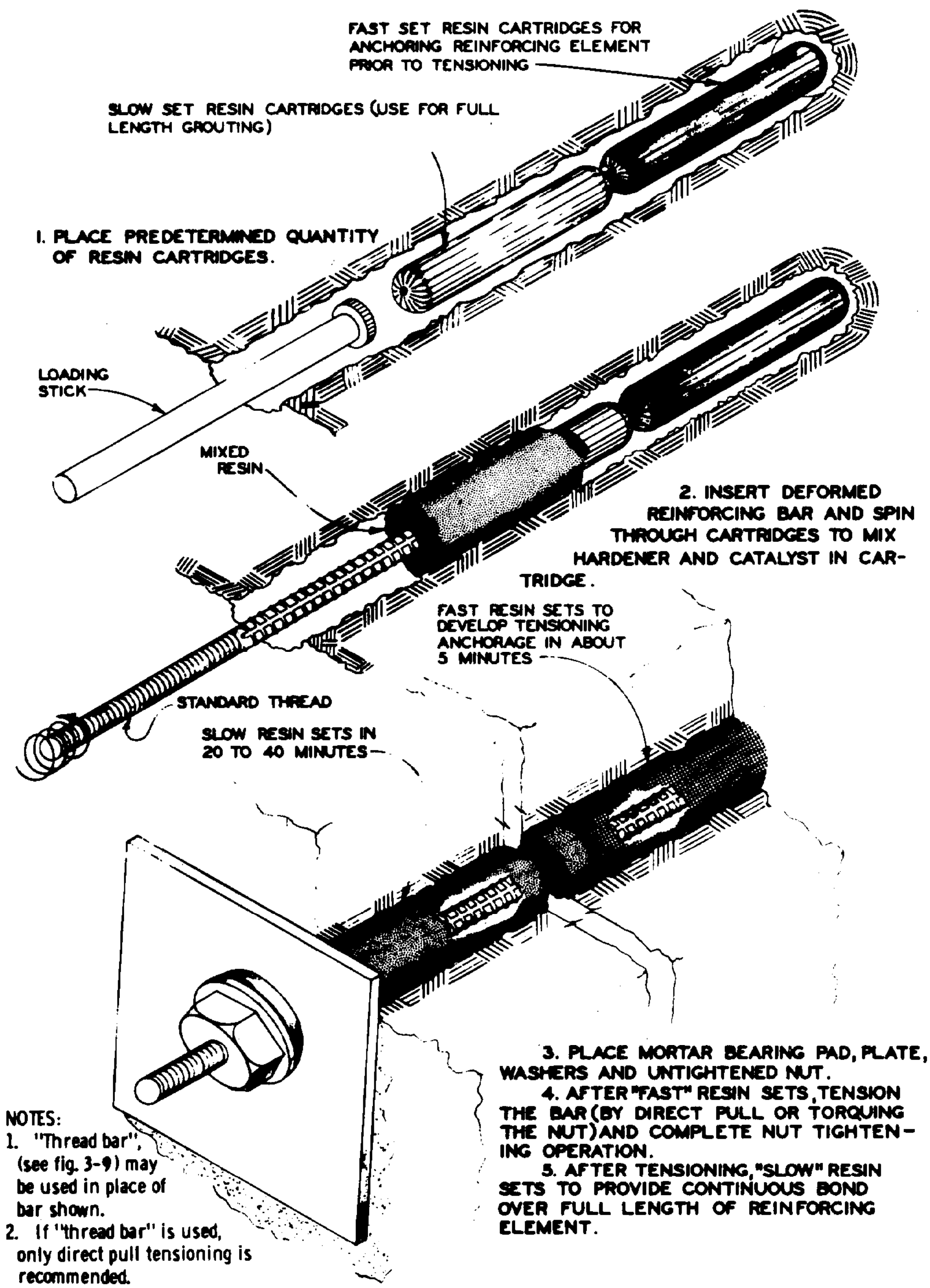


Figure 3-8. Grouted end anchorage, polyester resin (includes full-length bonding technique).

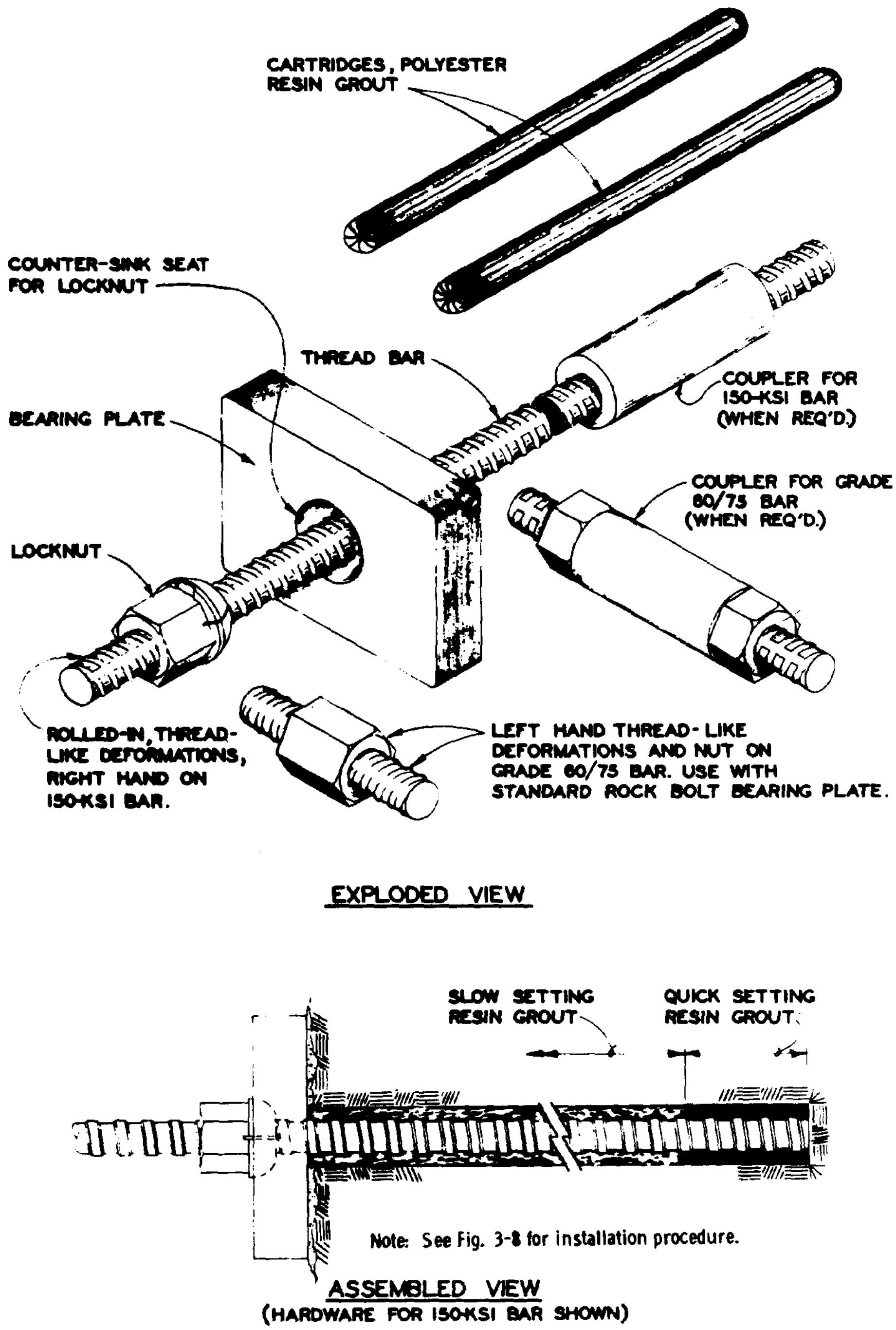


Figure 3-9. "Thread bar" rock bolt with polyester resin grouted anchorage.

(6) A variation of grouted anchorage which can be used in soft rocks involves belling the bottom of the drill hole using commercially available belling tools. The bottom end of the reinforcing element is threaded and a plate with a nut on each side is attached. This type of anchor reduces stress concentrations between the anchor and the rock. In this case, anchorage strength is developed by utilizing the bearing capacity of the rock rather than the shear strength at the grout-rock interface.

(7) Table 3-2 lists sources of commercially available grouting equipment, grout retention aids, and materials in common use for bonding reinforcing elements to the rock to form grouted end anchorages. Also listed, are rock bolt systems offered by suppliers which utilize specially constructed reinforcing elements in combination with a bonding medium. Most of the products can also be used to accomplish full length bonding of tensioned rock bolts or untensioned reinforcing elements.

3-3. Bolts and Accessories.

a. General. Bolts and other components required are listed in the following paragraphs along with a brief discussion of the purpose of each component.

b. Bolts. Steel bars used to connect the anchorage to the bearing plate at the collar of the hole are either smooth rods or deformed bars, solid or hollow (groutable), threaded one or both ends, or threaded one end and headed at the other, depending on the type of anchorage and the type of hardware at the collar. Manufacturer's literature should be checked to determine exact size and specified minimum strengths of the bars, since sizes sometimes vary from the nominal sizes given. Specified minimum yield strengths of commonly used bars usually vary between 30,000 psi and 75,000 psi with tensile strengths ranging from 60,000 psi to 100,000 psi. A minimum elongation in 8-inch-gage length of 8 percent is considered acceptable for the high strength steel ranging up to 17 percent minimum for the lower strength steel. Examples of smooth bar rock bolts are shown in figures 3-1 through 3-3. Groutable types, which are manufactured to simplify the task of pumping liquid grout to completely fill the annulus around the rock bolt, are shown in figure 3-4 (smooth bar with integral grout/vent tube) and in figure 3-5 (hollow core deformed bar). Deformed bars are ordinarily used with the types shown in figure 3-6 and 3-7 because a shorter bond length is required than with smooth bars. For the resin anchor types, examples of which are shown in figures 3-8 and 3-9, deformed bars or specially designed smooth bars must be used to achieve good mixing of the resin components. The "thread bar" shown in figure 3-9 is a specially manufactured deformed bar with a continuous rolled-in pattern of threadlike

Table 3-2. Some of the Commercially Available Equipment and Materials for Installing Grouted Reinforcing Elements

Item	Bonding Medium	Type and Size of Reinforcing Element Recommended
Perfo sleeves	Portland cement or gypsum mortar	Smooth or preferably deformed bars, 3/4-in. - 1-3/8-in. diam (No. 6-No.
Moyno grout pump	Neat cement or gypsum grout, chemical grouts	Smooth or deformed bars, all sizes
Williams grout pump	Neat cement or gypsum grout. Chemical grouts	Offered as part of Williams Groutabl Rock bolt system. Can be used similar to Moyno
Sulfa-Set, F-181	Neat gypsum grout or mortar	Use with perfo sleeves or grout pump and elements listed with each
ROC-LOC 540 Mining Kit	Polyester resin. Placed via transfer tube	Any size deformed bar or threaded smooth bar
Celtite System	Polyester resin. Prepackaged in cartridges	Deformed bars, No. 6 through No. 11, No. 14
Dywidag Threadbar Rock Bolt	Celtite resin cartridges	Dywidag high alloy deformed "threadbar." 5/8-in. diam (230 ksi), 1-in. 1-1/4 in., 1-3/8-in. diam (all 150 ksi)
Celtite resin anchor system. Dywidag Threadbar Rock Bolts	Celtite resin cartridges	Deformed bars, No. 6 through No. 11, No. 14. Dywidag Threadbar, 22 mm, Grade 60
FASLOC resin anchored bolt system	Polyester resin prepackaged in cartridges	Specially manufactured deformed bar headed bolt, 3/4-in. diam
Resin-anchor roof bolt	Epoxy resin and stone aggregate in glass cartridges	Specially manufactured smooth bar bolt with threaded ends. 5/8-in. diam - 1-1/4-in. diam

deformations along its entire length. Bars can be cut to the desired length in the field with a portable band saw. The deformations serve as threads to fit specially supplied anchorage nuts or couplings. One supplier offers a rock bolt system which uses a high alloy deformed "thread bar" having an ultimate strength of 150,000 psi. Another supplier offers "thread bar" steel with a yield strength of 60,000 psi. To avoid confusion in the field, the high alloy steel bar is rolled with right-hand thread deformations whereas left-hand deformations are used on the lower strength bars. Direct pull tensioning is recommended with this type of bar. With other resin anchor systems, specially designed deformed bars are sometimes offered and recommended by the suppliers. Types of available bolts and bars are listed in tables 3-1 and 3-2.

c. Bearing Plate and Mortar Pad. Bearing plates are used to spread out and transfer the concentrated bolt load to the rock around the collar of the hole. The bearing capacity of the rock and the prestress load in the elements will govern the size of the bearing plate but 6 inch by 6 inch by 3/8 inch to 8 inch by 8 inch by 3/8 inch for 1-inch bars or 8 inch by 8 inch by 1/2 inch for 1-3/8-inch bars have been found satisfactory in hard rock. If plate deformation is excessive, double plates may be used but increasing plate thickness is better. Some commercial plates have a keyhole slot for passage of grout tubes, or holes may be drilled for grout tubes. The bearing plate should be seated on a pad of quick-setting mortar to provide a uniform bearing surface and to adjust the angle of the plate with the bolt to a more normal position.

d. Bevel Washers. Bevel washers should be used between the bearing plate and the hardened washer to create a uniform bearing surface for the nut normal to the bolt axis. This provides for efficiency in tensioning the bolt by torquing the nut or in transferring the load to the nut when the bolt is tensioned by direct pull. Also, lack of uniform contact between the nut and bearing surface will result in combined stresses in the bolt which will tend to reduce its strength. The bevel angle of washers varies with manufacturer, but angles of 2 degrees, 7 degrees, and 9 degrees are typical examples of those available. By using washers in pairs, the total bevel angle can be varied by rotating one washer relative to the other.

e. Hardened Flat Washer and Thread Lubricant. A hardened flat washer should be used between the bevel washers and the nut in all cases. In addition, the nut bearing surface and the bolt thread should be treated with a molybdenum disulfide base lubricant because this type of lubricant is highly efficient for reducing friction. The hard washer-lubricant combination greatly reduces friction, thereby requiring less torque on the nut to achieve a given tension in the bolt, and also reduces torque induced in the bar as the nut is tightened. These considerations are equally important when the bolt is tensioned by direct pull since the nut must still be tightened under load to achieve maximum load transfer from the loading jack to the nut.

f. Nut. The nut should develop the ultimate strength of the bar; generally a heavy duty nut is required. A hexagon nut should be used which is double chamfered or washer faced.

g. Grouting Tubes. Tubes installed in the drill holes for transmitting pumped liquid grout or for venting air should be semirigid. Diameters of tubes which have been used successfully are 3/8-inch outside diameter (OD) and 1/4-inch inside diameter (ID) for cement grout and 1/2-inch OD and 3/8-inch ID for resin grout.

h. Bond Breaker.

(1) A short bond breaker or covering (approximately 6 inches long) over the bolt installed immediately behind the bearing plate is recommended to prevent bond between the bar and the mortar of the bearing pad or grout seal and also to keep the threads clean of mortar. Each of these conditions would tend to reduce the stress in the main part of the bolt relative to the applied stress during the tensioning operation. Lubricants specified for use on threads to reduce friction are also effective as bond breakers. However, to assure better quality control, an additional bond breaker should be specified such as a commercially available waterproof paper mailing tube, several wraps of aluminum foil or even kraft paper fitting snugly around the bolt behind the plate.

(2) Although used infrequently in normal fully grouted rock bolt installations, long bond breakers are useful for distributing working stresses in the steel over a greater length or for making maximum use of the steel ductility. Except for the portion of the bar needed to develop end anchorage, the bond breaker can extend over the full bar length. If a mechanical anchor or plate with two nuts is installed at the anchor end, the bond breaker can be installed full length since forces exerted through the bar on the end anchor will be resisted by the shear force developed along the full area of the grout-rock interface.

i. Couplings. Couplings are available for splicing bolts. These are especially useful when installing bolts from a top heading in a large chamber since bolt lengths required will often exceed the height of the opening. Coupling sizes vary with manufacturer and style and should therefore be checked for compatibility with drill hole size and grouting method used. Outside diameters of standard threaded couplings are approximately 1.4 to 1.5 times the thread diameter. Couplings for "thread bars," figure 3-9, are larger and outside diameters range from 1.75 to 2.0 times the nominal bar size. All couplings should be designed to exceed the full strength of bolt and in the case of deformed hollow core rebar rock bolts, the couplings should be "stop-type" coupling to ensure the free flowing of the grout through the coupled connection.

j. Miscellaneous. A variety of other accessories such as angle washers, spherical washers, rubber bolt hole sealers, grout tube adapters, plastic washers for holding resin cartridges in place, and impact wrench adapters are available and described in manufacturer's catalogs.

3-4. Installation Methods.

a. General. The success of a tensioned rock bolt installation is largely dependent on the installation techniques employed by the contractor. Since close attention to details is a necessity to attain maximum advantage of the rock bolt system, supervisors with knowledge and appreciation of the basic principles of rock bolting should be employed. Installation methods recommended in the following paragraphs have been developed by trial and error, by laboratory experiments, and primarily by on-the-job experience on several projects. Deviations from these recommendations by inexperienced personnel should be held to a minimum in order to avoid unnecessary problems and the expense of remedial work.

b. Drill Holes.

(1) Close control of hole drilling operations during installation of rock bolts is extremely important for achieving successful rock reinforcement. Hole size, length, condition, location, and alignment are all factors which can significantly affect the installation of particular systems. Hole size is critical for most installations. The length of hole is critical only for slot and wedge and certain grouted and resin anchorages, but for economy reasons, the hole should not be longer than necessary. Irregular, dog-legged, rifled, or undersized holes usually have less serious effects on good anchorage than oversized holes but for maximum bolt effectiveness, these should also be held to a minimum.

(2) Oversized holes, which have a most serious effect on good anchorage, are caused by allowing the bit to spin at the end of the hole, by using incorrectly sharpened bits, by using the wrong size bit, or by using bits which are incorrectly marked as to size. Bit sizes should

always be measured and drill hole sizes checked routinely with a hole measuring gage throughout the construction period, such as the Ohio Brass gage. The standard gage is 10 feet long and measures from 1-1/4 inch to 1-5/8 inch in diameter, but practically any size or diameter will be supplied on special order. The overall hole size range, however, cannot exceed 3/8 inch for each gage. The Williams gage also supplies a drill hole gage which is available in any length with one model for 1-1/2- to 2-inch holes and another model for 2- to 3-1/2-inch holes. The Ohio Brass and Williams devices are both calibrated in 1/16-inch increments but measurements are easily made to 1/32 inch by estimating between markings.

(3) For mechanical anchorages, the hole size should be the smallest diameter that will allow the assembled unit to be pushed into the hole so that a minimum amount of expansion ability is lost. When hole size tolerances are not given by manufacturers, hole diameters should not be more than 1/32 inch larger than the specified diameter and no smaller. For grouted anchorages, except for the pumpable grout types, size is also critical relative to the bar size and quantity of preplaced grout, but a tolerance of up to 1/16 inch oversize can be allowed.

(4) Some knowledge regarding the capability of rock drills and bits in common use is useful to avoid the possibility of specifying hole diameters and lengths which are difficult or expensive to achieve. Particularly with the harder rocks, the best economy is achieved with the use of percussive or rotary-percussive type drills. The drilling of smaller diameter holes ranging from 1-1/4 inches to 2 inches in diameter is possible with the use of jacklegs utilizing integral drill steel (bit permanently brazed to tip of rod). However, much higher drilling rates are possible with the use of heavier drills, called drifters, mounted on multiple booms on jumbos. With these drills, 1- to 1-1/4-inch-diameter drill steel (rod) is used with detachable bits which drill 1-5/8- to 1-3/4-inch-diameter holes (1-3/4 inches preferable) as a minimum. Drill rods are normally supplied in lengths varying in increments of 2 feet but normally do not exceed 12 feet. For holes longer than 12 feet (or where less headroom exists) extension rods with couplings are required which further limit the minimum size to 1-7/8 inches or 2 inches in diameter. Holes can also be started with a 2-inch diameter or larger bit and finished off in the anchorage area with a smaller bit. This technique works well with mechanical anchorages or grouted end anchorages where subsequent full length bonding is with pumpable grout. However, grouted systems which require or work best with no variation in size throughout the entire hole length will require the use of 1-7/8- or 2-inch-diameter holes as a minimum when installed to depths greater than 12 feet. This, in turn, will influence

the selection of bolt or bar size used to reinforce the rock.

(5) Drill holes must be cleaned just prior to installation of the bolt to remove sludge, rock dust and particles, and debris present in the hole. Cleaning can be accomplished by introducing compressed air (minimum of 50 psi) at the bottom of the hole or by washing with water. (In the case of slaking shales, use air only.) If expansion shell anchorages are being used, down-holes can be overdrilled in length to trap particles, particularly when drilling through high fractured rock.

c. Mortar Pads.

(1) Once positive anchorage is achieved and the bolt tensioned, a significant steel stress loss will occur unless a firm bearing surface exists at the rock face to resist the load in the bolt. Mortar should always be placed under the bearing plate and kept as thin as possible so that high points of the rock surface will remain in contact with the plate to help resist the plate pressure. The plate should be installed as near normal as possible to the long axis of the bolt. In this regard, drillers should be instructed to avoid collaring the drill bit in angular recesses or niches in the rock surface when drilling rock bolt holes. Although advantageous from the driller's standpoint, proper seating of the bearing plate at a workable angle and on a thin mortar pad is made very difficult on the highly irregular rock surface.

(2) Two parts of quick-setting cement and one part Type III portland cement mixed with sufficient water to form a stiff mix has proven to be a good mortar mix. The same mix with the addition of two parts sand will also provide good results. Only the quantity of mortar sufficient for one installation should be mixed at a time. For proper placement, the mortar is packed in a ball around the bar at the collar of the hole and the bearing plate and nut installed. In preparation for pressure grouting around the bolt steel, if required, the collar of the hole is also sealed with mortar at this time. Pressure is then applied to the mortar by rotating the nut until the mortar is evenly distributed under the plate. Adjustments in the angle of the plate are also possible at this time. Setting time required varies with condition and angle of the rock surface and the value of the tensioning load. If the surface is fairly normal to bolt axis and uniformly irregular, and a very thin pad is placed, the bolt can be tensioned in less than a minute just as soon as the mix sets initially. If a large adjustment in the plate angle is necessary and a thick pad results under a portion of the plate, a setting time of 5 minutes-15 minutes or more may be necessary. Tensioning during the first few minutes following initial set should be avoided since the pad has a tendency to break up during this time.

Setting times to fit different conditions can be quickly determined with a small amount of experimentation.

d. Expansion Anchor. Rock bolt installation should be accomplished immediately after the drill hole is cleaned. The following procedures are for a fully grouted bolt:

(1) The protective sleeve over the shell is removed just prior to insertion.

(2) The threads should be checked by screwing the cone onto the bolt until the threads project through the hole. The cone should spin freely on the threads. If it does not, a small amount of grease should be placed on the threads. Care must be taken to keep grease off the surface of the shell.

(3) Insert assembled rock bolt in the drill hole and set the expansion shell with a calibrated preset impact wrench preferably with an automatic cutoff. Check the correct setting torque with a hand torque wrench periodically. If the rock bolt does not have a hollow core, insert a long plastic tube along the bolt extending to the expansion shell.

(4) Apply coating of molybdenum disulfide base lubricant to threads of bolt.

(5) Install waterproof paper tube bond breaker over threads flush with surface of mortar pad.

(6) Install quick-set mortar around collar of hole to provide a uniform bearing surface for bearing plate. For later pressure grouting operations, position the grout tube or tubes at this time and seal the collar of the hole. Position grout tubes so that weight of bar or shifting of bar will not seal or restrict the tube opening.

(7) Seat bearing plate against mortar before it sets up.

(8) Install bevel washers as necessary to provide uniform bearing surface for hardened washer and nut.

(9) Clean exposed threads and apply thread lubricant over threads and contact surfaces of washer and hex nut.

(10) Install hardened washer and hex nut. Advance nut and position plate on mortar pad as described in paragraph 3-4c.

(11) As soon as sufficient bearing strength is developed, tension

the bolt as described in paragraph 3-5 and grout full length as described in paragraph 3-6.

e. Slot and Wedge Bolt Installation. Slot and wedge bolts should be installed according to the manufacturer's recommendations as modified by the results of the field test program at the site with the following supplementary steps:

(1) The hole must be cleaned as discussed in paragraph 3-4b(5).

(2) The rock bolt (with the wedge inserted in the slot) and the hole depth must both be measured to ensure that adequate length for driving and installing bearing plate, washers, and nut remains. For pressure grouting, a long plastic tube extending to the back of the bolt can be taped in position at this time. Tube should be positioned along slot so pinching of tube will not occur as bolt ears are expanded into rock.

(3) The bolt is driven with an air-powered stoper or drifter until there is no further movement of the bolt into the hole. A driving dolly (available from rock bolt manufacturers) must be used to protect the exposed threads and to keep the bolt from rotating. If preferred, the long grout tube can be inserted along bolt to back of hole at this time rather than as described in step 2.

(4) Continue with installation steps 4 through 11 listed in paragraph 3-4d for the expansion shell type.

f. Grouted End Anchorages. The desirability of using grouted end anchorages over mechanical anchorages increases as rock quality decreases. Once the embedment length for a particular rock type or condition is determined, positive anchorage is possible in 100 percent of all installations. However, experience has shown this to be achievable only with close attention to installation detail. Installation methods and techniques for creating grouted end anchorages are very similar to those used for accomplishing full length bonding of tensioned or untensioned elements. Manufacturer's literature and data sheets as well as practices recommended in this manual should be carefully studied and followed. Field tests must be conducted to determine embedment lengths, to determine required set or cure time of the bonding medium before tensioning of the element is attempted, and for establishing procedures to be used during the construction period. Checks on all installation crews should be conducted routinely throughout the entire construction period for conformity to established procedure.

(1) Pumpable grout type. This type is prepared as shown in

figure 3-6. For full length grouting (after the end grout has set and the bolt has been tensioned) an additional long plastic tube beginning at the threaded end and terminating just short of the packer is also taped to the bar. Polyfoam rubber is usually cut to size and fastened to the bar with a narrow band of tape to form the packer. In up-holes, wooden wedges at the hole collar are used to hold the bar until the grout sets up. Except for providing clearance for the grout tubes and making adjustments to the packer thickness, hole size is not critical and can be almost any size larger than the bar diameter. Normally, a hole diameter equal to the bar diameter plus 5/8 inch to 3/4 inch should be used as a minimum. In softer rocks where bond strength is low at the rock-grout interface, the bonding area can be increased by drilling larger rather than longer holes. Procedures and materials for placing the end anchorage grout are covered in paragraph 3-6. Once the end grout is set, steps 4 through 11 in paragraph 3-4d should be followed to complete the installation.

(2) Perforated sleeve and mortar type.

(a) When installing this type, figure 3-7, the drill hole depth should not exceed the depth to which the reinforcing element will extend into the rock. The size relationship between the drill hole, the reinforcement element, and the perforated sleeve is critical and should be such that the combined cross-sectional area of the bar and the sleeve packed with mortar be 10 percent to 15 percent greater than the area of the drill hole. (Sleeves are of 20-gage metal with perforations over approximately one half the surface area.) Poor anchorage strength may result if a smaller volume of excess mortar is used. A greater volume of excess mortar will make driving of the bar difficult through the mortar. A listing follows which shows the size relationships recommended by the sleeve manufacturer for installing standard deformed bars. Other combinations must be based on separate computations.

Deformed Bar Size	Drill Hole Diam, in.	Perforated Sleeve Diam, in.
No. 6	1-1/4	1-1/16
No. 8	1-1/2	1-1/4
No. 9	1-3/4	1-1/2
No. 10	2	1-3/4
No. 11	2-1/4	2

The following portland cement mortar mix is recommended for packing the sleeves when a setup time of two days or more is acceptable.

Portland Cement Mortar for Packing Perforated Sleeves

Type III portland cement (2 sacks)	188 lb
Sand	188 lb
Admixture (CE Specification CRD-C566)[19]	2 lb

Water: Approximately 0.3 water-cement ratio by weight. Sufficient mixing water should be added to produce a mortar with a flow of approximately 85 percent when tested in accordance with CE Specification CRD-C 116.[16] (Note: A good mix is one that will "pack like a snowball," without exuding free water.)

Shorter setup times are possible with the use of portland cement and accelerators, but four hours or more time is usually required before the element can be tensioned depending on the rock temperature. The quantity of accelerator should be determined on the basis of pull tests conducted at hourly intervals on bars embedded in mortar. A mixture of one part Type I portland cement, one part sand, and sufficient water and accelerator (one part Sika-Set, or equal, to five parts water) to produce a mortar with a flow of 85 percent in accordance with CRD-C 116[16] is recommended as a starter mix for testing. Tests should also demonstrate that the mortar will remain plastic for a sufficient length of time (usually 20 minutes or less) to allow driving of the bar through the mortar. With the use of gypsum cement and sufficient water to form a stiff plastic mix, bars can be tensioned after a setup time of approximately thirty minutes. However, the long term stability (years) characteristics have not been completely investigated. Laboratory tests have shown that when submerged in water, gypsum grout did not increase in bond strength over the seven-day strength. After three months, the water-immersed sample exhibited less than one half the bond strength of a sample cured in dry air.

(b) Installation procedure is identical with that shown in figure 4-1 except that a bar with a threaded end is used. The back end of the bar is ground smoothly to a hemispherical shape to facilitate driving through the mortar. Where holes are not drilled straight, the sleeve may bind in the hole and cause premature extrusion of mortar by the bar. To prevent this, small-diameter copper wire can be laced across the mouth of the sleeve to temporarily restrain the bar until the sleeve reaches the back of the hole. If holes are drilled oversize, immediate correction should be made to the drilling procedure. However, compensation for an oversize hole is possible by overfilling the sleeve to form an elliptical shape when the two halves are tied together.

(c) In preparation for subsequent full length grouting of the element, a long plastic tube stopping short of the final embedment should be taped to the bar before driving or the tube may be inserted just prior to tensioning of the element. Once the end anchorage has developed sufficient strength, the installation is completed by following steps 4 through 11 in paragraph 3-4d.

(3) Polyester resin type.

(a) Resins are available in bulk form or packaged in cartridges as shown in table 3-2. The bulk resin is available in a shipping container which also serves as a mixing container for paste, hardener, and activator packed in separate plastic bags. A polyethelene tube and a plunger are used to load the tube and to transfer the resin to a drill hole prior to inserting a deformed bar through the resin to form the end anchorage. Drill hole size is not critical but should be as small as possible to conserve resin. Resin cure time prior to tensioning the bar varies from 1 hour at 90° F to 24 hours at 45° F. Detailed data sheets are available from the manufacturer which show installation details, resin volume requirements for various deformed bar/hole size combinations, resin gelation and curing times required at different temperatures with varying amounts of activator, and storage instructions. These are not repeated here because this type has not gained wide acceptance for use in civil engineering works. However, the resin has been used successfully in a number of installations, particularly in Canadian metal mines. Special safety precautions are necessary for handling the resin, because it is combustible (flash point 150° F) and component vapors or contact may cause skin and eye irritation.

(b) Methods for installing end anchorages with the use of cartridges will pertain primarily to the types shown in figures 3-8 and 3-9 because a wide range of cartridge diameters with various set times is available. However, installation techniques are similar for all cartridge types listed in table 3-2. Since manufacturers are in the process of expanding existing lines of polyester resin products, the latest data sheets should be obtained from suppliers during the design of rock reinforcement. Steps in the installation sequence and other information are given in figures 3-8 and 3-9. As indicated in the figures, holes are also loaded with sufficient resin to bond the element full length as well as to form the end anchorage. Once the bar has been spun to the end of the drill hole, steps 4 through 11 of paragraph 3-4d are followed after eliminating all reference to grout tubes or full length grouting.

(c) For installing the polyester cartridge types, detailed data sheets are available which show the recommended bar size/hole size/cartridge size combinations and the respective unit embedment length.

These relationships are critical not only to assure filling the space around the deformed bar but also for other reasons. Shredding of the cartridge is best assured when hole and cartridge diameters are only slightly larger than the bar diameter. With these size relationships, the best mixing of components is achieved and formation of air pockets is prevented because the resin component particles are forced to translate and interact along the bar length. For achieving a good installation an excess resin quantity of 15 percent is allowed. As a general rule, a successful installation will result if the cartridge and drill hole diameters exceed the nominal bar diameter by approximately 1/4 inch and 3/8 inch respectively, i.e., No. 8 rebar, 1-1/4-inch- (32-mm-) diameter cartridge, 1-3/8-inch-diameter hole. This relationship will result in an embedment length of approximately 20 inches for a standard 12-inch-long cartridge. Correct size relationships as well as embedment yields must be positively determined at the time field tests are conducted to establish embedment lengths required for positive anchorage. Overdrilling of hole depth up to 2 inches is acceptable.

(d) Since the amount of resin required is somewhat sensitive to hole size, a problem may occur when longer bolts requiring a coupling are utilized. The hole enlargement required to accommodate the coupling may make the area to be filled with resin grout uneconomical. To remedy this situation, a cement groutable rock bolt has been developed (Fox Industries) incorporating a pressed-on washer which acts as a resin stop at the point of anchorage (figure 3-10). An integral grout tube then permits the remainder of the bolt to be grouted in the usual manner. In this case, the air bleeder tube is terminated just under the resin stop. Bars incorporating these types of features are available in 1-inch, 1-1/4-inch, and 1-3/8-inch sizes.

(e) Cartridges are available in several viscosities and setting times for use at different temperatures or construction conditions. Standard setting times are 1 minute, 2 to 4 minutes, 5 to 10 minutes, and 15 to 20 minutes. Insertion of bar and mixing of resin must be complete within the range of the set time of resin at the end anchorage. The bar can then be tensioned after 5 minutes but tensioning must be complete before the full length bonding resin sets. Because of the rapid set times, hole straightness is critical for rotating the bar without binding in the hole, particularly in long holes. Driving of the bar must not be allowed. Setting times are also sensitive to temperature and steel must be stored to assure temperature compatibility with the resin formulation and viscosity. Checks must also be established to ensure that resin is used within the limit of its storage life.

3-5. Tensioning Methods.

a. General. There are two methods of tensioning and these methods

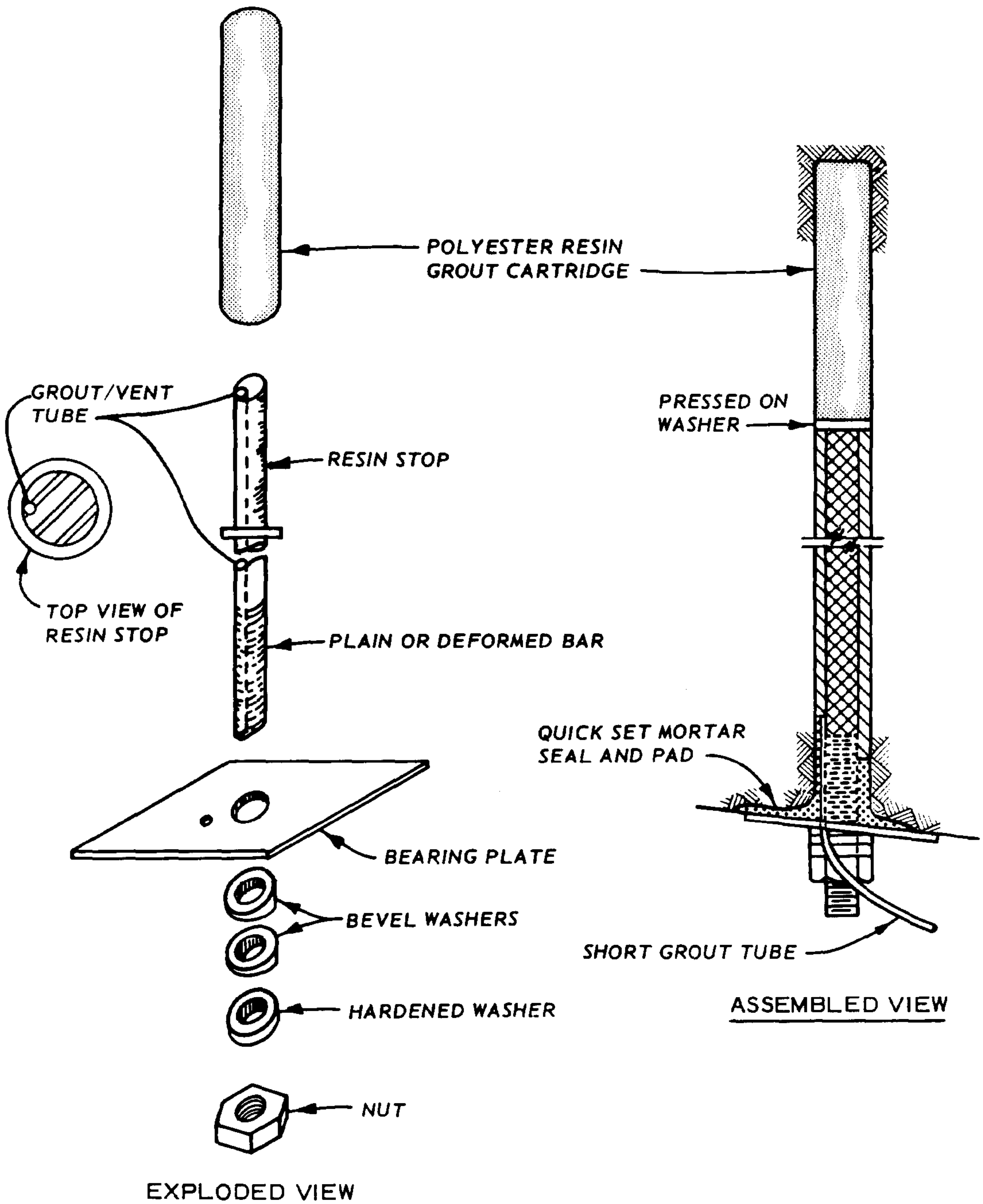


Figure 3-10. Cement groutable rock bolt with resin anchor stop and integral grout tube.

are not dependent on the type of anchorage used or the type of bolt used. These are direct pull tensioning using a hydraulic system and torquing of the nut using a torque wrench. Direct pull tensioning has two advantages over torquing. The first is that torsional stresses do not combine with tensile stresses to reduce the strength of the bar. This advantage is not so great if friction reducing materials, as discussed in paragraph 3-3e, are placed prior to torque tensioning. The other advantage is that direct pull tensioning gives a positive indication of the capacity of the anchorage within the range of the tensioning load for every bolt installed. With either method, a deformeter (hollow core rock bolt modified to act as an instrument for monitoring strain in the steel) should occasionally be installed in place of a rock bolt to check the efficiency of the tensioning method. Improvements in the equipment and hardware generally used by contractors with both tensioning methods would be desirable.

b. Direct Pull Tensioning. Installation of the bearing plate as near normal as possible to the long axis of the bolt is a prerequisite to tensioning with presently available equipment. Direct pull tensioning can be accomplished with commercially available rock bolt pullers which are self-contained and attach directly to the rock bolt or by the use of a bridging device along with a bolt extension and a center hole ram with remote hydraulic pump as schematically indicated in figure 3-11. Any commercially available system should be satisfactory but the system should incorporate an easily read dial gage with a large face calibrated throughout the range of the tensioning load on a scale extending at least over a semicircular arc. Jack systems equipped with preset load indicators which give no indication of load prior to or after the preset load is reached are not recommended. Some load loss can be expected when transferring the load from the jack to the bolt nut. In most cases, a large load loss is traceable to insufficient tightening of the nut and controls are necessary to prevent this condition. Using the dial gage as a control, transfer losses can be greatly minimized by tightening the nut until the jack load is reduced by approximately 15 percent prior to releasing the jack load. An alternative method which may be particularly necessary on larger bolts would be to apply a jacking load some 10 or 15 percent larger than design loading to ensure no load loss when transferring the load. Tests should be conducted with the use of a rock bolt deformeter to determine the efficiency of the load transfer operation in the field. The jack system should also be recalibrated frequently during the course of a project to assure accurately recorded tensioning loads. The seating loss for a jack system can also be determined by a lift-off test. Once this is known, then the bolt can be tensioned to the desired working load plus the load for seating loss.

c. Torquing. Tensioning by torquing the nut is acceptable

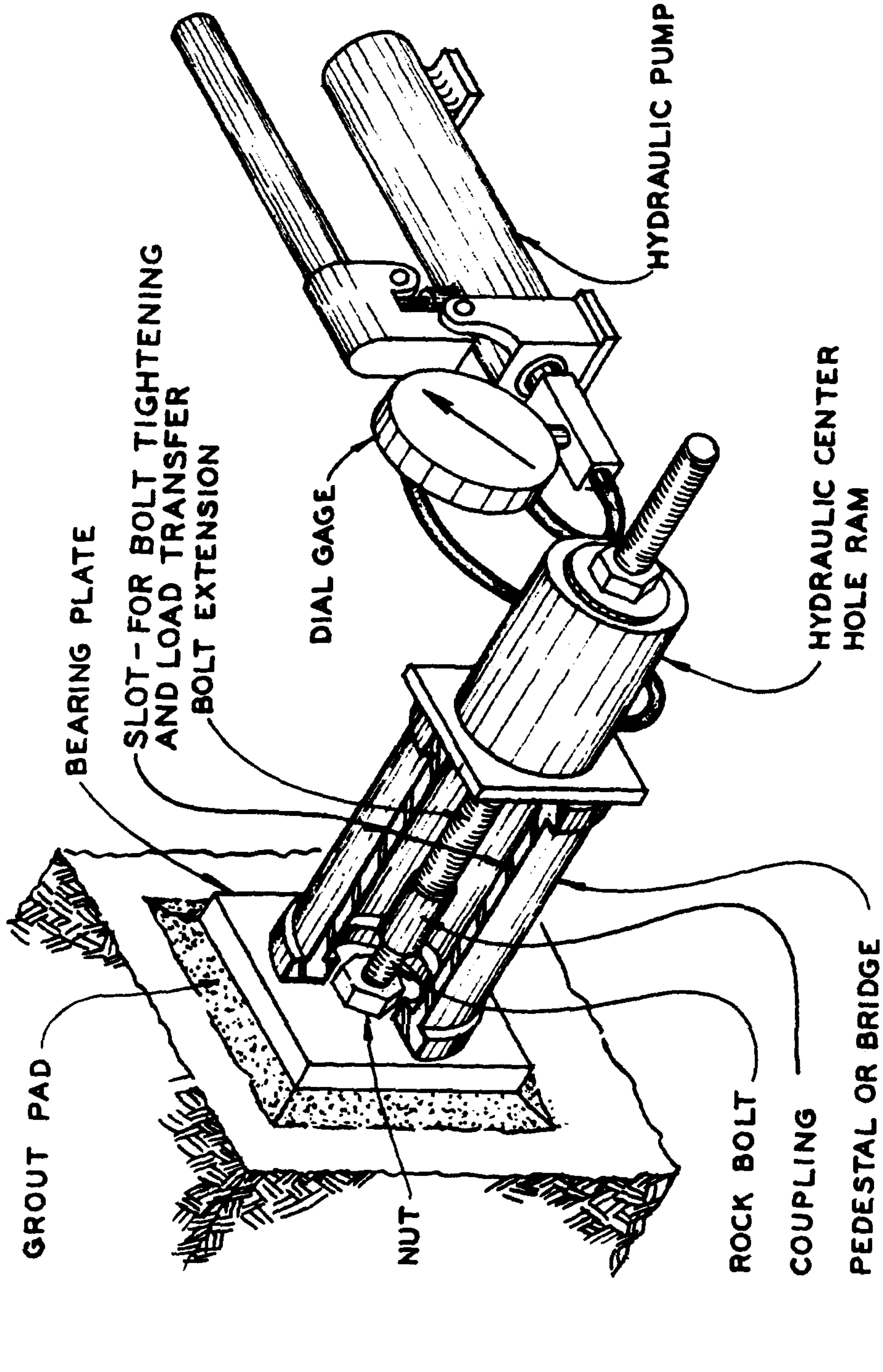


Figure 3-11. Center hole ram for direct tensioning of rock bolts.

particularly when tensioning loads under 30,000 pounds. The installed load is subject to wide variation due to a number of conditions related to the control of alignment and friction between mating parts as well as to size of reinforcing element and undetectable anchor slip. The torque required to produce a specified load is usually expressed empirically as:

$$\text{Bolt tension, lb} = C \times \text{torque, ft-lb}$$

Although C may be defined within narrower limits under controlled laboratory conditions, C can be expected to range from a value of 40 to a value of 80 under actual field conditions, excluding anchor slip and provided installation techniques presented in this chapter are specified. Torquing is usually accomplished with a calibrated air-driven impact wrench. For reliability of results, these required frequent recalibration and a great deal of maintenance. The wrench output is also subject to variations in air line pressure. In all cases, a hand torque wrench must be used as a check on the powered torque wrench. With the hand torque wrench, the nut should be in motion at the time the reading is made. Hand torque wrenches also need to be recalibrated periodically. The manufacturer of a given bolt system will usually provide recommended C-values for his particular product.

d. Retensioning. Rock bolts should be tensioned and grouted full length prior to continuing blast operations near the bolt installations. If nearby blasting is permitted before grouting, retensioning becomes necessary just before grouting to eliminate any load losses caused by the blasting vibrations. In any event, bolts should be grouted as soon as possible and normally should not be left ungrouted for more than one day.

3-6. Grouting of Reinforcing Elements. Installation methods and techniques for creating grouted end anchorages or for accomplishing full length grouting after tensioning of elements are very similar. In both cases, successful installations will result by following recommended procedures with close attention to all of the installation details. Manufacturer's literature and data sheets should also be consulted.

a. Pumpable Liquid Grout. Liquid grout is usually used for the anchorage shown in figure 3-7 and for full length bonding of the types shown in figures 3-1 through 3-7. Grout should always be injected at the lowest point in the hole so that air will escape as the grout level builds up. With hollow core bolts installed in up-holes, the hollow core is used for venting and a short length of plastic tube is grouted in the collar of the hole for injecting the grout. The process is reversed for down-holes. For the type in figure 3-6, a similar procedure is followed for creating the end anchorage except that the long tube substitutes for the hollow core. For full length grouting of the types shown in figures 3-6 and 3-7, a long tube terminating at the anchorage must be taped in place prior to placing the reinforcing element and a short tube installed at the collar. The same two-tube technique is used for full length grouting of types shown in figures 3-1 through 3-3. With these types, the long tube is best placed after the bolt is anchored to avoid crimping the tube as the bolt is spun. A long thin removable rigid rod or tube inserted in the plastic tube will simplify the placement. This method is particularly useful when it becomes necessary to upgrade older installations where ungrouted bolts were installed.

(1) Portland cement grout. Although various bonding materials are possible, the most common is neat portland cement grout. The following mix is recommended for forming grouted end anchorages where a fast setup time (less than 2 days) is not required or for full length grouting of elements previously tensioned.

Type III portland cement (2 sacks)	188 lb
Flyash (optional) - check for possible alkali reactivity	75 lb
Admixture (CE Specification CRD-C566)[19] Grout Fluidifier and Expanding Agent	2.6 lb
Water: Approximately 0.4 water-cement by weight. Quantity of water sufficient to produce a grout efflux time of 20 seconds when tested in accordance with CE Specification CRD-C79-58.[15]	

For pumping neat cement grout, all grout pipes, tubes, and fittings should be free of dirt, grease, hardened grout, or other contamination before grouting commences. All wash water and diluted grout should be flushed from the lines. The grout line should be attached to the injection point such that leakage is entirely prevented. As a general rule, grout pressure at the collar should not exceed 25 psi and grouting should be continued until there is a full return through the vent. To avoid clogging the tubes, the grout must be passed through a No. 8 screen prior to injection because the cement will sometimes "ball-up" to form obstructions. If, during grouting, grout leaks to the surface through open joints and appreciable grout is being lost, the grouting operation should be temporarily suspended and the joints caulked with quick-setting mortar or other caulking material. If, during grouting of any bolt, the hole accepts more grout than required to fill the nominal volume of the annular space without return of grout through the vent tube and if no leakage is visible at the surface, then grouting operations should be temporarily suspended, the grout line disconnected, and the bolt hole allowed to drain. No earlier than one hour and not later than two hours after suspension of grouting, the grout lines should be reconnected and grouting completed. If excess leakage still occurs, sand should be added to the grout to stiffen it. When grout flows in a steady stream from the vent tube, the vent tube should be plugged (with a golf tee or other plug) while pressure is maintained on the injection tube. The grout line should then be removed and the injection tube plugged.

(2) Polymer and gypsum grout. Epoxy or polyester grout may be substituted for neat cement grout for pumping around the reinforcing element. These have not been used extensively because specialized equipment is necessary which has not been fully developed for economical operation. Gypsum grout can also be pumped using one part gypsum to three to five parts water. However, as mentioned previously, the long term stability of gypsum has not been completely investigated.

b. Packaged Polyester Grout. Installation techniques for accomplishing full length bonding with cartridges are covered in paragraph 3-4f(3).

3-7. Anchorage Capability.

a. As emphasized throughout this manual, determination of anchorage capability in a particular rock type must finally be determined by conducting field pull tests at the site. Figure 3-12 illustrates equipment commonly used to perform a pull test and monitor the behavior of a system as the load is applied. When making direct pull tests of rock bolt it is recommended in underground and bedded formations that safety props be installed to protect the man conducting the test.

b. For making the initial choice of anchorage (prior to the time a site is available for making tests), past experience is useful for selecting alternate systems or for making preliminary estimates of mechanical or grouted end anchorage capabilities. Manufacturer's data sheets also provide some guidance.

c. Table 3-3 summarizes results of anchorage tests conducted at various projects. Work in this area is only partially complete and additional information needs to be gathered and summarized.

3-8. Quality Control. The quality control program for a rock reinforcement system should begin with a test installation at the project

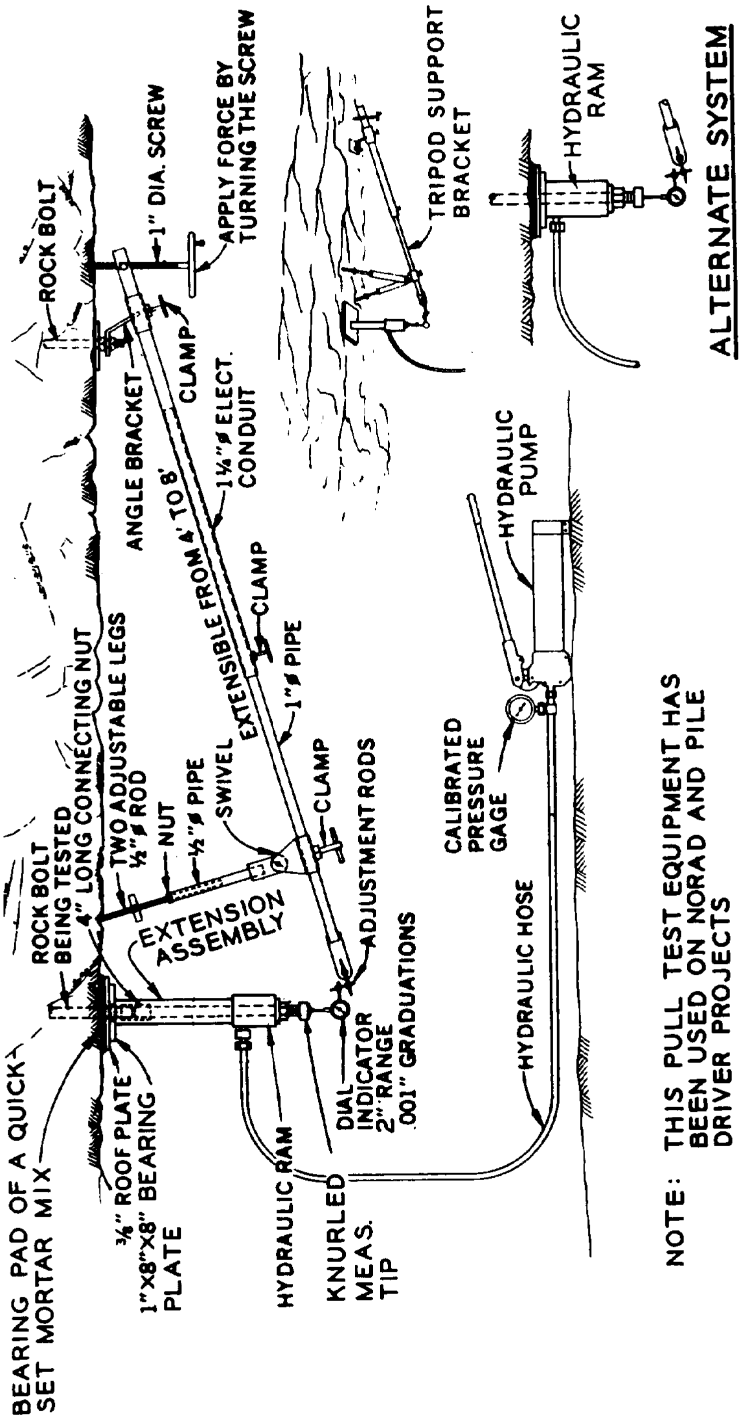


Figure 3-12. Pull test equipment.

Table 3-3. Summary Sheet - Rock Bolt Pull Tests

Project Location	Testing Agency	Rock Type and Description	General Comments
Nevada Test Site	Omaha District, Corps of Engineers	Quartz Monzonite, light gray, dense, porphyritic with a fine- to medium-grained ground mass. G_s = 2.67; q_u = 30,500 psi and tensile strength = 1,450 psi. Rock was competent for all but 3 bolts tested.	A quick setting mortar pad was used under all bearing plates. No. 8 and No. 11 bolts were hollow-core. Testing was performed in 1965.

Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	Failure Load, kips Anchor Failure	Failure Load, kips Bolt Failure	Remarks
14 Smooth		170	6	Williams Shell	750	None	--	--	--	--	--	--	--	170	
14 Smooth		170	6	Williams Shell	750	None	--	--	--	--	--	--	--	175	
14 Smooth		170	6	Williams Shell	750	Neat Cement					4	34 days	--	160+	Loading discontinued. No failure
14 Smooth		170	6	Williams Shell	0	Neat Cement					4.5	34 days	--	160+	Loading discontinued. No failure
11 Deformed	71	118	16	Williams Shell	400	None	--	--	--	--	--	--	72	--	Manufacturer's rating is 74 kips at yield and 100 kips at ultimate. Lab tests showed 71 and 118 kips, respectively, through bolt and 59 and 98 through threads
11 Deformed	71	118	16	Williams Shell	400	None	--	--	--	--	--	--	84	--	
11 Deformed	71	118	16	Williams Shell	400	None	--	--	--	--	--	--	--	62	
11 Deformed	71	118	6	Williams Shell	400	None	--	--	--	--	--	--	36	--	
11 Deformed	71	118	6	Williams Shell	400	None	--	--	--	--	--	--	82	--	
11 Deformed	71	118	6	Williams Shell	400	None	--	--	--	--	--	--	76	--	
11 Deformed	71	118	6	None	--	Cem-Sand & Perfo Shell	1:1	0.29 Max.			4	28 days	--	96	Grout mix was 188 lb of cement, 188 lb sand, 56 lb water (max), and 2 lb fluidifier
11 Deformed	71	118	6	None	--	Cem-Sand & Perfo Shell	1:1	0.29			3.66+	28 days	--	92	
11 Deformed	71	118	8	None	--	Cem-Sand & Perfo Shell	1:1	0.29			7.75	28 days	--	92	
11 Deformed	71	118	6	None	--	Gypsum (S-1) Perfo					4	1.1 hr	--	90	
11 Deformed	71	118	6	None	--	Gypsum (S-1) Perfo					3	2 hr	--	94	
11 Deformed	71	118	6	None	--	Gypsum (S-1) Perfo					2	2 hr	--	92	
11 Deformed	71	118	6	None	--	Gypsum (S-1) Perfo					1	3 hr	84		
11 Deformed	71	118	8	Williams Shell	0	Neat Cement		0.4	10:36	23:00	5	28 days	--	96	Grout mix was 188 lb of Type III cement, 75 lb flyash, and 2.6 lb fluidifier. Water was added to produce efflux time of 20 sec
11 Deformed	71	118	16	Williams Shell	0	Neat Cement		0.4	10:36	23:00	4	28 days	--	92	
8 Deformed	38	59	8	Williams Shell	0	Gypsum (S-1) (pumpable)					6.5	1.5 hr	--	40	

(Continued)

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Table 3-3. (Continued)

Project Location			Testing Agency			Rock Type and Description					General Comments					
Nevada Test Site (Continued)											Test equipment included 10,000-psi capacity hydraulic land pump, 60-ton and 100-ton center pull rams, extension assembly, and support bracket for dial gage used to measure bolt displacement during testing.					
Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	Failure Load, kips Anchor Failure	Failure Load, kips Bolt Failure	Remarks
11 Deformed	71	118	8	Williams Shell	0	Neat Cement		0.4	10:36	23:00	4	28 days	--	90	
11 Deformed	71	118	8	Williams Shell	0	Neat Cement		0.4	10:36	23:00	4	28 days	--	92	
11 Deformed	71	118	6	Williams Shell	0	Neat Cement		0.4	10:36	23:00	3	28 days	--	90	
11 Deformed	71	118	16	Williams Shell	0	Epoxy					5	30 days	--	84	A packer was used to control grouted length and grout was pumped in
11 Deformed	71	118	8	Williams Shell	0	Epoxy					5	30 days	--	80+	Loading discontinued. No failure
11 Deformed	71	118	8	Williams Shell	0	Epoxy					4	30 days	--	90	
11 Deformed	71	118	8	Williams Shell	0	Epoxy					3	30 days	--	94	
11 Deformed	71	118	8	Williams Shell	0	Epoxy					1.5	30 days	50	--	
11 Deformed	71	118	8	None	--	Polyester (P-1)					4	7 days	--	92	Resin inserted in back of hole and bolt is driven through it. A transfer tube is used to place the resin
11 Deformed	71	118	8	None	--	Polyester (P-1)					3	7 days	40	--	
11 Deformed	71	118	8	Williams Shell	0	Polyester (P-1)					3	7 days	25	0	
11 Deformed	71	118	8	Williams Shell	0	--					2	7 days	1	--	
11 Deformed	71	118	8	None	--	Polyester (P-2)					4	7 days	--	100	P-2 polyester is furnished in sausage-shaped plastic bags which are placed at back of hole. Bolt is driven through and rotated to mix
11 Deformed	71	118	6	None	--	Polyester (P-2)					3	8 days	65	--	
11 Deformed	71	118	8	None	--	Polyester (P-2)					2	7 days	32	--	
8 Deformed	38	59	6	Williams Shell	250	None					--	--	38	--	Manufacturer rates yield at 37 kips and ult at 50^k. Lab tests indicated 38 and 59 for bolt and 28 and 44^k through threads. Badly fractured rock
8 Deformed	38	59	6	Williams Shell	250	None					--	--	14	--	Rock was badly fractured
8 Deformed	38	59	6	Williams Shell	250	None					--	--	2	--	Rock was badly fractured
8 Deformed	38	59	8	Williams Shell	0	Gypsum (S-1) pumpable					6.5	2 hr	30	--	

(Continued)

(Sheet 2 of 9)

Table 3-3. (Continued)

Project Location	Testing Agency	Rock Type and Description	General Comments
Nevada Test Site (Continued)			For additional information about this series of tests, refer to "Investigation of Rock Bolt Anchorages in Quartz Monzonite Rock," pile-driver project, Technical Report No. 1, July 1965, Omaha District.

Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	Failure Load, kips Anchor Failure	Failure Load, kips Bolt Failure	Remarks
8 Deformed	38	59	8	Williams Shell	250	Gypsum (S-1) pumpable					6.5	4 hr	--	40	
8 Deformed	38	59	8	Williams Shell Xtra-Long	250	None	--	--	--	--	--	--	0	--	Improperly installed
7 Deformed			4	C.F.I. Shell	200	None								44	
7 Deformed			4	C.F.I. Shell	200	None							8		
7 Deformed			4	C.F.I. Shell	200	None							4		Improperly installed. Hole drilled 1/8 in. oversize
7 Deformed			4	C.F.I. Shell	200	None								40	
7 Deformed			4	Williams Shell	200	None							8		
7 Deformed			4	Williams Shell	200	None							10		
7 Deformed			4	Williams Shell	200	None							6		
7 Deformed			6	None		Gypsum (S-1) Perfo					4	1 hr	37		
7 Deformed			6	None		Gypsum (S-1) Perfo					4	2 hr		40	

Key

Table Name	Manufacturer's Name	Manufacturer
Epoxy	Epoxy, Formulation 62E2	George W. Whitesides Co. Louisville, Ky.
Polyester (P-1) Polyester (P-2)	ROC-LOC 60 ROC-LOC 20	American Cyanamide Co. Stanford, Conn.
Gypsum (S-1)	F-181 Bolt Anchor Sulfaset	Ranco Industrial Products Corp., Cleveland, Ohio
Rock Bolts	Hollow Core Rock Bolts	Williams Form Eng. Co. Grand Rapids, Mich.
Perfo	Perfo Sleeves	Sika Chemical Co. Passaic, N. J.

(Continued)

(Sheet 3 of 9)

Table 3-3. (Continued)

Project Location			Testing Agency			Rock Type and Description					General Comments				
Stockton Dam, Missouri			Kansas City District Corps of Engineers			Limestone					All bolts tested in this series of tests were Williams High Strength Hollow Groutable, Type US-H-HC. The bolt has a nongrouted rating of 47 kips, maximum working load of 74 kips, and ultimate of 100 kips.				
Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	Failure Load, kips Anchor Failure	Failure Load, kips Bolt Failure	Remarks
11	74	100	9.4	None	--	Sand-Cement (Perfo)					4	3 days	55 (no failure)	--	Grout for perfo shells was 1 part sand, 2 parts Type I cement and enough solution of 3 parts water to 1 part Sika-Set to make mortar workable
11	74	100	10	None	--	Sand-Cement (Perfo)					4	6 hr 7 min	70.3	--	No failure but spreading of threads froze the nut
11	74	100	10	None	--	Sand-Cement (Perfo)					4	7 hr 5 min	70.3	--	No failure but spreading of threads froze the nut
11	74	100	10	None	--	Sand-Cement (Perfo)					4	7 hr 19 min	73.1	--	No failure but spreading of threads froze the nut
11	74	100	10	None	--	Sand-Cement (Perfo)					4	23 hr 10 min	73.8	--	No failure but spreading of threads froze the nut
11	74	100	10	None	--	Sand-Cement (Perfo)					2	18 hr	56.4	--	No failure and no displacement
11	74	100	10	None	--	Sand-Cement (Perfo)					3.8	6 hr	56.4	--	No failure and no displacement
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	300	None					--	--	51	--	No failure. Testing stopped at 0.78 in. displacement
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	300	None					--	--	51	--	No failure. Testing stopped when displacement reached 0.95 in.
11	74	100	10	Williams Short (1-5/8 in.) Cone Type	300	None					--	--	52.5	--	No failure. Displacement reached 0.8 in.
11	74	100	10	Williams Short (1-5/8 in.) Cone Type	800	None					--	--	82	--	No failure. Threads were spread out at 82 kips. Displacement was 1.4 in.
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	700	None					--	--	45.2	--	No failure. Displacement was 0.55 in. when testing discontinued
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	700	None					--	--	39	--	Complete failure 30 sec after reaching 39 kips
11	74	100	10	Williams Short (1-5/8 in.) Cone Type	700	None					--	--	36	--	Complete failure in 45 sec

(Continued)

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Table 3-3. (Continued)

Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	Failure Load, kips Anchor Failure	Failure Load, kips Bolt Failure	Remarks
Project Location: Stockton Dam, Missouri (Continued)															
11	74	100	10	Williams Short (1-5/8 in.) Cone Type	700	None					--	--	41	--	No failure
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	500	None					--	--	41	--	Displacement exceeded 1.6 in.
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	500	None					--	--	46.7	--	No failure. Displacement was 0.9 in.
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	300	None					--	--	46.7		
11	74	100	10	Williams Short (1-5/8 in.) Cone Type	300	None							41.2	--	No failure. 0.7-in. displacement
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	300	None							41.2	--	No failure. 1.7-in. displacement

(Continued)

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Table 3-3. (Continued)

Project Location			Testing Agency			Rock Type and Description					General Comments				
Stockton Dam, Missouri (Continued)											Reference for this series of tests: "Rock Bolt Anchor Design," Supplemental Design Memorandum No. 7B, Stockton Dam and Reservoir, Kansas City District, Corps of Engineers, February 1966.				
													Failure Load, kips		
Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	Anchor Failure	Bolt Failure	Remarks
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	300	None							50	--	No failure. Early stress loss
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	300	None							40	--	Complete failure in 3 min
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	300	None							50	--	No failure
11	74	100	10	Williams Short (1-5/8 in.) Cone Type	300	None							28.7	--	Complete failure in 1-1/2 min
11	74	100	10	Williams Short (1-5/8 in.) Cone Type	300	None							50	--	No failure
11	74	100	10	Williams Short (1-5/8 in.) Cone Type	300	None							46.7	--	No failure
11	74	100	11	Williams Long (3 in.) Cone Type	500	None							55	--	No failure
11	74	100	11	Williams Long (3 in.) Cone Type	880	None							69	--	No failure
11	74	100	11	Williams Long (3 in.) Cone Type	800	None							59.2	--	No failure
11	74	100	11	Williams Long (3 in.) Cone Type	800	None							59.2	--	No failure
11	74	100	21	Williams Long (3 in.) Cone Type	800	None							59.2	--	No failure
11	74	100	21	Williams Long (3 in.) Cone Type	560	None							45.2	--	No failure

(Continued)

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Table 3-3. (Continued)

Project Location		Testing Agency				Rock Type and Description					General Comments				
Nevada Test Site		Fenix and Scisson, Inc.				Alternating layers of red and white tuff. The tuff is commonly pumiceons and zeolitized.					This test program was conducted at NTS by Fenix and Scisson in cooperation with Reynolds Electrical Engineering Company.				
													Failure Load, kips		
Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	Anchor Failure	Bolt Failure	Remarks
1-in.-diam		83	16	None	--	Sulfaset (pumped)					7		--	83 (coupling failure)	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	16	None	--	Sulfaset (pumped)					7		--	74 (thread failure)	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	16	None	--	Sulfaset (pumped)					7		10 (poorly grouted)	--	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	32	None	--	Sulfaset (pumped)					7		0 (poorly grouted)	--	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	32	None	--	Sulfaset (pumped)					7		53 (poorly grouted)	--	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	32	None	--	Sulfaset (pumped)					7		--	82 (thread failure)	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	32	None	--	Sulfaset (pumped)					7			73 (coupling failure)	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	32	None	--	Sulfaset (pumped)					7		--	57 (coupling failure)	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	32	None	--	Sulfaset (pumped)					7		--	63 (no failure)	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	40	None	--	Sulfaset (pumped)					7		10 (poorly grouted)	--	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	40	None	--	Sulfaset (pumped)					7		--	72 (coupling failure)	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	12	None	--	Sulfaset in perfo sleeve					1		0	--	Bolt was smooth, A431 steel
1-in.-diam		83	12	None	--	Sulfaset in perfo sleeve					5		10	--	Bolt was smooth, A431 steel
8		72	12	None	--	Sulfaset in perfo sleeve					2		53	--	Bolt was No. 8 rebar, A431, NC threads
8		72	12	None	--	Sulfaset in perfo sleeve					4		--	72	Bolt was No. 8 rebar, A431, NC threads
9	80	96	16	None	--	Sulfaset (pumped)					7		--	94	Bolt was No. 9 rebar, A431, NC threads

(Continued)

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Table 3-3. (Continued)

Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	Failure Load, kips: Anchor Failure	Failure Load, kips: Bolt Failure	Remarks
Project Location: Nevada Test Site (Continued)			Testing Agency			Rock Type and Description					General Comments				
9	80	96	24	None	--	Sulfaset (pumped)					5		10	--	Bolt was No. 9 rebar, A431, NC threads
9	80	96	24	None	--	Sulfaset (pumped)					5		81	--	Bolt was No. 9 rebar, A431, NC threads
9	80	96	32	None	--	Sulfaset (pumped)					7		63	--	Bolt was No. 9 rebar, A431, NC threads
9	80	96	32	None	--	Sulfaset (pumped)					7		87	--	Bolt was No. 9 rebar, A431, NC threads

(Continued)

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Table 3-3. (Concluded)

Project Location			Testing Agency			Rock Type and Description					General Comments				
Nevada Test Site (Continued)											Results of tests are from a report by E. J. Cording, F. D. Patton, and D. U. Deere, "Rock Bolt Tests in U12g Tunnel, Nevada Test Site," Fenix and Scisson, Inc., July 1965.				
													Failure Load, kips		
Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	Anchor Failure	Bolt Failure	Remarks
9	80	96	32	None	--	Sulfaset (pumped)					9		--	96	Bolt was No. 9 rebar, A431, NC threads
11	57	70	12	None	--	Sulfaset (pumped)					7		--	69	Bolt was No. 11 rebar, A431, NF threads (1-1/8 in.)
11	57	70	12	None	--	Sulfaset (Perfo)					4		--	72	Bolt was No. 11 rebar, A431, NF threads (1-1/8 in.)
11	57	70	12	None	--	Sulfaset (Perfo)					5		35	--	Bolt was No. 11 rebar, A431, NF threads (1-1/8 in.)

(Sheet 9 of 9)

site before the contract is advertised. The purpose of this initial program is to familiarize field personnel with installation procedures and to assure the designers that the system selected meets design criteria. Another test program should be included as part of the contract. The purpose of this test is to train contractor's personnel in correct installation procedures and impress upon them the importance of attention to details. If direct pull tensioning is used during construction, the anchorage of every bolt is verified. If torquing is specified for tensioning, a small percentage (0.5 percent is customary) of the elements should be selected at random and tensioned by direct pull as a check on the anchorage. Immediate full length grouting of elements will eliminate requirements for checking for tension loss. Otherwise periodic checks with the use of a torque wrench will be required until grouting is completed. Special rock bolt load cells are available for measuring stress in bolts over lengthy periods and for measuring restrained relaxation of rock strata. During grouting, an inspector should be detailed to observe and ensure that specified procedures are followed and that all elements are fully grouted.

3-9. Specifications. Samples embodying most of the concepts and procedures recommended in this manual are included as Appendices B and C. Some minor revisions or additions to these are anticipated.

3-10. Sample Problem to Demonstrate a Method of Underground Rock Anchor Reinforcement.

a. Background.

(1) Despite extensive theoretical and model studies of the behavior of a single bolt or anchor when embedded in rock or of the behavior of a system of such reinforcing elements, the practical design of rock reinforcement continues to involve application of empirical rules, tempered by the experience, judgment, and observations of the designer. Studies show that when an underground opening is made in rock, deformation occurs in the near vicinity of the opening creating a loosened zone with compressive stresses being concentrated further back within the rock mass creating a supportive ground arch. Timely and proper installation of rock reinforcement restrains the loosened rock and prevents further loosening at greater distances that could lead to local or general fallout of rock. The depth of the loosened zone depends on several factors such as geologic conditions, size of opening with respect to spacing of rock joints, shape of opening, orientation of opening with respect to orientation of rock joints, groundwater conditions, construction procedure, and the engineering properties of the rock mass, particularly the rock mass strength. Although these factors

are important to varying degrees, current Corps of Engineers guidance (EM 1110-2-2901[3]) emphasizes the size of the rock block to be supported, the size of the opening, and overburden rock load pressure.

(2) Although progress is being made in more completely understanding rock reinforcement/rock interactive behavior, it is unlikely that the empirical approach will ever be completely replaced. The inability to predict such important factors as geologic conditions and engineering properties of the rock mass will undoubtedly foster the continued use of empirical rules which, in turn, will require the designer to exercise considerable judgment. This requirement does not particularly detract from the use of rock reinforcements for underground support, especially when consideration is given to the adaptability of the reinforcement to meet unforeseen rock conditions or special construction procedures.

(3) The following example illustrates the application of current CE guidance in the design of rock reinforcement of an underground opening by the use of tensioned rock anchors.

b. Description of Problem. A 10-foot-diameter circular tunnel is to be constructed in a jointed granitic rock formation. Geologic investigation has shown the rock mass to contain two closely spaced conjugate joint sets (figure 3-13). One joint set has an average joint spacing of 12 inches, while the intersecting joint set has an average joint spacing of 18 inches; the strike of the joint set parallels the tunnel axis. From experience or observation it is assumed that the depth of the loosened zone will be on the order of 2 feet.

c. Analysis.

(1) Table 3-4 lists the current design guidance and shows the stepwise procedure to determine minimum length, maximum spacing, and minimum average confining pressure for rock reinforcement. The spacing of the intersecting joint sets and their resulting orientation with respect to the tunnel axis has been determined to yield critical and potentially unstable blocks in the crown of the tunnel (figure 3-13) with dimensions of approximately 2 feet. From the assumed behavior of the rock, no significant loosening is anticipated below the spring line. Therefore, the empirical rules are not used to determine required rock reinforcement below the spring line. In weaker rocks some loosening could occur and support, as determined in the field, might be required.

(2) In trial 1, the minimum length of rock reinforcement is

Table 3-4. Determination of Minimum Length, Maximum Spacing, and Minimum Average Confining Pressure for Rock Reinforcement (Reference Tables 3-7 and 3-8; EM 1110-2-2901[3])

Parameter • Empirical Rules	Trial 1	Trial 2
Minimum length greatest of:		
a. Two times the bolt spacing	Undetermined	2 × 3 = 6 ft ← Use
b. Three times the width of critical and potentially unstable blocks	3 × 2 = 6 ft	As before
c. For elements above the spring line:		
1. Spans less than 20 ft; one-half span	0.5 × 10 = 5 ft	As before
2. Spans from 20 to 60 ft; interpolate between 10- to 15-ft lengths, respectively	NA	As before
3. Spans from 60 to 100 ft; one-fourth span	NA	As before
d. For elements below the spring line:		
1. For openings less than 60 ft high, use lengths determined in c. (above)	NR	As before
2. For openings greater than 60 ft high; one-fifth the height	NR	As before
Maximum spacing least of:		
a. One-half the bolt length	0.5 × 6 = 3 ft	As before ← Use
b. One-and-one-half times the width of critical and potentially unstable rock blocks	1-1/2 × 2 = 3 ft	As before
c. Six feet	6 ft	As before
Minimum average confining pressure at yield point of elements greatest of:		
a. For elements above spring line:		
1. Pressure equal to a vertical rock load of 0.20 times width of opening	0.2 × 10 × 170/144 = 2.4 psi	As before
2. Six pounds per square inch	6 psi	As before ← Use
b. For elements below spring line:		
1. Pressure equal to a vertical rock load of 0.10 times the opening height	NR	As before
2. Six pounds per square inch	NR	As before
c. For elements at intersections, twice the greatest confining pressure determined in a. or b. (above)	NR	As before

Note: NA, not applicable.
NR, not required.

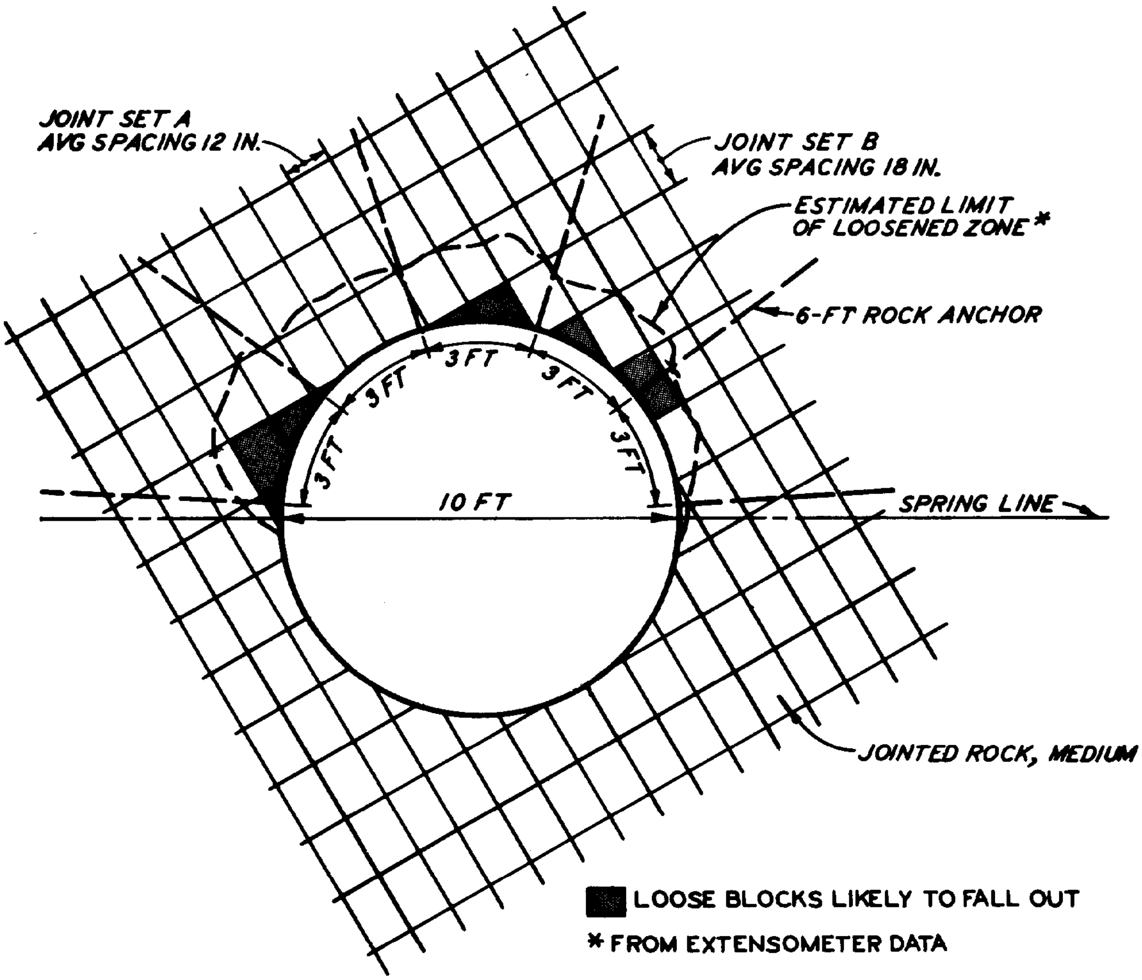


Figure 3-13. Example of underground reinforcement.

tentatively shown to be 6 feet with a maximum spacing of 3 feet. Assuming the unit weight of granite to be 170 pounds per cubic foot (pcf), then as shown in table 3-4, the minimum confining pressure at yield point of the reinforcing elements should be 6 pounds per square inch (psi). The final selection of parameters is determined from trial 2.

(3) From the selected parameters listed in table 3-4, the tension in each bolt should be sufficient to cause an average confining pressure on the tunnel surface of 6 psi. Since the selected anchor spacing is 3 feet on centers, each anchor must support 3 × 3 = 9 square feet (1296 square inches). Therefore, each anchor must be stressed to a minimum load of 1296 × 6 = 7800 pounds. Since any anchor size selected can easily be tensioned to several times this minimum load and still be below the yield strength, the tensile force in the rock anchor is not a critical factor in the anchor selection. Guidance is given in EM 1110-2-2901;[3] tables 3-5 and 3-6, on commercially available rock bolts with mechanical anchorage and grouted reinforcing elements.

3-11. <u>Sample Problem to Demonstrate a Method of Surface Slope Reinforcement by Rock Anchors</u>.

a. <u>Description of Problem</u>. Figure 3-14 shows a planned intake channel for pumped storage units. The 1000-foot-long channel will be blasted in sedimentary rock with bedding planes dipping from right to

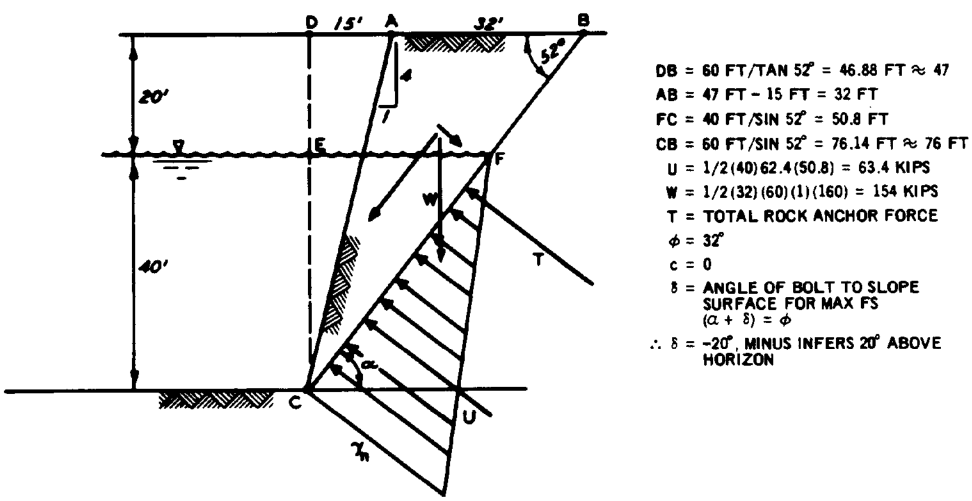


Figure 3-14. Example problem (planned intake channel for pumped storage units).

left at 52 degrees as shown. Presplit side slopes at 1H on 4V are planned to minimize rock breakage on the surface and to minimize disturbance at depth along weak bedding planes. From a cursory inspection it appears the right bank may be unstable since bedding planes dipping into the excavation daylight along the right bank. Accordingly, a rock bolt stabilization scheme will be designed to reinforce the dipping strata in the right bank cut.

b. Analysis.

(1) From inspection it is obvious that the bedding plane which intersects the toe of the slope (BC, figure 3-14) is the most critical when considering the stability of the right bank. Therefore, it is proposed to design a rock bolt system to overcome the forces tending to promote sliding on BC or outward movement from hydrostatic pressure. The 40-foot-deep pool shown can be raised or lowered in a matter of hours due to fluctuating peak power demands. Because of characteristically low joint and bedding plane permeabilities, it is assumed that full pool hydrostatic pressure will exist in both banks during rapid pool drawdowns.

(2) From an inspection of the sketch and from calculations, it is obvious that excavating the intake channel to bottom grade without incremental rock bolt reinforcement would result in a natural slope adjustment at least from AC to BC during blasting on the right bank. For this reason, it is necessary to excavate the channel in lifts, utilizing full rock bolt reinforcement on the cut slopes at each excavation stage. If no adjacent structures are located critically near the right bank, the economics of cutting the right bank channel parallel to the natural bedding planes (BC in sketch) should be considered. In this case, the rock bolt reinforcing could be substantially reduced. However, in this example problem some 36,000 cubic yards of additional rock excavation would be required. Since the bedding planes in the left bank dip advantageously into the cut slope, this bank can probably be stabilized by using only short rock bolts intended mainly to prevent surface ravelling.

(3) Because the intake channel leads directly to the forebay of power generating units, it is often prudent (for erosion control or hydraulic requirements) to provide permanent protection to the rock surfaces in the sides and bottom of the cut. For these cases, either a thin dowelled concrete facing or a wire-mesh-shotcrete design might be specified, each constructed with appropriate drain relief holes drilled through the concrete and into rock. Where extensive ravelling of the slope is anticipated or the slope height needs to be reduced

for stability, benching may be required. Bench width, also calculated for stability to prevent subtoe or failure beneath the bench, should be adequate for access by maintenance equipment, with the bench height determined from stability calculations.

(4) The following procedures should generally be followed to determine the rock bolt stabilization scheme:

(a) Calculate factor of safety (FS) of the unreinforced slope using equation 1 below to determine the need for rock bolts. The FS should be calculated for both the wet and dry conditions.

(b) Decide upon the minimum FS necessary to insure slope stability. Generally, a lower FS can be tolerated for temporary slopes or for slopes that are continuously monitored by instrumentation. Note that the contribution of the rock bolt shear strength to the slope stability is ignored in determining the FS.

(c) Calculate the total rock bolt force T (equation 2) required for a given FS to stabilize a 1-foot length of the cut. Decide upon bolt strength.

(d) From T determine rock bolt spacing.

(e) The rock bolt lengths should, in general, be determined such that when using the design bolt spacing, the force cone generated from the bottom of the anchor overlaps the cone from adjacent anchors when projected to the discontinuity surface. In the example problem however, the geometry is such that this is not a practical approach. Past experience and a good knowledge of the slope geology are used to determine bolt length. The bolt should penetrate the plane of the critical discontinuity to a depth that insures anchorage in sound rock. It is usually prudent to stagger the bolt depths so as to avoid defining a weak plane at the anchor tips. When using resin-type grouts for bolt anchors, the manufacturer's recommendations for length of resin column should be followed. This, in part, helps determine the length of bolt. When deciding upon an anchor system, pull-out tests are advisable to verify anchorage capacity. The following equation[28] should be used in calculating the FS:

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

where

W = weight of rock

δ = angle of rock bolt with horizontal; bolt sloping up above horizontal is of a negative angle

α = angle of discontinuity to the horizontal

ε = angle of resultant force due to earthquake or blasting forces and W

c = cohesion of rock

H = vertical distance from point where discontinuity daylights on slope to top of slope

U = groundwater force

T = total force due to rock anchors

The force polygon for earthquake or blasting force is shown in figure 3-15.

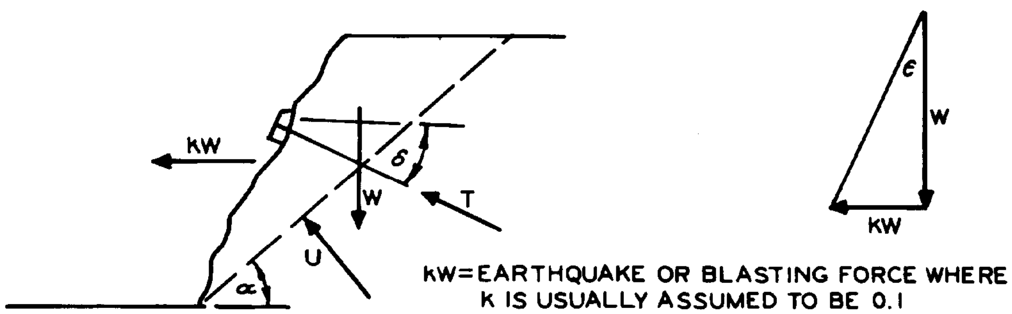


Figure 3-15. Force polygon for earthquake or blasting force.

Equation 1 was selected for the calculation of the FS since the equation contains most of the parameters required for slope-stability analysis and is straightforward to use. The existence of other equations and techniques, which define FS differently, are recognized; however, their use changes the numerical value of the FS only and not the slope stability. To determine the optimum direction of the rock bolts, equation 1 is differentiated with respect to δ . This gives a maximum FS when

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

c. Solution.

(1) Step 1. Check for stability:

Dry Static Case

Assumptions: $T = 0$; $U = 0$; earthquake and blasting effects ignored ($\varepsilon = \delta$); $\alpha + \delta = \phi$

From equation 1

$$FS = \frac{1}{154 \sin 52°} (154 \cos 52°) \tan 32°$$

$$FS = 0.49$$

Since the calculated FS is less than 1.0 there is no need to check for hydrostatic case. The slope is unstable, therefore, excavate and bolt in stages.

(2) Step 2-3. Calculate T .

For this example T will be calculated for $FS = 1.0$, 1.1, and 1.2. Solve equation 1 for T . This gives:

For $FS = 1.0$

$$T = \frac{FS[W \sin(\alpha + \varepsilon)] - cH \operatorname{cosec} \alpha - W \cos(\alpha + \varepsilon) \tan \phi + U \tan \phi}{\cos(\alpha + \delta) + \sin(\alpha + \delta) \tan \phi} \quad (2)$$

For $FS = 1$; $(\alpha + \delta) = \phi$; $\varepsilon = 0$; $U = 63.4$ kips

$$T = \frac{1(154) \sin 52° - 154 \cos 52° \tan 32° + 63.4 \tan 32°}{\cos 32° + \sin 32° \tan 32°}$$

$$T = 86.2 \text{ kips/foot of cut length}$$

(3) Step 4. Calculate bolt spacing.

Use a bolt working capacity of 102 kips and a discontinuity surface area of 76 sq ft (CB × 1). The bolt working capacity was assumed for this problem.

$$\text{Required bolt force/sq ft} = \frac{86.2 \text{ kips}}{76 \text{ sq ft}} = 1.13 \text{ kips/sq ft}$$

$$\therefore \text{ each bolt will strengthen } \frac{102 \text{ kips}}{1.13 \text{ kips/sq ft}} = 90.27 \text{ sq ft}$$

$$S_B = \text{bolt spacing} = \sqrt{90.27 \text{ sq ft}}$$

$$S_B = 9.5 \text{ feet}$$

Bolt should be placed on 9.5-foot centers on the plane of the discontinuity inclined at an angle of -20° ($52° + \delta = 32° \;\therefore\; \delta = -20°$) for FS = 1.

<u>For FS = 1.1</u>

$$T = \frac{1.1(154) \sin 52° - 154 \cos 52° \tan 32° + 63.4 \tan 32°}{\cos 32° + \sin 32° \tan 32°}$$

T = 96.5 kips/foot of cut length

$$\text{Required bolt force/sq ft} = \frac{96.5 \text{ kips}}{76} = 1.27 \text{ kips/sq ft}$$

$$\therefore \text{ each bolt will strengthen } \frac{102}{1.27} = 80.32 \text{ sq ft}$$

$$S_B = \sqrt{80.32 = 8.96 \text{ feet}}$$

$$S_B = 9.0 \text{ feet}$$

Bolt should be placed on 9-foot centers on the plane of the discontinuity inclined at an angle of -20° for FS = 1.1.

<u>For FS = 1.2</u>

$$T = \frac{1.2(154) \sin 52° - 154 \cos 52° \tan 32° + 63.4 \tan 32°}{\cos 32° + \sin 32° \tan 32°}$$

T = 106.8 kip/ft of cut length

Required bolt force/sq ft = $\frac{106.8}{76}$ 1.41 kips/sq ft

$\therefore$ each bolt will strengthen $\frac{102}{1.40}$ = 72.86 sq ft

$S_B = \sqrt{72.86} = 8.53$ feet

$S_B = 8.5$ feet

Bolts should be placed on 9-foot centers, on the plane of the discontinuity inclined at angle of -20° for FS = 1.2. The required bolt spacing on the plane of the cut slope is determined from figure 3-16 using the law of sines.

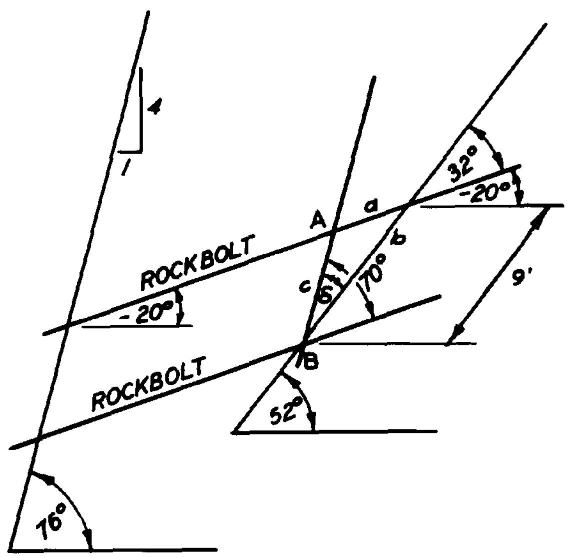


Figure 3-16. Calculation of actual bolt spacing for FS = 1.1.

Construct line AB parallel to cut slope and intersecting discontinuity at point c.

$\gamma = 56° - \delta = 52° - 20°$ $\quad\alpha = 76° - 52°$ $\quad B = 180° - (\alpha + \gamma)$

$\gamma = 32°$ $\quad\alpha = 24°$ $\quad B = 180° - (56°) = 124°$

From law of sines:

$$c = \frac{b \sin \gamma}{\sin B} = \frac{9 \sin 32^\circ}{\sin 124^\circ}$$

$$c = 5.75 \text{ feet}$$

∴ actual bolt pattern on cut surface should be 6 feet vertical by 9 feet horizontal for FS = 1.1.

d. Summary.

Figure 3-17 shows the relationship between the angle of inclination of the bolt and the bolt force required for a given FS for the example problem. The curves demonstrate that for small changes in the inclination of the bolt (in the range from $\delta = -20^\circ$ to $\delta = 0^\circ$), the FS would not change significantly. As an example, changing the bolt angle from $\delta = 20^\circ$ to $\delta = 0^\circ$ and using T as calculated for $\delta = -20^\circ$, the FS changes from 1.1 to 1.04 (from equation 1). Installation of the bolt at $\delta = -20^\circ$ may be impractical; if so, and if a $\delta = 0^\circ$ is desired, the bolt force and spacing would be calculated as outlined above. Bolt inclinations below the horizontal ($\delta > 0^\circ$) should be avoided. The bolt length can be determined graphically from figure 3-17 adding a suitable length beyond the plane of the discontinuity to satisfy anchorage requirements.

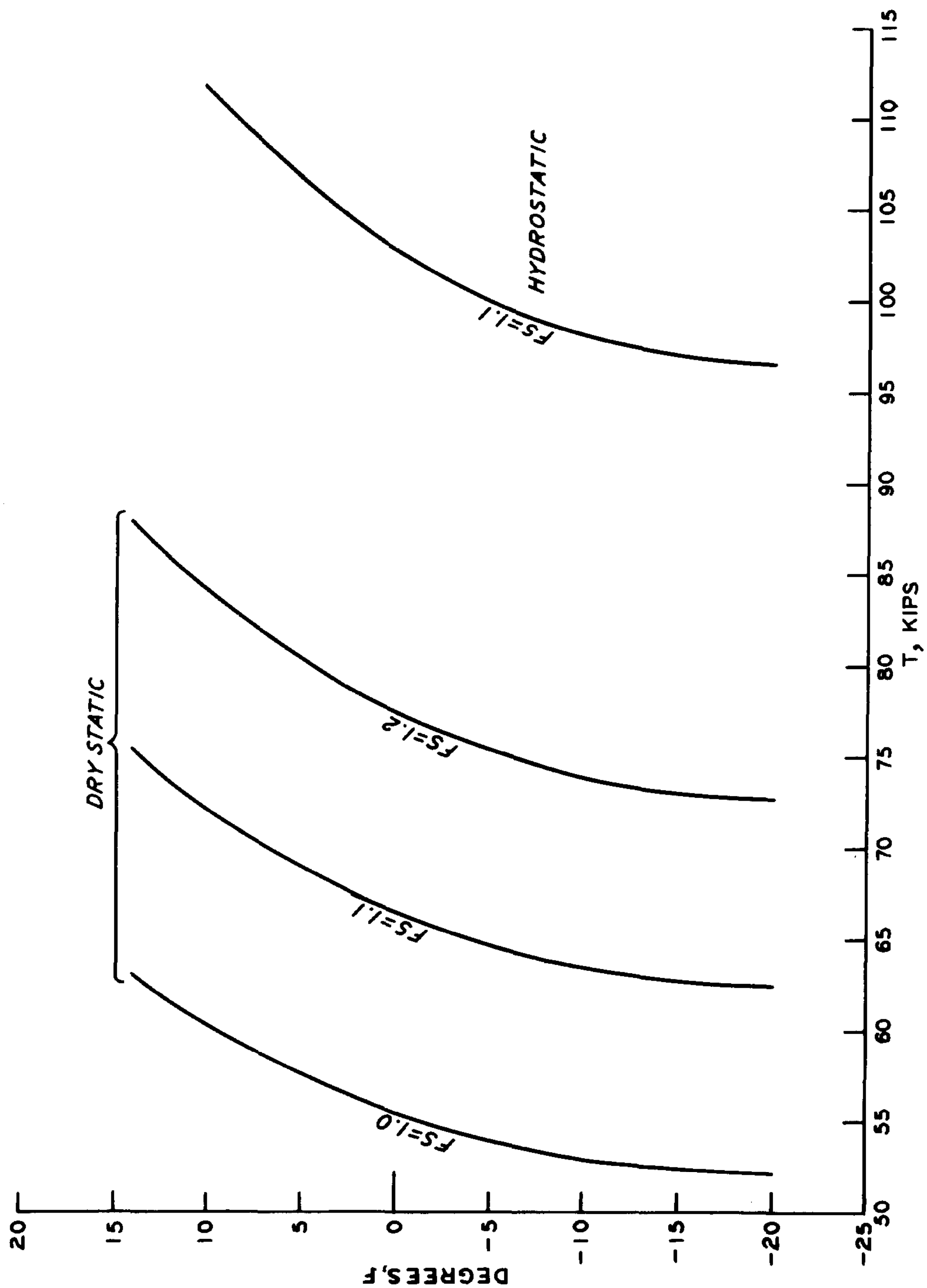


Figure 3-17. Relationship between angle of inclination and bolt force for a given FS in example problem.

CHAPTER 4
UNTENSIONED REINFORCEMENT ELEMENTS

4-1. General. Although the installation of tensioned fully grouted elements is preferred for rock reinforcement, some conditions may make desirable the installation of fully grouted untensioned elements (commonly referred to as rock anchors, anchor bars, or rock dowels). Once installed, stress is developed passively in the element as rock movements take place. Anchoring through and behind burden (recessed rock anchor) prior to removal by later blasting is a desirable application because rock movements are immediately controlled to maintain stability upon exposure of the final excavation line. Additional tensioned reinforcement can then be installed. Installation of grouted untensioned elements also becomes necessary when it is difficult to achieve anchorage for tensioning in soft or highly fractured rock. Other uses are for economically reinforcing areas that are essentially stable (downward installations or as supplemental reinforcement to existing reinforcement or shotcrete, for example) or for anchoring structures to rock.

4-2. Types and Installation Methods.

a. Except that embedment is initially made full length, commonly used types are identical to those shown in figures 3-6, 3-7, and 3-8. Additional details are shown in figures 4-1 and 4-2. Installation techniques and grouting materials, except for obvious differences, are likewise identical to those described in chapter 3 for forming grouted end anchorages or for full-length element grouting. Untensioned elements of all types may be installed without hardware at the face as shown in figures 4-1 and 4-2a or with hardware as shown in figure 4-2b, depending on the application. Nuts tightened against bearing plates are important for providing restraint and to prevent loosening of surface rock. Threaded bar ends are also needed for fastening chain link fabric or other surface treatment, when used. Where surface rock is highly fractured but temporarily stable, unthreaded bars could be installed followed by a shotcreted surface treatment. On the other hand, shotcrete might be used to provide initial support and reinforcement installed through the shotcrete. In this case, bearing plates could be installed at the surface to increase the support capability of the shotcrete.

b. Figure 4-2a shows a typical recessed installation. Once this type is driven to the existing excavation line, an additional bar equipped with a driving dolly is used for pushing the bar to the final line. Some contractors prefer to drill the hole through the burden oversize by approximately one-half inch or more. This reduces the

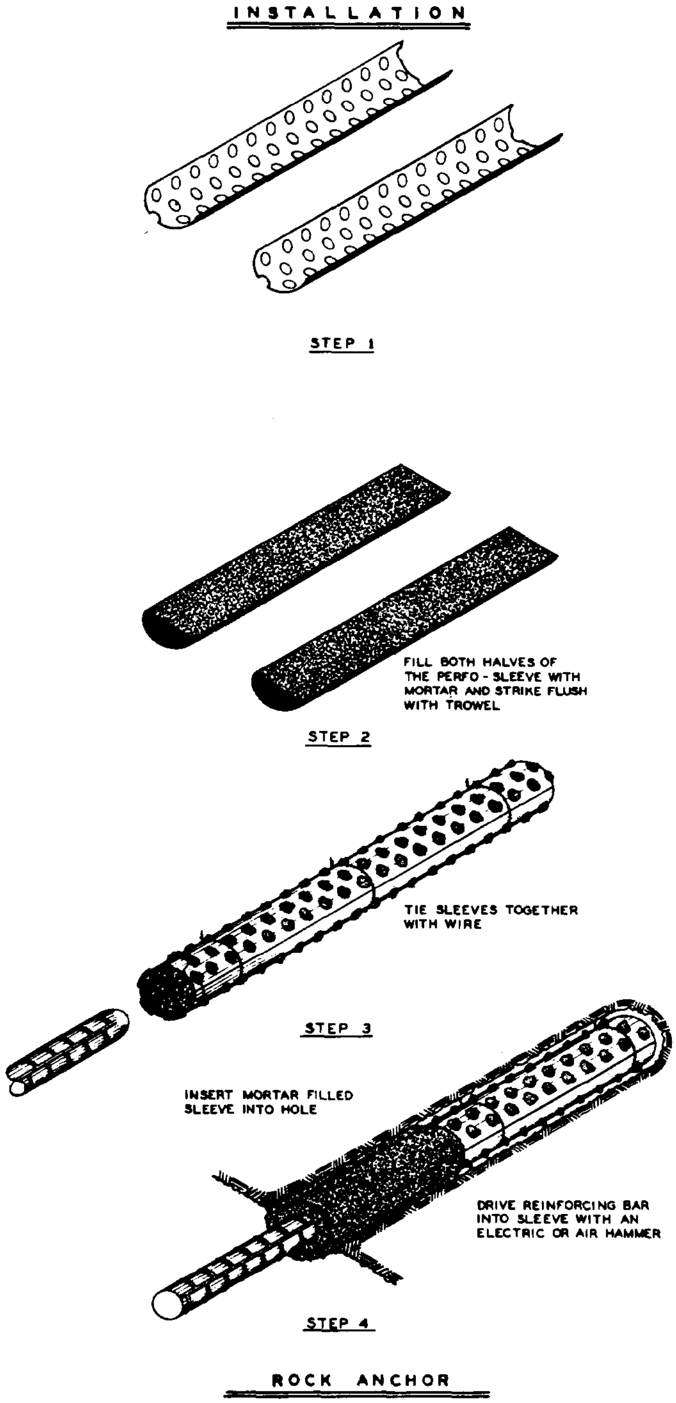
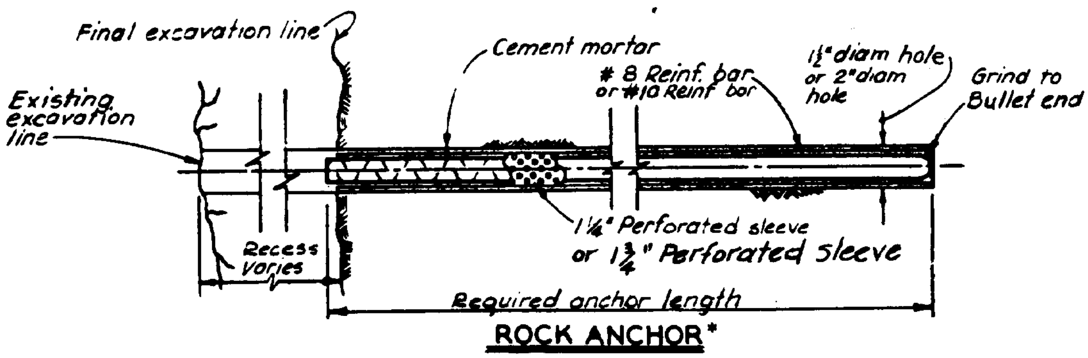
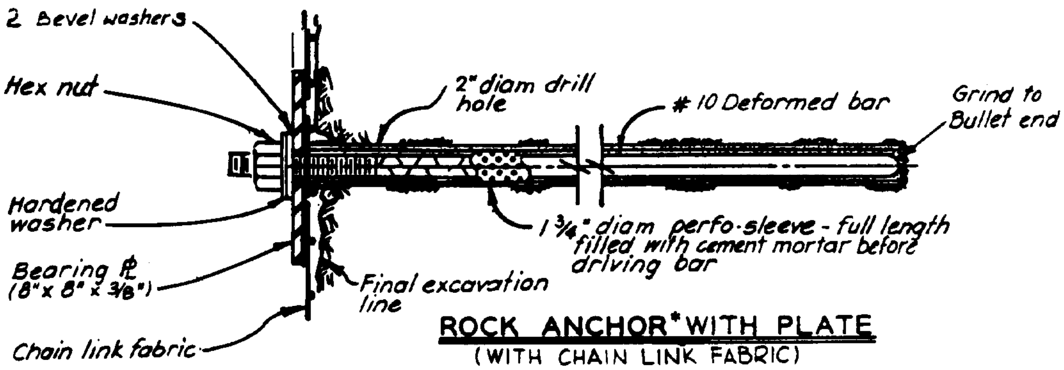


Figure 4-1. Rock anchor, perforated sleeve and mortar type.



a. Without plate.



* ON LONG ANCHORS, CENTER LUGS SHOULD BE WELDED TO THE BAR SO THAT THEY WILL CENTER THE BAR AND ALLOW GROUT TO FLOW COMPLETELY AROUND THE ANCHOR BAR.

b. With plate.

Figure 4-2. Typical recessed installations, with and without plate.

possiblility of bars binding in the hole and prevents bonding of the bar to the burden rock in the event a bar does bind and cannot be fully driven.

c. Although not shown, untensioned elements can be installed through grout placed by gravity in down-holes. Also, it may be convenient at times to inject grout through untensioned hollow-core rock bolts when the mechanical anchor does not hold because of the existence of poor rock. Other methods, more common in Europe, involve the use of compressed air to force plastic portland cement mortar into the drill hole. The element is then inserted to force excess mortar to flow out the hole. One variation of this method is commonly referred to as the SN method because it was used to reinforce rock on the Stor-Norrfors underground powerhouse in Sweden.

4-3. Sample Specification. A sample specification based on experience with fully grouted untensioned elements during excavation at the NORAD Cheyenne Mountain Complex is included as Appendix D. The sample may be used as a guide and revised to include other types described in this manual and to fit conditions at other projects.

CHAPTER 5
HIGH CAPACITY TENSIONED ROCK ANCHORS

5-1. General. High strength and long length cable and rod type bolts have been used in a number of cases to solve special rock reinforcement problems. Many patented systems have been allowed by specifications generally to encourage competitive bidding. The designer specifies only the hole length and orientation and the amount of effective confining force required. Table 5-1 lists some sources and sizes of available systems which have been used for rock reinforcement. Systems are made up of wires, strands, or rods. High strength reinforcement systems all have three basic features: (1) an anchoring device, (2) a tensioning device, and (3) a tendon or bar connecting these two together. The available systems differ mainly in the gripping or holding mechanisms, which are patented.

5-2. Anchorage. The use of grouted anchorages is universal with high strength systems. Several systems use a spreader plate on the end of wire, strand, or bars, although stranded cables have been installed effectively without spreader plates. The grouted anchorage should be capable of withstanding the force of the connecting element at the ultimate load of the steel. The length of grouted anchorages may be 30 feet or more depending upon local rock conditions, capacity of the anchor, and the drill hole diameter. Generally, a grout length is arbitrarily selected and proof-tested. If excess anchor slip occurs, the length of anchorage is increased.

5-3. Types of Tensioning Devices.

a. General. Tensioning of high capacity reinforcement systems is accomplished using hydraulic jacks. The transfer of the load from the connecting element to the bearing plate is accomplished by three methods: button heads, wedge grip, and threads.

b. Button Heads. The wire systems listed in table 5-1 use the cold-formed button head for load transfer. The head is formed after the wire passes separately through a machined plate. Shims are placed between the machined plate and the bearing plate. One system also has a lock nut which is threaded on the machined plate. Button heads are considered to be positive transfer mechanisms.

c. Wedge Grips. All strand systems listed in table 5-1 use wedge-grip devices for load transfer. These devices grip the strand in a machined plate (either individually or in groups) and the machine plate bears against the bearing plate. Some load elongation loss occurs while setting the grips but can be compensated for when tensioning.

Table 5-1. High Capacity Rock Anchor Systems

Type of Anchor System	Name	Ultimate Stress, ksi	Nominal Diam in.	No. of Wires	No. of Strands	Surface Condition	Max Eff Force 0.60 f's K	Tension Load Transfer Mechanism	Anchorage
Wire	BBRV	240	0.25	8-52	--	--	56.6- 367.6	Button heads	Spreader plate and grout, plus button heads
		240	0.25	2-43	--	--	14.1- 304.0	Button heads	Spreader plate and grout, plus button heads
	Soil and rock assemblies	240	0.25	18-108	--	-- 763.6	127.3-	Button heads plus button heads	Spreader plate and grout,
Strand	Con a (Single) in Ryco (Mult)	270	0.50	--	1-48	— 1189.5	24.8-	Wedge grip plus wedge grip	Spreader plate and grout,
	Stressteel SEEE	270	0.50 0.60	-- --	1 1-12	-- -- 388.8	24.8 32.4-	Extrusion Wedge grip grip	Spreader plate and grout, plus extrusion wedge
	Stressteel S/H wedge	270	0.50	--	3-54	-- 1338.0	74.3-	Wedge grip plus wedge grip	Spreader plate and grout
	Monostrand rock anchors	270	0.50 0.60 0.70	-- -- --	1 1 1	-- -- --	25 35 52	Wedge grip	Grout only
	Rock and soil anchors	270	0.50	—	1-52	-- 1288.9	24.8-	Wedge grip plus diverging and verging of strands	Spreader plate and grout, con-
Bar	Dywidag	150	0.625 1.250	-- --	-- --	Deformed 116.6	26.1 deformation and nut	Thread type	Grouted, plus anchor ring
		160	0.625 1.250	-- --	-- —	Deformed 124.3	27.8		
	Stressteel Bay System	145	0.750 1.375	-- --	— --	Smooth 129	39 and nut (tapered), Howlett Grip nut	Wedge, thread wedge or threaded	Grouted, plus plate
		160	0.750 1.375	-- --	-- —	Smooth 143	42		
	Hollow Groutable	--	2.000	--	--	Deformed	120 nut	Thread and	Expansion
	Solid bar Rock bolts	--	2.000	--	--	Smooth			
	Solid bar		1.875	--	--	Smooth			

d. Threads. Each bar system listed in table 5-1 uses a different thread holding mechanism. The Dywidag Bar System uses threads which are formed as deformations on the rod. Stress Steel Bar Systems use wedge, tapered thread and nut, and grip nut and sleeve holding mechanisms. The Williams Bar System uses a system similar to that used with their No. 8 and 11 rock bolts.

5-4. Types of Connecting Elements. The connecting elements which connect the anchorage to the tensioning device are classified as wires, strands, or rods. The systems listed in table 5-1 are arranged by connecting elements and each is a complete system (patented).

5-5. Tentative Recommendations. "Tentative Recommendations for Prestressed Rock and Soil Anchors," prepared by an ad hoc committee of the Prestressed Concrete Institute's Post-Tensioning Committee, is presented as Appendix E of this manual.

CHAPTER 6
SURFACE TREATMENT

6-1. General. Surface treatment of rock can be accomplished by a variety of methods. Chain link fabric, welded wire fabric, mine ties, steel strapping, and shotcrete are the most common methods used. The value of each method has been demonstrated in laboratory tests and in numerous tunnels, natural or excavated slopes, and chambers. Therefore, the choice of which method or combination of methods to use must be based on experience, other projects, and economics. Examples of surface treatment are shown in figures 6-1 through 6-3.

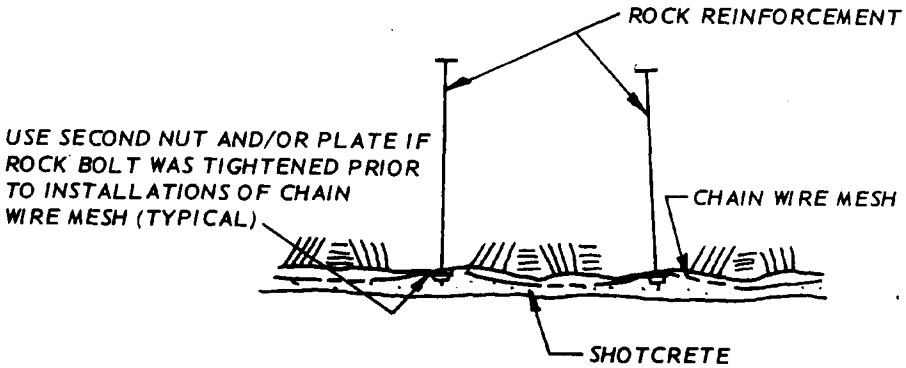


Figure 6-1. General surface treatment-wire mesh and shotcrete.

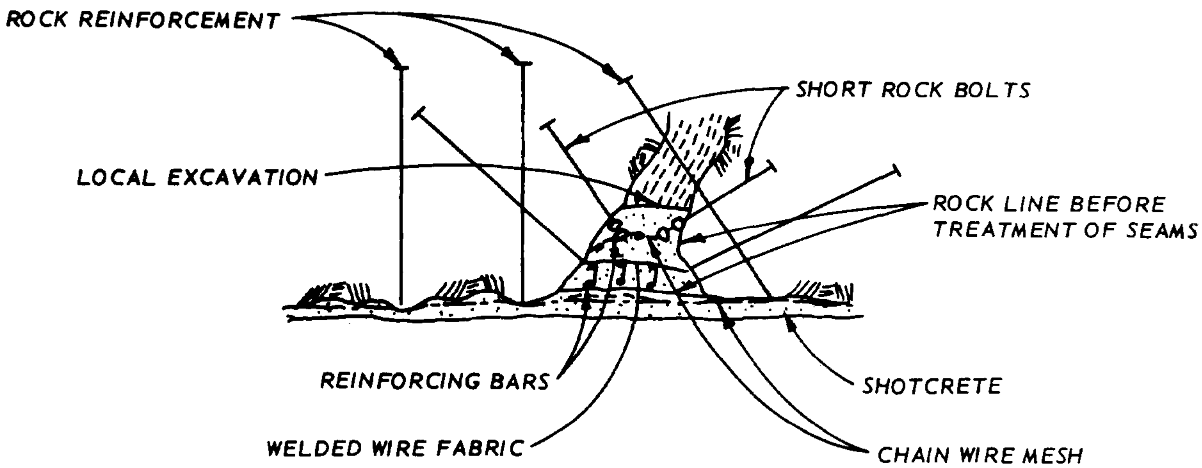
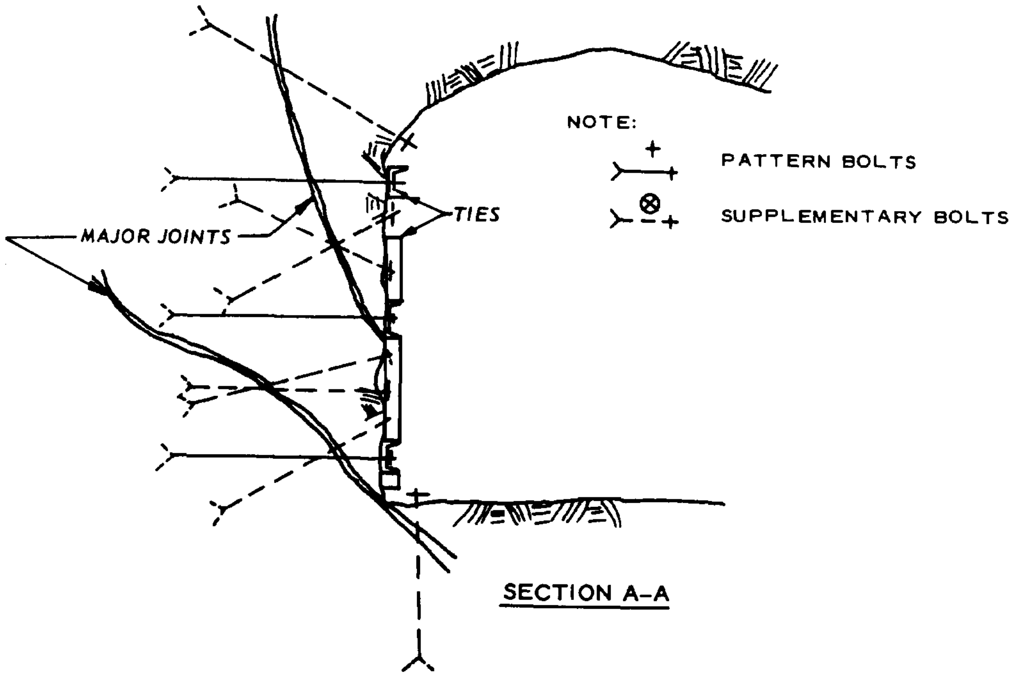


Figure 6-2. Typical local structural treatment of wide seams and fractured zones.



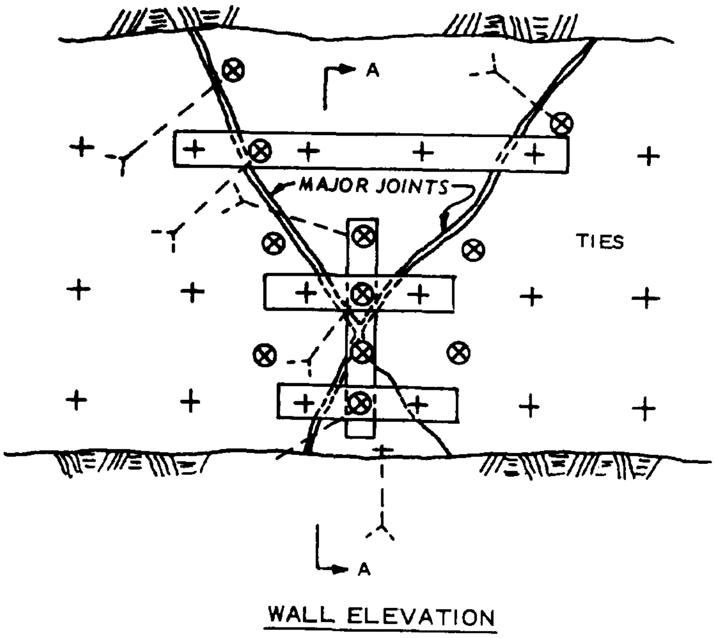


Figure 6-3. Typical use of ties and supplementary bolts.

6-2. Use of Wire Mesh and Fabric.

a. The most common type of surface treatment is wire mesh or fabric attached directly to the reinforcement elements. Such surface treatment can and should be used as a routine part of construction. The benefits include stabilization of deeper rock by holding loosened rock in place and control of unstable areas by containing loose rock rather than allowing it to fall. The safety benefits are obvious. The fabric should be galvanized if it is permanently exposed but may be ungalvanized if it is to be eventually covered by shotcrete or concrete.

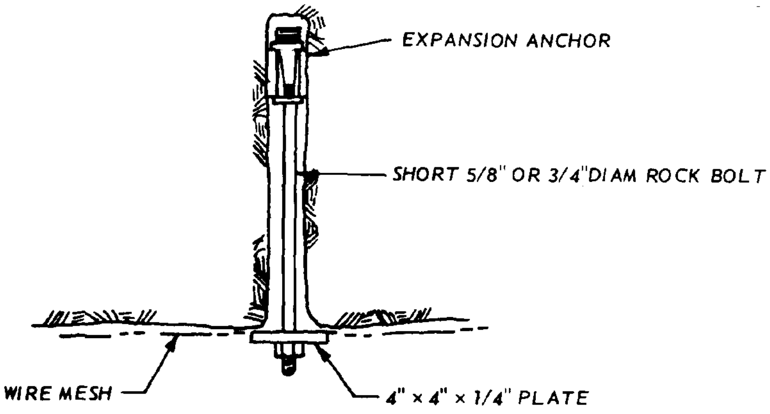
b. If maximum benefit is to be gained from wire mesh and fabric, intermediate attachment to the rock between primary reinforcement elements is necessary. Methods for such attachment are shown in figure 6-4.

6-3. Mine Ties and Strapping. These serve as a type of lagging between primary reinforcement elements. The surface restraint is more rigid than in the case of wire mesh. Strapping is very effective when it is continuous around exterior corners of cavity intersections, etc. A combination of strapping with wire mesh is most effective in preventing progressive loosening. An example of the use of mine ties is given in figure 6-3.

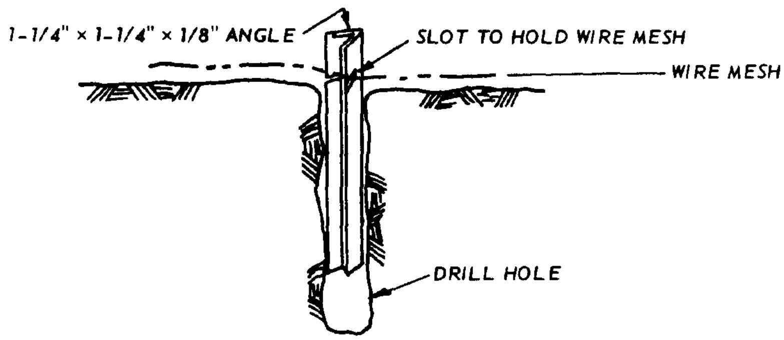
6-4. Shotcrete.

a. As with chain link fabric, a rational design method is not available for shotcrete. Its effectiveness, however, has been demonstrated on numerous projects around the world in recent years. In combination with rock bolt systems, the use of shotcrete as surface treatment is very effective in preventing surface ravelling. As with rock bolts, shotcrete is generally most effective (except in swelling ground) if it is applied shortly after excavation. When permanently exposed, the shotcrete should always be placed in combination with welded wire fabric or chain link fabric. Drain holes drilled through the shotcrete into the rock should be constructed to relieve hydrostatic pressure.

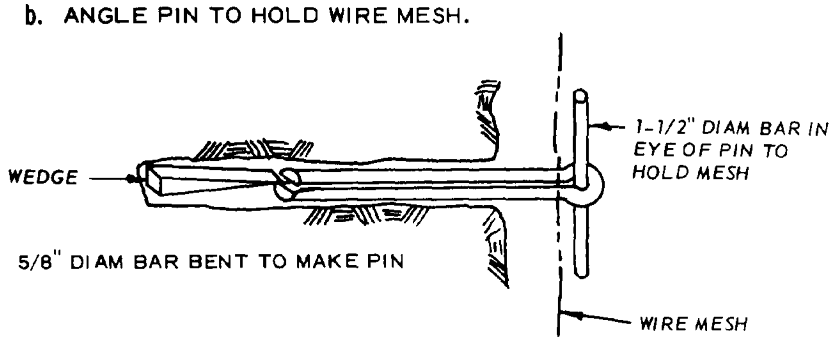
b. The use of shotcrete has provided savings on several projects and its successful use depends on the experience and judgment of the engineers and construction personnel at the project. As a method of surface treatment when used in combination with rock bolts, it is far superior to other methods of surface treatment but is also much more expensive. Because of these numerous variables, strict guidelines as to when to use shotcrete on a project cannot be presented in this manual. It can only be recommended that on each project its use should



a. SHORT EXPANSION ANCHOR.



b. ANGLE PIN TO HOLD WIRE MESH.



c. WEDGE MESH PIN.

Figure 6-4. Methods for intermediate attachment of wire mesh.

be considered when the method of surface treatment is chosen.

c. As in the case of wire fabric, shotcrete serves as a monitoring tool. The shotcrete is a brittle coating that will crack if there is major movement in the rock structure.

d. The following discussion pertaining to the material properties and placement of shotcrete is partially excerpted from EM 1110-2-2901.[3]

6-5. Material Properties and Placement of Shotcrete.

a. General.

(1) Shotcrete is defined by the American Concrete Institute as mortar or concrete conveyed through a hose and pneumatically projected at high velocity onto a surface. The force of the jet impinging on the surface compacts the material. A relatively dry mixture is generally used, and the material is capable of supporting itself without sagging or sloughing, even for vertical and overhead applications (ACI[20]).

(2) Shotcrete has been in use for more than 50 years for various types of construction. However, it has only recently been applied to tunnels and other underground support, following the development of high volume equipment and accelerating admixtures which cause the concrete to set up very rapidly. The state-of-the-art is best described in the proceedings of an Engineering Foundation Conference (ACI[21]).

(3) Shotcrete, applied immediately after excavation, seals and ties down the rock and provides a thin, strong, flexible membrane. It frequently speeds up the tunneling operation and may result in cost savings as compared with other tunnel support methods. It is best suited for tunnels excavated by drilling and blasting. Where appropriate, it should be listed as one of the bidding alternates.

(4) In Corps of Engineers tunnel construction, shotcrete will generally be used for initial support, prior to placing the permanent concrete lining. Occasionally, it may be used for final support, as in tunnels infrequently used and designed for low-velocity flows. Shotcrete is also used in a nonstructural capacity to seal rock surfaces and to keep them from air slaking, while at the same time preventing fallouts.

b. Material Properties.

(1) Shotcrete for rock support consists of portland cement, water,

sand, coarse aggregate graded up to about 1/2 inch, and a strong accelerator if the shotcrete is to be applied for tunnel support. The accelerator causes the shotcrete to start hardening immediately after application so that it will give early support to the rock. A minimum compressive strength of 700 psi at the age of 7 hours is readily achievable, and should generally be required. The accelerator also provides two other advantages.

(a) It permits the shotcrete section to be built up quickly in fairly thick layers (even in overhead applications) without falling out or excessive rebound.

(b) It enables the shotcrete to bond to wet surfaces. By increasing the amount of accelerator, shotcrete can be successfully applied even in local areas of light seepage. Any running water should be controlled by drains before shotcreting.

(2) Unfortunately, all present day accelerators which give good early strengths reduce the values at later ages, the amount of the reduction depending on the "compatibility" of the accelerator and cement used. This effectively places a ceiling of about 4000 psi on the 28-day strength which may be specified.

(3) As part of the mix design studies at the beginning of the job, tests should be made to arrive at the best combination of brand of accelerator and cement, and percent of accelerator to be used. Normally, molded specimens are made just to establish the approximate range of proportions. Then a few test panels or representative areas in the structure are gunned, and cubes or cores extracted for determining 7-hour and 28-day strengths. All values are reported in terms of core strengths which have been corrected to a length to diameter ratio of two. Cube strengths should be reduced by 15 percent. Strengths in these preconstruction tests should exceed the specified strength by at least 20 percent.

(4) Regulated set cement, which gives accelerated strength development without the need for an accelerator, has been used on an experimental basis with promising results.

c. Quality Control.

(1) It must be recognized that shotcreting is a specialized process, quite different from conventional concreting. There are many variables (conditions under which it is placed, equipment used, competence of the application crew, and others) which affect the quality of shotcrete in general construction (Reading[42]). Even more

attention must be given to quality control in underground support construction, because present accelerators contribute to variability and because quality requirements are higher than for most other shotcrete work (Reading[43]). Designers should familiarize themselves with the material, and inspection personnel should be properly instructed at the beginning of the job. The contractor should be required to demonstrate (usually as part of the mix design studies) that his personnel, equipment, materials, mix design, and procedures will produce shotcrete of the quality required. Even at best, the degree of quality control can hardly be expected to equal that for concrete.

(2) Tests should be made throughout the job to assure that the in-place product is satisfactory. A program of routine coring from the structure is normally recommended. As an alternative, test panels gunned periodically may be supplemented with occasional cores from the structure.

d. Equipment and Application.

(1) The dry-mix process (where the mixing water is added at the nozzle) is preferred for tunnel support shotcrete because it is better adapted to the use of accelerators. Particular care should be taken to see that the admixture is accurately proportioned and uniformly mixed with the other materials. The wet-mix process is more difficult to control because it requires that the accelerator be added before the mix enters the hoseline. Therefore, the mix might set up before it reaches the nozzle. However, the use of the wet-mix process without accelerators is satisfactory for other applications as is open excavation where high early strength is not required.

(2) In tunnel work, it is normal practice to apply a 2-inch layer of shotcrete as soon as practicable (usually within 2 to 3 hours) after the section is mined, and before mucking out is completed. It is often a safety hazard to have the nozzle operator working on the muck pile; a better method is to have the nozzle operator work from a platform on an adjustable hydraulic crane boom. Where the tunnel is large and it is judged that there is some risk of rock fallout from the crown before the application is completed, a "robot" may be used; here the nozzle operator sits in the protected cab of a crane 15 to 20 feet away, and controls the nozzle from that location. Good lighting, so that the nozzle operator can see the area being shotcreted, is essential. This is particularly important where the nozzle operator is a considerable distance from the work (where a robot is used).

(3) A second 2-inch layer is generally applied within 8 hours after mining, and a third layer (where required) within 24 hours.

(4) If the tunnel is excavated by drilling and blasting, the surface is likely to be irregular, with considerable overbreak. Care should be taken to get the full design thickness of shotcrete on the inside surfaces and particularly at the outer edges or corners of the overbreak. The shotcrete will normally round out the base of the cavity.

(5) The mix and application procedures should be carefully controlled so that the in-place shotcrete is dense and sound, and rebound is held to the lowest practicable quantity. The best results are obtained by keeping the nozzle normal to the surface of application.

(6) Shotcreting, particularly the dry-mix process, is accompanied by considerable dust, which is a health hazard and obstructs the visibility of the nozzleman. Good ventilation is needed to keep dust at a minimum. Other necessary safety precautions include face protection and a respirator for the nozzleman, and protection of the skin against cement and especially against caustic accelerators.

(7) Curing requirements (moisture retention and temperature) are about the same as for conventional concrete used in tunnels. However, no curing medium need be applied if the relative humidity in the tunnel is above 85 percent.

CHAPTER 7
CASE HISTORIES AND APPLICATIONS

7-1. General. This section summarizes underground rock reinforcement experience on a number of major underground projects completed since the year 1950. A more general background and history on rock reinforcement is presented in Appendix A. In this section, detailed information is presented which may be useful because the design and construction of new underground projects is largely based on previous experience. The data presented are not intended to substitute for the extensive study of past projects required of rock reinforcement designers. However, one of the objectives of summarizing the tabulated data is to present design parameters which are presently useful or which may become useful as additional data become available from new projects.

7-2. Data on Underground Chambers, Tunnels, and Shafts.

a. Table 7-1 presents data patterned after the manner used by Cording, Hendron, and Deere,[29] except that somewhat more detail is included in table 7-1. The information presented was gathered from the literature or from the designers of projects. In addition to factual information identified by the column headings, calculated values useful for establishing design parameters are also shown. These include element length to opening span (width or height) ratios, estimated average confining pressure exerted on the rock surface by the reinforcing element when initially tensioned, and a projected value for average confining pressure at the yield strength of the element. The confining pressure is then related to the unit weight of the rock and the opening width (or height) by a factor n (or m), which is defined below.

b. Cording, et al., stated that in a rock mass having both frictional resistance and cohesion, acted upon by its own body forces, the internal pressure, P_i, required to maintain stability would be:

$$P_i = nB\gamma - \text{(rock mass cohesion)}$$

where n is a function of the frictional resistance of the rock mass, B is the width of the opening, and γ is the unit weight of the rock. The rock mass cohesion should be related to the ratio of joint spacing to B. For a given joint spacing, increasing B will decrease the rock mass cohesion. Thus, a small opening may be stable with no internal pressure while a large opening may require support.

c. For large openings, if the rock mass cohesion is assumed to be zero, the relation becomes $P_i = nB\gamma$. In terms of measurable rock properties, n can be considered a function of (1) rock quality, (2) displacement along discontinuities, (3) strength and orientation of discontinuities, and (4) the ratio of intact unconfined compressive strength to the maximum natural stress, q_u/σ . Values of (q_u/σ) less than 5.0 are indicative of stress conditions where new fractures will form around the opening upon excavation and where shearing and crushing of irregularities on discontinuities is likely to take place as rock wedges displace.

d. Figure 7-1 compares cavern width, B , with the average support pressures which have been used in the crowns of chambers. The support pressure, P_i , is defined as the yield capacity of the bolt divided by the square of the bolt spacing. Support pressures generally fall within the range of n = 0.1 to 0.25 , equivalent in a 100-foot-wide cavern to supporting 10 to 25 feet of rock. Figure 7-2 compares height of caverns with average pressure used on walls. Wall support pressures are generally lower than crown pressures with values of m ranging from 0.04 to 0.13 and above.

e. Figures 7-3 and 7-4 summarize bolt lengths used in underground openings. Most of the bolts used in the crowns of large openings fall within the range of 0.2B to 0.4B with the ratio increasing for tunnels under 25 feet in diameter. Bolt lengths used in sidewalls range, widely, from 0.1H to 0.5H.

7-3. Anchorage of Structures. Rock bolts and rock anchors have been used to anchor several types of structures. In civil engineering projects, uses have included anchoring concrete walls to rock, anchoring retaining walls, anchoring spillway weirs and stilling basins, and anchoring several other types of miscellaneous structures. Stability analyses for these types of structures are covered in other Engineering Manuals and will not be discussed here.

Table 7-1. Reinforcement in Underground Chambers, Tunnels and Shafts

Item No.	Project Location Responsible Agency	Designer Construction Contractor	Location of Reinforcing Elements Construction Period	Excavation Dimensions H = Height, ft B = Width, ft L = Length, ft D = Depth, ft Shape	Rock Properties q_u = Unconfined Compressive Strength, γ = Unit Weight, RQD = Rock Quality Designation	Description and Properties of Reinforcing Elements (F_y = Yield strength of Element, T_a = Anchorage Setting Torque Used, T_n = Tensioning Torque Used, F_d = Direct Pull Tension Force Used, F_i = Estimated Initial Tensile Stress in Installed Element, S = Spacing of Elements, k = 1000 lb)	Average Confining Pressure on Rock Surface, P_i, psi $P_i = \frac{F_i}{S^2}$ or $\frac{F_y}{S^2}$		$n = P_i/B\gamma$ $m = P_i/H\gamma$		Bolt Length / Span		Comments
							Initial	Yield	Initial	Yield	$\frac{\ell}{B}$	$\frac{\ell}{H}$	
1	East Delaware Tunnel New York, N. Y.		Aqueduct 1950	H = B = 13 L = 63,360 D = Circular	Shale, red, flat bedded Sandstone, gray, thinly laminated	Ungrouted, 1-in.-diam by 6-in.-long slot and wedge rock bolts. S = approximately 3 ft but varied to meet local conditions. Steel channels and ties used on rock surface					0.46		Approximately 30,000 ft of tunnel reinforced. Gunite placed after bolts were installed
2	Nechako-Kemano-Kitimat British Columbia, Canada Aluminum Co. of Canada, Ltd.	British Columbia International Co., Ltd.	Kemano power development, underground hydro-electric power-plant chamber 1952	H = 120 B = 82 L = 700 D = 1500, cover Parabolic arch roof mined with rise of 37 ft and span of 103 ft Vertical walls	Granodiorite containing many small feldspar - aplite and diorite dikes; two dominant and two minor faults intersect chamber q_u = 16-25 ksi γ = 171 pcf	Ungrouted "fishtail" slot and wedge bolts, 12-16 ft long in walls. Following a massive fallout in west wall, over 1500 "fishtail," 1-1/2-in.-diam by 30- to 40-ft-long bolts placed in inclined diamond drilled holes filled with fluid sand cement grout, wedges driven tight, and bolts stressed with torque wrench						0.25 0.33	Rock bolts provide permanent reinforcement of walls. Continuous reinforced concrete arch supports roof. 2,200 temporary "fishtail" rock bolts and 11,000-cu-yd gunite were used during roof mining to assure personnel safety
3	Glendo Dam Glendo, Wyo. U.S.Buréau of Reclamation	U. S. Bureau of Reclamation C.F. Lytle Co. Green Construction Co.	Intake tunnel 1955-1958	H = B = 24.5 L = 1150 D = Circular	Sandstone, shale γ = say 165 pcf	Ungrouted, 3/4-in.-diam by 6-ft-long expansion shell rock bolts. 6,267 lin ft installed at 4-ft spacing where considered necessary. Bearing plates: 6 in. triangular by 1/2 in. thick. F_y = 15k		7		0.25	0.24		
4	Hills Creek Dam Oak Ridge, Ore. CE, Portland District	Corps of Engineers Shea Co.	Diversion tunnel 1956	H = B = 27 L = 1150 D = Horseshoe	Lapilli tuff, with areas of weak faults and joints γ = 134 pcf q_u = 5-7 ksi	Ungrouted, 1-in.-diam slot and wedge rock bolts, 10 ft long in crown, 8 ft long to springline, placed in 800 ft of tunnel. S = 5 ft each way. T_n = 250-300 ft-lb, F_i = 10k, F_y = 24k	3	7	0.12	0.28	0.30 0.37		Remainder of tunnel supported with steel sets. Tunnel is lined with reinforced concrete
5			Regulating tunnel 1959	H = B = 18 L = 545 D = Circular	Lapilli tuff, massive, with minor faults	Ungrouted, 1-in.-diam-slot and wedge rock bolts, 10 ft long in crown, 6 ft long to springline. S = 4 by 5 ft. T_n = 200-300 ft-lb, F_i = 8k, F_y = 24k	3	8	0.12	0.32	0.33 0.44		Tunnel is lined with reinforced concrete
6			Penstock tunnel 1959	H = B = 16 L = 667 D = Circular	Lapilli tuff with 20-ft-wide fault gouge zones	Ungrouted, 1-in.-diam slot and wedge, 10 ft long in crown and faults, 8 ft long to springline, 6 ft long in faults below springline. S = 4 by 4 ft above springline and 4 by 6 ft below. T_n = 100-200 ft-lb in fault zones, F_i = 4k; T_n = 200-300 ft-lb in intact rock, F_i = 8k, F_y = 24k	4 2 (faults)	13	0.19	0.45	0.50 0.63		Tunnel is lined with reinforced concrete

(Continued)

Table 7-1. (Continued)

Item No.	Project Location Responsible Agency	Designer Construction Contractor	Location of Reinforcing Elements Construction Period	Excavation Dimensions H = Height, ft B = Width, ft L = Length, ft D = Depth, ft Shape	Rock Properties q_u = Unconfined Compressive Strength, γ = Unit Weight, RQD = Rock Quality Designation	Description and Properties of Reinforcing Elements (F_y = Yield Strength of Element, T_a = Anchorage Setting Torque Used, T_n = Tensioning Torque Used, F_d = Direct Pull Tension Force Used, F_i = Estimated Initial Tensile Stress in Installed Element, S = Spacing of Elements, k = 1000 lb)	Average Confining Pressure on Rock Surface, P_i, psi $P_i = \frac{F_i}{S^2}$ or $\frac{F_y}{S^2}$ Initial	Yield	$n = P_i/B\gamma$ $m = P_i/H\gamma$ Initial	Yield	Bolt Length / Span $\frac{\ell}{B}$	$\frac{\ell}{H}$	Comments
7	Cougar Dam Eugene, Ore. Corps of Engineers	CE, Portland District Merritt, Chapman, & Scott, Inc.	Diversion tunnel 1956	H = B = 20 L = 1834 D = Horseshoe	Bedded Lapilli tuff Basalt near portal γ = 160 pcf q_u = 7-21 ksi	Ungrouted, 1-in.-diam slot and wedge rock bolts, 8 ft long in crown, 6 ft long to springline, and below springline in faulted areas. S = 4 by 5 ft. F_y = 24k, T_n = 250-300 ft-lb, F_i = say 10k	3.5	8.4	0.19	0.45	0.30 0.40		Temporary diversion tunnel, unlined
8			Penstock tunnel 1960	H = B = 16 L = 1028 D = Circular	Part Lapilli tuff, bedded; part basalt with vertical joints and faults	Ungrouted, 3/4-in.-diam expansion shell rock bolts, 6 ft long, S = 5 by 5 ft, T_n = 150-200 ft-lb, F_i = say 6k, F_y = 15k	1.7	4.2	0.11	0.28	0.33		Tunnel is lined with reinforced concrete
9			Regulating tunnel 1960	H = B = 22 L = 993 D = Horseshoe	Basalt, some widely spaced vertical joints with some oxidation and water seeps	Same as penstock tunnel, except 8-ft-long bolts used in crown, 6 ft long to springline	1.7	4.2	0.07	0.20	0.27 0.36		Tunnel is lined with reinforced concrete
10	Haas Hydroelectric Power Project 50 miles north of Fresno, Calif. Pacific Gas & Electric Co.	Pacific Gas & Electric Co. Morrison-Kaiser-Macco-Ferini	Underground power plant chamber 1957	H = 100 B = 56 L = 173 D = 500	Massive granite (widely spaced fractures) γ = say 170 pcf	Fully grouted perforated-sleeve rock anchors, 1-in.-diam deformed reinforcing bar with Portland cement-sand grout, 10, 12.5, and 15 ft long spaced at 3.5 ft each way. Installed in arch roof. Gunite mesh, 4 by 4 in. by No. 6, attached to bars and 4-in.-thick gunite to surface of roof. 2,000 rock anchors and 13,000-sq-ft gunite placed. Steel yield unknown, say 36k	0	20	0	0.30	0.18- 0.27		First large underground hydroelectric power plant constructed in the U.S.A.
11	Binga Hydro-electric Project Luzon, Republic of Phillipines National Power Corp.	National Power Corp. & Engineering Development Corp. of Phillipines Phillipine Engineers Syndicate, Inc., & Platzer, AB, Stockholm	Underground power plant chamber Prior to 1959	H = 93.5 B = 50 L = 255 D = 300	Metamorphosed andesites and sedimentary rocks intruded in places by diorite and other igneous rocks	Rock bolts consisting of deformed reinforcing bar, 10-20 ft long, inserted in grout mortar placed by the Swedish SN method. Lower portions of walls reinforced at 3.3-ft spacing each way. Generally installed pointing upward, 30-45 deg from horizontal, with some horizontal		12.9			0.11- 0.21		Size, strength, and initial load on bars unknown. In pull tests, an embedment of 3-5 ft developed full bar strength (in excess of 20k) in a variety of rock types, including the softer rocks
12	Broken Bow Dam Broken Bow, Okla. CE, Tulsa District	Corps of Engineers Nello L. Teer	Diversion tunnel 1962	H = B = 18 L = 1090 D = Circular	Argillite, shale, limestone	Ungrouted 3/4-in.-diam headed rock bolts with expansion shell, S = 5 ft longitudinal by 4.5 ft perimetrical, 6- by 6- by 3/8-in.-bearing plate, 4- by 4-in.- by No. 6 wire mesh on rock surface, F_y = 15k, T_n = 150 ft-lb, F_i = 6.5k	2	4.5	0.1	0.22	0.44		

(Continued)

Table 7-1. (Continued)

Item No.	Project Location Responsible Agency	Designer Construction Contractor	Location of Reinforcing Elements Construction Period*	Excavation Dimensions H = Height, ft B = Width, ft L = Length, ft D = Depth, ft Shape**	Rock Properties q_u = Unconfined Compressive Strength, γ = Unit Weight, RQD = Rock Quality Designation	Description and Properties of Reinforcing Elements (F_y = Yield Strength of Element, T_a = Anchorage Setting Torque Used, T_n = Tensioning Torque Used, F_d = Direct Pull Tension Force Used, F_i = Estimated Initial Tensile Stress in Installed Element, S = Spacing of Elements, k = 1000 lb)		Average Confining Pressure on Rock Surface, P_i, psi $P_i = \frac{F_i}{S^2}$ or $\frac{F_y}{S^2}$ Initial	Yield	$n = P_i/B\gamma$ $m = P_i/H\gamma$ Initial	Yield	Bolt Length / Span $\frac{\ell}{B}$	$\frac{\ell}{H}$	Comments
13	NORAD Cheyenne Mountain Complex Colorado Springs, Colo. North American Air Defense Command	CE, Omaha District; Parsons, Brinkerhoff, Quade & Douglas; A. J. Ryan & Associates Utah Mining & Construction Co.	Parallel chambers A, B, and C, typ.	H = 60.5 B = 45 L = 1368, total	Coarse-grained biotite granite intruded by fine-to-medium-grained granite and thin basalt dikes Pegmatite dikes may occur locally. Granite is closely fractured (3 in.-3 ft) and mostly unaltered. Predominant fracture system composed of two steeply dipping sets, striking NE and NW γ = 174 pcf q_u: Coarse-grained 19-29 ksi Fine- to- medium grained, 24-35 ksi Coarse-grained, highly altered, 4.8-8.6 ksi RQD: Coarse-grained, good Fine-grained, fair to good	S, Length Arch: 4 by 4 ft, 10 ft Walls: 5 by 5 ft, top 1-12 ft, bottom 2-8 ft, remainder-10 ft	Fully grouted slot and wedge rock bolts, 3/8-in.-thick bearing plates by 8-in. square or equilateral triangle. Chain link fabric, 2 by 2 in. by No. 6 gate, over rock surface. Some mine ties used to support rock between bolts. T_n = 275 ft-lb, F_i = 9k, F_y = 26k	Arch 3.9 Walls 2.5	Arch 11.3 Walls 7.2	n 0.07 m 0.03	n 0.21 m 0.10	0.22	0.17	*Construction period: 1961-1964 **Shape of chambers and tunnels on this sheet consists of semicircular arch with vertical sidewalls At intersection corners of large chambers, 12-, 14-, and 18-ft-long bolts used in lieu of those shown for a distance of 16 ft from the corners
14			Power plant chamber	H = 56 B = 50 L = 132				"	"	n 0.07 m 0.04	n 0.19 m 0.11	0.20	0.18	
15			Parallel building chambers 1, 2, and 3, typ.	H = 56 B = 32 L = 600, total		Arch: 4 by 4 ft, 8 ft Walls: 5 by 5 ft, 10 ft except bottom 2-8 ft		"	"	n 0.10 m 0.04	n 0.29 m 0.11	0.25	0.18	
16			Chambers A and B between 3 and 4	H = 36 B = 45 L = 200		Arch: 4 by 4 ft, 10 ft Walls: 5 by 5 ft, Upper one-12 ft, Others - 8 ft		"	"	n 0.07 m 0.06	n 0.21 m 0.17	0.22	0.22 0.33	
17			Chamber 4	H = 36 B = 32 L = 200		Arch: 4 by 4 ft, 8 ft Walls: 5 by 5 ft, 8 ft Upper two-10 ft		"	"	n 0.10 m 0.06	n 0.29 m 0.17	0.25	0.22 0.28	
18			North access tunnel	H = 22.5 B = 29 L = 1094, total			Same as above except bolts ungrouted and no chain link fabric	3.9	11.3	n 0.11	n 0.32			North access lined with 2-ft-8-in.-thick reinforced concrete for 269 ft beginning at portal
a			Pattern No. 1 (minimum reinforcement)	L = (498)		4 by 4 ft, 3-8 ft at crown, 6 ft at haunches and one below springline						0.21 0.28	0.27	
b			Pattern No. 2	L = (328)		4 by 4 ft, 14 ft over arch and one below springline						0.48	0.62	
19			North access transition tunnel	H = 22.5-25 B = 29-45 L = 324		4 by 4 ft, three 8 ft at crown, 6 ft at haunches	Same as north access tunnel, except 192 ft covered with chain link fabric	"	"	n 0.07 0.11	n 0.21 0.32	0.18 0.28	0.24 0.27	
20			Turn-around tunnel	H = 20.5 B = 32 L = 50		4 by 4 ft, 8 ft over arch to within 5 ft of springline. 10 ft long at springline	Same as north access tunnel, except chain link fabric over entire length	"	"	n 0.10	n 0.29	0.25	0.39 0.49	

(Continued)

Table 7-1. (Continued)

Item No.	Project / Location / Responsible Agency	Designer / Construction Contractor	Location of Reinforcing Elements / Construction Period*	Excavation Dimensions (H = Height, ft; B = Width, ft; L = Length, ft; D = Depth, ft; Shape**)	Rock Properties (q_u = Unconfined Compressive Strength, γ = Unit Weight, RQD = Rock Quality Designation)	Description and Properties of Reinforcing Elements (F_y = Yield Strength of Element, T_a = Anchorage Setting Torque Used, T_n = Tensioning Torque Used, F_d = Direct Pull Tension Force Used, F_i = Estimated Initial Tensile Stress in Installed Element, S = Spacing of Elements, k = 1000 lb)		Average Confining Pressure on Rock Surface, P_i, psi ($P_i = \frac{F_i}{S^2}$ or $\frac{F_y}{S^2}$): Initial	Yield	$n = P_i/B\gamma$, $m = P_i/H\gamma$: Initial	Yield	Bolt Length / Span: $\frac{\ell}{B}$	$\frac{\ell}{H}$	Comments
21	NORAD Cheyenne Mountain Complex Colorado Springs, Colo. North American Air Defense Command	CE, Omaha District; Parsons, Brinkerhoff, Quade & Douglas; A. J. Ryan & Associates Utah Mining & Construction Co.	Central access tunnel Pattern No. 1	H = 25 B = 45 L = 591 L = (156)	See page 7-3	S, Length 4 by 4 ft, 16 ft long at crown and upper haunch, 14 ft @ midhaunch, 12 and 10 ft lower haunch	Ungrouted 1-in.-diam slot and wedge rock bolts, 3/8-in.-thick bearing plates by 8-in.-square or equilateral triangle. Chain link fabric, 2- by 2-in. by No. 6 gage, over the rock surface T_n = 275 ft-lb, F_i = 9k, F_y = 26k	3.9	11.3	n 0.07 m 0.13	n 0.21 m 0.37	0.31 0.31	0.40 0.48	*Construction period: 1961-1964. **Shape of all tunnels or adits on this sheet consists of semicircular arch with vertical sidewalls **Portions of central access tunnel were supported with steel sets and lined with reinforced concrete**
a			Pattern No. 2	L = (93)		4 by 4 ft, 10 ft @ crown, 8 ft @ upper haunch, 6 ft @ midhaunch				n 0.07	n 0.21	0.13 0.18 0.22		
b			Pattern No. 3	L = (138)		4 by 4 ft, 18 ft long over entire arch	Grouted expansion shell hollow core deformed bar rock bolts, 1-in.-diam, 3/8- by 8-in.-square bearing plates on mortar pads T_a = 250 ft-lb, T_n = 275 ft-lb, F_i = 9k, F_y = 36k	3.9	15.6	n 0.07 m 0.13	n 0.29 m 0.52	0.40	0.72	200 ft of central access tunnel reinforced with Patterns Nos. 3 and 4 was treated with 2-in. thickness of shotcrete reinforced with 2- by 2-in. by No. 12 wire mesh over entire arch
c			Pattern No. 4	L = (120)		Same as Pattern No. 3	**Same as Pattern No. 3, except solid deformed bar used**							
22			South access tunnel	H = 17.5 B = 15 L = 2171										
a			Pattern No. 1	L = (1423)		4 by 4 ft; 5 each, 6 ft long over crown and upper haunch	Ungrouted slot and wedge rock bolts, 1 in. diam. No fabric over rock surface T_n = 275 ft-lb, F_i = 9k, F_y = 26k	3.9	11.3	n 0.21	n 0.62	0.40		
b			Pattern No. 2	L = (346)		4 by 4 ft, 8 ft in arch, 6 ft in walls				n 0.21 m 0.18	n 0.62 m 0.54	0.53	0.34	270 ft of south tunnel was supported with steel sets and lined with reinforced concrete
c			Pattern No. 3	L = (132)		4 by 4 ft, 8 ft over arch and walls	Fully grouted perforated sleeve rock anchors, 1-in.-diam deformed bars. Rock surface covered with 8-in.-thick shotcrete reinforced with 1 layer 2 by 2 in. by No. 12 and 2 layers 2 by 2 in. by No. 10 wire mesh. F_y = 30k	0	13.0	0	n 0.73 m 0.62	0.53	0.46	
23			Pedestrian and access adit	H = 24 B = 32 L = 390		4 by 4 ft, 8 ft in crown and upper haunch, 6 ft at midhaunch	Ungrouted 1-in.-diam slot and wedge rock bolts. Chain link fabric over rock surface	3.9	11.3	n 0.10	n 0.29	0.19 0.25		
24			**Reservoir** adit	H = 16 B = 20 L = 246		Ungrouted 1-in.-diam slot and wedge rock bolts, 6 ft long in arch. Chain link fabric over rock arch. S = 4 by 4 ft. T_n = 275 ft-lb, F_i = 9k, F_y = 26k		3.9	11.3	n 0.16	n 0.47	0.30		
25			Reservoir access adit	H = 16 B = 32 L = 48		Same as reservoir adit, except 8-ft-long rock bolts used		3.9	11.3	n 0.10	n 0.29	0.25		

(Continued)

Table 7-1. (Continued)

Item No.	Project Location Responsible Agency	Designer Construction Contractor	Location of Reinforcing Elements Construction Period*	Excavation Dimensions H = Height, ft B = Width, ft L = Length, ft D = Depth, ft Shape**	Rock Properties q_u = Unconfined Compressive Strength, γ = Unit Weight, RQD = Rock Quality Designation	Description and Properties of Reinforcing Elements (F_y = Yield Strength of Element, T_a = Anchorage Setting Torque Used, T_n = Tensioning Torque Used, F_d = Direct Pull Tension Force Used, F_i = Estimated Initial Tensile Stress in Installed Element, S = Spacing of Elements, k = 1000 lb)	Average Confining Pressure on Rock Surface, P_i, psi $P_i = \frac{F_i}{S^2}$ or $\frac{F_y}{S^2}$ Initial	Yield	$n = P_i/B\gamma$ $m = P_i/H\gamma$ Initial	Yield	Bolt Length / Span $\frac{\ell}{B}$	$\frac{\ell}{H}$	Comments
26	NORAD Cheyenne Mountain Complex Colorado Springs, Colo. North American Air Defense Command	CE, Omaha District; Parsons, Brinkerhoff, Quade & Douglas; A. J. Ryan & Associates Utah Mining & Construction Co.	Adits A and B	H = 18.75 B = 17.5 L = 679	See page 7-7 for general rock description at NORAD Cheyenne Mountain Complex	Fully grouted hollow core deformed bar, No. 8 (1-in.-diam) 10-ft-long expansion shell rock bolts over 195 ft of adits. S = 4 by 4 ft over arch. Ungrouted solid deformed bar No. 8 expansion shell rock bolts, 10 ft long, over 484 ft of adits. S = 4 by 8 ft over arch. T_a = 250 ft-lb, T_n = 275 ft-lb, F_i = 9k, F_y = 36k	3.9 2.0	15.6 7.8	n 0.18 n 0.09	n 0.74 n 0.37	0.57		*Construction period: 1961-1964. **Shape of tunnels, adits and chambers on this sheet consist of semi-circular arch with vertical walls
27			Air exhaust tunnel	H = 12 B = 12 L = 4675									108 ft of tunnel supported with steel sets and 2-in. shotcrete on rock surface
a			Pattern No. 1	L = (3900)		Ungrouted 1-in.-diam slot and wedge rock bolts, 6 ft long. 3 bolts at 4 ft installed in upper arch at locations determined in field					0.50		
b			Pattern No. 2	L = (449)		Ungrouted slot and wedge bolts, 8 ft long in arch, 6 ft long at springline. S = 4 by 4 ft	3.9	11.3	n 0.27	n 0.78	0.67		
c			Pattern No. 3	L = (158)		Fully grouted perforated sleeve-type anchor bars. No. 8 by 8-ft-long deformed bars. S = 4 by 4 ft. Rock surface covered with 6-in.-thick shotcrete. F_y = 30k	0	13	0	n,m 0.90	0.67	0.67	Rock surface covered with 6-in.-thick shotcrete reinforced with one layer 2- by 2-in. by No. 12 and two layers of 2- by 2-in. by No. 10 welded wire mesh.
d			Pattern No. 4	L = (60)		Same as Pattern No. 3, except that at every 12-ft interval along tunnel, anchor bar length increased to 12 ft and spacing decreased to 2 ft	0	13 26	0	n,m 0.90 1.80	0.67 1.0	0.67 1.0	Rock surfaced covered with 1/2-in. minimum thickness shotcrete
28			Cooling tower chamber	H = 30 B = 45 L = 53		Fully grouted slot and wedge rock bolts, 12 ft long in arch, 10 ft long at springline, 8 ft long in wall. S = 4 by 4 ft. Shotcrete on rock surface, 2 in. thick, reinforced with one layer 2- by 2-in. by No. 12 welded wire mesh	3.9	11.3	n 0.07 m 0.11	n 0.21 m 0.31	0.27	0.27	Shotcrete, 2-in. thick, reinforced with 2- by 2-in. by No. 12 wire fabric over rock surface
29			Exhaust shaft adit	H = 34.5 B = 22 L = 36		16 ft of adit reinforced 12-ft-long fully grouted No. 11 (1-3/8-in.-diam) hollow core deformed bar rock bolts with expansion shells. T_a = 375 ft-lb, T_n = 600 ft-lb, F_i = 25k, F_y = 70k 20 ft of adit reinforced with fully grouted perforated sleeve-type rock anchors, No. 10 (1-1/4-in.-diam) by 12-ft-long deformed bars. S = 4 by 4 ft, F_y = 52k	10.8 0	30.4 22.6	n 0.41 m 0.25 0	n 1.14 m 0.70 n 0.85 m 0.52	0.55	0.35	Shotcrete, 4 in. thick, reinforced with two layers wire fabric over rock surface. Surface of shotcrete painted with epoxy
30			Central exhaust shaft 1961-1964	Vertical shaft, 12 ft square by 91 ft high		Fully grouted 1-in.-diam slot and wedge rock bolts, 6 ft long. S = 4 by 4 ft. T_n = 275 ft-lb, F_i = 9k, F_y = 26k	3.9	11.3	Does not apply		Does not apply		Surface of rock covered with 4-in. thickness of shotcrete reinforced with two layers 2- by 2-in. by 12-gage welded wire fabric

(Continued)

Table 7-1. (Continued)

Item No.	Project Location Responsible Agency	Designer Construction Contractor	Location of Reinforcing Elements Construction Period	Excavation Dimensions H = Height, ft B = Width, ft L = Length, ft D = Depth, ft Shape	Rock properties q_u = Unconfined Compressive Strength, γ = Unit Weight, RQD = Rock Quality Designation	Description and Properties of Reinforcing Elements (F_y = Yield Strength of Element, T_a = Anchorage Setting Torque Used, T_n = Tensioning Torque Used, F_d = Direct Pull Tension Force Used, F_i = Estimated Initial Tensile Stress in Installed Element, S = Spacing of Elements, k = 1000 lb)	Average Confining Pressure on Rock Surface, P_i, psi $P_i = \frac{F_i}{S^2}$ or $\frac{F_y}{S^2}$ Initial	 Yield	$n = P_i/B\gamma$ $m = P_i/H\gamma$ Initial	 Yield	Bolt Length / Span $\frac{\ell}{B}$	 $\frac{\ell}{H}$	Comments
31	NORAD Cheyenne Mountain Complex Colorado Springs, Colo. North American Air Defense Command	CE, Omaha District; Parsons, Brinkerhoff, Quade & Douglas, A. J. Ryan & Associates Utah Mining & Construction Co.	Intersection of Chamber B and Chamber 2 1963-1964	Chambers initially mined to dimension shown in Items 13 and 15, page 7-7	Intersection of two shear zones created the necessity for special reinforcement and support of the B-2 intersection See page 7-7 for general rock properties	In circular chamber sections: Fully grouted recessed rock anchors, perforated sleeve type, No. 10 (1-1/4-in.-diam) by 18-ft-long deformed bars over roof and walls to midway between springline and invert. S = 8 by 8 ft. Superposed at S = 4 by 4 ft are recessed rock anchors, No. 10 by 12 ft long in area beginning midway between invert and springline and extending to midway between springline and crown. Superposed in remaining roof at S = 4 by 4 ft are No. 8 by 12-ft-long fully grouted hollow core deformed bar expansion shell rock bolts							B-2 intersection consists of circular tunnels leading to an intersection mined with a domed roof and domed invert Recessed rock anchors were placed before enlargement of the excavation
a			Chamber B	H = B = 72 L = 58 (total in two legs) circular			Roof No. 8 3.9 No. 10 0	Roof No. 8 15.6 No. 10 5.6 Total 21.2	n 0.05 m 0	n 0.24 m 0.32	0.17 0.25	0.17 0.25	
b			Chamber 2	H = B = 63 L = 40 (total in two legs)			Walls and invert 0	Walls and invert 28.2	n 0.05 m 0	n 0.28 m 0.37	0.19 0.29	0.17 0.29	Following reinforcement of rock, intersection and chambers were lined with reinforced concrete
c			Intersection	H = 84 B = 104 (at diagonal) Invert and roof consist of spherical segments See comments		In intersection: Domed roof reinforced with recessed rock anchors, No. 10 deformed bar with perforated sleeve by 24 ft long. S = 5 ft-8 in. by 5 ft-8 in. Superposed are fully grouted hollow core deformed bar bolts, No. 8 by 8 ft long with S = 4 by 4 ft. For No. 10 deformed bars, F_y = 52k. For No. 8 rock bolts, T_a = 250 ft-lb, T_n = 275 ft-lb, F_i = 9k, F_y = 36k	No. 8 3.9 No. 10 0	No. 8 15.6 No. 10 11.2 Total 26.8	n 0.03 m 0.04	n 0.21 m 0.27	0.17 0.23	0.21 0.29	Walls created by chamfering the intersection corners are reinforced similar to domed roof except that adjacent anchor lengths alternate from 24 to 18 ft Upper portion of domed invert reinforced with 18- and 24-ft-long recessed rock anchors; No. 10 gravity grouted deformed bars, at 4-ft spacing
32			Corner of intersection, Chamber A and pedestrian adit 1963-1964	H = 60.5	Rock movement along fractures cutting and dropping into the corner required additional rock excavation and specially designed reinforcement	173 rock bolts, 24 and 30 ft long, consisting of No. 11 (1-3/8-in.-diam) deformed bars grouted in 12-ft-long perforated sleeves installed to stabilize rock at corner. Bolts tensioned with T_n = 600 ft-lb and grouted with neat cement or epoxy grout. Bearing plates: 8 by 8 by 1/2 in. set on mortar pad. F_i = 25k, F_y = 117k, S^2 varies from 4.5-36 sq ft. 36 additional fully grouted (neat cement or epoxy) hollow core deformed rock bolts used. T_a = 250 ft-lb, T_n = 375 ft-lb, F_i = 20k	Max. 39	Max. 1.80	Max. 0.54	Max. 2.47	Does not apply	0.40	After the corner was stabilized with rock bolts, consolidation grouting of open fractures was done by pumping epoxy grout through bore holes and through hollow core rock bolts. Some filling of open fractures also took place during neat cement grouting of rock bolts intersecting open fractures
33	Snowy Tumut Development, Tumut-1 Snowy Mountains, Australia Snowy Mountain Hydro-Electric Authority	Snowy Mountain Hydro-Electric Authority, Australia CITRA Enterprises	Machine hall, underground power plant 1955-1957	H = 110 B = 77 L = 306 D = 1100 Curved roof, vertical sidewalls	Biotite granite, granite gneiss q_u = 20,000 psi RQD = fair to good	Arch: Ungrouted 1-in.-diam slot and wedge rock bolt, 15 ft long. S = 4 by 4 ft. F_y = 23k		10		n 0.11	0.20		Ungrouted rock bolts installed in roof prior to installation of permanent concrete ribs
						Walls: Grouted 1-in.-diam slot and wedge, 12 ft long. S = 5 by 5 ft		6		m 0.05	0.11		

(Continued)

Table 7-1. (Continued)

Item No.	Project Location Responsible Agency	Designer Construction Contractor	Location of Reinforcing Elements Construction Period	Excavation Dimensions H = Height, ft B = Width, ft L = Length, ft D = Depth, ft Shape	Rock Properties q_u = Unconfined Compressive Strength, γ = Unit Weight, RQD = Rock Quality Designation	Description and Properties of Reinforcing Elements (F_y = Yield Strength of Element, T_a = Anchorage Setting Torque Used, T_n = Tensioning Torque Used, F_d = Direct Pull Tension Force Used, F_i = Estimated Initial Tensile Stress in Installed Element, S = Spacing of Elements, k = 1000 lb)	Average Confining Pressure on Rock Surface, P_i, psi $P_i = \frac{F_i}{S^2}$ or $\frac{F_y}{S^2}$ Initial	 Yield	n = P_i/B γ m = P_i/H γ Initial	 Yield	Bolt Length / Span $\frac{\ell}{B}$	 $\frac{\ell}{H}$	Comments
34	Eklutna Dam Anchorage, Alaska U. S. Bureau of Reclamation	U. S. Bureau of Reclamation A & B Construction Co.	Intake tunnel 1952-65		Mostly sound argillite and greywacke in rock bolted sections	Expansion shell rock bolt, 1 in. diam							Rock bolts used during early stages of excavation, but due to numerous seams, thin bedding planes and poorly cemented joints, bolts proved to be ineffective and use was discontinued
35	Gilham Dam Gilham, Ark. CE, Tulsa District	CE, Tulsa District	Outlet works tunnel 1965	H = 13 B = 13 L = 1100	Sandstone	Ungrouted expansion shell type rock bolts, 7/8-in.-diam by 6-15 ft long. Rock surface covered with 2- by 2-in. by No. 9 gage wire mesh							
36	Carters Dam Carters, Ga. CE, Mobile District	Corps of Engineers; Tippetts, Abbett, McCarthy, and Stratton Cowin & Co.	Diversion tunnel 1963	H = 23 B = 23 L = 2410 D = Horseshoe	Quartzite, phyllite q_u = 10,000-27,000 psi γ = 165-170 pcf	Ungrouted 1-in.-diam by 8-ft-long slot and wedge rock bolts, S = 6 by 4.7 ft, 6- by 6- by 3/8-in. bearing plate. F_y = 26k, T_n = 170 ft-lb, F_i = 7k	2	6	0.07	0.22	0.35		
37			4 penstock tunnels 1968	H = B = 23 L = 3044, total D = Circular	Quartzite q_u = 11,000-43,000 psi γ = 170 pcf	Ungrouted 1-in.-diam by 8-ft minimum length expansion shell rock bolts, no set spacing, 6- by 6- by 3/8-in. bearing plate, 2- by 3-in. by No. 6 wire mesh on rock surface					0.35		
38	Morrow Point Dam Montrose, Colo. U.S. Bureau of Reclamation	U. S. Bureau of Reclamation Al Johnston Construction Co., Morrison-Knudsen Co.	Underground power plant chamber 1963	H = 100 to 138 B = 57 L = 206 D = 400	Micaceous quartzite, mica schist, some pegmatite intrusions. Two shear zones present in vicinity of chamber. RQD = good to excellent q_u = 6000-16,000 psi γ = say 165 pcf	Fully grouted 1-in.-diam expansion shell rock bolts, S = 4 by 4 ft, 20 ft long in roof, 12 ft long in walls. Special reinforcement to stabilize sidewall: 9 each, 135k grouted rebars, 60-100 ft long; 25 each, 250k tendons; 27 each, 1-3/8-in.-diam rock bolts, 26-78 ft long tensioned to 60k. Based on 7500-sq-ft wall area reinforced, P_i estimated at 8 psi. Average length of specified reinforcement approximately = B		Crown Walls, 13 Spec Wall 8		n 0.20 m 0.11 Spec Wall 0.07	0.35	0.12 Spec 1.0	Special reinforcement on sidewall intersected by 1- to 5-ft-thick shear zone dipping 32 deg. A 100-ft-wide wedge moved 2 in. into chamber, along a 17-deg component of shear zone dip. Grouted rebars installed during excavation stopped movement; tendons and long bolts added after excavation was complete
39	Laurel Dam Kentucky CE, Nashville District	CE, Nashville District Feenix & Scisson	Power tunnel 1966	H = B = 24.5 L = 670 Circular	Sandstone, occasional zones of coal and shale	Fully grouted, 1-in.-diam by 12-ft deformed bar, perforated sleeve rock anchor, S = 3 ft along and 5 ft around 70 ft of tunnel	0				0.47		
					Sandstone, occasional coal stains γ = say 160 pcf	Fully grouted, 1-in.-diam by 10-ft hollow core deformed bar rock bolt, S = 5 ft along and 7 ft around 600 ft of tunnel. F = 36k	7			n 0.26	0.41		

(Continued)

Table 7-1. (Continued)

Item No.	Project Location Responsible Agency	Designer Construction Contractor	Location of Reinforcing Elements Construction Period	Excavation Dimensions H = Height, ft B = Width, ft L = Length, ft D = Depth, ft Shape	Rock Properties q_u = Unconfined Compressive Strength, γ = Unit Weight, RQD = Rock Quality Designation	Description and Properties of Reinforcing Elements (F_y = Yield Strength of Element, T_a = Anchorage Setting Torque Used, T_n = Tensioning Torque Used, F_d = Direct Pull Tension Force Used, F_i = Estimated Initial Tensile Stress in Installed Element, S = Spacing of Elements, k = 1000 lb)	Average Confining Pressure on Rock Surface, P_i, psi $P_i = \frac{F_i}{S^2}$ or $\frac{F_y}{S^2}$ Initial	 Yield	$n = P_i/B\gamma$ $m = P_i/H\gamma$ Initial	 Yield	Bolt Length / Span $\frac{\ell}{B}$	 $\frac{\ell}{H}$	Comments
40	Oroville Dam and Reservoir State of California	California Department of Water Resources Oro Dam Constructors, McNamara-Fuller	Underground power plant chamber 1964-1966	H = 88 to 137 B = 69 L = 550 D = 300 Parabolic arch roof, 24 ft high by 69 ft wide vertical sidewalls	Amphibolite, fine-to-coarse grained, generally massive, but with a slight schistosity. Steeply dipping fractures, spaced 5-20 ft apart with most 1-6 in. wide containing crushed rock, schist, and lenses of clay gouge γ = 185 pcf RQD = fair to good	Fully grouted expansion shell rock bolts, 1 in. diam by 20 ft long. S = 4 by 4 ft in roof, 6 by 6 ft in walls. Additional bolts angled across slabby rock where shears intersect roof. Surface covered with chain link fabric or steel headers. Pneumatic mortar, 4 in. thick, F_y = say 36k		Roof 16 Walls 7		n 0.18 m 0.04- 0.06	0.29	0.15- 0.23	40-ft-long rock bolts installed at junction of access tunnel with power plant chamber where large overbreak occurred Specifications called for installation of pattern bolts within 5 ft of advancing face and within 3 hr of blasting permanent surface. Later lengthened to 48 hr
41	Foster Dam	CE, Portland District	Diversion tunnel 1965	H = 32 B = 32 L = 565 Horseshoe	Basalt. Tuff at downstream portal	Ungrouted expansion shell-type rock bolts, 5/8 and 3/4 in. diam by 6-10 ft long. S = 5 by 5 ft. T_n = 250-300 ft-lb					0.19 to 0.31		Temporary tunnel
42	Blue River Dam	CE, Portland District	Diversion and regulating tunnel 1965	H = 24 B = 24 L = 7797 Horseshoe	Andesite with platy joints and vertical fault zones	Ungrouted 1-in.-diam slot and wedge rock bolts over 1547 ft of the tunnel. Length = 6-10 ft, S = 5 by 5 ft. T_n = 200-250 ft-lb					0.25 to 0.41		
43	Boundary Dam Metaline Falls, Wash. City of Seattle	Bechtel-Leeds-Hillard City of Seattle Mannix Contractors, S.G.S. Constructors, Frontier Construction Co., McLaughlin, Inc.	Underground power plant chamber 1965	H = 175 B = 76 L = 477 D = 500 Arched roof, vertical side walls	Dolomitic limestone q_u = 10,000 psi RQD = good to excellent (est.)	Fully grouted, 1-in.-diam hollow core or solid expansion shell rock bolts. In roof, S = 6 by 6 ft, length = 15 ft. Three rows of rock bolts, 15 and 20 ft long, S = 5 by 5 ft at extreme top of sidewalls. Remainder of walls bolted only where required In draft tube pillars: 8-636k tendons per pillar.							Additional 30-ft-long bolts placed in roof where joints appeared to form wedges Wire mesh and some shotcrete used Attention given to reinforcement of rock around reentrants 636k tendons placed to reinforce jointed and slickensided rock

(Continued)

Table 7-1. (Continued)

Item No.	Project Location Responsible Agency	Designer Construction Contractor	Location of Reinforcing Elements Construction Period	Excavation Dimensions H = Height, ft B = Width, ft L = Length, ft D = Depth, ft Shape	Rock Properties q_u = Unconfined Compressive Strength, γ = Unit Weight, RQD = Rock Quality Designation	Description and Properties of Reinforcing Elements (F_y = Yield Strength of Element, T_a = Anchorage Setting Torque Used, T_n = Tensioning Torque Used, F_d = Direct Pull Tension Force Used, F_i = Estimated Initial Tensile Stress in Installed Element. S = Spacing of Elements, k = 1000 lb)	Average Confining Pressure on Rock Surface, P_i, psi $P_i = \frac{F_i}{S^2}$ or $\frac{F_y}{S^2}$ Initial	Yield	$n = P_i/B\gamma$ $m = P_i/H\gamma$ Initial	Yield	Bolt Length / Span $\frac{\ell}{B}$	$\frac{\ell}{H}$	Comments
44	Ranier Mesa Nevada Test Site, Mercury Atomic Energy Commission	Fenix & Scisson, Inc. Reynolds Electrical Engineering Co.	Test Cavity I 1964-65	H = 140 B = 80 (because of cavity geometry, span is taken as 100 ft) L = diam = 120 D = 1300 Shape approximate spherical segment with plane surface inclined 68 deg from horizontal	**Alternating layers of red to yellow-white porous tuff of low intact strength.** Tuff is predominantly thick-bedded (dipping 8-15 deg) and massive, except for occasional thin beds (3-18 in.) of soft, friable white tuff q_u = 1500 psi γ = 125 pcf RQD = 95-100 percent	Grouted anchorage type rock bolts, No. 9 (1-1/8-in.-diam) deformed bars, grouted length = 7-9 ft with gypsum ("Sulfaset," Ranco F-181) liquid grout mix pumped to back of hole. Grout length controlled with use of polyfoam packer In curved surface: Roof, bar length = 32 ft, S = 3 by 3 ft Middle portion, length = 24 ft, S = 3 by 3 ft Lower portion, length = 16 ft, S = 6 by 6 ft In plane surface: Length = 24 ft, S = 6 by 6 ft Bearing plate size = 8 by 8 by 1/2 in. with cement mortar pad under plate T_n = 400 ft-lb, F_i = 30k, F_y = 59k	23 23 6 6	45 45 11 11	n,0.26 n,0.26 m,0.05 m,0.05	0.52 0.52 0.09 0.09	0.32 0.24	0.11 0.17	**In Cavities I and II: Extra bolts were spotted as required particularly where joint sets were present. Shotcrete was applied in roof to prevent drying and to support rock slabs between rock bolts. Some chain link fabric supported from** 6-ft-long perforated sleeve-type grouted **rock bolts was also placed to provide temporary support prior to placing long rock bolts**
45			Test Cavity II 1965	Same as Cavity I.	**Same as Cavity I, except high angle joint systems were much more prevalent**	**Initially reinforced the same as Cavity I. During construction, instability of the plane** face required additional reinforcement, and 48-ft-long rock bolts (same type as in Cavity I), S = 3 by 3 ft, were installed over large areas	**Same as Cavity I except in plane surface** 23	45	m,0.19	0.37		0.34	**Grouted rock bolt anchorages were used in both cavities because the tuff was incapable of supporting standard expansion shell anchors** In Cavity II, stabilization was required because deep-seated movements occurred **along the steeply dipped joint and bedding planes intersecting the plane surface. In addition to placing additional reinforcement, approximately 1100 bags of cement (neat cement grout) were pumped into open joints and fault zones**
46			Test Cavity III 1965	H = 80 B = 50 (because of cavity geometry, span is taken as 60 ft) L = 75 D = 350 Similar to Cavity I except plane surface inclined 74 deg from horizontal	Quartz monzonite, iron-stained joints form rock blocks approximately 2 ft on a side. Major joint set parallels plane face of cavity q_u = **27,000 psi** γ = **167 pcf** RQD = 63-85 percent	**Fully grouted** 1-1/8-in.-diam deformed bar rock **bolts with expansion shells. Grouted with gypsum grout after tensioning** **In curved surface:** Roof, bar length = 24 ft, S = 3 by 3 ft Middle portion, bar length = 16 ft, S = 3 by 3 ft Lower portion, bar length = 8-16 ft, S = 6 by 6 ft In plane surface, bar length = 16 ft, S = 6 by 6 ft T_n = 275 ft-lb, F_i = 20k, F_y = 59k	15 15 4 4	45 45 11 11	n,0.29 n,0.29 m,0.03 m,0.03	0.86 0.86 0.06 0.06	0.40 0.27	.13-.27 0.20	**Extra bolts placed normal to predominant joints and fault zones as required**
47	Poatina Power Station Tasmania Hydro-Electric Commission of Tasmania	Hydro-Electric Commission of Tasmania Same	Underground power plant chamber 1962	H = 85 B = 45 L = 300 D = 500 Trapezoidal arch, vertical sidewalls	Mudstone. Roof is thinly bedded, highly fossiliferous, calcareous mudstone. Remainder is massive mudstone with occasional thin shale bands. No faults; joints sealed with calcite and are watertight. In situ compressive strength = 5000 psi, measured parallel to bedding γ = 163 pcf	Fully grouted slot and wedge rock bolts Roof: 14 ft long, S = 3 by 3 ft Roof, near haunches, slabby rock: 14 ft long, S^2 = 4.5 sq ft Haunches and top of walls: 12 ft long, S = 3 by 3 ft Midheight of walls: 14 ft long, S = 3 by 3 ft Lower walls: 8 ft long, S = 3 by 3 ft Mesh and 4-in. gunite over rock surface		10 20 10 10 10		0.20 0.40 0.20	0.31 0.31	0.14 0.16 0.09	**For roof rock, q_u = 5,000 psi. saturated,** q_u = 10,000 psi, air dried. For wall rock, q_u = 16-19 ksi, air dried. 3-ft-deep stress relief slots cut at junction of flat roof and sloping haunches. Shear failure developed on horizontal bedding plane at intersection of haunch and crown and caused 1/8-in. displacement of haunch into chamber

(Continued)

Table 7-1. (Continued)

Item No.	Project Location Responsible Agency	Designer Construction Contractor	Location of Reinforcing Elements Construction Period	Excavation Dimensions H = Height, ft B = Width, ft L = Length, ft D = Depth, ft Shape	Rock Properties q_u = Unconfined Compressive Strength, γ = Unit Weight, RQD = Rock Quality Designation	Description and Properties of Reinforcing Elements (F_y = Yield Strength of Element, T_a = Anchorage Setting Torque Used, T_n = Tensioning Torque Used, F_d = Direct Pull Tension Force Used, F_i = Estimated Initial Tensile Stress in Installed Element, S = Spacing of Elements, k = 1000 lb)	Average Confining Pressure on Rock Surface, P_i, psi $P_i = \frac{F_i}{S^2}$ or $\frac{F_y}{S^2}$ Initial	 Yield	$n = P_i/B\gamma$ $m = P_i/H\gamma$ Initial	 Yield	Bolt Length / Span $\frac{\ell}{B}$	 $\frac{\ell}{H}$	Comments
48	El Toro Chile			H = 126 B = 80 L = 335	Granodiorite, orthogonal joints	Roof: Tendons, 400k, S = 20 by 20 ft, length = 49-55 ft		7		0.08	0.61-0.69		
						Roof: Rock bolts, 40k, S = 8 by 8 ft, length = 13 ft		6		0.06	0.16		
						Roof, total		13		0.14			
						Walls: Tendons, 400k, S = 20 by 20 ft, length = 50 ft		7		0.05		0.4	
49	Churchill Falls Labrador, Canada Churchill Falls Labrador Corp.	Acres Canadian Bechtel	Surge chamber 1968-1971	H = 150 B = 64 L = 760 D = Circular shaped arch, vertical walls	Gneiss, intruded by gabbro, diorite, syenite, and pegmatites Excellent quality mass rock, no major fault zones. Rock cut by persistent joint sets. Minor shear zones, joints generally planar and rough, alteration by gypsum, nematite, chlorite, and talc	Arch: Fully grouted hollow core deformed bar rock bolts with expansion shells, No. 11 (1-3/8-in.-diam) by 15-20 ft long, S = 5 by 5 ft. Rock surface covered with 2- by 2-in. by No. 10 gage wire mesh. F_i = 45k, F_y = 68k	12.4	19	n 0.16	n 0.25	0.24 to 0.31		One half of bolt pattern installed within 8 hr of blasting and to within 10 ft of working face. Remainder installed within three days following first half installation and to within 60 ft of working face. In poor rock, bolts specified to be installed within 5 ft of working face In walls, bolts were angled 20 deg from vertical and 20 deg from horizontal to enable reinforcement of maximum number of rock joints Direct pull tensioning or by torquing specified, but contractor elected to tension bolts by torquing. Torque values up to 1000 ft-lb used to achieve tension loads of 45k in 1-3/8-in.-diam bolts
						Walls: Fully grouted (1-1/8-in.-diam), expansion shell type solid, bar rock bolts. Length = 15-25 ft, S = 7 by 7 ft. F_i = 30k, F_y = 45k. No mesh on walls	4.2	6.4	m 0.02	m 0.04		0.10 to 0.16	
50			Powerhouse chamber 1968-71	H = 145 B = 81 L = 1000 Circular shaped arch, vertical walls	q_u = 16,000 psi γ = say 170 pcf RQD = > 94 percent	Arch (15 ft each side of crown): Fully grouted No. 11 hollow core, S = 10 by 5 ft, length = 25 ft. Superposed on No. 11 pattern are fully grouted No. 9 solid rock bolts, S = 10 by 5 ft, length = 15-25 ft	6.2 4.2 10.4	9.3 6.3 15.6	n 0.14	n 0.21	0.31 0.18 to 0.31		Additional rock bolts, up to 75 ft long, installed and tensioned to 60k in surge chamber wall to prevent possible movement of rock wedges formed by intersection of joint sets and "foliation shears" dipping about 50 deg into excavation
						Remainder of Arch: Same as above, except No. 11 bolts are 20 ft and No. 9 bolts are 15-20 ft. Entire arch covered with 2- by 2-in. by No. 10 wire mesh	10.4		n 0.14	n 0.21	0.18 to 0.24		
						Walls: Fully grouted No. 9 solid bar rock bolts, S = 7 by 7 ft, length = 15-20 ft	4.2	6.4	m 0.02	m 0.04	0.10 to 0.14		
51			Transformer gallery 1968-71	H = 55 B = 45 L = 800 D =		Arch: Fully grouted No. 11 hollow core rock bolts, S = 5 by 5 ft, length = 15 ft. Arch covered with 2- by 2-in. by No. 10 wire mesh	12.4	19	n 0.24	n 0.35	0.33		
52	Libby Dam, Flathead Tunnel Libby, Mont. Great Northern Railroad	CE, Seattle District	Railroad tunnel 1969	H = 29 B = 21 L = Semihorseshoe	Argillite moderately fractured to blocky, with local zones of closely shattered rock	Ungrouted, 3/4-in.-diam, high-strength, expansion shell-type rock bolts, S = 4 by 5 ft, length = 10 ft in arch, 6 and 8 ft in walls, welded wire fabric over arch. T_n = 200-250 ft-lb, F_i = 8k, F_y = 21.5k							Bolts were installed right after blasting and carried within 5 ft of face before shooting next round

(Continued)

Table 7-1. (Continued)

Item No.	Project Location Responsible Agency	Designer Construction Contractor	Location of Reinforcing Elements Construction Period	Excavation Dimensions H = Height, ft B = Width, ft L = Length, ft D = Depth, ft Shape	Rock Properties q_u = Unconfined Compressive Strength, γ = Unit Weight, RQD = Rock Quality Designation	Description and Properties of Reinforcing Elements (F_y = Yield Strength of Element, T_a = Anchorage Setting Torque Used, T_n = Tensioning Torque Used, F_d = Direct Pull Tension Force Used, F_i = Estimated Initial Tensile Stress in Installed Element, S = Spacing of Elements, k = 1000 lb)		Average Confining Pressure on Rock Surface, P_i, psi $P_i = \frac{F_i}{S^2}$ or $\frac{F_y}{S^2}$ Initial	Yield	$n = P_i/B\gamma$ $m = P_i/H\gamma$ Initial	Yield	Bolt Length / Span $\frac{\ell}{B}$	$\frac{\ell}{H}$	Comments
53	NORAD Cheyenne Mountain Complex Addition Colorado Springs, Colo. North American Air Defense Command	CE, Omaha District Tiro Construction Corp.	Air intake delay path tunnel	H = B = 18 L = 502 **Circular**	See page 7-7 for general rock description at NORAD Cheyenne Mountain Complex	No. 8 by 10-ft rock bolts, S = 4 by 4 ft over upper 204-deg arc of tunnel surface. Tensioned by direct pull or by nut torquing. Chain link fabric over a portion of roof	Rock bolts: Fully grouted hollow core deformed bar, expansion shell type rock bolts, No. 8 (1-in.-diam) with 8- by 8- by 3/8-in. bearing plates. T_a = 325 ft-lb, F_y = 36k, T_n (where used) = 350 ft-lb, F_d (where used) = 24k, F_i = 20k Rock anchors: No. 10 (1-1/4-in.-diam) deformed bars grouted full length with use of 1-3/4-in.-diam perforated sleeves in 2-in.-diam drill holes. Generally recessed behind trim burden prior to shooting trim round. Trim burden varied from 15- to 45-in. thickness. F_y = 51k Rock anchors: No. 8 (1-in.-diam) deformed bars grouted full length with use of 1-1/4-in.-diam perforated sleeves in 1-1/2-in.-diam drill holes. F_y = 32k Chain link fabric: 2 by 2 in. by No. 6	8.7	15.6	n 0.4 m 0.4	n 0.72 m 0.72	0.56	0.56	The NORAD Cheyenne Mountain Complex was constructed during 1961-1964. During 1971, the size of the complex was increased by mining additional chambers, tunnels and shafts to house a new power plant, air-cooling and air-handling facilities Rock bolts were placed and tensioned to within 2 ft min. and 10 ft max. of heading. Initially, rock bolts were retensioned and grouted to within 10 ft min and 30 ft max of heading. This procedure was modified to allow grouting of bolts immediately after the first tensioning
54			Air exhaust delay path tunnel and raise	H = B = 19 L = 374 **Circular**		No. 8 by 10-ft rock bolts, S = 4 by 4 ft over upper 194-deg arc of tunnel surface and over 360 deg of raise. Tensioned by torquing nut		8.7	15.6	n 0.38	n 0.68	0.53		
55			Air exhaust valve chamber	H = 27 B = 20 L = 140 Semicircular **roof, vertical walls**		No. 8 by 10-ft-long rock bolts, S = 4 by 4 ft over roof and walls. Tensioned by direct pull or by nut torquing. 30 ft of chamber rock covered with chain link fabric		8.7	15.6	n 0.36 m 0.22	n 0.65 m 0.48	0.50	0.37	
56			Cooling tower chamber	H = 45 B = 38 L = 185 Semicircular roof, vertical **walls**		No. 10 by 16-ft-long recessed rock anchors installed over 135-deg arc of roof. Installed inclined at 60 deg from direction of excavation advance. S = 4.67 by 4.67 ft		0	16.3			0.42		
						After shooting trim round, No. 8 by 16-ft-long rock bolts installed and tensioned by direct pull in roof. S = 4.67 by 4.67 ft		$\frac{+6.4}{6.4}$	$\frac{+11.5}{27.8}$	n 0.14	n 0.6	0.42		
						Walls: No. 8 by 16-ft rock bolts, S = 4.67 by 4.67 ft tensioned either by direct pull or by nut torquing		6.4	11.5	m 0.12	m 0.21	0.36		
						Chain link fabric over roof and 65 percent of wall area								

(Continued)

Table 7-1. (Continued)

Item No.	Project Location Responsible Agency	Designer Construction Contractor	Location of Reinforcing Elements Construction Period	Excavation Dimensions H = Height, ft B = Width, ft L = Length, ft D = Depth, ft Shape	Rock Properties q_u = Unconfined Compressive Strength, γ = Unit Weight, RQD = Rock Quality Designation	Description and Properties of Reinforcing Elements (F_y = Yield Strength of Element, T_a = Anchorage Setting Torque Used, T_n = Tensioning Torque Used, F_d = Direct Pull Tension Force Used, F_i = Estimated Initial Tensile Stress in Installed Element, S = Spacing of Elements, k = 1000 lb)	Average Confining Pressure on Rock Surface, P_i, psi $P_i = \frac{F_i}{S^2}$ or $\frac{F_y}{S^2}$ Initial	 Yield	n = $P_i/B\gamma$ m = $P_i/H\gamma$ Initial	 Yield	Bolt Length / Span $\frac{\ell}{B}$	 $\frac{\ell}{H}$	Comments
57	NORAD Cheyenne Mountain Complex Addition Colorado Springs, Colo. North American Air Defense Command	CE, Omaha District Tiro Construction Corp.	Power plant chamber	H = 53 B = 67 L = 172 Circular segment > semicircle	See page 7-7 for general rock description at NORAD Cheyenne Mountain Complex	No. 10 by 16-ft-long recessed rock anchors installed over 110-deg arc of roof. Anchors inclined at 60 deg from direction of advance. S = 4.67 by 4.67 ft No. 8 by 24-ft-long rock bolts spaced at 4.67 by 4.67 ft installed over roof and walls after shooting trim rock. Bolts tensioned by direct pull Chain link fabric over roof and 35 percent of wall area below springline	0 +6.4 6.4	16.3 +11.5 27.8	 n 0.08 m 0.10	 n 0.34 m 0.43	0.24 0.36	 0.45	See page 7-23 for general description of reinforcing elements used in the NORAD Cheyenne Mountain Complex Addition
58			Power plant access adit	H = 25 B = 32 L = 113 Semicircular roof, vertical walls		Typical Section: No. 8 by 10-ft-long rock bolts spaced at 4.67 by 4.67 ft over arch and one row in walls below springline. Chain link fabric over arch	6.4	11.5	n 0.17 m 0.21	n 0.30 m 0.38	0.31	0.4	
59			Cooling tower access adit	H = 16.5 B = 12 L = 120 Semicircular roof, vertical walls		No. 8 by 8-ft-long rock bolts spaced at 4 by 4 ft over arch and one row in walls below springline	8.7	15.6	n 0.6 m 0.44	n 1.1 m 0.78	0.67	0.49	
60			Air intake shafts	Vertical shafts, 8 ft diam 3 at 46 ft Total L = 138		No. 8 by 6-ft-long rock anchors. S = 4 by 4.2 ft Shafts were later steel lined	8.3	14.9	Does not apply		Does not apply		Shafts mined mechanically by raised bore drilling
61			Air exhaust shafts	Vertical shafts 8 ft diam 4 at 31 ft Total L = 124		Bottom 14 ft of each shaft: No. 8 by 6-ft-long rock bolts tensioned by torquing nut. S = 4 by 4.2 ft Remainder of each shaft: No. 8 by 6-ft-long rock anchors. S = 4 by 4.2 ft. Top 15 ft of each shaft was later steel lined	8.3 0	14.9 13.2	Does not apply		Does not apply		Shafts mined mechanically by raised bore drilling
62			Pipe adit No. 1	H = 12 B = 12 L = 47		No. 8 by 8 ft rock bolts, S = 4 by 4 ft over arch and walls	8.7	15.6	n 0.6 m 0.6	n 1.1 m 1.1	0.67	0.67	

(Continued)

Table 7-1. (Concluded)

Item No.	Project Location Responsible Agency	Designer Construction Contractor	Location of Reinforcing Elements Construction Period	Excavation Dimensions H = Height, ft B = Width, ft L = Length, ft D = Depth, ft Shape	Rock Properties q_u = Unconfined Compressive Strength, γ = Unit Weight, RQD = Rock Quality Designation	Description and Properties of Reinforcing Elements (F_y = Yield Strength of Element, T_a = Anchorage Setting Torque Used, T_n = Tensioning Torque Used, F_d = Direct Pull Tension Force Used, F_i = Estimated Initial Tensile Stress in Installed Element, S = Spacing of Elements, k = 1000 lb)	Average Confining Pressure on Rock Surface, P_i, psi $P_i = \frac{F_i}{S^2}$ or $\frac{F_y}{S^2}$ Initial	Yield	$n = P_i/B\gamma$ $m = P_i/H\gamma$ Initial	Yield	Bolt Length / Span $\frac{\ell}{B}$	$\frac{\ell}{H}$	Comments
63	**NORAD Cheyenne Mountain Complex Addition** **Colorado Springs, Colo.** **North American Air Defense Command**	CE, Omaha District **Tiro Construction Corp.**	**Pipe** adit **No.** 2	H = 18 B = 12 L = 48	See page 7-7 for general rock description at NORAD Cheyenne Mountain Complex	No. 8 by 10-ft rock bolts, S = 4 by 4 ft, and chain link fabric over arch and walls	8.7	15.6	n 0.6 m 0.4	n 1.1 m 0.72	0.83	0.56	See page 7-23 for general description of reinforcing elements used in the NORAD Cheyenne Mountain Complex Addition
64			New to old power plant adit	H = 25 B = 20 L = 57		No. 8 by 10-ft rock bolts, S = 4.67 by 4.67 ft, and chain link fabric over arch and walls	8.7	15.6	n 0.36 m 0.29	n 0.65 m 0.52	0.5	0.4	
65			Diesel exhaust tunnel (2)	H = 12 B = 12 L = 87 + 78=165		No. 8 by 8-ft rock bolts, S = 4 by 4 ft, over arch with one row in walls below springline	8.7	15.6	n 0.6 m 0.6	n 1.1 m 1.1	0.67	0.67	
66			**Combustion** air tunnel	H = 6.25 B = 6.25 L = 84 **Circular**		No. 8 by 6-ft rock bolts, 3 bolts per station in roof, S = 4 by 4 ft	8.7	15.6	n 1.1	n 2.1	0.96		
67	**Northfield Mountain Pumped Storage Project** **Northfield and Irving, Franklin County, Mass.** Connecticut Light **& Power Co., The Hartford Electric Light Co.,Western Massachusetts Electric Company**	**Stone and Webster Engineering** Corp. **Morrison-Knudsen-Northfield Associates**	Underground powerhouse chamber 1968-1970	H = 155 B = 70 L = 328 Circular arch roof (19-ft rise), vertical walls	**Interbedded layers of gneiss and schist with varying amounts of quartz and mica. Two major joint sets, generally widely spaced, dipping steeply and striking** NE and NW predominant q_u = 16,000-22,000 psi γ = say 175 pcf	Roof: 1-in.-diam high strength expansion shell fully grouted rock bolts, S = 5 by 5 ft, 35 ft long in central part of arch, 25 ft long in lower arch. Bolts tensioned by direct pull Walls: Same except top row of bolts, 20 ft long, remainder are 16 ft long. Bolts tensioned by torquing the nut							
68	**Bloomington Lake Dam** Mineral County, W. Va./Garrett County, Md. CE, Baltimore District	CE, Omaha District **L.G. Defelice, Inc.**	Outlet works tunnel 1973-74	H = B = 19.33 L = 1619 D = 200 **Circular**	Sandstone, thin medium bedded (1-4 ft), fine-to-medium grained sub-graywacke, lightly jointed q_u = 30,000 psi γ = 165 pcf Shale, sandy-clayey	Fully grouted rock bolts, 12 ft long, consisting of No. 8 (1-in.-diam) deformed bars anchored in 1-1/2-in.-diam holes with the use of 1-1/4- by 12-in.-long polyester resin cartridges. Approximately 4-ft fast-setting grout at back of hole set (hardened) prior to stressing bars. Slower setting grout in remainder of hole. Fy = 39k, Fd = 36k, Fi = 30k, S = 4 by 4 ft over top 135 deg of tunnel surface. Rock bolts anchored primarily in sandstone Fast-setting grout: 1-min set Slow-setting grout: 25 to 30-min set	10.8	14.0	n 0.49	n 0.63	0.62		Rock bolts carried to 1-4 ft of heading. Bolts installed, tensioned and grouted prior to shooting next advance Portal face immediately above and extending to 45 ft above portal crown reinforced with No. 11 (1-3/8-in.-diam) rock bolts, 25-50 ft long, S = 10 by 10 ft, installed downward from face at 45 deg from horizontal. Deformed bars fully grouted with 1-1/2-in.-diam polyester resin cartridges installed in 1-3/4-in.-diam holes. Fd = 70k. Tunnel lined with 1-1/2-in.-thick reinforced concrete following excavation and rock reinforcement

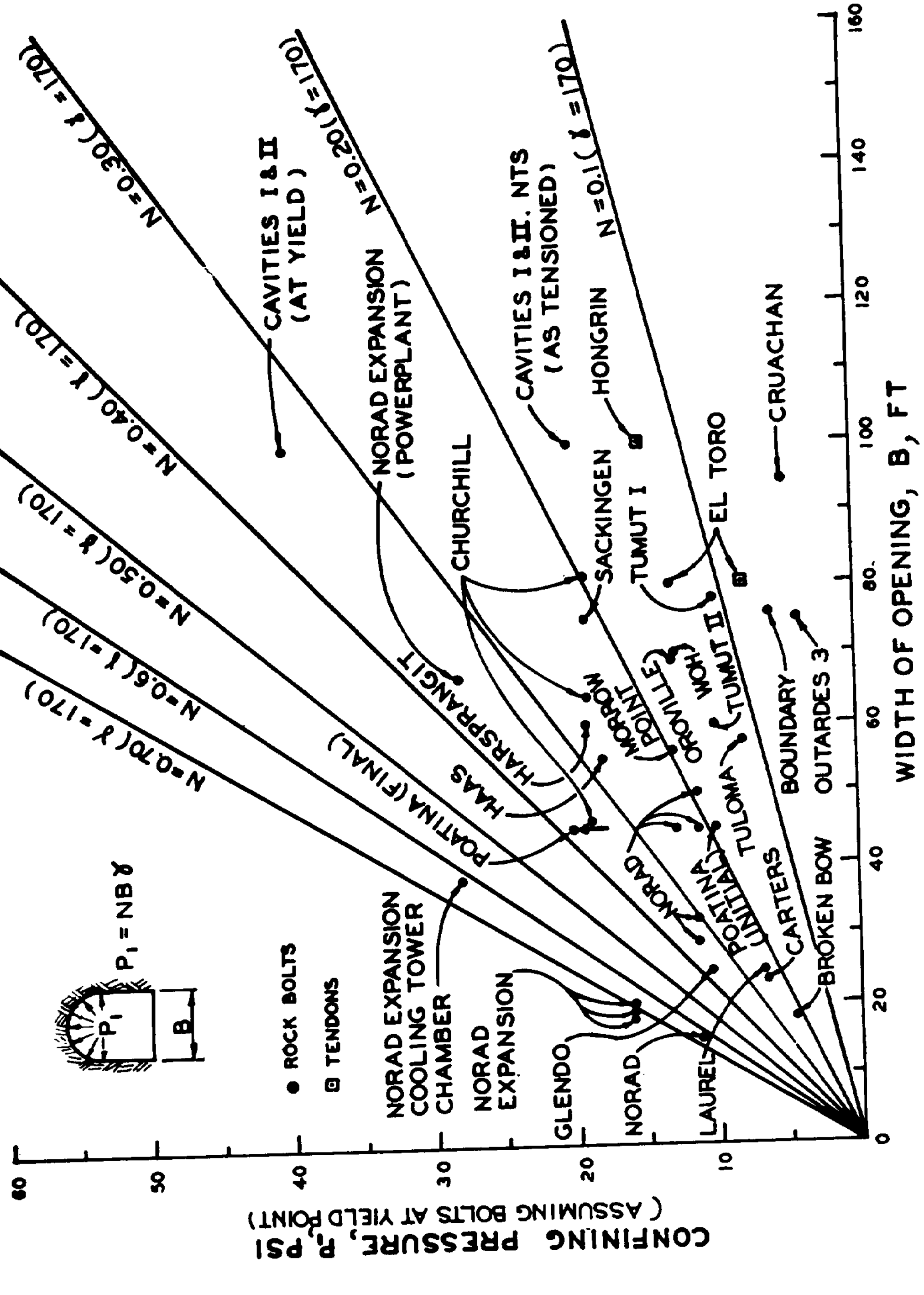


Figure 7-1. Confining pressures used in crown of caverns.

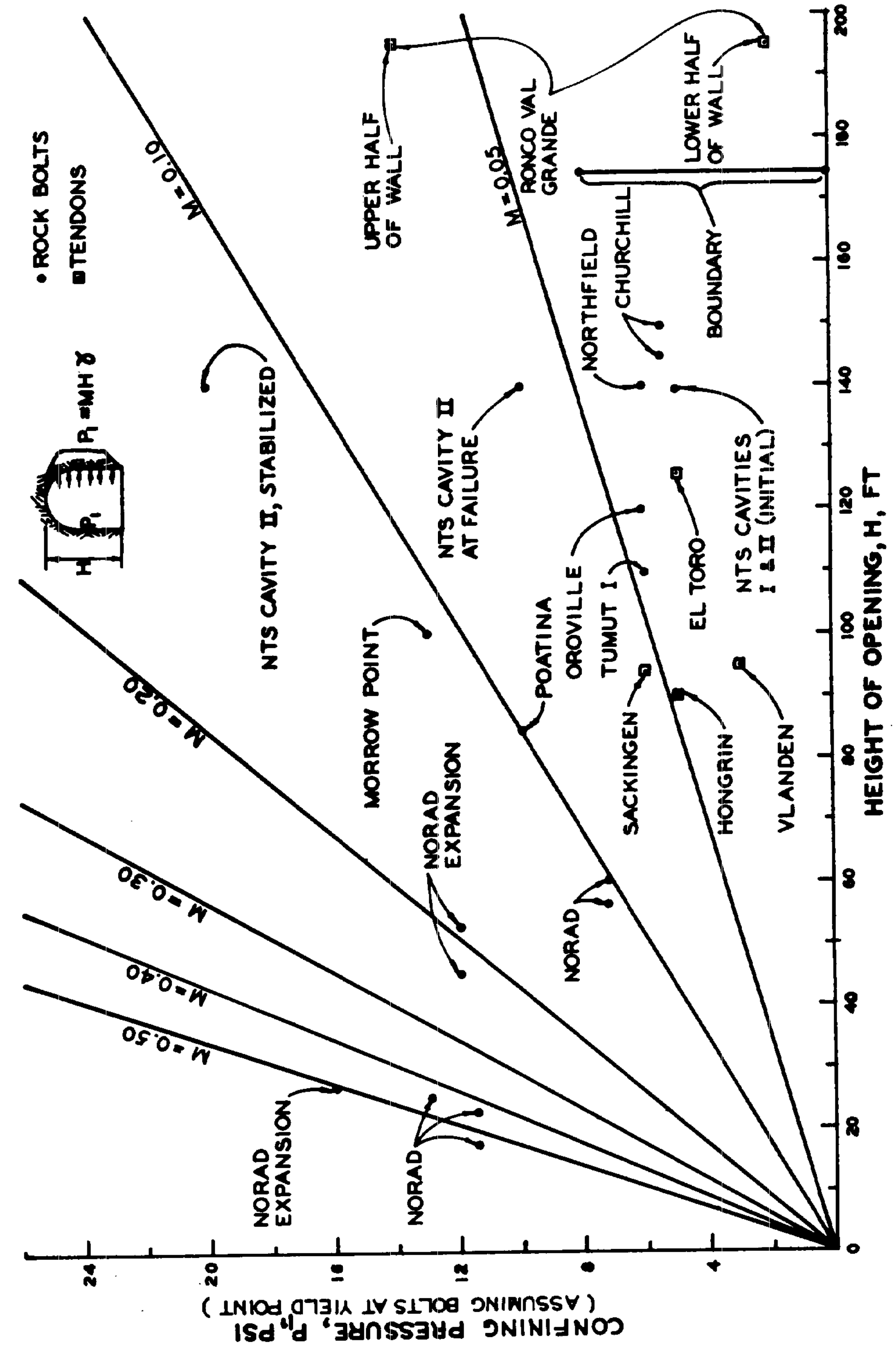


Figure 7-2. Confining pressures used on cavern walls.

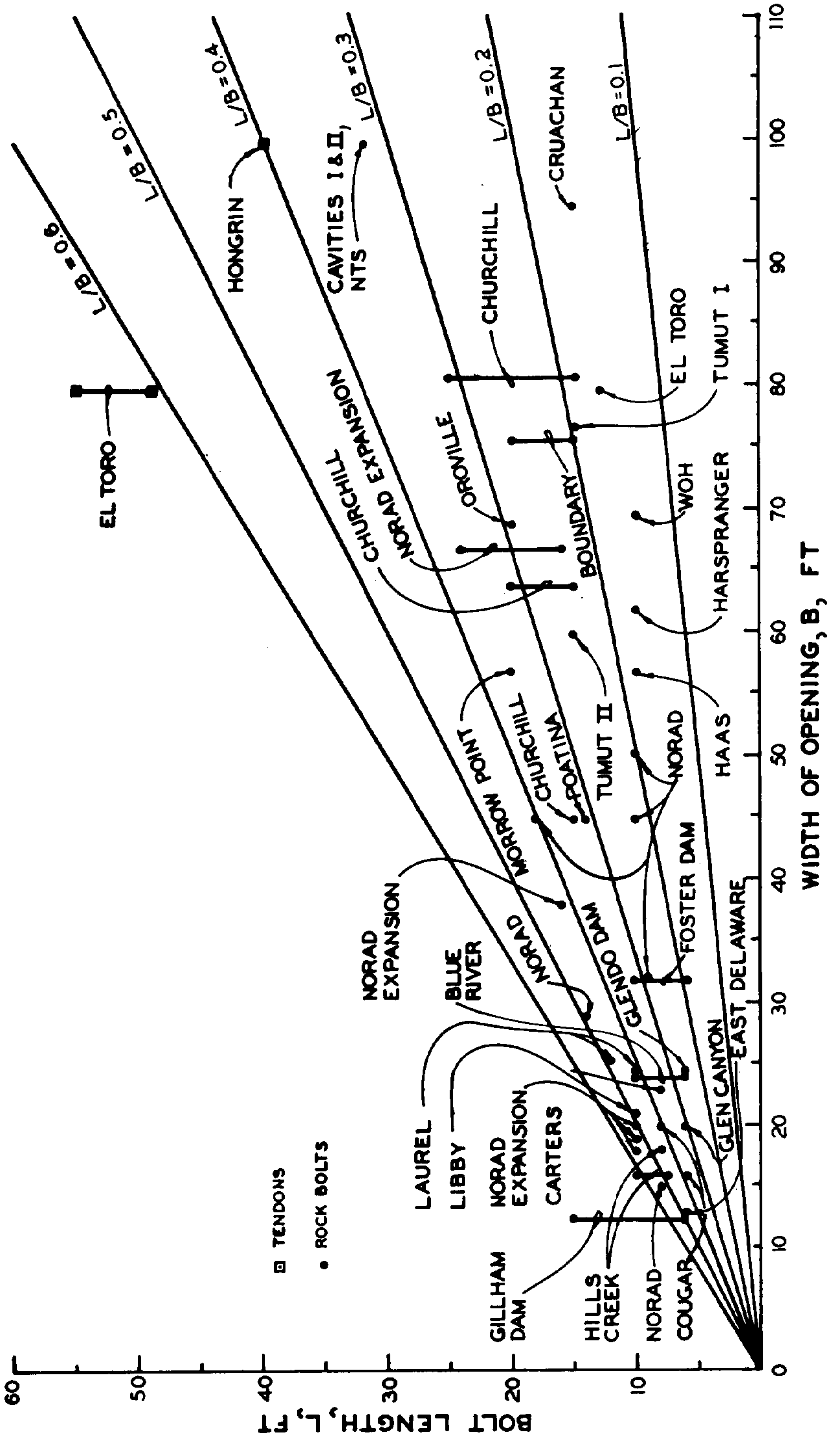


Figure 7-3. Bolt lengths used in crowns of caverns.

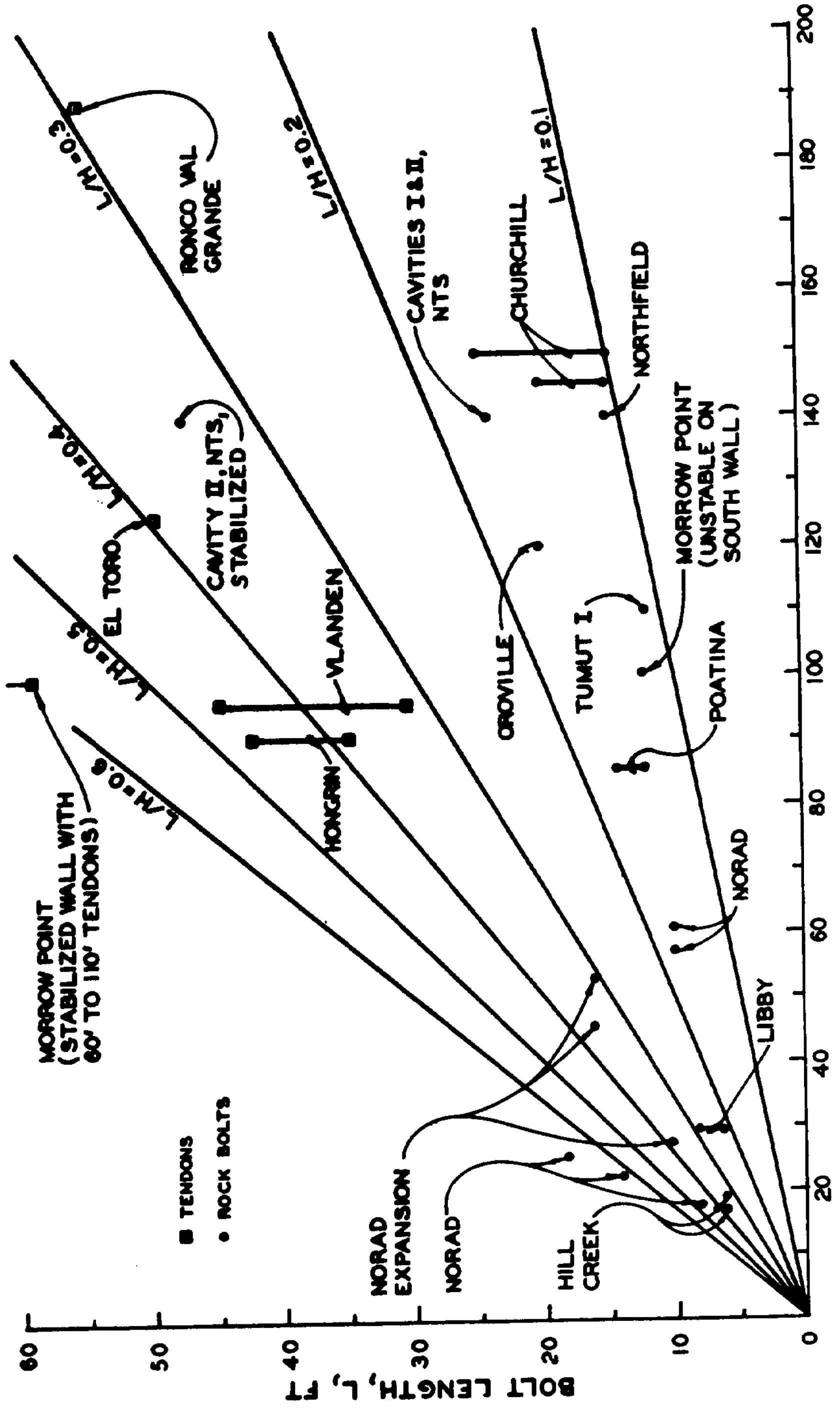


Figure 7-4. Bolt lengths used on cavern walls.

APPENDIX A
BACKGROUND AND HISTORY OF ROCK REINFORCEMENT

A-1. The use of rock bolts and anchor bars to reinforce rock was initiated by the mining industry. Although the exact origin is unknown, it is said that a form of the slot and wedge type bolt was in existence[50] during the days of the Roman Empire. In more recent times, isolated instances of the use of rock bolts or pins to attach insecure rock to secure rock have been reported in the late Nineteenth Century. In North Wales prior to 1890 steel pegs or bolts were used for reinforcing overhanging brows in slate quarries. In the United States, rock (roof) bolts were reportedly used in coal mine roofs as early as 1905. In 1917 rock bolts were successfully installed over the main haulway in the Sagamore mine of the Pocahontas Fuel Company. The bolt installation was still intact when studied by the U. S. Bureau of Mines some 30 years later (Gibson[31]).

A-2. The first known published account of bolting was a German article titled, "Versuche und Verbesserungen geim Bergwerksbetriebe in Preussen Wahrend des Jahres 1918,"[51] which described experiments made in mines in Upper Silesia prior to the end of World War I. This short article discussed the use of bolts in conjunction with the support of concrete reinforced roadways in longwall areas and the bolting of weak shale or self-supporting sandstone above. The experiments were halted by the war and were not resumed.

A-3. The St. Joseph Lead Company, operating in southeastern Missouri, is credited with being the first large mining company to demonstrate the practicability of reinforcing mine roofs through the use of systematic bolting. The interest in rock bolting was largely due to the introduction of mechanical, full-revolving loading shovels which required maneuver room free of the conventional timber posts used to support bad ground in the roof of stopes. The results of this work, which began in the late 1920's, included the development of a technique for reinforcing the rock below the natural arch line and anchoring it to the solid rock above the opening and to the rock above the pillars. In this technique, lengths of 4-inch channel iron were bent to conform to the immediate contours of the mine roof and bolted to the roof with 1-inch-diameter slot and wedge bolts (6 to 10 feet long) anchored in firm rock above the insecure rock. The bolts were installed inclined to the roof through holes in the channels at about 4-foot centers. In average bad ground, the channels were placed 5 to 10 feet apart. Because the bedded dolomitic rock formation had numerous randomly oriented vertical slips, the suspension supports were sometimes placed to cross a maximum number of slips rather than parallel to each other at constant distance intervals. The combination of channel iron and rock bolts later became known as "suspension roof supports." Small areas of insecure roof were suspended with the use of square bearing plates and single rock bolts placed either vertically or at an angle. The single bolt installations were known as "shin plasters."

A-4. In this early work, the success of the suspension supports in preventing roof failure was attributed to two functions. One was the suspension of the insecure rock below the natural arch from the secure rock above to prevent loosening of the insecure rock. The other recognized that roof failure was progressive in action. Quoting from W.W.Weigel's article,[53] "If the lower layers are caught and held tight, the upper ones do not cause trouble. The succession of thin layers (of rock) thus become one thick, heavy beam of sufficient strength to carry from pillar to pillar." The same article also described the strengthening of pillars by bending channel irons around the pillars and then bolting the channels to the pillars with 4- to 6-foot-long bolts.

A-5. Other mining companies were also installing or experimenting with rock bolts and rock anchors during the same period of time. The Homestake Mining Company in South Dakota used steel pins grouted into holes to reinforce the hanging walls of shrinkage stopes during the 1920's. The Empire Zinc Company at Gillman, Colorado, used rock bolts to support the large openings which housed its underground mill constructed during the late 1920's. The Anaconda Mining Company of Butte, Montana, began making rock bolt installations in 1939. In that year, an exhaust air crosscut was rock bolted and shotcreted at the time it was driven. The crosscut was still being used to exhaust hot humid air from the 3400 level of the Belmont mine many years later. In 1942, rock bolting experiments were carried out at the Washington-Glebe Colliery in England, but were not entirely successful. In the Forchaman Colliery in South Wales, a 60-foot-long section of roadway roof was systematically rock bolted with slot and wedge bolts in 1944. The work was terminated because of a shortage of materials, but the installation was still in good condition in 1958.

A-6. Examples of early attempts to devise mechanical anchorages for firmly anchoring rock bolts are interesting. The external area around the slot was sometimes roughened with chisel cuts with the hope of improving the holding power. Circumferential beads were also sometimes welded near the back of the bolt to increase the bolt diameter for use in 1-1/2-inch diameter holes. Both of these techniques were later discarded. A technique was developed during the same period of time by the Missouri Portland Cement Company at Sugar Creek, Missouri, for improving the slot and wedge anchorage where soft rock was encountered. After the slotted rod was driven over the wedge, a steel tube, 8 to 14 inches long and with three fourths of its length slit into four sections, was slipped over the rod and driven tightly between the wedged end and the rock to reinforce the anchorage.

A-7. Although slot and wedge bolts were the most common in this early work, others were experimenting with expansion shell anchorages. In a series of articles appearing in Colliery Engineering in 1945 and 1946, Z. S. Beyl, a mining engineer of Delft, Holland, proposed a method of reinforcing the roof in longwall mining that incorporated the use of vertical rods with expansion shell anchors. Beyl's method was based on experiments made in British mines during World War II. An early use of specially designed expansion shells was in the No. 7 mine of the Consolidated Coal Co., Staunton, Illinois.

A-8. Following the publishing of Mr. Weigel's article in 1943, a great deal of interest in rock bolting applications was generated throughout the mining industry. Because of the steel shortage during World War II however, further development was delayed until 1947. At this time, the U. S. Bureau of Mines, in coordination with the mining industry and State agencies, initiated rock bolt developmental work in an effort to reduce the existing high rate of accidents resulting from roof falls. By May 1949, 114 mines were involved in experimental rock bolting work, primarily in connection with suspension roof supports. Research into the theory and practice of rock bolting, as well as other means of roof control, was by now also being conducted by the Bureau of Mines and other agencies. The results of this work demonstrated that rock bolting offered the mining industry a safe, efficient method of roof support. This is not to say that all roof problems could be solved through the application of rock bolts. It became apparent that careful study of the rock conditions, accompanied with closely supervised experimentation and rock bolt installation, greatly increased the chances of making successful installations. The failures that did occur served to stimulate further interest among the researchers and members of the mining industry.

A-9. Beginning with 1949, rock bolts began replacing timber supports in U. S. mines at a rapid rate. By the end of 1952, over 2,000,000 rock bolts per month were being installed. By 1954, 800 mines had adopted rock bolting for systematic reinforcement. By 1957, over three million rock bolts per month were being installed. Almost 90 percent of these were being installed in bituminous coal mines east of the Mississippi River. Of the remaining 10 percent used in the western United States, two thirds were being used in coal and nonmetallic mines, and one third in metal mines.[46] In Canada, systematic bolting of rooms with slot and wedge bolts began in 1950 in Breton coal and metal mines and developed rapidly in the middle 1950's. In Europe the mining of coal generally by the longwall system provided less opportunity for applying rock bolts than did the room-and-pillar system used in American mines. In 1958 approximately 33,000 bolts were used in the Saar Basin, 112,000 in France, and 250,000 in Great Britain.[54] Most of these were used in strenthening the roof of roadways. In the metal mines of Lorraine, over 100,000 rock bolts per month were being used in 1959.[33] Rock bolts were reported being used in the Kolar Gold Field, South India, at depths of 9,000 to 10,000 feet prior to 1957.[41] In South Africa, expansion shell rock bolts were used on a minor scale as far back as 1951. In 1964, 120,000 to 140,000 rock bolts per month were being used in South African mines.[49]

A-10. By 1957, rock bolting had been generally adopted in U. S. mines and had gained acceptance in many other countries. The "explosion" in rock bolt use over a span of a few years, after laying dormant for so long, can be attributed to other significant mining developments.

A-11. For centuries prior to the early 1950's, the mucking of ore and fractured material had always been done by hand. Following the introduction of pneumatic loaders in the United States, the mechanization of mining equipment gained momentum so that by 1950, many mines throughout the world had discontinued manual mucking. Mechanized mining equipment created a need for maneuverability space which was satisfied by the substitution of rock bolts for timbering.

A-12. The time and expense involved in drilling holes for installing rock bolts, however, was another matter. Although a steam driven rotary drill had been invented as early as 1813, the first major use of mechanical drills was made in the 1860's in the Frejus, the first long Alpine tunnel, and the Hoosac, a long railroad tunnel in Massachusetts.[45] The difficulties encountered in these early projects directed the interest of numerous inventors in Great Britain, Germany, Italy, France, and the United States to the development of rock drilling machines. Many improvements were made in the years that followed. Up until 1945 the drilling of rock was still time-consuming in spite of almost a century of developmental work. In that year a historic conversion from alloyed steels to tungsten carbide steel for drill bits took place in a Swedish power tunnel. Tungsten carbide was almost unknown at the start of World War II, except in Germany where it was being used primarily to speed up machine tool operations. Tungsten carbide tipped steels, first tried in German mines in 1928, and developed into a stable production item in Sweden in the late 1940's, were supplanting steel drills in mines throughout the world in 1950. Holes for installing rock bolts could be drilled much more rapidly and less expensively. With this impetus the phenomenal growth of rock bolt application and manufacturing was on its way.

A-13. The quantity of use of a method is not necessarily related to progress in its development and application. Although the coal mining industry was consuming over 90 percent of the rock bolts used in the United States in the 1950's, the contributions made in connection with the metal mining industry were equally significant. By the same token the relatively small number of rock bolts used in Europe and other parts of the world did not limit the number of rock reinforcement techniques that came from those countries.

A-14. The work done by the U. S. Bureau of Mines and other agencies in connection with the reinforcement of mine roofs did not go unnoticed by the designers and builders of civil engineering structures. Just as the miners had adapted the techniques developed by the tunnelers of using explosives and mechanized equipment to drill, breakup, and haul away rock, so did the tunnelers adapt mining rock reinforcement techniques to their uses.

A-15. The first major use of rock bolts in civil engineering underground installations was in the Keyhole Dam Diversion Tunnel[40] in Wyoming in 1950, and in one 238-foot section of the 6-mile Duchesne Tunnel in Utah, both U. S. Bureau of Reclamation projects. At the Keyhole Dam, 1-inch diameter by 6-foot-long slot and wedge type bolts were used to tie loose blocks of rock in the roof of the 650-foot-long outlet tunnel driven through sandstone. At the start of tunneling operations, four bolts were installed at 4-foot intervals along the tunnel. After improved rock conditions were encountered and after experimenting with the bolt spacing, the use of two rock bolts for each 6 feet of tunnel proved satisfactory. Additional bolts were installed where seams or joints angled across the tunnel. In this project 27 longer rock bolts were also installed to reinforce the rock at the outlet portal face. At the Duchesne tunnel, 1-inch-diameter by 5- and 6-foot slot and wedge bolts spaced at 4 to 5 feet were used to control "popping" and slabbing of the rock in the tunnel. The U. S. Bureau of Mines provided assistance in both cf these projects.

A-16. The largest early installation of rock bolts in a tunnel, from 1950 to 1952, was in the east Delaware Tunnel, part of New York's Delaware Aqueduct, where 1-inch-diameter, 6-foot slot and wedge type bolts and steel channels were used in approximately 3-foot centers to stabilize the rock in over 30,000 feet of tunnel.[40] The number, location, and spacing of bolts were varied to meet the condition of the generally flat-bedded red shale and thinly laminated gray sandstone that existed in the tunneling zone.

A-17. The use of rock bolting for permanent support was greatly advanced by construction of the Snowy Mountain Scheme[39] in Australia between 1952 and 1962. Experimental work led to greater acceptance of rock reinforcement as permanent support. The successful use of grouted slot and wedge and hollow core groutable rock bolts provided a strong argument for future use of permanent reinforcement in tunnels and large caverns.

A-18. The Haas Hydroelectric Power Project, California, was the first large underground power plant built in the United States. Constructed in 1957, the rock of the underground chamber was reinforced with fully grouted untensioned deformed bar elements.

A-19. In 1961 and 1962 an underground complex was mined in Cheyenne Mountain near Colorado Springs, Colorado, for the NORAD defense installation.[52] Rock bolts and anchors were used almost exclusively for stabilization of the jointed granite around the chambers. Experience gained on this project provided impetus to the use of fully grouted rock reinforcement on many projects in the United States.

A-20. The construction of other large underground power plant chambers followed in the 1960's. These were the Morrow Point, Oroville, Boundary, Churchill Falls, and Northfield projects. Rock reinforcement on all of these projects consisted of long tensioned fully grouted expansion shell rock bolts, mostly of the hollow core type.

A-21. The use of fully grouted rock reinforcement developed more slowly in tunneling and slope stability work, but by 1970 very few ungrouted reinforcement elements were being installed on civil engineering works.

A-22. During the 1960's new types of rock bolts were developed, some on an experimental basis. These included explosively anchored bolts, "strippable" or "yieldable" bolts, and bolts utilizing epoxy or polyester resin for the element bonding medium or for the element itself. None of these were widely used on civil engineering works.

A-23. By 1972, prepackaged polyester resin systems were developed, tested, and marketed. These systems made possible the development of positive anchorage, tensioning, and full-length bonding within minutes in almost any type of rock. These systems quickly gained acceptance on civil engineering works and are being widely used along with groutable rock bolts of the hollow core type.

A-24. The future may see a greater utilization of fully grouted, <u>untensioned</u> rock anchor systems (reference Appendix E) for underground work where a minimum amount of initial rock movement is acceptable or desirable at times to fully develop arch action. However, for surface slope reinforcement, <u>pretensioned</u> rock bolts or tendons are the only practical means by which significant increases of normal forces on incipient failure planes can be achieved to prevent first movement. By preventing first movement of unstable rock slopes, existing asperities are preserved on the sliding surface, thereby ensuring that peak strength will be utilized in lieu of the lower residual strength.

APPENDIX B
SAMPLE ROCK BOLT SPECIFICATION

B-1. General. The following specification was written specifically for the NORAD Expansion Project (1970-71) and is specific about both excavation procedures and bolt installation procedures. Other projects will have their own unique requirements which will require modifications to this specification.

B-2. Tensioned Rock Bolts.

Index

B-3. Applicable Publications. Applicable references are listed in Appendix F.

B-4. Materials.

a. Rock Bolts, Hollow Bar Groutable Type. (See figure B-1.)

(1) Williams US-8-HC-SCS-158 or US-8-SCS-175, hollow-core, high bond, high strength rebar groutable type rock bolt as manufactured by Williams Form Engineering Corporation, Grand Rapids, Michigan, or equal, complete with standard expansion shell head assembly, bearing plate, bevel and flat washers, and hexagonal nut; or the same groutable rebar equipped with shells and accessories listed in paragraph B-4a(2) (a)

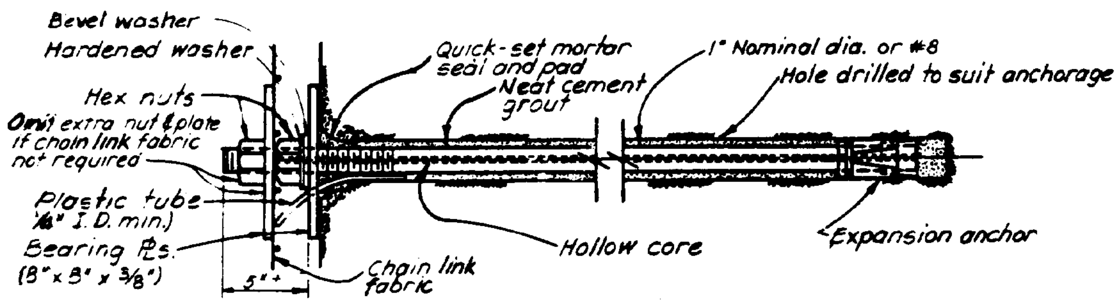


Figure B-1. Grouted rock bolt (with chain link fabric).

and (b). The various lengths required are indicated on the drawings. The use of rock bolt coupling (manufacturer's standard long couplings with hole for grout passage) for splicing rock bolts where required to provide the lengths indicated, will be permitted.

(2) Titan. Nominal 1-inch hollow-core, high bond, high strength groutable deformed rebar conforming to the tensile requirements of ASTM A 615,[25] grade 40. The hollow core shall be 1/4-inch nominal diameter with countersink at ends of rebar. Cross-sectional area of the hollow core and countersink at end of bar shall be equal to that of the Williams rebar specified in B-4a(1). The various lengths required are indicated on the drawings. The use of rock bolt couplings (manufacturer's standard long couplings with hole for grout passage) for splicing rock bolts where required to provide the lengths indicated, will be permitted. Titan rebars are available from Tower Pacific Corporation, 101 Townsend Street, San Francisco, California 94107. The following expansion shell anchorage units are suitable for use with Titan rebars.

(a) D-5 Pattin expansion shell anchorage available from Colorado Fuel and Iron Corporation, P. O. Box 1920, Denver, Colorado, or equal.

(b) K-4 expansion shell anchorage available from Bethlehem Steel Corporation, Lebanon, Pennsylvania, or equal.

Bearing plates, bevel and flat washers, and hexagonal nuts shall be as specified herein.

b. <u>Rock Bolt Deformeter</u> shall consist of rock deformeter assembly available from Williams Form Engineering Corporation, Grand Rapids, Michigan, or equal size, complete, including brass protective cap, grout and vent tubes, 8- by 8- by 3/8-inch-square steel bearing plate, hard steel washer, bevel washer, and hex nut but excluding the dial gage. Deformeter lengths are indicated in the drawings. (See figure B-2.)

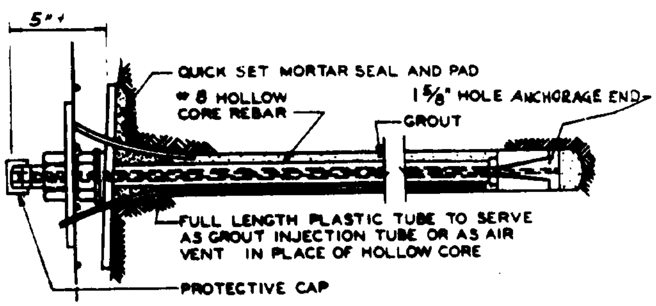


Figure B-2. Rock bolt deformeter.

c. Cement shall conform to Federal Specification SS-C-192g,[7] Type 1, Type II, or Type III, as indicated. The cement shall meet the requirements for low alkali and for control of false set contained therein.

d. Bearing Plates shall be of steel conforming to ASTM A 36,[22] with holes for installation over rock bolts and for accommodating grout and vent tubes, where necessary. Bearing plates shall be 8 inches by 8 inches by 3/8 inch.

e. Hex Nuts shall conform to ASTM A 307,[23] Grade B, heavy-duty.

f. Flat Washers shall conform to ASTM A 325,[24] quenched and tempered to a Rockwell hardness of C 38 to C 45. A quenched and tempered flat washer shall always form the seat for a heavy-duty hex nut.

g. Bevel Washers shall be ASTM A 36[22] steel, circular, standard slope, and minimum diameter to accommodate hardened flat washer above.

h. Thread Lubricant shall be a molybdenum base lubricant, similar and equal to Molykote as manufactured by Alpha Molykote Corporation, Stamford, Connecticut, or Molub-Alloy 298 as manufactured by Imperial Oil and Grease Company, Inc., Los Angeles, California.

i. Grout and Vent Tubes shall be semirigid polyvinyl chloride or polyethylene plastic tubes 3/8-inch OD and 1/4-inch ID, or larger at the contractor's option. Tubes for grouting rock bolt deformeters shall be as supplied by the deformeter manufacturer.

j. Water for mixing mortar and grout shall be fresh and free from injurious amount of oil, salt, acid, alkali, organic matter, or other deleterious substance as determined by Corps of Engineers Specification CRD-C 400.[18]

k. Fluidifier Admixture shall conform to Corps of Engineers Specification CRD-C 566.[19]

l. Fly Ash shall conform to Corps of Engineers Specification CRD-C 262, Type F.[17]

m. Quick-Setting Mortar Mix for packing collar of drill hole and forming base for bearing plates shall be a mixture of Type III portland cement, sand, quick-setting admixture and water, or an approved proprietary quick-setting cement and water that when mixed will produce a quick-setting mortar with the necessary handling properties and of sufficient strength to resist grouting pressures and stressing of rock bolts. (Sika-Plug as manufactured by Sika Chemical Corporation, or

Wil-Kwik-Set manufactured by Williams Form Engineering Corporation will meet these requirements.)

n. Sand for mortar or for grout, if a sanded grout mix is required, shall conform to Federal Specification SS-A-281b,[6] Class 1, except that the gradation shall be as specified herein. Particle shape shall be generally rounded or cubical. The sand shall be well graded from fine to coarse within the following limits:

Sieve Designation (U. S. Standard Square Mesh)	Cumulative Percentage by Weight Passing
No. 8	100
No. 16	95-100
No. 30	60-85
No. 50	20-50
No. 100	10-30
No. 200	0-5

o. Chain Link Fabric shall conform to Federal Specification RR-F-191g,[7] Type I, Grade A, No. 6 (0.1920-inch) steel wire gage, woven into 2-inch diamond mesh. Width of chain link fabric shall be coordinated with the rock bolt and rock anchor installation pattern.

p. Expansion Bolts for supporting chain link fabric from rock at intermediate points shall be suitable commercially available steel bolts with steel expansion shields. Expansion bolts shall be 3/4 inch in diameter and of a length to extend approximately 1 foot into sound rock with enough projection from the rock for the proper application of the fabric. (See figure B-3.)

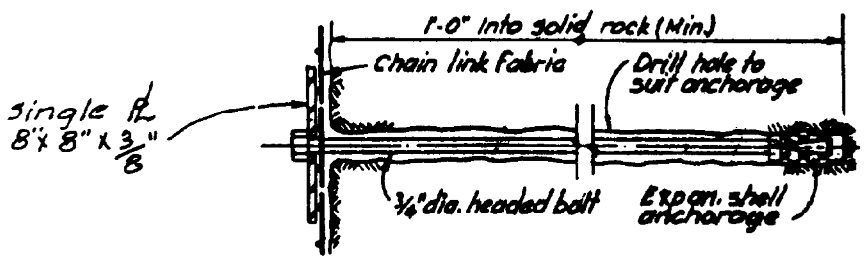


Figure B-3. Supplementary bolts for chain link fabric.

B-5. Certificates. The contractor shall submit certificates of compliance in accordance with SPECIAL PROVISIONS, attesting proof of compliance with the specifications prior to delivery of the certified material to the project site. Certificates are required for the following materials:

Cement	Washers, flat and bevel
Sand	Nuts
Bearing plates	Chain link fabric

B-6. Planned Installation Pattern for Rock Bolts. The planned installation pattern, sizes, and lengths of the rock bolts are usually indicated on the drawings. Rock conditions encountered as the work progresses may require the actual pattern, sizes, and lengths to vary from the planned installation indicated and the specific location, attitude, size, and length of each rock bolt is subject to adjustment in the field by the contracting officer. In those instances where the rock condition in or behind the burden of the trim cut is such as to be hazardous, prebolting using safety bolts of adequate size and length shall be temporarily installed for the safety of the workman. Safety bolts shall be removed or tension released before shooting the trim cut. Rock bolts, in addition to those shown on the drawings, shall be installed as directed by the contracting officer.

B-7. Test Program. At a time prior to major underground excavation, the contracting officer will designate a test section in rock representative of that to be bolted for conducting a test program designed to provide data for installing rock bolts. The contractor shall notify the contracting officer a minimum of 7 days in advance of starting the test program. A representative of the contractor in charge of installing rock bolts shall witness and actively cooperate in conducting the tests. The installation of the rock bolts for the tests and the tests shall be performed in the presence of a representative of the contracting officer. The test program will consist of:

a. The contractor shall furnish and install a minimum of eight No. 8- by 12-foot-long, hollow-bar, groutable-type rock bolts complete with anchorage shell, bearing plate, bevel and flat washers, and hex nut representative of the units proposed for use in the work.

b. Units shall be installed as specified hereafter to include setting the anchorage, bedding the bearing plate in quick-setting mortar with vent of grout tube in place, adding bevel washers as necessary, flat washer and nut.

c. The contractor will tension the rock bolt within the range of 24,000 pounds to 30,000 pounds, using a direct pull rock bolt tensioning device to verify the anchorage capability and the installation technique.

d. Any bolt installation that fails in the test program shall be dismantled and the cause of failure determined.

e. The error or defect shall be corrected in the subsequent installation and the tests repeated until eight installations are satisfactory.

f. Bolts in the test pattern shall be grouted and will be included for payment as a pattern bolt installation.

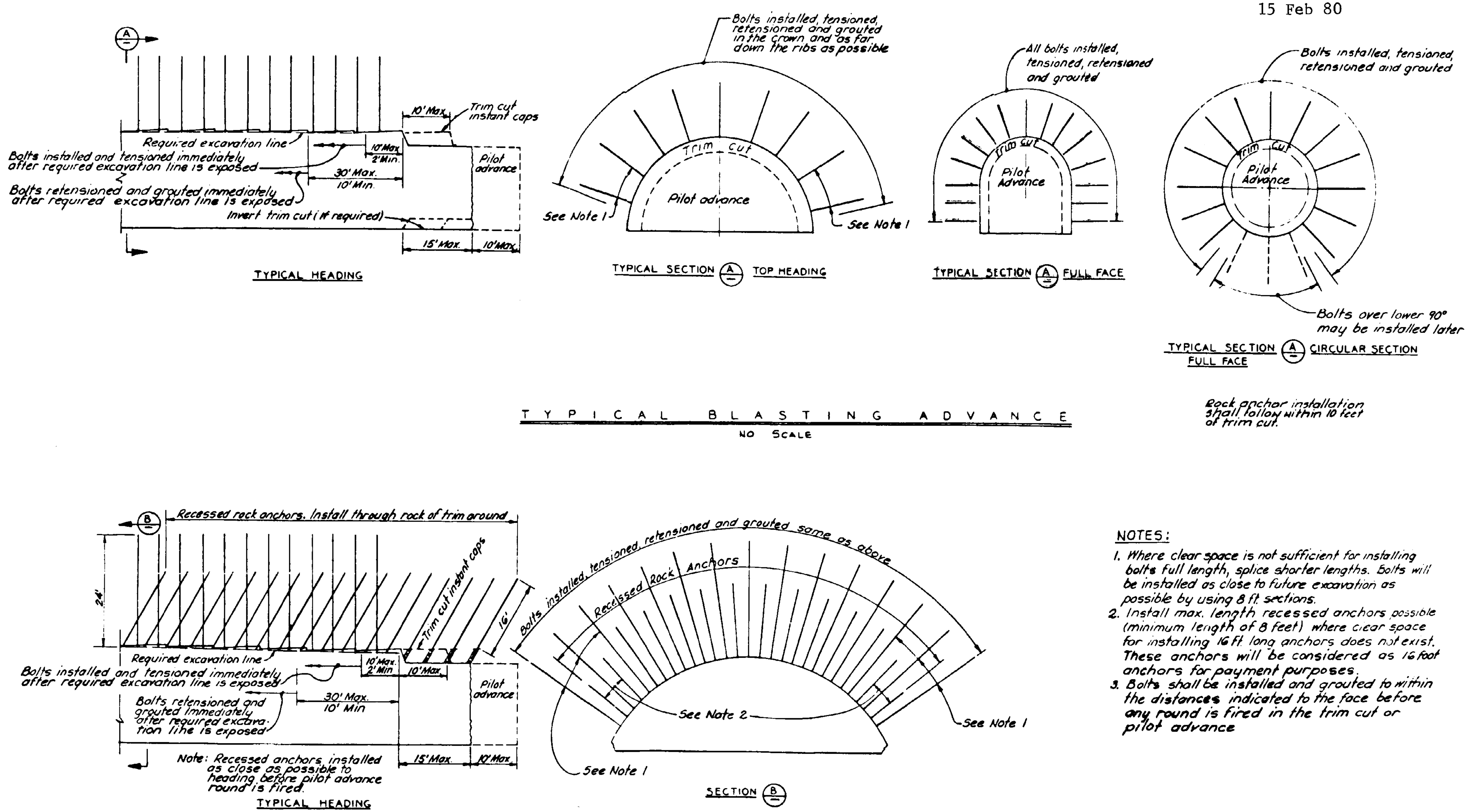
g. The procedures and methods resulting in satisfactory installations shall be used in all bolt installations.

B-8. Drilling Holes. Holes for the installation of rock bolts shall be drilled into the rock to the lengths as shown on the drawings or as directed and to such inclination as will permit bolting generally normal to the rock surface, except when otherwise indicated or as directed. All drilled holes shall be blown clear with compressed air, minimum of 50 psi introduced at the back of the hole, upon completion of drilling. In addition, all horizontal and downwardly inclined holes shall be blown clean immediately before installation of the bolt. Diameter of the drilled hole for expansion shell type rock bolts shall be as recommended by the manufacturer of the expansion shell and this diameter shall not be exceeded in the anchorage area. Holes shall be drilled using properly sharpened bits operated in such manner as to produce straight holes with smooth walls. The diameter of the drilled hole in the anchorage area shall be checked with a hole gage (Ohio Brass Company, Mansfield, Ohio 44902, or equal), and all holes exceeding the recommended diameter by more than 1/16 inch will be considered outside and not acceptable. Such holes shall be redrilled or replaced with a new hole at no additional cost to the Government. The depth of the hole shall be not less than the full length of the bolt plus the anchorage shell.

B-9. Installation of Rock Bolts.

a. All rock bolts shall be installed within 8 hours after shooting the round, except in rare instances where it may be necessary for the contracting officer to adjust the 8-hour limitation. (See figure B-4 for additional requirements for bolt installation.)

b. Hollow-Bar Groutable Type of rock bolts shall be installed in accordance with the recommendations of the manufacturer, dependent on the type expansion shell used, subject to the following modifications: (1) set expansion shell using not less than 275 ft-lb of torque or as directed by the contracting officer applied by a calibrated preset



No Scale

Figure B-4. Bolting, blasting and prebolting sequence for crown of power plant chamber.

impact wrench of proven capacity; (2) use bevel washers to limit the maximum thickness of the quick-setting mortar under the bearing plate to less than 2 inches; (3) clean threaded end of rock bolt with thread chaser full length of thread; (4) apply thread lubricant over entire thread and contact surfaces of hex nut and flat and bevel washers; (5) tension the rock bolt between 24,000 pounds minimum and 30,000 pounds maximum loading applied by a direct pull tensioning device; and (6) advance hex nut to contact bearing plate in a tight, solid fit. Any installed bolt that cannot be tensioned to the specified loading shall be replaced by the contractor at no additional cost to the Government. Replacement shall be effected by a new installation.

(1) No additional payment will be made for the replacement installation. After each heading advance, the bolts in that portion of the planned installation pattern as indicated on the drawings shall be retensioned and grouted, as specified hereafter prior to shooting the next round. Bolts shall be retensioned within the range specified herein for the original installation.

(2) If any rock bolt anchorage fails to withstand a retension load of 24,000 pounds, the contracting officer will indicate either (a) grout the bolt in place, (b) add a complete new rock bolt installation, or (c) install a perforated sleeve type rock anchor. Payment for (b) and (c) above will be made based on the installation indicated. Direct pull rock bolt tensioning devices, ELBROC, Mark IX (will require a factory modification to accommodate the 5-inch plus projection) are available from Soiltest, Inc., 2205 Lee Street, Evanston, Illinois 60202. Commerical center pull hydraulic jacks, of the required capacity, modified to provide the required operation will be permitted, subject to the approval of the contracting officer.

(3) Each direct pull rock bolt tensioning device, furnished by the contractor, shall be individually identified to enable the Government to establish and maintain a record calibration schedule verifying the accuracy of the output of each unit. Each unit will be checked by the Government against a load cell at least once in every 7-day period (time to check a unit will average 2 hours). The use of unchecked units will not be permitted. The contractor shall furnish individually identified direct pull rock bolt tensioning devices in sufficient number to permit the Government to accurately verify the output of each unit without disrupting the scheduled rock bolt installation operation. Defective or unreliable units shall either be repaired to indicate a satisfactory operation or replaced. Grout or vent tubes shall be installed as indicated on the drawings.

B-10. Installation of Rock Bolt Deformeters. Rock bolt deformeters shall be installed at the locations indicated on the drawings and as specified above for installation of hollow bar groutable type rock bolts with the addition of a full length plastic tube to serve as a grout injection tube or as an air vent in place of the blocked hollow core. Tensioning, retensioning, and grouting will be required. Grout injection of deformeter inclined upward, when viewed from the collar of the hole, shall be through the short tube. When inclined downward, grout injection shall be through the long tube. Tubes shall be color coded to differentiate between short and long tubes. Time of grouting of rock bolt deformeters will be the same as for pattern bolts in the same area unless the contracting officer specifies certain ones to be left ungrouted. (Installation and grouting of solid bar rock bolts would be identical to installation and grouting of rock bolt deformeters.)

B-11. <u>Portland Cement Grouting of Rock Bolts and Deformeters</u>.

a. <u>General</u>. Rock bolts and deformeters shall be pressure grouted as specified herein. The annular space around each rock bolt, tie bolt, or deformeter shall be filled by pumping grout through the injection tube at pressures not to exceed 25 psi measured at the collar of the hole. The annular space will be considered grouted when there is a full flow return of grout through the vent. For up bolts and approximately horizontal bolts, the short plastic tube sealed in the collar of the hole shall be used as the grout injection tube and the hollow core of the bolt used as a vent tube. For units inclined downward more than 3 degrees from the horizontal, the injection and venting processes shall be reversed. A grout tube adapter available from Williams Form Engineering Corporation, Grand Rapids, Michigan, or equal, shall be used when grout is injected through the hollow core.

b. <u>Grouting</u>.

(1) Grout mixing and pumping equipment. All equipment used for mixing and injecting grout shall be of a type and capacity approved by the contracting officer and shall be maintained in first class operation condition at all times. The selection of equipment and the determination of its suitability to the work shall be based upon a maximum grouting pressure of 100 psi at the pump. The minimum grouting equipment to be furnished shall include the following:

(a) One grout pump capable of delivering grout at the pressure required by the grouting procedures. The grout pump shall be similar and equal to a "Moyno" double helical screw type pump as manufactured by Robbins and Myers, Inc., of Springfield, Ohio. The intake to the grout pump shall consist of an open hopper to allow visual observation of the grout at the intake. The hopper shall be fitted with a No. 8

screen through which all the grout must pass. An accurate pressure gage shall be provided at the discharge of the pump.

(b) One mechanically driven paddle-type grout mixer of standard make capable of effectively mixing and stirring the type of grout required by these grouting operations. The mixer shall be equipped with a gravity feed water measuring device mounted over the mixer with a discharge tube and control valve for discharging the specified quantities of water to the mixer. The water container shall be made with clear plastic and shall have a capacity suitable for the size of batch. The plastic container shall be calibrated to read to the nearest five-hundredth (0.05) cubic foot. The calibrated increments shall be imprinted on the plastic container wall so that the column of water can be read by eye. A readily adjustable overflow pipe shall be provided in the plastic container to positively regulate the amount of water available for discharge into each batch.

(c) A mechanically agitated sump, so designed as to effectively stir and hold in suspension all solid matter in the grout. If grout mixer and grout pump are suitably arranged and operated so as to preclude need for a sump, it may be omitted if specifically approved.

(d) An approved "gun" or arrangement of valves shall be provided in the grout lines not more than 12 feet from the nozzle end to be attached to the injection tube for each individual bolt. A return line of at least 3/4-inch ID shall be provided from this point to the mixer, or sump if used, and grout shall be continually circulated through the return line whenever grout is in the pump and lines, except when actually injecting grout into each individual bolt hole. Supply line to the "gun" or arrangement of valves shall be at least 3/4-inch ID and line for connection to each individual bolt shall be at least 1/2-inch ID and not over 12 feet long.

(e) Valves, pressure gages, pressure hose, small tools, and accessories as may be necessary to provide a continuous supply of grout and accurate pressure control will be required.

(f) The contractor shall keep on hand at the work site a supply of extra hoses, tubes, accessories, and small tools as required to minimize work stoppages due to need for replacement items during grouting operations.

(2) Grout mixture. The grout mixture shall consist of the following materials in these proportions:

Portland cement, Type III	(2 sacks)	188 lb
Fly ash	(1 sack)	75 lb
Interplast-C or equal	(1 percent cement + fly ash)	2.6 lb
Water		(see note)

Note: Water shall be in such quantity that grout will have an efflux time of 20 seconds or more when tested in accordance with Corps of Engineers Test Method CRD-C 79-58.[15]

(3) Mixing and handling. All grout shall be mixed in the specified type mixer. Dry materials charged into the mixer shall be measured by volume except that the admixture for each batch shall be furnished to the mixer in individual containers of preweighed material. Water shall be measured by the specified measuring device on the mixer. All mixing, pumping, and packing operations shall be carefully coordinated so that no delay occurs in the process. Any grout which commences to set or indicates an appreciable change in consistency before grouting operations are complete shall be completely removed from mixer, sump, pump, and lines and discarded as directed.

(4) Grouting of rock bolts. All grout pipes, tubes, and fittings shall be clean and free from dirt, grease, hardened grout, or other contamination before grouting is commenced for any bolt. All wash water and diluted grout shall be flushed from all lines and wasted before commencing operations. The grout line shall be attached to the grout injection tube for the individual bolt with suitable fittings such that leakage is entirely prevented. The grout shall be injected at a rigidly controlled pressure, as approved. Equipment which cannot maintain a uniform pressure as directed shall be removed and replaced with suitable equipment. Care shall be taken to avoid premature clogging of pipes or tubes and any pipe or tube that becomes clogged or obstructed, before completion of grouting operations, shall be removed, cleaned, and replaced in an approved manner at no cost to the Government. The grouting of any bolt shall not be considered complete until the grout flows from the vent in a steady stream. The vent tube shall be plugged and the grout injection tube sealed, with pressure maintained on the injection tube. In any case the hole shall be left completely filled. All actual grout injection shall take place in the presence of a representative of the contracting officer.

(5) Leaks and grout loss. If during grouting of any bolt hole grout leaks to the surface of the rock through an open seam or other void and visibly indicates appreciable loss of grout, grouting operations shall be temporarily suspended on this bolt hole and the seam calked with quick-setting mortar or as otherwise approved. If during

grouting of any bolt hole the hole accepts more grout than required to fill the nominal volume of the annular space in the hole, and if no grout leakage is visible at the surface, grouting operations shall be temporarily suspended on this bolt hole. Not earlier than 1 hour and not later than 2 hours after suspension of grouting on the bolt hole, the grout lines shall be reconnected and grouting of the bolt hole completed. If leakage still continues, grouting shall be continued until termination by the contracting officer. A sanded mix may be required if excess leakage occurs.

B-12. Chain Link Fabric Rock Support.

a. General. Chain link fabric shall be installed where indicated on the drawings or directed by the contracting officer as a secondary support of rock. Installation shall be made following grouting of rock bolts and final scaling, or earlier if directed by the contracting officer.

b. Installation. Installation shall be as shown on the drawings. Fabric shall be lapped at the rock bolt a minimum of three mesh openings and shall be supported from rock bolts. At intermediate points, the fabric shall be supported with 3/4-inch steel expansion bolts installed in holes drilled approximately 1 foot into solid rock where directed. The fabric is not to be lapped at intermediate points. Holes shall be drilled into rock to such depth as to allow the bolt to protrude only far enough beyond the rock surface for proper application of the fabric.

B-13. Measurement and Payment.

a. General. The contract prices for the various items under this section shall constitute full compensation for furnishing all materials, labor, tools, equipment, and incidentals necessary to accomplish the work herein, including cleanup of the area and disposal of waste water and grout.

b. Rock Bolts, hollow bar groutable type, will be measured by the unit each for the various lengths involved and will be paid for at the contract unit price each for "Rock Bolts, Hollow Bar Groutable Type" to include drilling the hole, installing the bolt, initial tensioning, final tensioning, and pressure grouting. Additional grout as required under paragraph "Leaks and Grout Loss" will be measured by the sack of portland cement used and will be paid for at the contract unit price per sack for "Portland Cement Grout Loss." The quantity for "Portland Cement Grout Loss" indicated in the Unit Price Schedule is estimated and the Government reserves the right to order any increase or decrease in the actual number of sacks of portland cement required without any restriction percentagewise from the estimated quantity stated in the contract without any recourse by the contractor to demand any adjustment in the contract unit price or to claim loss of anticipated profit by reason of such change.

c. Chain Link Fabric will be measured by the square foot, computed from the length and width of the rock surface covered, computed in accordance with the design configuration indicated on the drawings, excluding any corner chamfer, and no allowance for laps, and will be paid for at the contract unit price per square foot for chain link fabric.

d. Expansion Bolts Supporting Chain Link Fabric will be measured by the unit each and will be paid for at the contract unit price each for "Expansion Bolts Supporting Chain Link Fabric."

e. Bearing Plates for Supporting Chain Link Fabric will be measured by the unit each and will be paid for at the contract unit price each for "Bearing Plates - Chain Link Fabric Support," such payment to include supplementary hex nuts as required.

f. Rock Bolt Deformeters will be measured by the unit each, of the various lengths involved, complete in place, including pressure grouting, and will be paid for at the contract unit price each for "Rock Bolt Deformeters."

APPENDIX C
SAMPLE MATERIAL SPECIFICATION FOR POLYESTER RESIN GROUTED ROCK BOLTS

C-1. Applicable Publications. Applicable references are listed in Appendix F.

C-2. Materials shall conform to the respective specifications and other requirements specified below.

a. Chain Link Fabric shall conform to Federal Specifications RR-F-191g,[5] No. 9 gage, 2-inch diamond mesh, with twisted and barbed selvage of top and bottom edges. Fabric shall be either zinc-coated, Type I, with 2.0 ounces of zinc per square foot of uncoated wire surface, or aluminum-coated, Type II, with 0.40 ounce of aluminum alloy per square foot of uncoated wire. Fabric width shall be 10-foot widths.

b. Tie Wire shall be 9 gage (0.148-inch-diameter) steel wire. All wire will be made from steel wire complying with Federal Specification QQ-W-461g,[4] finish 5, zinc coated with class 3 heavy zinc coating.

c. Rock Bolt Units. All rock bolts and accessories shall be the standard product of Celtite, Inc., of Cleveland, Ohio, or approved equal. The bolts shall be No. 11 deformed steel bars conforming to the requirements of ASTM A 615,[25] grade 60, threaded on one end as shown on the drawings. The contractor shall furnish anchor and encapsulating polyester resin cartridges having sufficient gel and cure times to provide time to place the bolt as required in these specifications and as shown on the drawings. Polyester resin rock bolt units shall be furnished with 8- by 8- by 1/2-inch bearing plates with hole for accommodating rock bolt, hardened flat washer, and hexagonal nut. Manufacturer's standard couplings for splicing rock bolts may be provided to provide the lengths required as indicated in the drawings.

d. Bearing Plates shall be of steel conforming to ASTM A 36[22] with holes for installation over rock bolts. Bearing plates shall be 8 inches by 8 inches by 1/2 inch for 1-3/8-inch-nominal-diameter rock bolts.

e. Hexagonal Nuts shall conform to ASTM A 325,[24] grade B, heavy-duty.

f. Flat Washers shall conform to ASTM A 325,[24] quenched and tempered to a Rockwell hardness of C38 to C45. A quenched and tempered flat washer shall always form the seat for a heavy-duty hexagonal nut.

g. Polyester Resin Cartridges. The resin shall be high-strength polyester containing nonreactive inorganic aggregate filler. The catalyst shall contain nonreactive inorganic filler. The compressive strength of the mixed and cured resin shall be 14,000 psi when tested in accordance with ASTM C 39-72, "Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens."[26] The material shall be thixotropic and of such viscosity that the bolt can adequately mix the material. The relative time of cure between end anchor cartridges and encapsulating cartridges should provide adequate time to place the bolt as required in the specifications and as shown on the drawings. All cartridges shall be inspected before insertion to see that the polyester resin components have not hardened. Resin that is older than six (6) months shall not be used.

APPENDIX D
SAMPLE ROCK ANCHOR SPECIFICATION

D-1. Applicable Publications. Applicable references are listed in Appendix F.

D-2. Materials.

a. Rock Anchors Perforated Sleeve-Type shall consist of one pair of perforated steel half-sleeves, filled with portland cement mortar tied together to form one sleeve extending from the bottom of the drilled hole to the rock surface at the final excavation line and one steel deformed rebar, end ground to a bullet nose for ease of driving. The anchors shall be No. 8 deformed bars with 1-1/4-inch-diameter steel half-sleeves and No. 10 deformed bars with 1-3/4-inch-diameter half-sleeves. Sizes shall be as indicated on the drawings. The perforated half-sleeves shall be similar and equal to units as manufactured by the Sika Chemical Corporation, Perfo Division, Passaic, New Jersey. The rebar shall extend, without splicing (coupling or weld) from the collar of the hole at the final excavation line to the bottom of the drilled hole. Thread extension shall be provided where indicated, or as directed by the contracting officer. Rock anchors shall be recessed from a rock surface other than at the final excavation as indicated on the drawings or as directed by the contracting officer. Deformed rebar shall conform to ASTM A 615,[25] grade 40.

b. Groutable Rebar Type shall be of the various sizes shown. Rebars shall conform to ASTM A 615,[25] grade 40, with bottom end shaped to a bullet nose for ease of driving. The length of the bar shall be such that it will extend without splicing to the bottom of the drilled hole and shall protrude beyond the rock surface with either (1) standard ACI hook end for embedment in concrete or (2) a sufficient distance for the proper attachment of metal plates and shapes as shown. The ends of bars to which metal shapes are to be attached shall be threaded 6 inches and approximately 1 inch of thread shall project beyond the finally installed nut.

c. Cement shall conform to Federal Specification SS-C-192g,[7] Type III. The cement shall meet the requirements for low alkali and for control of false set contained therein.

d. Reinforcement Bars shall conform to ASTM A 615,[25] grade 40.

e. Bearing Plates shall be of steel conforming to ASTM A 36,[22] with holes for installation over rock bolts and for accommodating grout and vent tubes, where necessary. Bearing plates shall be 8 inches by

8 inches by 3/8 inch for rock bolts or rock anchors and 8 inches by 8 inches by 1/2 inch for 1-3/8-inch tie bolts.

f. Hex Nuts shall conform to ASTM A 307,[23] grade B, heavy duty.

g. Flat Washers shall conform to ASTM A 325,[24] quenched and tempered to a Rockwell hardness of C38 to C45. A quenched and tempered flat washer shall always form the seat for a heavy-duty hex nut.

h. Bevel Washers shall be ASTM A 36[22] steel, circular, standard slope, and minimum diameter to accommodate hardened flat washer above.

i. Thread Lubricant shall be a molybdenum base lubricant, similar and equal to Molykote as manufactured by Alpha Molykote Corporation, Stamford, Connecticut, or Molub-Alloy 298 as manufactured by Imperial Oil and Grease Company, Inc., Los Angeles, California.

j. Grout and Vent Tubes shall be semirigid polyvinyl chloride or polyethylene plastic tubes 3/8-inch OD and 1/4-inch ID, or larger, at the contractor's option.

k. Water for mixing mortar and grout shall be fresh and free from injurious amount of oil, salt, acid, alkali, organic matter, or other deleterious substance as determined by Corps of Engineers Specification CRD-C 400.[18]

l. Fluidifier Admixture shall conform to Corps of Engineers Specification CRD-C 566.[19]

m. Fly Ash shall conform to Corps of Engineers Specification CRD-C 262,[17] Type F.

n. Quick-Setting Mortar Mix for packing collar of drill hole and forming base for bearing plates shall be a mixture of Type III portland cement, sand, quick-set admixture, and water or an approved proprietary quick-setting cement and water that mixed will produce a quick-setting mortar with the necessary handling properties and of sufficient strength to resist grouting pressures and stressing of rock bolts. (Sika-Plug as manufactured by Sika Chemical Corporation, or Wil-Kwik-Set manufactured by Williams Form Engineering Corporation will meet these requirements.)

o. Sand for mortar or for grout, if a sanded grout mix is required, shall conform to Federal Specification SS-A-281b,[6] Class 1, except that the gradation shall be as specified herein. Particle shape shall be generally rounded or cubical. The sand shall be well graded from fine to coarse within the following limits:

Sieve Designation (U. S. Standard Square Mesh)	Cumulative Percentage by Weight Passing
No. 8	100
No. 16	95 - 100
No. 30	60 - 85
No. 50	20 - 50
No. 100	10 - 30
No. 200	0 - 5

D-3. Certificates. The contractor shall submit certificates of compliance, in accordance with Special Provisions, attesting proof of compliance with the specifications prior to delivery of the certified material to the project site. Certificates are required for the following materials: reinforcing steel, cement, sand, bearing plates, washers (flat and bevel), and nuts.

D-4. Planned Installation Pattern for Rock Anchors. The planned installation pattern, sizes, and lengths of the rock anchors and recessed rock anchors is indicated in the drawings. Rock conditions encountered as the work progresses may require the actual pattern, sizes, and lengths to vary from the planned installation indicated and the specific location, attitude, size, and length of each rock anchor is subject to adjustment in the field by the contracting officer. In those instances where the rock condition in or behind the burden of the trim cut is such as to be hazardous, prebolting using safety bolts of adequate size and length shall be temporarily installed for the safety of the workman and/or recessed rock anchors shall be installed through the burden where deemed necessary for stabilizing the excavation and as approved or directed by the contracting officer. Safety bolts shall be removed or tension released before shooting the trim cut. Rock anchors in addition to those shown in the drawings shall be installed as directed by the contracting officer.

D-5. Test Program.

a. Test Section. At a time prior to any underground excavation, the contracting officer will designate a test section in the south access tunnel representative of the rock to be bolted for conducting a test program designed to provide data for installing rock bolts and rock anchors. Test locations will be approximately 4 feet above the tunnel invert. The contractor shall notify the contracting officer a minimum of 7 days in advance of starting the test program. A representative of the contractor in charge of installing rock anchors shall witness and actively cooperate in conducting the tests. The installation of the rock anchors for the tests and the tests shall be performed in the presence of a representative of the contracting officer. The test program will consist of:

b. Rock Anchors, One End Threaded.

(1) The contractor shall furnish eight No. 10 by 10-foot-long and four No. 10 by 16-foot-long threaded rebar Perfo sleeve rock anchors complete with bearing plate, bevel and flat washers, and hex nut representative of the units proposed for use in the work. Units shall be installed as specified hereafter to include the Perfo sleeve and mortared-in rebar, but no recess is required. The contractor shall install eight No. 10 by 10-foot-long and four No. 10 by 16-foot-long rock anchors using the specified mix for rock anchors unless the mortar mix is varied by the contractor, with approval of the contracting officer. The tests should demonstrate that bars can be driven full length through the mortar, with the mix selected, within a time frame consistent with that required by the contractor during the pattern installation of recessed rock anchors and that rock anchors will be capable of providing positive reinforcement to the rock behind the trim round before the trim round is fired. The Government will bed the contractor-furnished bearing plate in quick-set mortar and, with a center-pull hydraulic jack, conduct tests on individual anchors at 2, 3, 4, and 5 hours following installation to the yield load of the anchor, if possible.

(2) Considering the above test results the mortar mix can be varied by the contractor, with approval of the contracting officer and with the installation and testing of four No. 10 by 10-foot-long and one No. 10 by 16-foot-long rock anchors accomplished to verify the revised mix. The final mortar mix and installation procedures to be used in the work will be based on the results of these tests. After the final mortar mix is selected the remaining rock anchors will be installed and tested to verify the final mix. Each rock anchor in the test pattern will be included for payment as a 16-foot-long pattern rock anchor installation and shall include the furnishing, installing of the bolt, and cutting off of satisfactory bolts.

D-6. Drilling Holes. Holes for the installation of rock anchors shall be drilled into the rock to the lengths as shown on the drawings or as directed and to such inclination as will permit anchoring generally normal to the rock surface, except when otherwise indicated or as directed. All drilled holes shall be blown clear with compressed air, minimum of 50 psi introduced at the back of the hole, upon completion of drilling. In addition, all horizontal and downwardly inclined holes shall be blown clean immediately before installation of the anchor. Size of drilled holes for rock anchors of the perforated sleeve type shall be 1-1/2 inches in diameter for the No. 8 deformed bars and 2 inches in diameter for the No. 10 deformed bars for the length of the mortar filled sleeve and may be larger in diameter for the recessed portion between the rock face and the final excavation line indicated on the drawings. The diameter of the drilled hole mortar filled sleeve area shall be checked with a hole gage and all holes exceeding the recommended diameter by more than 1/16 inch will be considered outside and not acceptable. Such holes shall be redrilled or replaced with a new hole at no additional cost to the Government. The hole shall be accurately drilled to ensure that the bar and mortar filled sleeve, when installed as specified hereafter, will completely fill the hole and will provide a tight bond of extruded mortar between the bar and the adjacent rock in the hole.

D-7. Installation of Rock Anchors. Rock anchors of the perforated sleeve type shall be installed as recommended by the manufacturer of the perforated sleeves subject to the following modifications: (a) mortar for packing the sleeves of recessed rock anchors when a short mortar set time is desirable, shall be a mixture of one part Type I or Type II portland cement, one part sand, and sufficient water and admixture to produce a mortar with a flow of approximately 85 percent when tested in accordance with Corps of Engineers Specification CRD-C 116-16[16] Admixture shall be "Sika Set," as manufactured by the Sika Chemical Corporation, or equal. The admixture shall be added to the mixing water in a ratio of one part admixture to five parts water or as modified in accordance with results of tests outlined in paragraph "Test Program." Mortar for packing sleeves of rock anchors located at distances greater than 50 feet from the nearest round being fired, or where a grout setting time of 2 days minimum can be allowed when located at distances less than 50 feet from the nearest round being fired, shall consist of Type III portland cement and sand in the ratio of 1 to 1 by weight and Interplast-C powder added at the rate of 1 pound of powder per sack of cement and sufficient water to produce a mortar with a flow of approximately 85 percent when tested in accordance with Corps of Engineers Specification CRD-C 116-16[16](Note: A satisfactory mix is one which will stick together on being molded into a ball by slight pressure of the hands and will not exude free water but will leave the hands damp.) All mixing, packing, and bar driving operations shall be coordinated so that no delay occurs in the process. Any mortar which commences to set or appreciably changes consistency before bar inserting operations are complete shall be completely removed from the sleeves and holes and discarded. Retempering of mortar will not be permitted. Each one-half sleeve shall be packed full with a convex surface on the mortar to assure sufficient material to completely fill the drilled hole during insertion of the rebar, (b) copper wire (gage as required) shall be threaded across the Perfo sleeve a few inches from the collar end forming a stop for placing the rebar. After the packed sleeve is inserted to the maximum depth possible, the rebar shall be placed against the copper wire stop and the entire assembly pushed to the back of the drilled hole, after which, drive the rebar into the sleeve, breaking the copper wire stop and displacing the mortar. Care must be taken to assure that the sleeve does not bind in the hole causing the rebar to punch through the wire stop and sleeve before reaching the back of the hole. The contractor may revise this procedure at no additional cost to the Government as approved by the contracting officer. Rock anchors with plates, as directed by the contracting officer, shall have a quick-set mortar seal and pad placed in the collar of the hole as indicated in the drawings. Bevel washers shall be used to limit the maximum thickness of the quick-set mortar pad to less than 2 inches. Bearing plate and hex nut shall be installed with the nut hand wrenched to a tight, solid fit.

D-8. Installation of Groutable Rebar Type Anchors. These anchors are the grouted rebar type and sizes, lengths, and locations are indicated in the drawings. When anchors are to be installed in tunnel inverts up to 30 degrees from vertical the hole shall be blown clean and filled with grout. The anchor shall then be pushed or driven through the grout to the bottom of the hole. When anchors are to be installed in walls or tunnel crowns, the grouting of the annular space around the rebar shall be as follows:

a. A plastic tube shall be inserted in the drilled hole the full depth, less 2 inches, by taping the tube to the rebar at approximately 2-foot intervals before the bar is inserted in the hole.

b. The tube shall be left in place and the projecting end cut off flush with the collar of the hole after completion of grouting.

c. For up-holes, a short plastic grout injection tube, together with the long vent tube, shall be sealed in the collar of the hole with quick-setting mortar. For horizontal and down-holes up to 60 degrees from horizontal, the short plastic tube shall be the vent tube and the long tube shall be used for grout injection.

d. Rebar shall be rigidly supported until grout hardens.

e. For bars with hook ends, after the grout has hardened for 4 days, the protruding end of the bar may be bent to shape shown. Heating of rebar for bending shall be controlled by approved means so as not to damage the rebar. Each rebar must be approved by the contracting officer before it is bent in a hook.

D-9. Measurement and Payment.

a. General. The contract prices for the various items under this section shall constitute full compensation for furnishing all materials, labor, tools, equipment, and incidentals necessary to accomplish the work herein, including cleanup of the area and disposal of waste water and grout.

b. Rock Anchors, perforated sleeve type, without bearing plate, bevel and flat washers, and hex nut will be measured by the unit each for each of the various lengths and sizes involved and will be paid for at the contract unit price each for "Rock Anchors-Perfo Sleeve Type" to include drilling of the hole and complete installation of the anchor. The 1-1/2-inch-diameter holes for recessing the rock anchors, where required, will not be separately measured for payment but will be considered a subsidiary obligation of the contractor included under the applicable Rock Anchor-Perfo Sleeve Type item.

c. Rock Anchors, Perfo type with bearing plate assembly, including bevel and flat washers and hex nut, indicated on the drawings as a substitute for an expansion shell rock bolt in problem areas, will be measured and paid for, on a size for size basis, as a "Rock Bolt, Hollow Bar Groutable Type."

d. Rock Anchors, Groutable Rebar Type will be measured by the unit each for each of the various lengths and sizes involved and will be paid for at the contract unit price for each length and size of Rock Anchors-Groutable Rebar Type to include drilling of the hole and complete installation of the anchor.

D-10. Quality Control. Quality control on untensioned rock anchor installation is more difficult than for expansion shell or slot and wedge rock bolts since a tensioning load is not applied during installation. On the Norad Project, as shown in the sample specifications above, the quality control was started by requiring a test program prior to any underground excavation. The purpose of such a test program is to have the workmen demonstrate to the contracting officer that they are capable of installing the anchors satisfactorily and to develop proficiency in installation methods. Since the test program is closely monitored by the contracting officer's representative in cooperation with the contractor, it is possible to educate the workmen on the importance of good workmanship and attention to details in achieving the best possible results. After the test program is over it is the responsibility of the contractor to continue to use the best techniques developed in the test program. Although it was not required on the Norad Project, since the anchors were used primarily in the crowns of only two chambers, on a lengthy project there is merit in considering a requirement for random pull tests during construction to ensure that the anchors are being installed as in the test program.

Tentative Recommendations for Prestressed Rock and Soil Anchors

PREPARED BY AN AD HOC COMMITTEE
of the
PCI POST-TENSIONING COMMITTEE

(After this report was published the Post-Tensioning Division of the Prestressed Concrete Institute left PCI in 1976 to establish the Post-Tensioning Institute. Accordingly, information regarding new and future developments on prestressed anchors may be obtained from either the Prestressed Concrete Institute or the Post-Tensioning Institute.)

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TENTATIVE RECOMMENDATIONS FOR PRESTRESSED ROCK AND SOIL ANCHORS

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1. SCOPE

This document has been prepared to provide guidance in the application of permanent and temporary prestressed rock and soil anchors utilizing high strength prestressing steel. It represents the present state of the art and outlines what are considered the most practical procedures for installation of prestressed rock and soil anchors. Typical applications are illustrated in the Appendix.

2. DEFINITIONS

Permanent Anchor: Any prestressed rock or soil anchor for permanent use. Generally more than a 3-year service life.

Temporary Anchor: Any prestressed rock or soil anchor for temporary use. Generally less than a 3-year service life.

Downward Sloped Anchor: Any prestressed anchor which is placed at a slope greater than 5° below the horizontal.

Upward Sloped Anchor: Any prestressed anchor which is placed at a slope greater than 5° above the horizontal.

Horizontal Anchor: Any prestressed anchor which is placed at a slope between ±5° with the horizontal.

Anchor Grout: *(Also known as primary injection)* Portland cement grout that is injected into the anchor hole to provide anchorage at the non-stressing end of the tendon. In case of a sheathed anchor, also included in the grout between the sheath and the anchor hole. Resins are also used as anchor grout. Their properties are not covered by this recommended practice.

Corrosion Protective Filler Injection: *(Also known as secondary injection)* Material that is injected into the anchor hole to cover the stressing length of the prestressed anchor, providing corrosion protection to the high strength steel. This material may be grout or other suitable materials.

Consolidation Grout: Portland cement grout that

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is injected into the hole prior to inserting the tendon to waterproof or otherwise improve the rock surrounding the hole.

Inserting: The physical placement of the anchor tendon in the prepared hole.

Lift-Off Check: Checking the force in the prestressed anchor at any specified time with the use of a hydraulic jack.

Proof Load: Initial prestressing per anchor, representing the proof loading.

Transfer (lock-off) Load: Prestressing force per anchor after the proof loading has been completed and immediately after the force has been transferred from the jack to the anchorage.

Design Load: Prestressing force per anchor after allowance for time dependent losses.

Tendon: The complete assembly consisting of anchorage and prestressing steel with sheathing when required.

Anchorage: The means by which the prestressing force is permanently transmitted from the prestressing steel to the rock or earth.

Prestressing Steel: That element of a post-tensioning tendon which is elongated and anchored to provide the necessary permanent prestressing force.

Coating: Material used to protect against corrosion and/or lubricate the prestressing steel.

Sheathing: Enclosure around the prestressing steel to avoid temporary or permanent bond between the prestressing steel and the surrounding grout.

Coupling: The means by which the prestressing force may be transmitted from one partial-length prestressing tendon to another.

Sheathed Anchor: An anchor in which the stressing length of the high strength steel is encased in a grout-tight sheath. The annulus between the sheath and the periphery of the drilled hole may be grouted together with the anchor grout.

Un-sheathed Anchor: An anchor in which the stressing length of the high strength steel is not encased in a sheathing.

Cohesive Soils: Soils that exhibit plasticity.

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In order to better define a soil as cohesive or

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Generally defined as composed of material more than half of which is smaller than the No. 200 size sieve.

Non Cohesive Soils: Granular material that is generally nonplastic, composed of material more than half of which is larger than the No. 200 size sieve.

3. ROCK ANCHORS

3.1 Description

A prestressed rock anchor is a high strength steel tendon, fitted with a stressing anchorage at one end and a means permitting force transfer to the grout and rock on the other end. The rock anchor tendon is inserted into a prepared hole of suitable length and diameter, fixed to the rock and prestressed to a specified force. The basic components of prestressed rock anchor tendons are the following *(see Fig. 1)*:

1. Prestressing steel which may be a single or a plurality of wires, strands or bars. *(Refer to PCI Guide Specifications for Post-Tensioning Materials.)* The total length of the prestressing tendon is composed of two parts:
   a. Bond length *(socket)*, is the grouted portion of the tendon that transmits the force to the surrounding rock.
   b. Stressing length, which is the part of the tendon free to elongate during stressing.
2. A stressing anchorage is a device which permits the stressing and anchoring of the prestressing steel under load.
3. A fixed anchor is at the opposite end of the tendon than the stressing anchor and is a mechanism which permits the transfer of the induced force to the surrounding grout.
4. Grout and vent pipes and miscellaneous appurtenances required for injecting the anchor grout or corrosion protective filler.

3.2 Design Considerations – Rock Anchors

Rock anchors can be installed in downward or upward positions, however, close to horizontal positions are not recommended because of grouting difficulties

Recommended Bond Stress: The ultimate bond stress values given in the table below are

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non-cohesive it is necessary to know the percentage of fines and also to know the Atterberg limits of soils containing more than 12 percent fines.

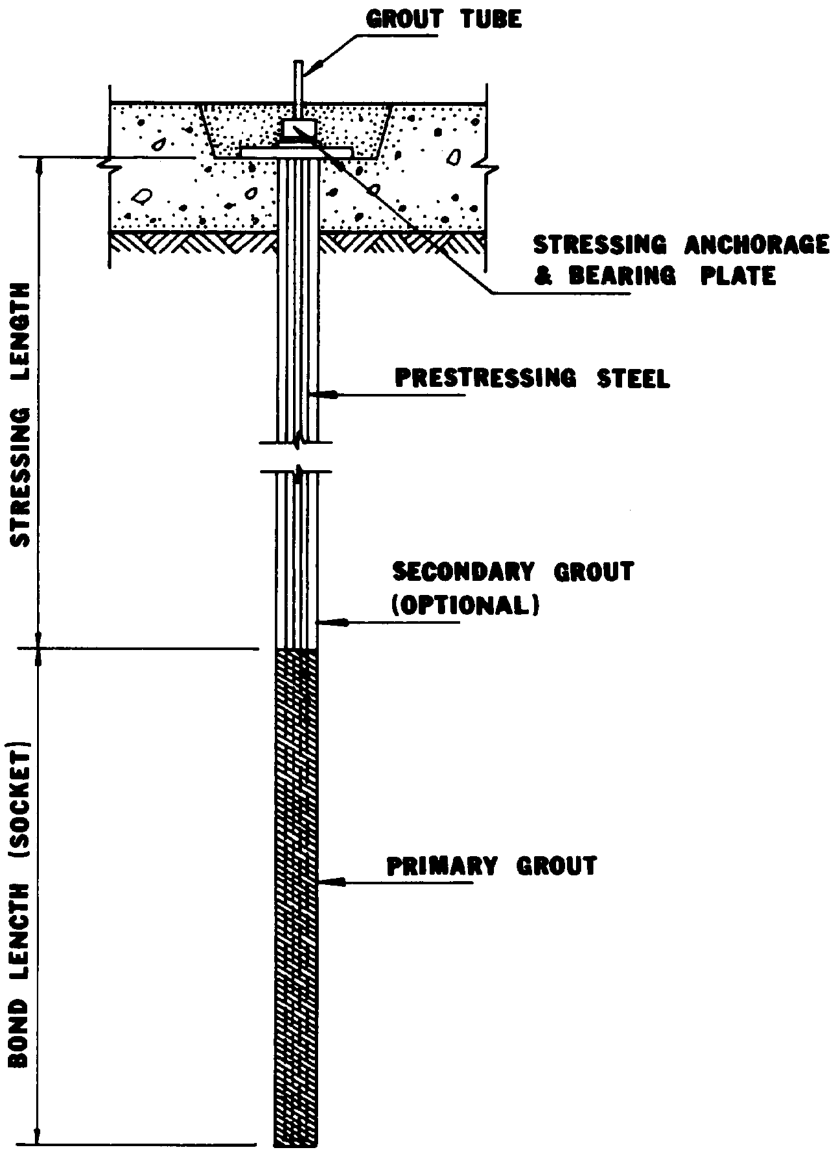

Figure 1 — Rock Anchor

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guide values only. Core drilling to explore the rock quality is an absolute necessity, and core testing together with pull-out tests of test rock anchors are strongly recommended to verify the design assumptions prior to installation of production anchors.

The values presented in the table must be used with a Safety Factor which will depend upon the type of application. The following are suggested methods of obtaining safe working loads:

a. Safety factor applied to the ultimate bond stress obtained from either pull-out tests or bond stress table. Safety factor should range from 1.5 to 2.5.

b. Proof loading of every anchor of not less than 115 percent of its transfer *(lock-off)* force. During the proof loading operation, the prestressing force shall not be more than 80 percent of the guaranteed ultimate tensile strength *(GUTS)* of the high strength steel. The duration of the proof loading is to be specified by the Engineer. Transfer *(lock-off)* the prestressing force at a level of between 50 and 70 percent of its guaranteed ultimate tensile strength. The difference between transfer load and design load shall include allowance for time dependent losses.

Typical Bond Stresses for Rock Anchors

Type	Ultimate Bond Stresses Between Rock and Anchor-Grout Plug **Sound, Non-decayed**
Granite & Basalt	250 PSI – 450 PSI
Dolomitic Limestone	200 PSI – 300 PSI
Soft Limestone*	150 PSI – 220 PSI
Slates & Hard Shales	120 PSI – 200 PSI
Soft Shales*	30 PSI – 120 PSI
Sandstone	120 PSI – 250 PSI
Concrete	200 PSI – 400 PSI

**Bond strength must be confirmed by pullout tests which include time creep tests.*

3.3 Drilling

Holes for anchors should be drilled to a diameter, depth, line, and tolerance as specified by the engineer. The hole shall be drilled so that its

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The duration of the proof loading is usually up to 15 minutes, in which case, the prestressing force is held by the jack. If longer duration is required, it is recommended to transfer the force to the anchorage and remove the jack.

For small load strand anchors *(such as single strand)* the bond between grout and strand might govern. The bond capacity between grout and strand is about 450 psi.

Core drilling, rotary drilling and percussion drilling may be employed as the conditions warrant Core drilling is generally slower and less

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diameter is not more than 1/8 inch smaller than the specified diameter.

3.4 Watertightness

The holes for some or all rock anchors may be tested for watertightness, if specified by the Engineer. When specified, the entire hole shall be tested for watertightness by filling it with water and subjecting it to a pressure of 5 psi. If the leakage rate from the hole over a period of 10 minutes exceeds 0.001 gallons per inch diameter per foot of depth per minute, the hole should be consolidation grouted, redrilled and retested. Should the second watertightness test fail, the entire process should be repeated.

Holes adjacent to a hole being tested for watertightness shall be observed during the test so that any inter-hole connection can be more easily detected.

3.5 Fabrication

3.5.1 Materials

Anchor material shall be in accordance with PCI Guide Specification for Post-Tensioning Materials.

Anchor material shall consist of either single or multiple units of the following:

a. Wires conforming to ASTM Designation A421, "Uncoated Stress-Relieved Wire for Prestressed Concrete."

b. Strand conforming to ASTM Designation A416 "Uncoated Seven-Wire Stress Relieved Strand for Prestressed Concrete."

c. High alloy steel bars, either smooth or deformed.

Stressing achorages shall be capable of developing 95 percent of the guaranteed minimum ultimate tensile strength of the anchor material when tested in an unbonded state.

Mill test reports for each heat or lot of pre-

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economical.

Drilling tolerances are controlled by the size of the drill steel, weight of the drill rig, the method of drilling, and the nature of the ground. Holes can be drilled to an angle tolerance of 3 percent of their planned location.

Holes are water tested to insure limited grout loss for proper anchoring of the tendon, and to insure corrosion protection by limiting loss of either anchor grout or secondary grout. Consistency of consolidation grout depends on the results of the water test. Should the water test indicate a high volume of leakage in the hole, a stiff consolidation grout should be used, such as, a maximum of five gallons water per sack of cement. Should the water test indicate a low volume of leakage, a very lean consolidation grout should be used, such as eight gallons of water per sack of cement.

It is normal practice to redrill a consolidation grouted hole after the grout has had 24 hours to set up.

Payment for consolidation grouting, redrilling and testing should be based on unit prices since these quantities are unpredictable. Typical payment units would be: water tests *(each)*; cement *(CWT)*; redrilling *(lin. ft.)*.

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stressing material used to fabricate tendons shall be submitted if required by the engineer.

3.5.2 Fabrication of Anchors

Anchors shall be either shop fabricated or field fabricated in accordance with approved details, using personnel trained and qualified in this type of work.

Anchors shall be free of dirt, detrimental rust or any other deleterious substance.

Anchors shall be handled and protected prior to installation in such a manner as to avoid corrosion and physical damage thereto.

Anchors may be either sheathed or unsheathed.

The sheathing may consist of tubes surrounding individual anchor elements *(bar, wire or strand)* or a single tube surrounding the elements altogether. A seal shall be provided to prevent the entry of grout into the sheath prior to stressing.

3.6 Insertion and Anchor Grouting

Anchors shall be placed in accordance with the recommendation of the manufacturer.

Anchors shall be securely fastened in place to prevent any movement during grouting.

Grout tubes and vent networks shall be checked with water or compressed air to insure that they are clear.

Care shall be taken to insure that the bond length of the anchor is centrally located in the hole.

If multi-unit tendons are used without a fixed anchorage at the lower end of the tendon, provision should be made for adequate spacing of the tendon elements to achieve proper grout coverage.

Grouting operations shall generally be in accordance with PCI "Recommended Practice for Grouting of Post-Tensioned Prestressed Concrete" and in accordance with the recommendations of the manufacturer.

Primary grout of the proper consistency shall be pumped into the anchor hole through a grout pipe provided for that purpose until the hole is filled to the top of the anchorage zone. The grout shall always be injected at the lowest point of the bond length.

Provisions shall be made for determining the level of the top of the primary grout to assure adequate anchorage.

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A light coating of rust on the anchor material is normal and will not affect the ability of the anchor to perform its function. Heavy corrosion or pitting should be cause for rejection of the anchor.

The sheathing material can be either steel, plastic or any other material non-detrimental to the high strength prestressing steel.

Centering devices are normally provided at about 10 ft. centers throughout the bond length.

It should be recognized that water separation or bleed creates a layer of water at the top of any grouting stage. For strand tendons where bleed is more pronounced, bleed water could be over 6 percent of the vertical height of the tendon. Chemical additives are available that will control the bleed. Collodial *(high energy)* grout mixers will reduce this phenomenon. In the case of two stage grouting, it is normal procedure to fill the void caused by bleed water at the top of the second stage by regrouting after the second stage grout has set.

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After grouting, the tendon shall remain undisturbed until the necessary strength has been obtained.

The following data concerning the grouting operation shall be recorded:

Type of Mixer
Water/Cement Ratio
Types of Additives
Grout Pressure
Type of Cement
Strength Test Samples
Volume of first and second stage grout

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In the case of sheathed anchors, the first stage grouting covers the full length of the anchor between the sheathing and the periphery of the hole, and may fill the space between the sheathing and tendon throughout the bond length. Second stage grouting may be used to fill the space between the sheathing and the tendon throughout the stressing length or throughout the entire anchor length.

For sheathed anchors, consideration should be given to force transfer through the grout in the annulus around the stressing length.

3.7 Stressing

Stressing shall generally be accomplished in accordance with "PCI Manual for Quality Control for Plants and Production of Precast Prestressed Concrete Products."

The anchor shall be first stressed to an initial load of about 10 percent of the test load, which is the starting point for elongation measurements.

Immediately thereafter, the anchor shall be stressed to the proof load and elongation is to be recorded. The magnitude of the proof load is to be determined by the engineer. If measured and calculated elongations disagree by more than 10 percent, an investigation shall be made to determine the source of the discrepancy.

When the above requirements are met, the anchor force shall be lowered and anchored at the transfer load. This load may be verified by a lift-off test and recorded, if required by the Engineer.

Stressing is normally carried out seven days after grouting for Type I or Type II cements and three days after grouting for Type III cement. At these times, grout with a water-cement ratio of 0.45 will have a compressive strength of about 3500 psi.

Movements of the bearing plate in excess of ½ inch shall be taken into consideration in comparing measured and theoretical elongations. For temporary rock anchors, elongation measurements are not usually required.

Usually, the proof load is specified as 115 percent to 150 percent of the transfer load. The proof loading of anchors is part of the stressing operation and occurs just prior to load transfer.

3.8 Testing

The stressing anchorages shall be capable of lift-off during the period of installation, in order to check the force.

The lift-off test, if any, is to be specified by the engineer. Allowances shall be made for time dependent losses when comparing the lift-off force with the previous transfer load.

The lift-off, if required, is usually done on a random basis. The engineer is to determine the percentage of tendons tested. Meaningful lift-offs can be taken as soon as 24 hrs. after the anchor is stressed. It is poor practice to require that the jack be left on an anchor since the jack bleeds off and the results are incorrect.

For most rock anchor applications, the primary time dependent loss is steel relaxation which can be as much as 3 percent of the transfer load in seven days depending on the type of steel. More exact values can be obtained from the rock anchor supplier.

3.9 Corrosion Protection

Prestressed rock anchors shall be protected against corrosion by procedures suitable for the

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intended service life.

3.9.1 Temporary Rock Anchors

Corrosion protection provided for temporary anchors shall be based on the intended service life of the anchor, and on the corrosion potential of the environment in which the anchor is to be installed. For wedge-type post-tensioning systems, protection shall be applied to the anchor head and wedge holes prior to insertion of wedges and stressing of tendons. Corrosion protection of temporary anchors shall be inspected and maintained throughout the service life of the anchor.

Appropriate spacers shall be provided to center the tendon in the hole throughout the bond length to insure adequate cover.

3.9.2 Permanent Rock Anchors

Permanent rock anchors shall be provided with protective corrosions seals over their entire length.

For tendons utilizing sheathing over the stressing length, the annulus between sheathing and tendon in the stressing length of the tendon shall be protected with a preplaced grease, powder corrosiion inhibitor or grout. A grout plug shall be provided to seal the end of the sheathing adjacent to the bond length. Grout shall be applied from the bottom of the anchor hole covering bond length and the annulus between sheathing and rock in the stressing length in one continuous operation.

COMMENTARY

When in rock where there is no apparent danger of corrosive attacks, temporary anchors with a service life up to 3 years are sometimes installed with no corrosion protection along the stressing length. However, normal practice for temporary anchors requires use of a ferrous metal or suitable plastic sheathing covering the stressing length to keep the prestressing steel dry and protect it from contact with the surrounding rock. A watertight seal should be provided between the sheathing and the grout in the bond length on one end and between the sheathing and anchorage device at the other end. The annular space between tendon and sheathing may contain preplaced grease or powder corrosion inhibitors. Asphaltic painting or grease corrosion protection of anchorage hardware is recommended. For wedge-type post-tensioning systems, a small amount of movement or travel of the wedges is required to develop force in the tendon above the transfer load. To develop the full tendon capacity, the required wedge movement may vary from approximately 1/32 inch to 1/8 inch depending on the wedge type and the transfer load level. Therefore, to assure that the tendons have capacity to sustain unanticipated loads substantially in excess of the transfer load, it is important that corrosion protection of anchorage hardware be provided and maintained.

Centering devices are normally provided at about 10 ft. centers throughout the bond length.

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Permanent rock anchors utilizing a two stage grout system may be fabricated without the use of sheathing above the bond length. Grout shall be injected from the bottom of the anchor to the top of the bond length. Grout quantity shall be continuously monitored. Secondary grouting shall be applied to the stressing length after stressing and any required stress monitoring are complete and accepted.

Special attention shall be given to assure corrosion protection of the tendon at the connection to the anchorage hardware. The anchorage hardware shall be protected by embedment in concrete or other suitable material.

4. SOIL ANCHORS

4.1 Description

A prestressed soil anchor is a high strength steel tendon, fitted with a stressing anchor at one end and an anchor device permitting force transfer to the soil on the other end. These anchors, which are used in clay, sand or other granular soils, are inserted into a prepared hole or driven into the soil. Concrete is gravity placed to form an anchorage, or grout is injected under pressure to form a bulb of grout to anchor the tendon. Pressure bulb soil anchors are usually equipped with a casing, which is withdrawn during the grouting operation. Subsequent to placement of anchor grout, the soil anchor is stressed and anchored at a specified force.

Soil anchors may be classified as follows depending on their use in cohesive or noncohesive soils.

Soil anchors in noncohesive material are generally pressure grouted *(See Fig. 2)*. They may be installed by two procedures:

1. Auger drilled - using hollow stem continuous flight augers normally of 6" to 10" diameter, the tendon is placed through the hollow stem of the auger before or after drilling is completed. Concrete or grout is then pumped under pressure through the hollow stem and the auger is withdrawn as the grout fills the hole.
2. Drilled or Driven Casing Pressure Grouted. In this type of anchor a 3" to 6" diameter casing is either drilled or driven into the ground to the final depth. The casing is then cleaned

COMMENTARY

A "lost point" on the bottom end of the casing is used in this method. The point remains in the ground during and after casing withdrawal.

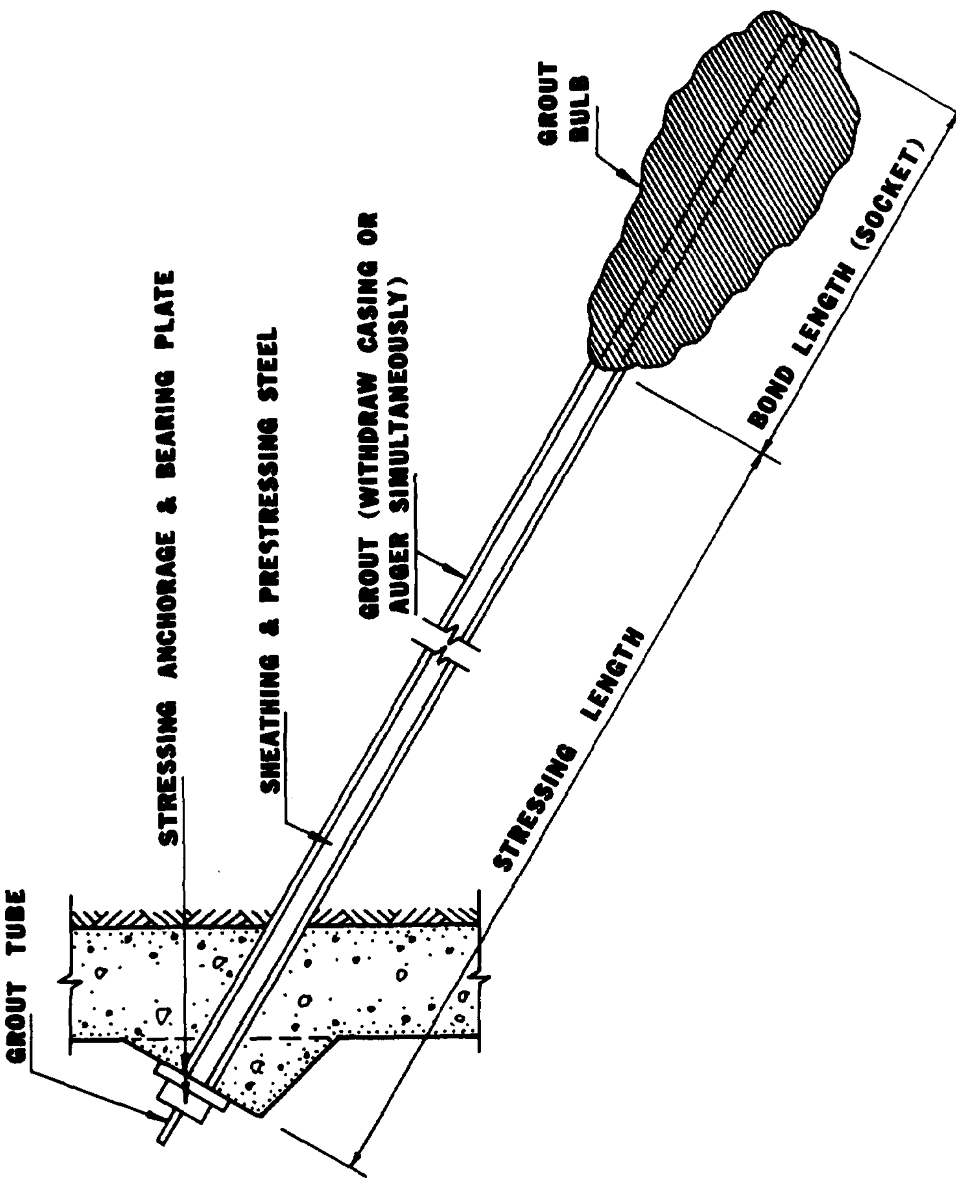

Figure 2 — Pressure grouted soil anchor

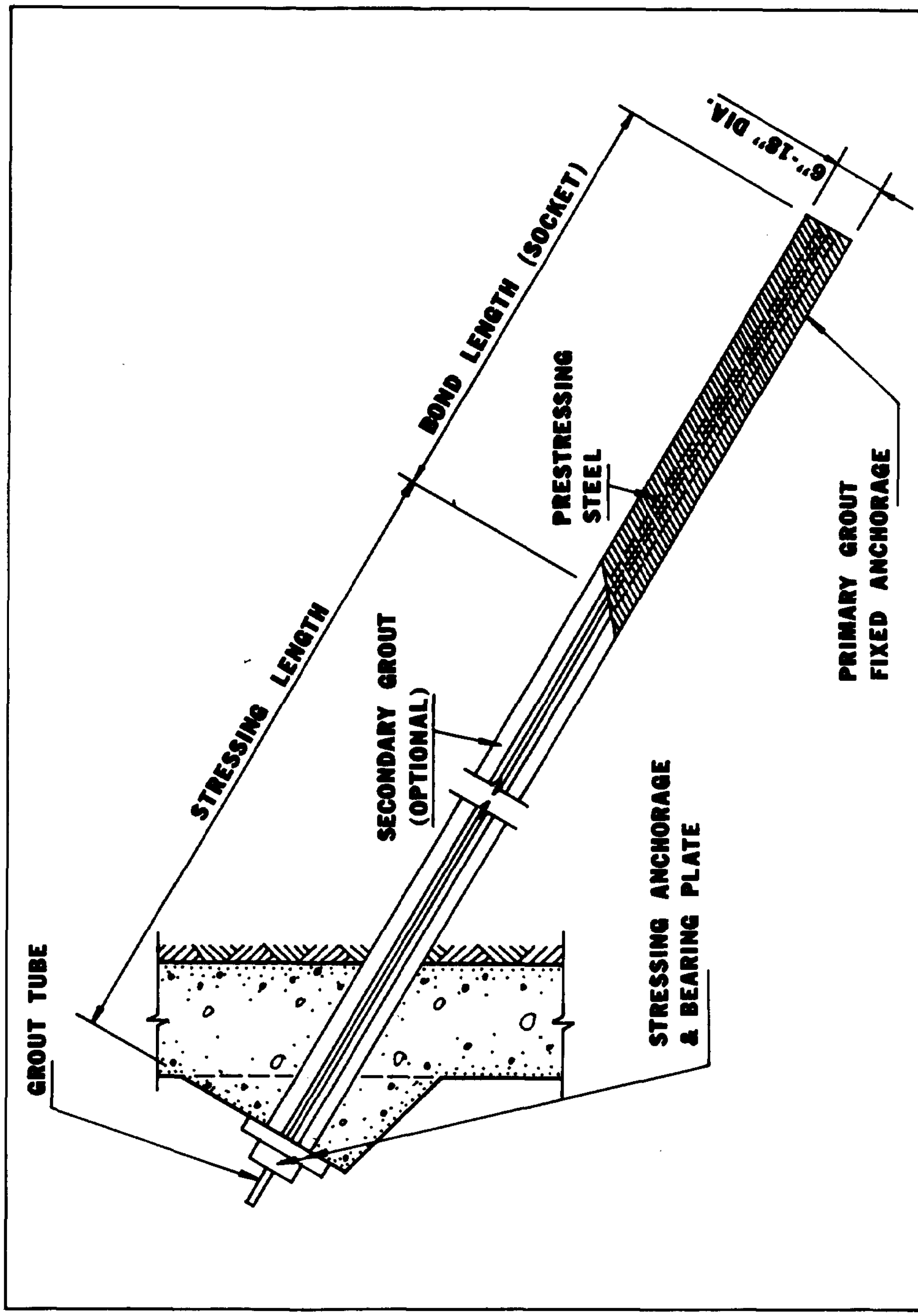

Figure 3 — Augered soil anchor

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out and the tendon inserted. The anchor is then pressure grouted over the anchoring zone as the casing is withdrawn. Grout pressures used vary from 50 to 200 psi.

Soil anchors in cohesive soils are generally of the following types:

1. Auger Drilled *(See Fig. 3)* - using either continuous flight augers or short augers on a Kelly Bar type of machine. These anchors differ from those drilled in cohesionless soil only in the way they are grouted. The auger is withdrawn before grouting, and pressure grouting is not used.
2. Belled Type Anchors *(See Fig. 4)* - Drilled either by a Kelly Bar type machine using augers and a standard caisson belling bucket or the drilled casing method which employs a small air or mechanically activated underreamer. The cuttings are removed by air or water flushing. Belled anchors rely on the bearing of the underream cones against the soil for resistance to pullout.

4.2 Design Considerations

The design of soil anchors is largely dependent on the soil conditions and upon the type of anchor used. Use of test anchors to determine the necessary bond length is strongly recommended for augered anchors and is essential for pressure bulb type soil anchors.

Minimum stressing lengths of 20 to 25 ft. are recommended.

4.3 Drilling

4.3.1 Augered holes

Augered holes may vary from 6 inches to 24 inches in diameter and lengths may be as much as 100 feet. Some augers have attachments which permit belling or enlarging the bottom of the hole. More than one bell may be provided in cohesive soils.

COMMENTARY

For large diameter holes, augered anchor bond stresses in the bond length are normally about 10 psi although there can be a wide variation in this figure. It is not practical to give typical bond stress values for pressure bulb type soil anchors. Pressure bulb anchors develop the tendon force partially through bond and partially through bearing of the bulb of the soil. The response of soils to the pressure grouting varies widely, and, for this reason, field anchor tests are necessary to properly design pressure bulb anchors.

The minimum stressing lengths recommended are necessary so that small movements of the stressing anchor will not result in large changes in load.

Augered holes are the fastest method of drilling a soil anchor.

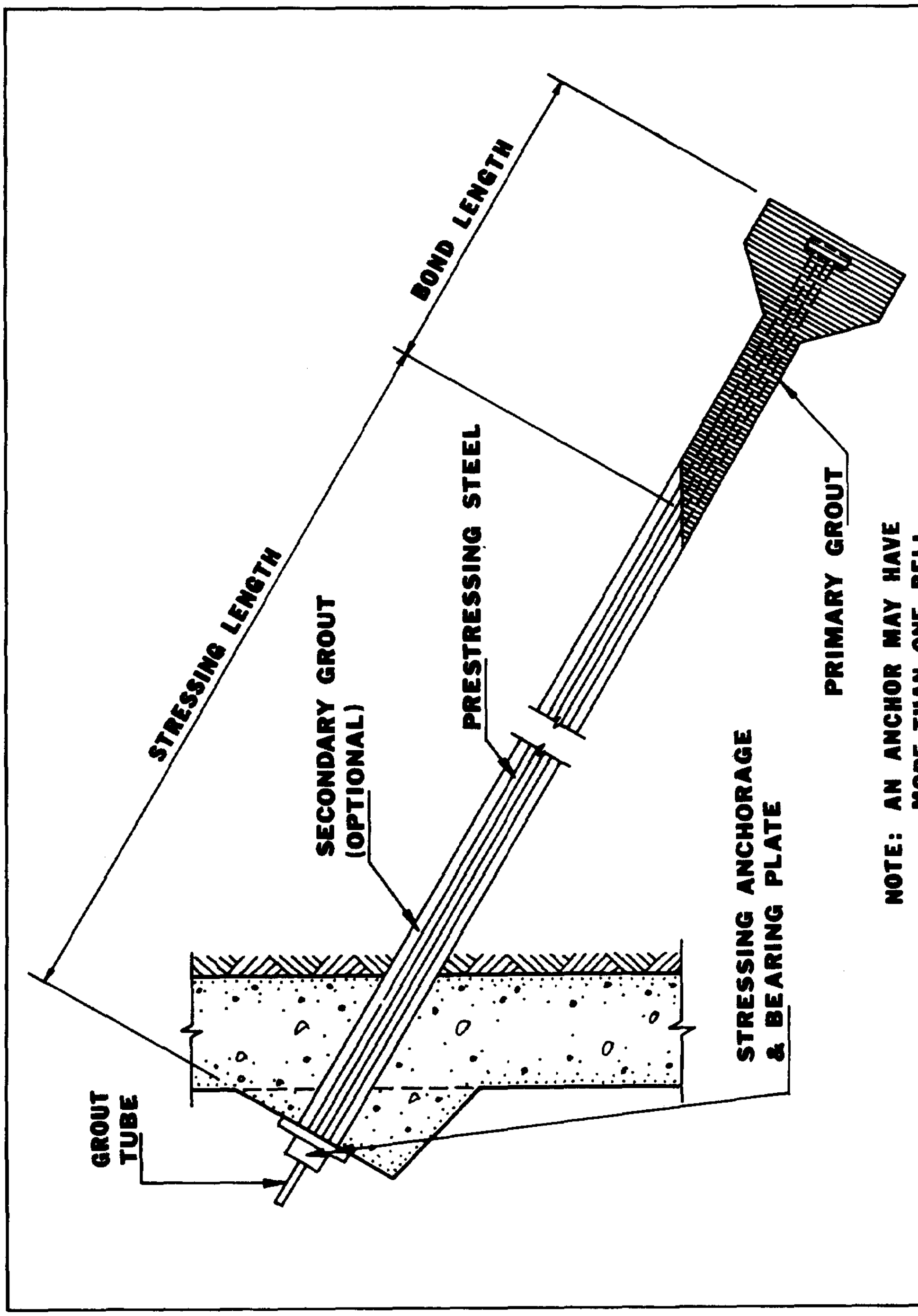

Figure 4 – Belled soil anchor

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COMMENTARY

4.3.2 Pressure Grouted Anchors

Pressure grouted anchors are installed by either ramming a casing with a detachable point using an air track, or by augering a small hole with a hollow stem continuous flight auger.

Ramming is usually only employed in fairly loose sands and gravels.

4.4 Fabrication

4.4.1 Materials

Soil anchor materials shall conform to the requirements of Section 3.4.1 Materials for prestressed rock anchors.

4.4.2 Fabrication of Anchors

Anchors shall be either shop fabricated or field fabricated in accordance with approved details, using personnel trained and qualified in this type of work.

Anchors shall be free of dirt, detrimental rust or any other deleterious substance. Anchors shall be handled and protected prior to installation in such a manner as to avoid corrosion and physical damage.

A light coating of rust on the anchor material is normal and will not affect the ability of the anchor to perform its function. Heavy corrosion or pitting should be cause for rejection of the anchor.

Spacers are normally provided at about 5 ft. centers in the bond length of augered anchors.

The sheathing material can be either steel, plastic or any other material non-detrimental to the prestressing steel.

Anchors may be either sheathed or unsheathed.

4.5 Insertion and Anchor Grouting

4.5.1 Augered or Belled Anchors

Soil anchors are manually inserted in augered holes. Concrete or grout is pumped or gravity placed into the bond length of the anchor.

4.5.2 Pressure Grouted Anchors

4.5.2.1 Rammed Anchors

The prestressing tendon is inserted in the casing and driven to its final position with the casing, or the tendon may be inserted after the casing is driven. Grout, under pressure, is pumped into the sealed casing as the casing is withdrawn from the hole by means of hydraulic jacks. After the casing has been withdrawn from the bond length, pressure grouting is discontinued and the casing may be withdrawn.

It is common practice to withdraw the casing and continue pumping grout at pressures high enough to result in a grout requirement of one bag of cement per foot of hole. However, the grout requirement depends greatly on the hole diameter, and the permeability and density of the soil.

4.5.2.2 Augered Pressure Anchors

A small diameter continuous flight auger is used to drill the hole. The procedure for installing this type of anchor is exactly the same as the

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driven anchor described above with the exception that the auger is always completely withdrawn.

4.5.2.3 Upward Sloped Soil Anchors

Pressure type soil anchors may be installed on upward slopes.

4.6 Stressing

Stressing shall generally be accomplished in accordance with "PCI Manual for Quality Control for Plants and Production of Precast Prestressed Concrete Products."

4.7 Testing

Soil anchors in cohesive soils normally require more testing than rock anchors since cohesive soils may creep under sustained load. Continuous monitoring systems may be employed when specified by the Engineer.

4.8 Corrosion Protection

Measures to provide corrosion protection for soil anchors vary depending on whether the anchor is intended for temporary or permanent use. In both cases, protective measures are similar to those for prestressed rock anchors presented in Sections 3.9.1 and 3.9.2.

COMMENTARY

Stressing is normally carried out seven days after grouting for Type I or Type II cements, and three days after grouting for Type III cement. At these times, grout with a water-cement ratio of 0.45 will have a compressive strength of about 3500 psi.

Soil anchors are normally stressed to 15 to 50 percent above design load, held at that load for 5 or 10 minutes, and then relaxed and anchored at the design load.

Lift-off tests are sometimes performed on selected anchors; these may be of 8-hour duration in the case of granular soils, but 24-hour duration may be called for on anchors in cohesive soils

The average monitoring system consists of a load cell placed behind the stressing anchorage. This load cell has SR4 strain gauges installed on it, and the results can be directly read on a Wheatstone bridge. A separate payment item should be set up for monitoring.

APPENDIX

TYPICAL APPLICATIONS OF PRESTRESSED ROCK AND SOIL ANCHORS

A.1 INTRODUCTION

The purpose of this appendix is to illustrate typical applications of prestressed rock and soil anchors as they have been used to date in the United States and Canada. The first application of a prestressed anchor dates back to 1935 when the late Andre Coyne, a French engineer, used prestressed anchors to stabilize the Cheurfas Dam in Algeria. Until recently, the 1100 ton anchors used in the Cheurfas Dam were the largest ever installed in a structure. This project generated a number of new systems and applications in Europe. However, the widespread use of prestressed rock and soil anchors is a relatively recent development in North America.

The examples presented below are representative of the techniques used most often in present-day construction. Additional applications and variations of the projects illustrated have been used, and no doubt more will be developed as engineers become more familiar with this construction technique.

A.2 BUILDING EXCAVATIONS

Rock and soil anchors make possible the effective use of modern excavation methods by eliminating internal bracing and allowing free movement of the excavating equipment. Prestressed tie-backs also make it possible to reduce or eliminate settlements of structures adjacent to the excavation because wall movements or deflections can be minimized. Many different wall types have been anchored with prestressed rock and soil anchors including: Steel H piles with wooden lagging; steel sheet piles; drilled concrete shafts; and, slurry walls. Tie-back applications for building excavations are illustrated in Figs. A-1, A-2, A-3, and A-4.

A.3 RETAINING WALLS AND REVETMENTS

The rather unusual structure in San Francisco shown in Figs. A-5 and A-6 is one of many walls with post-tensioned tie-backs built under plans developed by the Bridge Department of the California Division of Highways. This retaining wall is a series of twenty-two thin horizontal arch sections supporting the earth between inclined buttresses that are anchored into the earth and rock by prestressed anchors. The wall has a maximum height of about 60 ft. and was built in a series of 12 ft. increments from the top down as excavation proceeded. The precast concrete blocks used for the inclined buttresses were anchored by 100 kip soil anchors with length varying from 30 to 45 ft. The post-tensioned wall was selected to minimize earth movements which might cause damage to the property on top of the wall. Also, because no footing was required, it was unnecessary to disturb or shore the property at the top of the wall during construction.

Fig. A-7 shows details of a retaining wall built on Interstate Highway I-96 in Detroit. The wall was built in extremely poor clay and rock anchors were used to minimize potential soil movement effects on the wall and the I-96 freeway. Figs. A-8 through A-10 illustrate installation and stressing of the 1575 kip rock anchors which utilized 54 half inch diameter 270k strands.

Fig. A-11 shows details of a retaining wall in San Diego which utilizes five foot diameter concrete "soldier beams" and soil anchors at 10 ft. centers to minimize potential soil movement and possible damage to the adjacent apartment building. The vertical beams and tie-backs were built first and then the concrete wall was built in front of the vertical beams. The 90 kip soil anchors were placed in a 10 inch diameter hole and varied in length from 24 to 31 ft.

Over a period of years, apparently stable rock faces can deteriorate by weathering. This can be prevented by prestressing a retaining wall against the rock using rock anchors as shown schematically in Feg. A-12. Fig. A-13 shows a Swiss application of a revetment of this type. In some cases, the concrete retaining wall may be replaced by a direct application of shot-crete to the rock face. The rock anchors are used in the same fashion.

A.4 SLOPE STABILITY

The availability of prestressed rock anchors installed as illustrated in Fig. A-14 often provide an economical means of stabilizing rock slopes. The other tools available to cope with stability problems are huge gravity retaining walls or cuts of slope angle to coincidde with the mechanical properties of the rock. Both of these solutions require large excavations.

The rock slope at Libby Dam in Montana shown in Feb. A-15 had three huge faults, minor, flat, and approximately 30° downward. After the cut section was excavated for highway construction, the upper bed of stratum slid down along the first fault, leaving an uneven rock surface exposed. The second bed then became dangerously unstable. A retaining structure of any kind would not have been practical or feasible. After extensive rock mechanics investigation, it was determined that an effective prestressing force of 18,000 kips, making an approximate 60% angle with the fault, would stabilize the surface of the rock. This force would tie the unstable portion to the main rock mass of the slope as well as increase the frictional forces in the failure plane. Fig. A-16 shows installation of the 200 kip anchors, and Fig. A-17 shows the concrete cover placed over the stressing anchorages to provide complete corrosion protection.

A.5 STABILIZATION OF UNDERGROUND EXCAVATION

Stabilization of tunnel excavations in rock with prestressed rock anchors is distinguished from the more conventional lining methods by the fact that the prestressing creates an active arch in the rock making it act as its own structural support. This technique avoids costly bracing and shoring and increases the speed of excavation.

Fig. A-18 shows drilling preparatory to placement of a prestressed rock anchor in a tunnel.

A.6 DAM STABILIZATION

Requirements for increased capacity or improved safety of concrete dams can often be facilitated by use of prestressed rock anchors. This method is commonly used when an existing dam is to be increased in height, and prestressed rock anchors may also be helpful in restoring the water-tightness of cracked dams or locks by compressing the structure and closing the cracks.

Fig. A-19 shows drilling in progress for installation of prestressed rock anchors on the Ocoee Dam in Tennessee. Fig. A-20 shows installation of the 170 wire tendons used to provide improved stability of the dam.

A.7 ANCHORAGE AGAINST UPWARD WATER PRESSURE

When basin-shaped structures located in an area of high ground-water are in an unloaded state, danger of uplift exists. A possible solution to this problem is mass concrete, but the cost of the extra excavation and material sometimes renders this method too expensive. Rock anchors may often be used in such a situation with a substantial reduction in cost.

A.8 ANCHORAGE OF FIXED POINTS

Suspension bridges, cableways, locations where pressure pipes change direction, pylons or other structures can often be advantageously anchored with rock or soil anchors. Most important here is the fact that the transfer of the anchoring force can be effected in a stable load-carrying ground or rock zone.

Fig. A-21 shows the Hudson Hope Suspension Bridge in British Columbia. Anchorage of the main suspension cables was achieved by use of the prestressed anchor details shown in Fig. A-22. In the case of the Hudson Hope Bridge, the post-tensioned anchors were preferred because the anchor plates would remain virtually motionless under all load conditions, and no anchor load would be transferred to the surface rock which was normally fissured or water bearing. In some applications, post-tensioned suspension bridge anchorages have shown an economic advantage over alternate methods, and, in general post-tensioned anchorages will provide a more accurate and constant zero point for connection of the cable strands.

The problem of supporting concentrated forces is often encountered in all types of underground construction. Rock anchors allow the transfer of these loads directly to the rock without the necessity of providing a heavier lining. Typical examples are crane rails, derricks and structural support beams.

In bridges where overturning forces due to earthquake or wind are large, post-tensioned rock anchors may be a more economical way to provide the necessary anchorage. The Pine Valley Creek Bridge near San Diego shown under construction in Fig. A-23 has piers ranging to 340 ft. in height, and stability under earthquake forces is obtained through use of post-tensioned rock anchors connecting the footing to the underlying rock mass. In this case, the rock anchors were considered more economical than additional excavation and use of large gravity-type concrete foundations.

A.9 MISCELLANEOUS APPLICATIONS

Rock or soil anchors may cut time and cost when used for pile testing, and often add safety and flexibility. Loads are applied to the pile through a beam loaded by prestressed anchors, thus eliminating the necessity of large testing weights. The anchor will give precise and continuous indications of pile movement as well as allow measurement of applied load.

Settlements of structures in compressible ground can be avoided by extending their foundations to a solid stratum, but sometimes this is not practical. In these cases, settlement can be induced prior to construction by loading the foundations with rock or soil anchors. The anchorage forces, of course, would be of the same magnitude as the future design load and must act on the stratum which causes settlement.

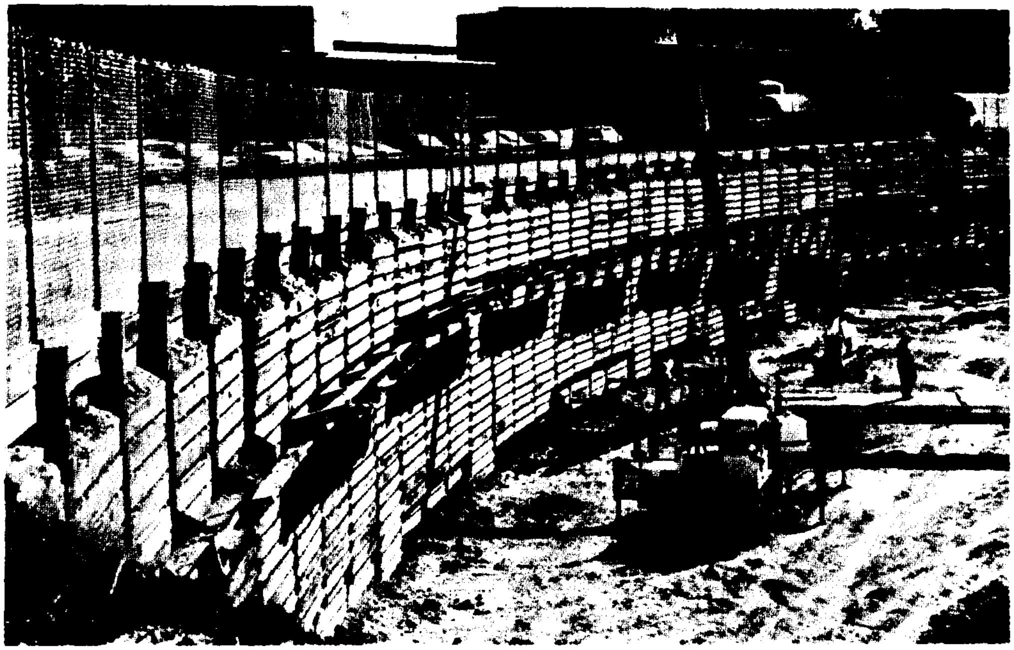

Figure A-1 — Prestressed Soil Anchors used to tie-back a wall of soldier piles with wooden lagging.

Figure A-2 — Drilled concrete shafts supported by three levels of tie-backs and walers.

Figure A-3 — Concrete slurry wall. Top level of anchors utilize steel walers to transmit anchor force to wall; bottom layer of anchors use concrete pedestals.

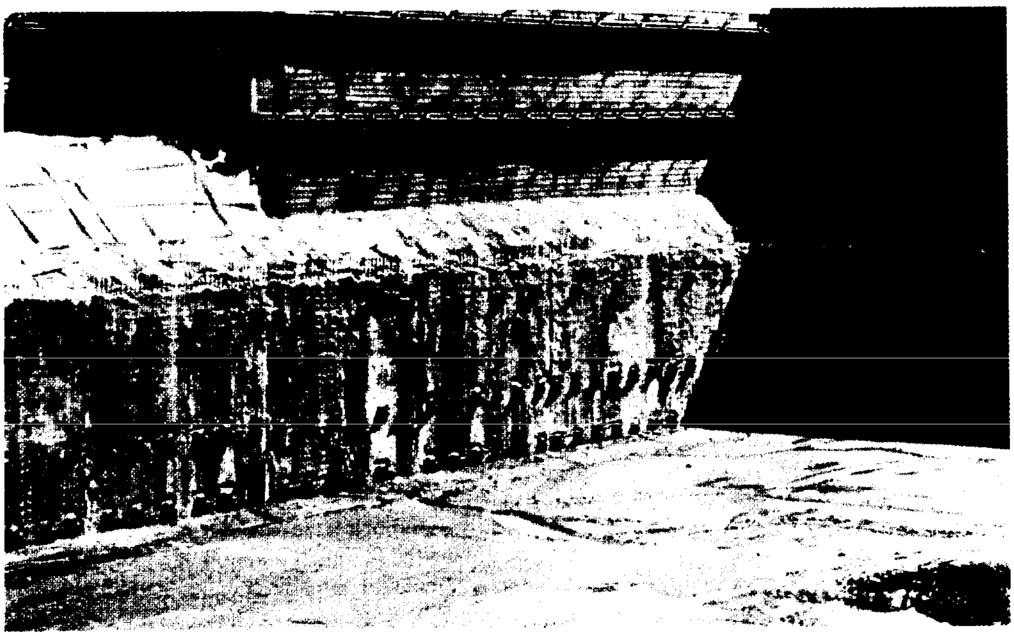

Figure A-4 — Upper wall of soldier piles and wooden lagging braced by two layers of tie-backs with steel walers. Lower level of reinforced slurry wall construction with multiple levels of tie-backs.

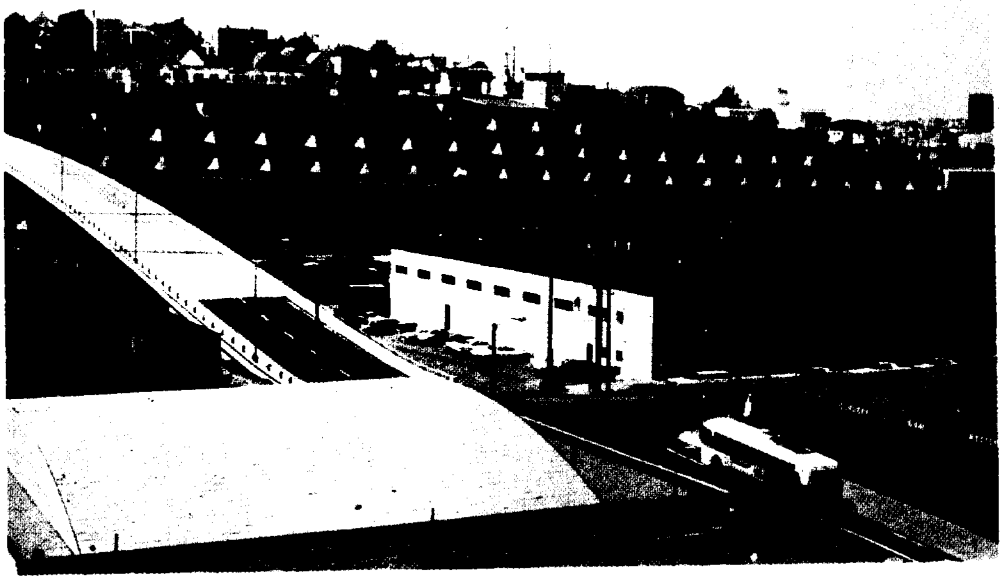

Figure A-5 — Protrero Hill retaining wall, San Francisco

Figure A-6 — Protrero Hill retaining wall, San Francisco

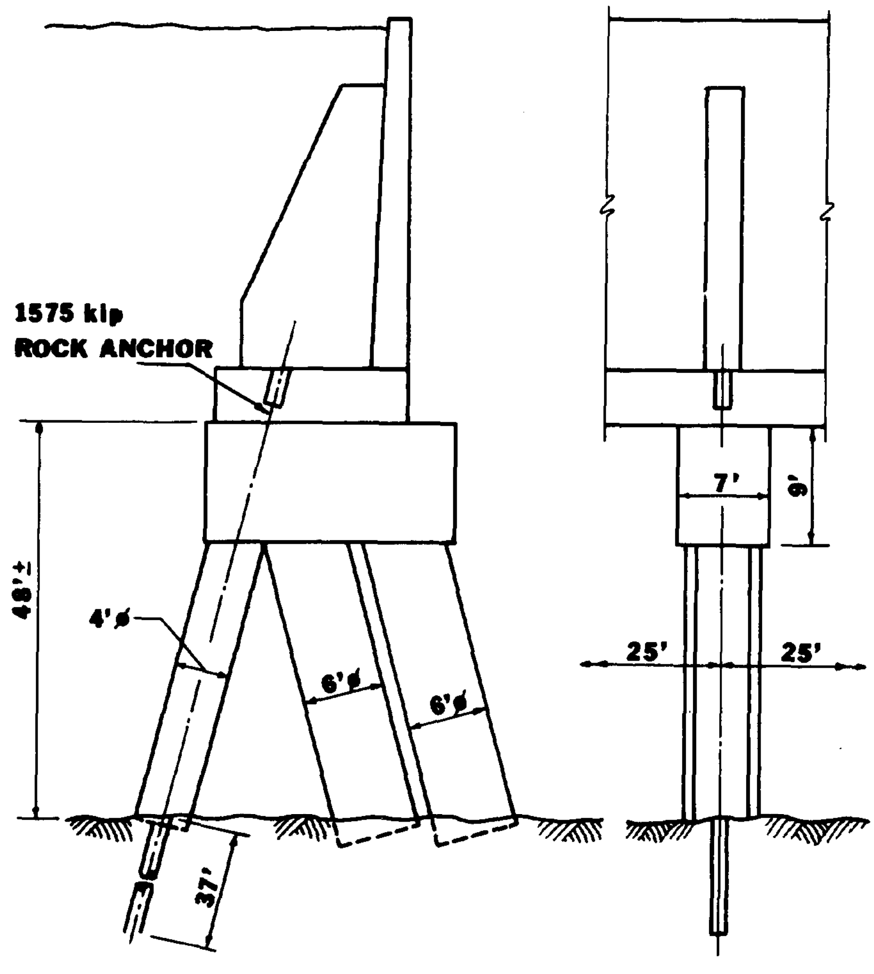

Figure A-7 — Retaining wall details, Interstate Highway I-96, Detroit

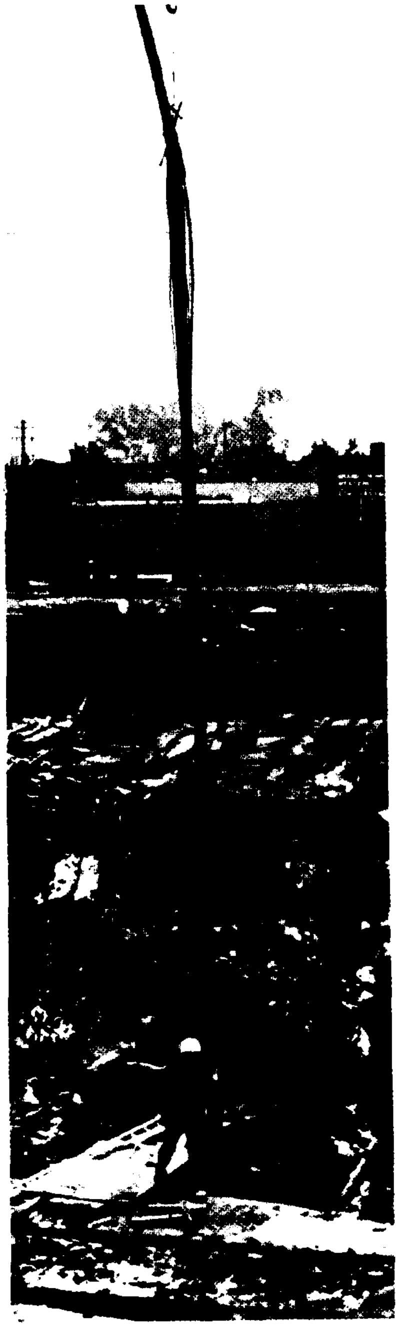

Figure A-8 – Tendon installation, Interstate Highway I-96 retaining wall, Detroit

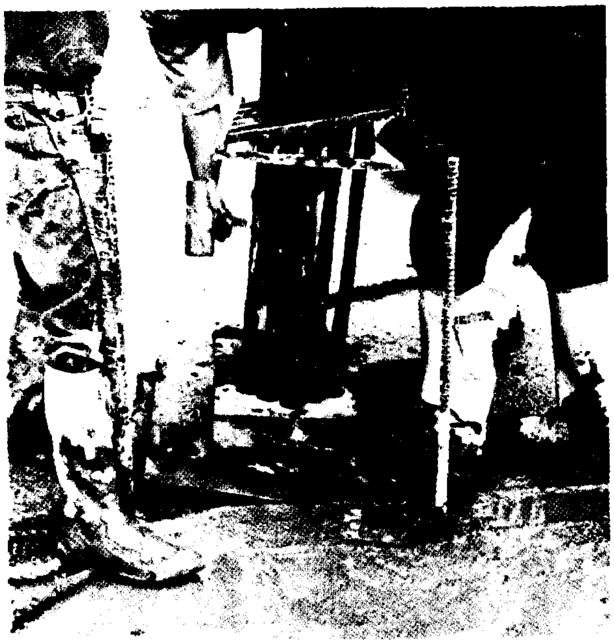

Figure A-9 – Placing wedge anchor plate, Interstate Highway I-96 retaining wall, Detroit

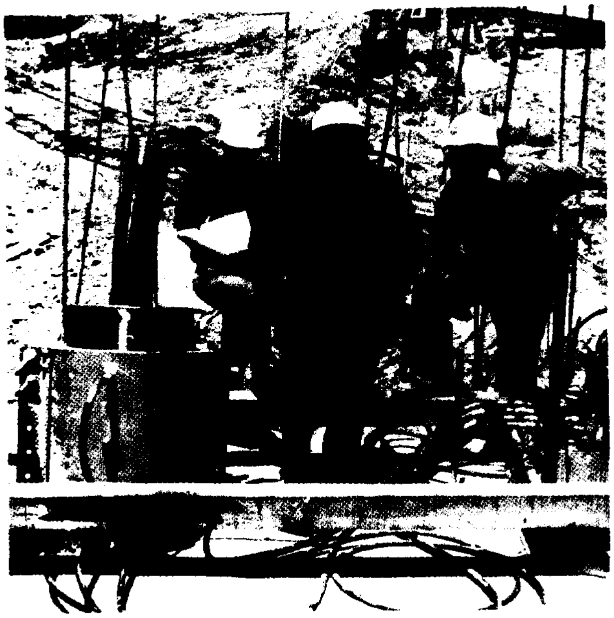

Figure A-10 – Stressing 54 strand tendon, Interstate Highway I-96 retaining wall, Detroit

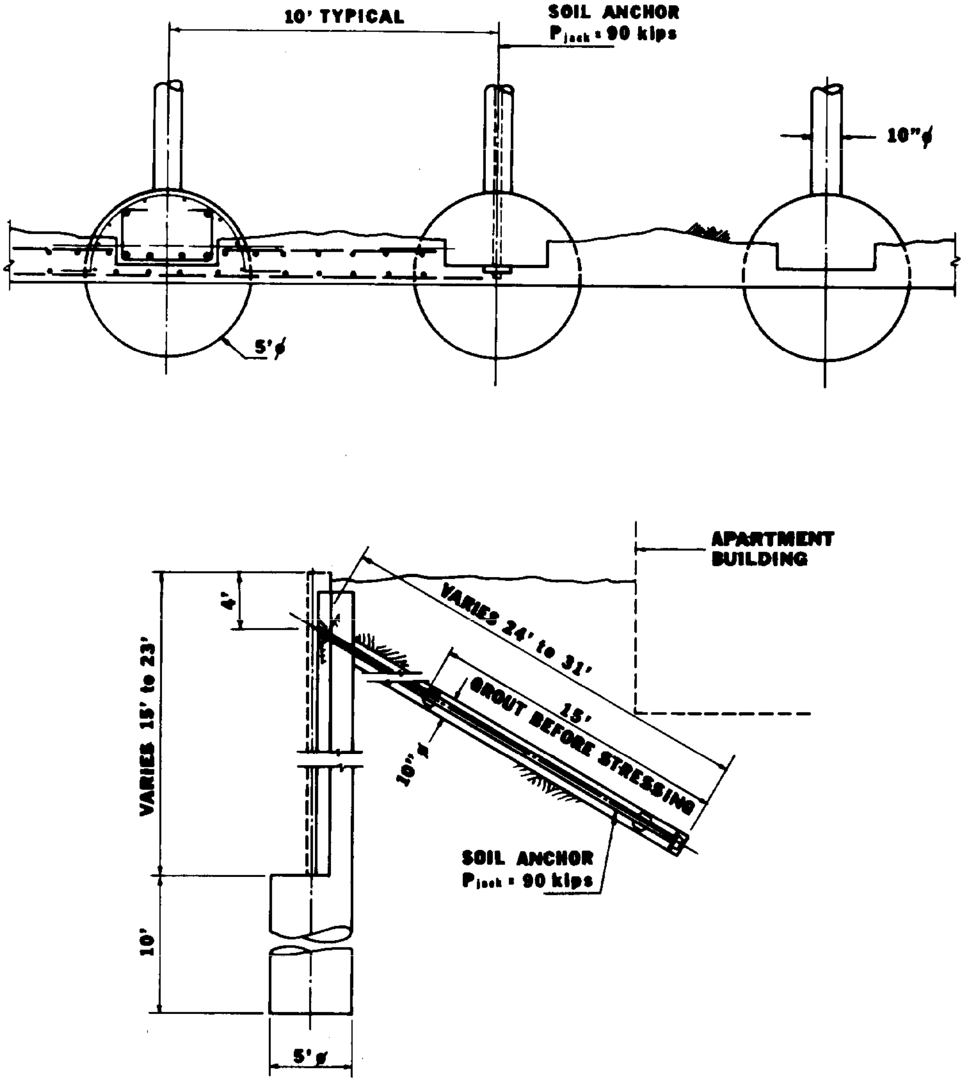

Figure A-11 – San Diego retaining wall details

Figure A-12 – Rock face revetment concept with rock anchors

Figure A-13 – Rock face revetment, Switzerland

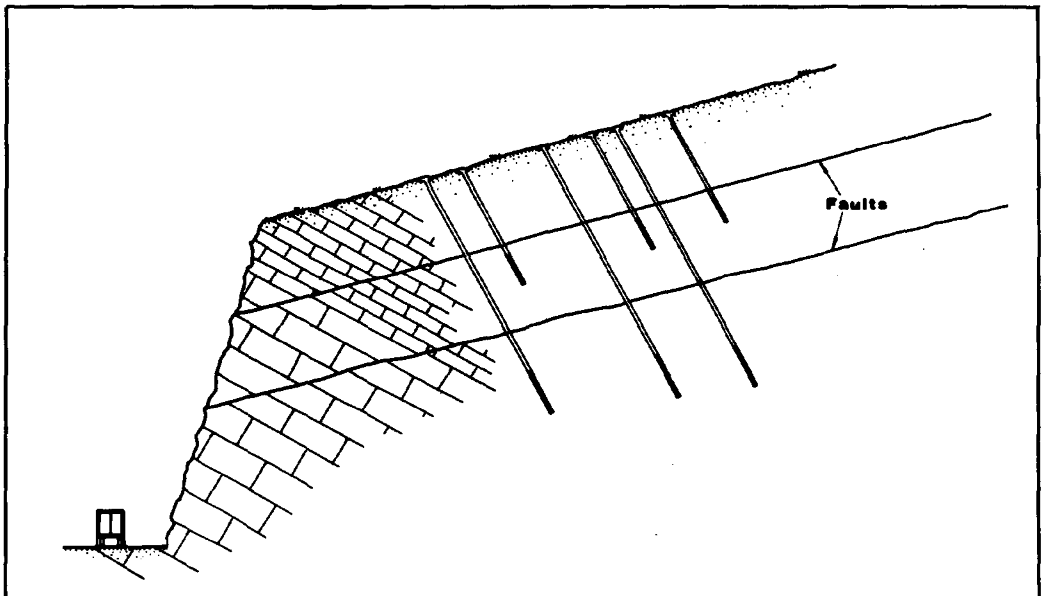

Figure A-14 – Concept of slope stabilization with rock anchors

Figure A-15 – Rock slopes at Libby Dam, Montana

Figure A-16 – Installation of 200 kip anchors, Libby Dam, Montana

Figure A-17 – Concrete cover over stressing anchorages, Libby Dam, Montana

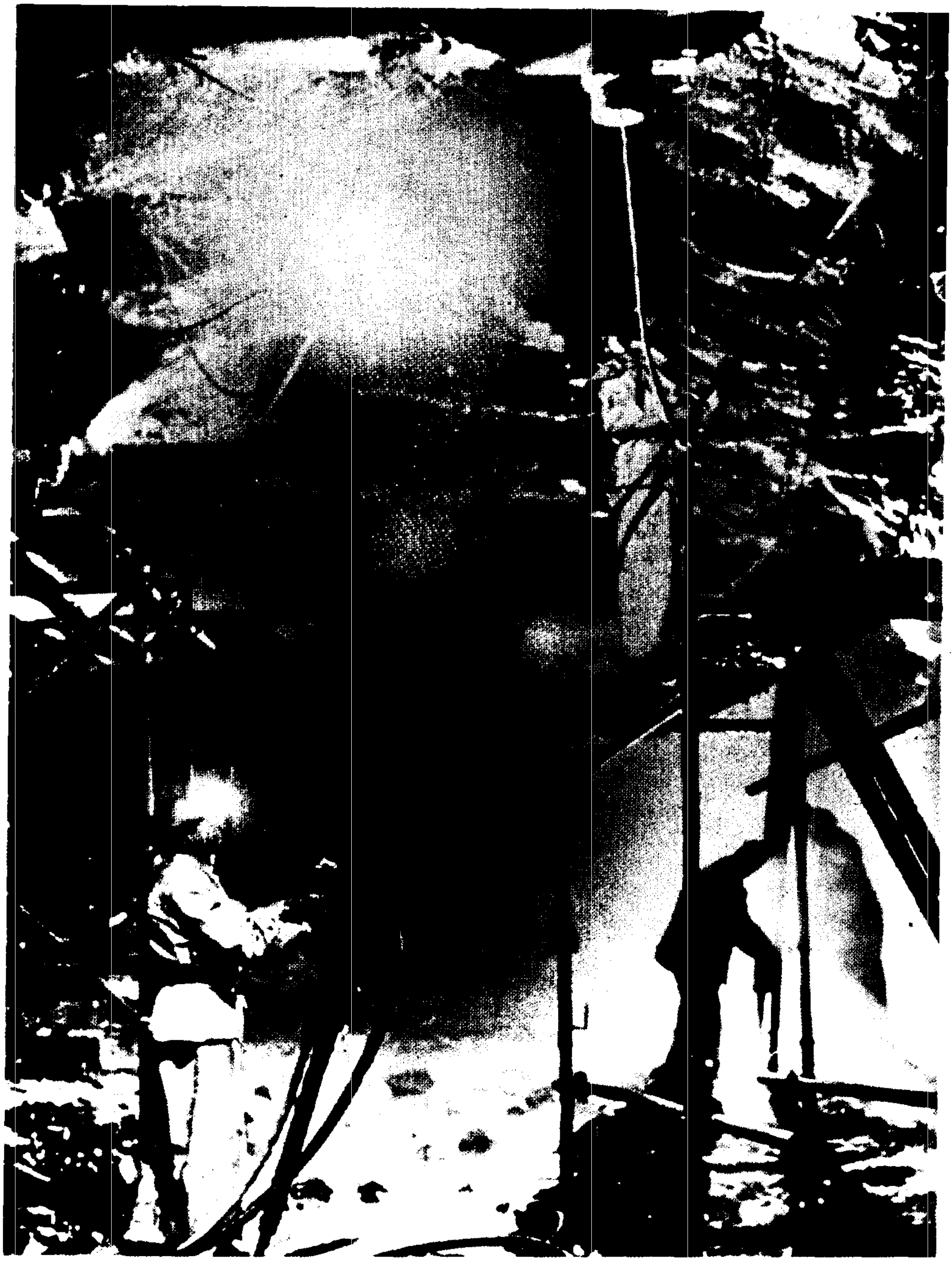

Figure A-18 – Drilling for rock anchors in tunnel excavation

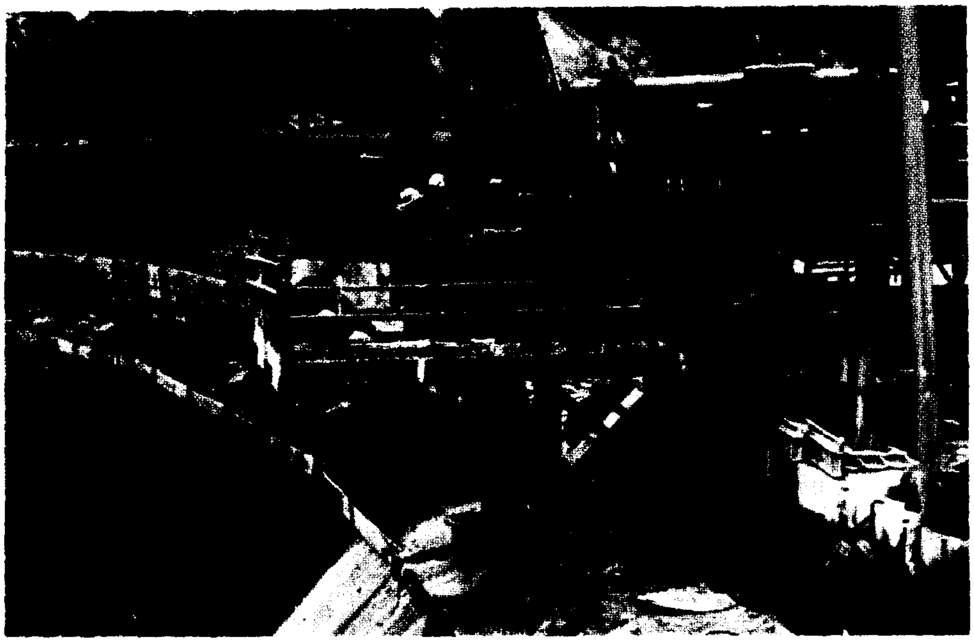

Figure A-19 — Drilling for rock anchor installation, Ocoee Dam, Tennessee

Figure A-20 — Installation of rock anchors, Ocoee Dam, Tennessee

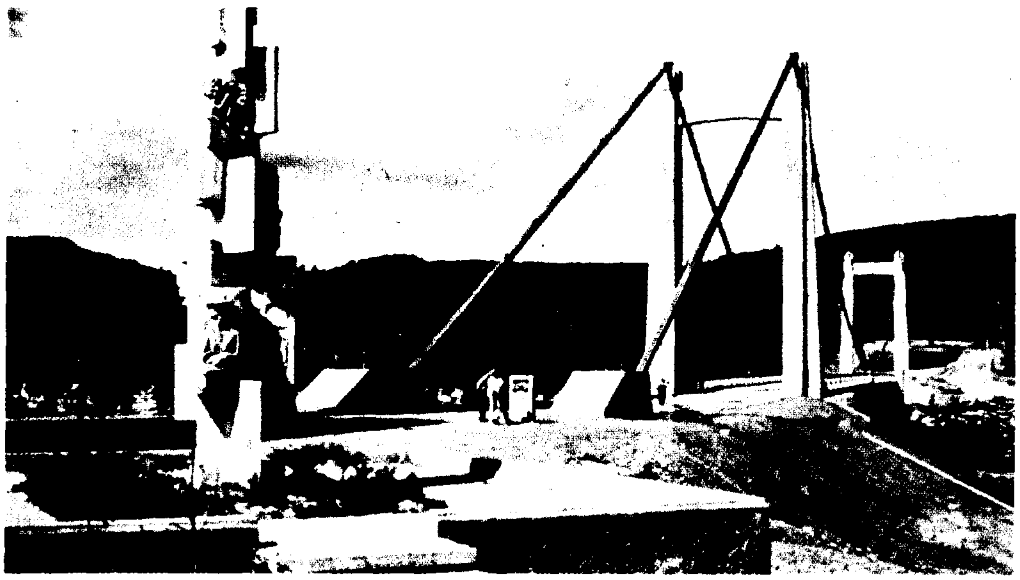

Figure A-21 – Hudson Hope Suspension Bridge, British Columbia

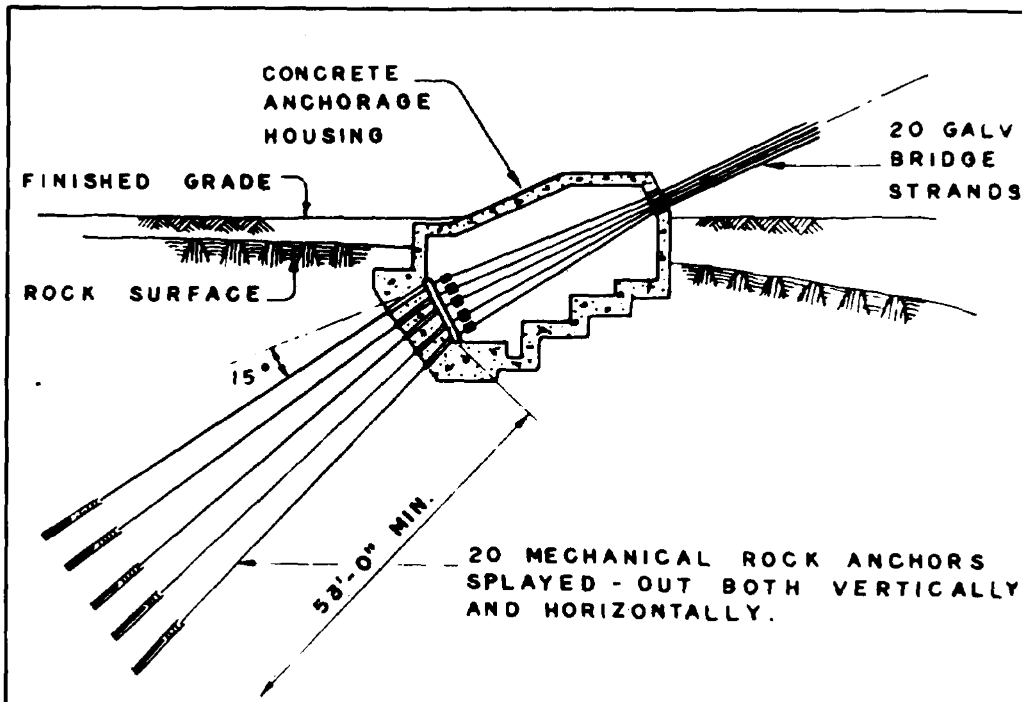

Figure A-22 – Prestressed suspension cable anchorages, Hudson Hope Suspension Bridge, British Columbia

Figure A-23 – Pine Valley Creek Bridge, California

APPENDIX F
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