Chapter 18

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STABILIZATION OF ROCK SLOPES

1. INTRODUCTION

A successful rock slope stabilization program requires the integration of a number of interrelated activities, including geotechnical engineering as well as environmental and safety issues, construction methods and costs, and contracting procedures. Modern methods for design and stabilization of rock slopes were developed in the 1970s (Brawner and Wyllie 1975; Fookes and Sweeney 1976; Groupe d'Etudes des Falaises 1978; Piteau and Peckover 1978; Hoek and Bray 1981) and continue to be refined and developed (Federal Highway Administration 1989; Wyllie 1991; Schuster 1992). These procedures can therefore be used with confidence for a wide range of geological conditions. However, as described in this chapter, it is essential that the methods used be appropriate for the particular conditions at each site.

1.1 Effect of Rock Falls on Transportation Systems

The safe operation of transportation routes in mountainous terrain often requires that measures be taken to control the incidence of rock falls and rock slope failures. Along highways, the types of events and accidents that can be caused by rock falls range from minor falls that damage tires and bodywork, to larger falls that affect vehicles or cause vehicles to swerve off the road, to substantial slope failures that block the facility. The effects of these events can be damage to vehicles, injury to or death of drivers and passengers, economic loss due to road closures, and possibly discharge of toxic substances when transporting vehicles are damaged. Figure 18-1 shows a rock slide that occurred from a height of about 300 m above the road and closed both the road and the railway. The removal of the rock on the road followed by stabilization of the slide area, which brought down further unstable rock, resulted in a total closure time of several weeks with significant economic losses for users of both the road and the railway.

Although the cost of a major slide such as that shown in Figure 18-1 is substantial, the cost of even a single-car accident can be significant. For example, costs may be incurred for hospitalization of the driver and passengers, for repair to the vehicle, and in some cases for legal costs and compensation. Often there are additional costs for stabilization of the slope that will involve both engineering and contracting charges, usually carried out at premium rates because of the emergency nature of the work. In cases where the rock slide results in closure of a road, indirect costs may be incurred, such as loss of revenue for businesses located along the highway. In the case shown in Figure 18-1, the costs included leasing a car ferry to move vehicles around the landslide. For railroads and toll highways, closures result in a direct loss of revenue that may amount to thousands of dollars per hour.

FIGURE 18-1 Railway and highway at Howe Sound, British Columbia, blocked by October 1990 rock fall that originated approximately 300 m above highway.
DENISE HOWARD, VANCOUVER SUN

1.2 Causes of Rock Falls

The California Department of Transportation (Caltrans) made a comprehensive study of rock falls that have occurred on the state highway system to assess both their causes and the effectiveness of the various remedial measures that have been implemented (McCauley et al. 1985). Because of the diverse topography and climate within California, Caltrans records provide a useful guideline on the stability conditions of rock slopes and the causes of falls. Table 18-1 shows the results of a study of 308 rock falls on California highways in which 14 different causes of instability were identified.

Of the 14 causes of California rock falls identified, 6 are directly related to water, namely, rain, freeze-thaw, snowmelt, channeled runoff, differential erosion, and springs or seeps. One cause is indirectly related to rainfall—the growth of tree roots in cracks, which can open fractures and loosen blocks of rock on the slope face. These seven causes of rock falls together account for 68 percent of the total falls.

These statistics are confirmed by the authors' experience in the analysis of rock-fall records over a 19-year period on a major railroad in western Canada, in which approximately 70 percent of the events occurred during the winter. Weather conditions during the winter included heavy rainfall, prolonged periods of freezing temperatures, and daily freeze-thaw cycles in the fall and spring. The results of a similar study carried out by Peckover (1975) are shown in Figure 18-2. They clearly show that the majority of rock falls occurred between October and March, the wettest and coldest time of the year in western Canada.

The other major group of factors affecting stability in the California study were the particular

Table 18-1
Causes of Rock Falls on Highways in California

Cause	Percentage of Total
Rain	30
Freeze-thaw	21
Fractured rock	12
Wind	12
Snowmelt	8
Channeled runoff	7
Adverse planar fracture	5
Burrowing animals	2
Differential erosion	1
Tree roots	0.6
Springs or seeps	0.6
Wild animals	0.3
Truck vibrations	0.3
Soil decomposition	0.3

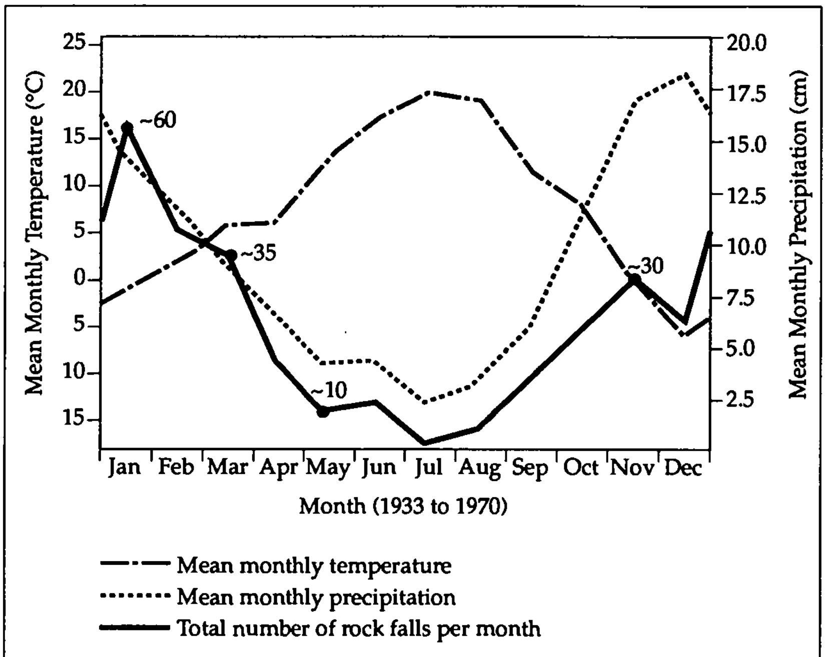


FIGURE 18-2 Correlation of number of rock falls with temperature and precipitation on railway lines in Fraser Canyon, British Columbia (Peckover 1975).

geologic conditions at each site, namely, fractured rock, adverse planar fracture (a fracture dipping out of the slope face), and soil decomposition. These three factors represented 17 percent of the falls, and the total rock falls caused by water and geologic factors accounted for 85 percent of the falls. These statistics demonstrate that water and geology are the most important influences in rock slope stability.

It appears that the study in California was carried out during a time when there were no significant earthquakes, which frequently trigger rock falls and cause displacement and failure of slopes (Van Velsor and Walkinshaw 1991). The mechanism by which earthquakes trigger landslides is described in Chapter 4, and methods of incorporating earthquake forces into the design of rock slopes are demonstrated in Chapter 15.

2. PLANNING SLOPE STABILIZATION PROGRAMS

In mountainous areas where there are numerous rock-fall hazards that may result in a significant cost to the operator of the transportation system, a stabilization program is often justified. The situation may be particularly severe where the slopes have been open for 20 to 30 years and are in an unstable condition either because of poor initial construction practices or because of weathering. Under these circumstances, stabilization work can take many years to complete. In order to make the best use of available funds, it often is beneficial to set up a systematic program that identifies the most hazardous slopes. Then annual stabilization work can be scheduled, with the most hazardous sites being worked on early in the program. The components of such a program make up six steps, as shown in Figure 18-3.

The objective of the program shown in Figure 18-3 is to be proactive in identifying and stabilizing slopes before rock falls and accidents occur. This requires a careful examination of the site to identify the potential hazard and estimate the likely benefit of the stabilization work. In contrast, a reactive program places the emphasis on areas in which rock falls and accidents have already occurred and the hazard may be significantly diminished. The benefits of a reactive program are likely to be less than those of a proactive program.

An effective proactive approach to stabilization requires a consistent, long-term program under the direction of a team experienced in both the engineering and construction aspects of this work. Another important component of this work is to keep accurate records, with photographs, of slope conditions, rock falls, and stabilization work. This information will document the location of hazardous areas and determine the long-term effectiveness of the program in reducing the incidence of rock falls. These records can be most conveniently handled using data-base programs that readily allow updating and retrieval (see Section 2.2).

2.1 Rock Slope Inventory Systems

The relative risk of rock falls at a site as compared with other sites can be used in selecting priorities. Early work on this topic by Brawner and Wyllie (1975) and Wyllie (1987) was developed by Pierson et al. (1990) into a process for the rational management of the rock slopes along transportation systems, which has been named the Rockfall Hazard Rating System (RHRS). The first step in this process is to make an inventory of the stability conditions of each slope so that they can be ranked according to their rock-fall hazard (Figure 18-3).

The rock-fall areas identified in the inventory are ranked by scoring the categories as shown in Table 18-2. These categories represent the significant elements of a rock slope that contribute to the overall hazard. The four columns correspond to logical breaks in the hazard represented by each category. The rating criteria scores increase exponentially from 3 to 81 points and represent a continuum of points from 1 to 100. An exponential system allows for a rapid increase in score, which quickly distinguishes more hazardous sites. Using a continuum of points allows flexibility in evaluating the relative impact of conditions that are variable by nature. Some categories require a subjective evaluation, whereas others can be directly measured and then scored. A brief description of each of the categories of the RHRS follows. It should be noted that the system was developed on the basis of customary units; however, conversion to metric specifications is generally straightforward.

2.1.1 Slope Height

Category 1 represents the vertical height of the slope. Measurement is to the highest point from which rock fall is expected. If rocks are coming from the natural slope above the cut, the cut height plus the additional slope height (vertical distance) are used.

2.1.2 Ditch Effectiveness

The effectiveness of a ditch is measured by its ability to prevent falling rock from reaching the traveled way. In estimating the ditch effectiveness, factors to consider are (*a*) slope height and angle; (*b*) ditch width, depth, and shape; (*c*) anticipated block size and quantity of rock fall; and (*d*) effect of slope irregularities (launching features) on falling rocks. A launching feature can negate the benefits expected from a fallout area. Valuable information on ditch performance can be obtained from maintenance personnel and reference to Figure 18-15 (see Section 5.1).

2.1.3 Average Vehicle Risk

Average vehicle risk (AVR) represents the percentage of time that a vehicle will be present in the rock-fall section, which is obtained from the following relationship:

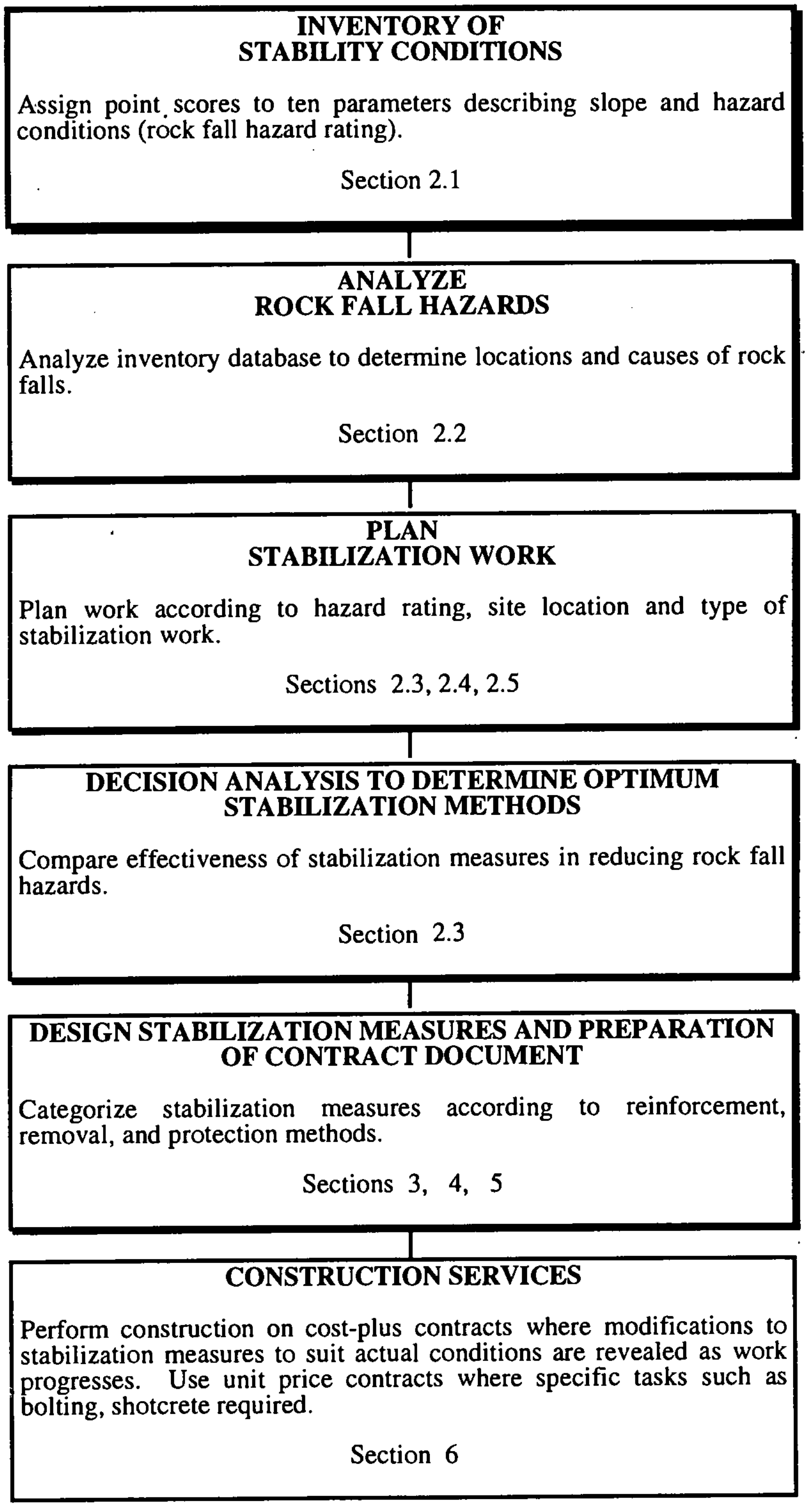


FIGURE 18-3 Rock slope stabilization program for transportation systems.

$$\text{AVR (\%)} = \frac{\text{ADT (cars/day)} \times \text{slope length (mi)}}{\text{posted speed limit (mph)} \times 24 \text{ (hr/day)}} \times 100\% \qquad (18.1)$$

Table 18-2
Summary Sheet of Rockfall Hazard Rating System (Pierson et al. 1990)

	Rating Criteria by Score			
Category	Points 3	Points 9	Points 27	Points 81
1. Slope height (m)	7.5	15	23	>30
2. Ditch effectiveness	Good catchment	Moderate catchment	Limited catchment	No catchment
3. Average vehicle risk (% of time)	25	50	75	100
4. Decision sight distance (% of design value)	Adequate	Moderate	Limited	Very limited
5. Roadway width including paved shoulders (m)	13.5	11	8.5	6
6. Geologic case characteristics				
Case 1				
Structural condition	Discontinuous joints, favorable orientation	Discontinuous joints, random orientation	Discontinuous joints, adverse orientation	Continuous joints, adverse orientation
Rock friction	Rough, irregular	Undulating	Planar	Clay infilling or slickensided
Case 2				
Structural condition	Few differential erosion features	Occasional erosion features	Many erosion features	Major erosion features
Difference in erosion rates	Small	Moderate	Large	Extreme
7. a. Block size (m)	0.3	0.6	1	1.2
b. Volume of rock fall or event (m^3)	3	6	9	12
8. Climate and presence of water on slope	Low to moderate precipitation; no freezing periods; no water on slope	Moderate precipitation or short freezing periods or intermittent water on slope	High precipitation or long freezing periods or continual water on slope	High precipitation and long freezing periods or continual water on slope and long freezing periods
9. Rock-fall history	Few falls	Occasional falls	Many falls	Constant falls

A rating of 100 percent means that on average a vehicle can be expected to be within the section 100 percent of the time.

2.1.4 Decision Sight Distance

Sight distance is the shortest distance along a roadway that an object is continuously visible to the driver. The sight distance can change appreciably throughout a rock-fall section. Horizontal and vertical curves, together with obstructions such as rock outcrops and roadside vegetation, can severely limit the available sight distance. Decision sight distance (DSD) is used to determine the length of roadway (in meters) needed to make a complex or instantaneous decision. DSD is critical when obstacles on the road are difficult to perceive or when unexpected or unusual maneuvers are required.

The relationship between DSD and the posted speed limit used in the inventory system has been modified from the *Policy on Geometric Design of Highways and Streets* of the American Association of State Highway and Transportation Officials (AASHTO 1984).

2.1.5 Roadway Width

The available maneuvering room to avoid a rock fall is measured perpendicular to the highway centerline from one edge of the pavement to the other and includes the shoulders if they are paved.

When the roadway width is not constant, the minimum width is used.

2.1.6 Geologic Case Characteristics

Generally, there are two cases of conditions that cause rock fall. Case 1 includes slopes in which joints, bedding planes, or other discontinuities are the dominant structural features. In Case 2 differential erosion or oversteepened slopes are the dominant conditions that control rock fall. The case that best fits the slope should be used in the evaluation. If both types are present, both are scored but only the worst case (highest score) is used in the rating.

Case 1

Structural condition is characterized by adverse joints, those with orientations and lengths that promote planar, circular, block, wedge, or toppling failures. Rock friction on a discontinuity is governed by the characteristics of the rock material as well as by the surface roughness and properties of any infilling (see Section 2 of Chapter 14).

Case 2

Structural condition is characterized by differential erosion or oversteepening, which is the dominant condition that leads to rock fall. Erosion features include oversteepened slopes, unsupported rock units, or exposed resistant rocks. The different rates of erosion within a slope directly relate to the potential for a future rock-fall event. The score should reflect how quickly erosion is occurring; the size of rocks, blocks, or units being exposed; the frequency of rock-fall events; and the amount of material released during a rock fall.

2.1.7 Block Size and Volume of Rock Fall per Event

The measurements in Category 7 should be representative of whichever type of event is most likely to occur. The scores should also take into account any tendency of the blocks of rock to break up as they fall down the slope.

2.1.8 Climate and Presence of Water on Slope

Water and freeze-thaw cycles both contribute to the weathering and movement of rock materials. If water is known to flow continually or intermittently on the slope, the slope is rated accordingly. Areas receiving less than 50 cm per year are low-precipitation areas. Areas receiving more than 125 cm per year are considered high-precipitation areas.

2.1.9 Rock-Fall History

Historical information is an important check on the potential for future rock falls. As a better data base of rock-fall occurrences is developed, more accurate conclusions for the rock-fall potential can be made.

2.2 Data-Base Analysis of Rock Slope Inventory

It is common practice to enter the results of the rock slope inventory into a computer data base. The data base can be used both to analyze the data contained in the inventory and to facilitate updating of the inventory with new information on rock falls and construction work. The following are some examples of data-base analyses:

- The slopes can be ranked in order of decreasing point score to identify the most hazardous slopes;
- Correlations can be found, for example, between rock-fall frequency and such factors as weather conditions, rock type, and slope location;
- The severity of rock falls can be assessed from analysis of delay hours caused by falls;
- The effectiveness of stabilization work can be assessed by determining how soon rock falls reoccur after stabilization work has been carried out.

The results of such analyses of the rock slope inventory data base can be used to plan future stabilization work. For example, mobilization costs for construction equipment can be minimized by selecting sites within one contract that are close together and that require similar types of equipment.

2.3 Selection of Stabilization Measures

Methods of slope stabilization fall into three categories: reinforcement, rock removal, and protection. Figure 18-4 includes 16 of the more common stabilization measures divided into these categories.

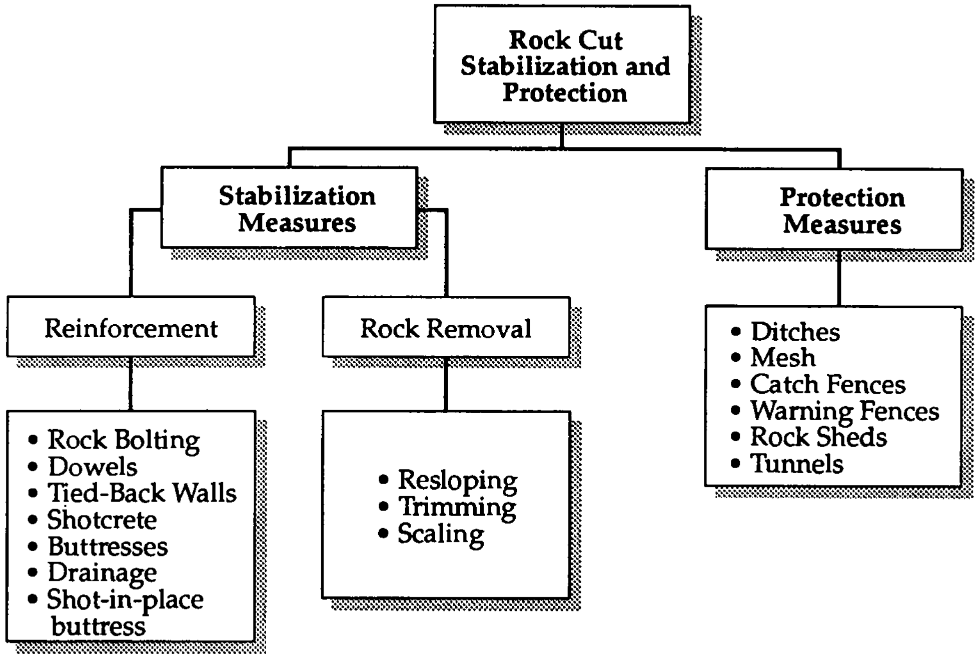


FIGURE 18-4 Categories of rock slope stabilization measures.

It is important that the appropriate stabilization method be used for the particular conditions at each site. For example, where the slope is steep and the toe is close to the highway or railway, there will be no space to excavate a catch ditch or construct a fence. Alternative stabilization measures may be to remove loose rock, secure it in place with bolts, or cover the slope with mesh. It is generally preferable to remove loose rock and eliminate the hazard, but only if this will form a stable face and not undermine other potentially loose rock on the face.

When stabilization measures that are appropriate for a site are selected and designed, geotechnical, construction, and environmental issues must be considered. The geotechnical issues—geology, rock strength, groundwater, and stability analysis—are discussed in Chapters 14 and 15. Construction and environmental issues, which can affect the costs and schedule of the work, must be addressed during the design phase of the project in order for the work to be carried out as efficiently as possible. Issues that are frequently important are equipment access, available work time during traffic closures, and disposal of waste rock and soil.

Another factor to consider in the selection of stabilization measures is the optimum level of work. For example, a minor scaling project will remove the loosest rock on the slope face, but if the rock is susceptible to weathering, this work may have to be repeated every 3 to 5 years. Alternatively, a more comprehensive program can be carried out using shotcrete and bolting in addition to scaling. Although the initial costs of this second program will be higher, its effects will last longer, perhaps for 20 years. Alternative stabilization programs such as these, as well as the alternative of doing no work, can be compared by the use of decision analysis, which is a systematic procedure for evaluating alternative courses of action taking into account both the cost and design life of the stabilization work, as well as the probability that rock falls will occur, causing accidents and their costs (Wyllie et al. 1980; Roberds 1991).

2.4 Construction Issues

The following is a brief discussion of some construction issues that may have a significant influence on stabilization work, depending on actual site conditions.

2.4.1 Blasting

Damage to rock faces by excessively heavy blasting is a frequent cause of instability in the years following excavation of a slope. Methods of controlled blasting, such as preshear and trim blasting, as described in Section 4.2, can be used to excavate a slope to a specified line with minimal damage to the rock behind the face.

2.4.2 Topography

The topography at a site can significantly affect both the slope design and the type and extent of stabilization work. For example, if there is a continuous slope above the crest of a cut, stabilization work that involves laying back the cut will have the effect of increasing the cut height. This increase in cut height will require a larger catch ditch and may result in additional stability problems, especially if there is a substantial layer of soil or weathered rock at the surface.

2.4.3 Construction Access

In the design of excavations and stabilization work, it is necessary to determine the type of equipment that is likely to be required to carry out the work and how this equipment will be used at the site. For example, if it is planned to excavate a substantial volume of rock in order to lay back a slope, it is likely that airtrac drills and excavators will have to work on the slope. In steep terrain it may be found that the construction of an access road for this equipment would be costly and cause additional instability. Alternatively, if stabilization work is planned using large-diameter rock bolts, it is essential that suitable drilling equipment have access to the site. For example, on steep faces, holes with a diameter larger than about 100 mm will have to be drilled with heavy equipment supported by a crane or anchored cables. If it is not possible to use heavy drilling equipment, it will be necessary to drill smaller-diameter holes with lightweight equipment, such as downhole hammers or hand-held percussion drills.

2.4.4 Construction Costs

Cost estimates for stabilization work must take into account both the costs of the work on the slope and indirect costs such as mobilization and traffic closures. For example, when minor stabilization work is carried out on a steep face, traffic closures can be minimized by having the construction crews work from ropes secured behind the crest of the slope rather than from a crane located on the road, which may block two to three lanes of traffic.

2.5 Environmental Issues

A number of procedures can be implemented during both the design and construction phases of a rock slope project to minimize the effects of this work on the environment. These procedures may add to the direct cost of the work, but can have benefits of producing a more visually attractive highway and reducing environmental impact and public opposition.

2.5.1 Waste Disposal

The least expensive method of disposing of waste rock produced by excavation and scaling operations in mountainous terrain is by dumping it down the slope below the highway. However, disposing of waste rock in this manner has a number of drawbacks. First, a steeply sloping pile of loose rock may be a visual scar on the hillside that can be difficult to vegetate. Second, the waste rock may become unstable if not adequately drained or keyed into the existing slope; the material may move a considerable distance if it fails, which could endanger facilities downslope. Third, when the road is located in a river valley, the dumped rock may fall into the river and have a deleterious effect on fish populations. In order to minimize these impacts, it is sometimes required that the rock excavated from the slope be hauled to designated, stable waste sites.

Another problem that may need to be addressed in the disposal of waste rock is acid-water drainage. In some areas of North Carolina and Tennessee, for example, some argillite and schist formations contain iron disulfides; percolation of water through fills constructed with this rock produces low-pH, acidic runoff. One method that has been tested to control this condition is to mix the rock with lime to neutralize the acid potential and then to place the blended material in the center of the fill (Byerly and Middleton 1981). Sometimes it is necessary to encase the rock-lime mixture in an impervious plastic membrane.

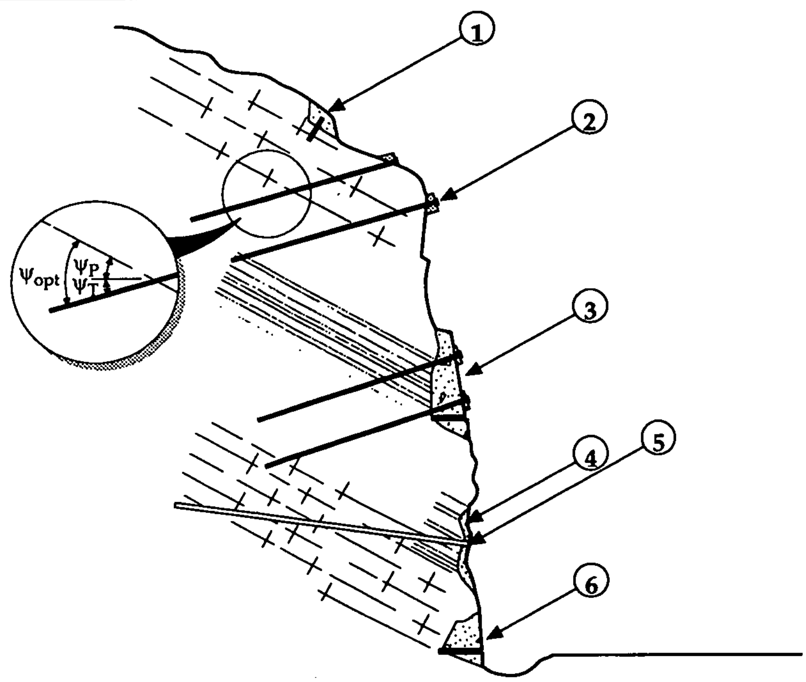
2.5.2 Aesthetics

A series of steep, high rock cuts above a highway may have a significant visual impact when viewed both by the road user and the local population. In scenic areas it may be desirable to incorporate appropriate landscaping measures in the design of the rock cuts in order to minimize their visual impact (Norrish and Lowell 1988).

3. ROCK REINFORCEMENT

Figure 18-5 shows a number of reinforcement techniques that may be implemented to secure potentially loose rock on the face of a rock cut. The common feature of all these techniques is that they minimize the relaxation and loosening of the rock mass that may take place as a result of excavation and unloading (Hoek 1983). Once relaxation has been allowed to take place, there is a loss of interlock between the blocks of rock and a significant decrease in the shear strength. Figure 14-8(*b*) shows the effect of installing rock bolts to maintain the interlock on high-angle, second-order asperities. Once relaxation has taken place, it is not possible to reverse the process. For this reason, reinforcement of rock slopes is most effective if it is installed before excavation—a process known as prereinforcement.

The different applications of untensioned (prereinforcement) bolts and tensioned bolts are shown in Figure 18-6. Prereinforcement of a benched excavation can be achieved by installing bolts as each bench is excavated. Installation of fully grouted but untensioned bolts at the crest of the cut before excavation [Figure 18-6(*b*)] prevents loss of interlock of the rock mass because the bolts are sufficiently stiff to prevent movement on the natural fractures (Moore and Imrie 1982; Spang and Egger 1990). However, when blocks have moved and relaxed, it is necessary to install tensioned bolts in order to prevent further displacement and loss of interlock [Figure 18-6(*a*)]. The advantages of using untensioned bolts are the lower costs and quicker installation compared with tensioned bolts.



1 Reinforced concrete dowel to prevent loosening of slab at crest
2 Tensioned rock anchors to secure sliding failure along crest
3 Tieback wall to prevent sliding failure on fault zone
4 Shotcrete to prevent raveling of zone of fractured rock
5 Drain hole to reduce water pressure within slope
6 Concrete buttress to support rock above cavity

FIGURE 18-5 Rock slope reinforcement methods.

3.1 Rock Bolts

Tensioned rock bolts are installed across potential failure surfaces and anchored in sound rock beyond the surface. The application of a tensile force in the bolt, which is transmitted into the rock by a reaction plate at the rock surface, produces compression in the rock mass and modifies the normal and shear stresses across the failure surface. The effect of the forces produced by the rock bolt when installed at a dip angle flatter than the normal to the potential failure surface is to increase the resisting force and decrease the driving force. It is found that the required bolt force to produce a specified factor of safety is minimized when the sum of the dip angle of the bolt (Ψ_T) and the dip angle of the failure surface (Ψ_P) is equal to the friction angle (see inset, Figure 18-5). Savings in bolting costs usually can be realized by installing bolts at the optimum angle (Ψ_{opt}) rather than at an angle normal to the failure surface.

The three main requirements of a permanent, tensioned rock-bolt installation are that

1. There be a method of anchoring the distal end of the anchor in the drill hole,
2. A known tension be applied to the bolt without creep and loss of load over time, and
3. The complete anchor assembly be protected from corrosion for the design life of the project.

These three aspects of rock-anchor design and installation are discussed below.

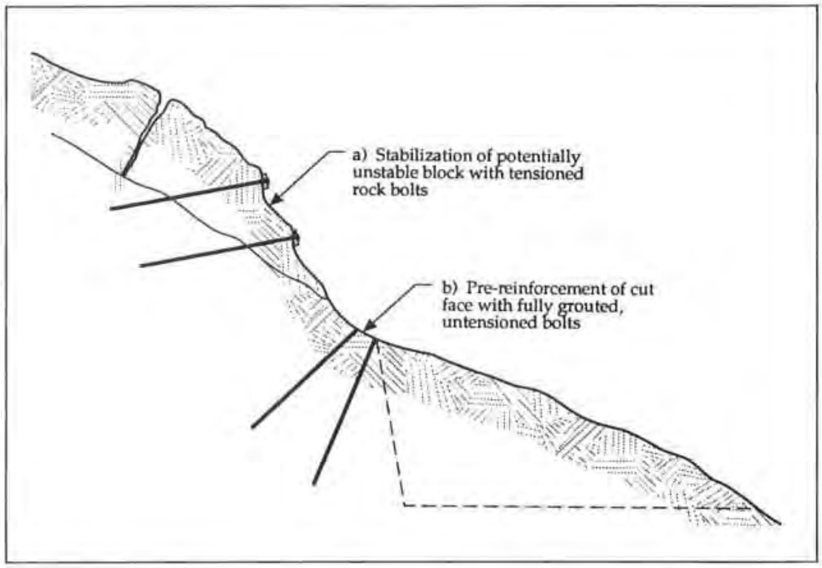


FIGURE 18-6 Reinforcement of a rock slope with (*a*) tensioned rock bolts and (*b*) fully grouted, untensioned rock bolts.

3.1.1 Anchorage

Methods of securing the distal end of a bolt in the drill hole include resin, mechanical, and cement-grout anchors. The selection of the appropriate anchor depends on such factors as the required capacity of the anchor, speed of installation, strength of the rock in the anchor zone, access to the site for drilling and tensioning equipment, and level of corrosion protection required.

3.1.1.1 Resin Anchors

The resin anchor system includes a plastic cartridge about 25 mm in diameter and 200 mm long that contains a liquid resin and a hardener that set when mixed together (Figure 18-7). Setting times vary from about 1 to 5 min to as much as 90 min, depending on the reagents used. The setting time is also dependent on the temperature, with fast-setting resin hardening in about 4 min at a temperature of -5°C and in about 25 sec at 35°C.

The installation method consists of inserting a sufficient number of cartridges into the drill hole to fill the annular space around the distal end of the bolt (the bar). It is important that the hole diameter in relation to the bar size be within specified tolerances so that complete mixing of the resin is achieved when the bar is spun. This fit usually precludes the use of coupled anchors because the hole diameter to accommodate the coupling would be too large for complete resin mixing. The bar is spun as it is driven through the cartridges to mix the resin and form a rigid solid that anchors the bar in the hole. The required speed of rotation is about 60 rpm, and spinning is continued for about 30 sec after the bar has reached the end of the hole. The maximum bolt length is about 12 m because most drills cannot rotate longer bars at sufficient speed to mix the resin. It is possible to install a tensioned, resin-grouted bolt by using a fast-setting (about 2 min) resin for the anchor and a slower-setting (30 min) resin for the remainder of the bar. The bolt is tensioned between the setting of the fast and slow resins.

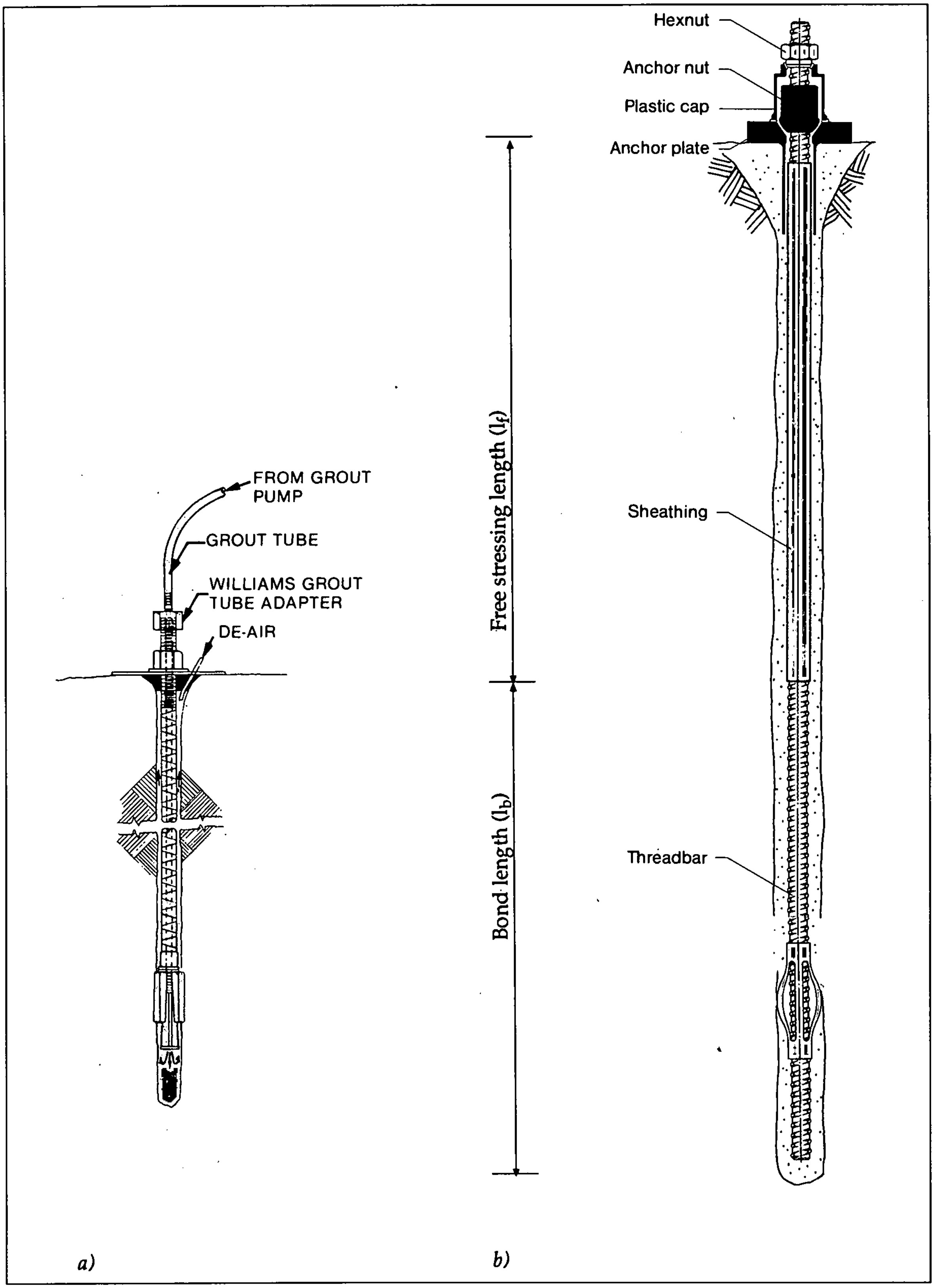
The primary advantages of resin anchorage are the simplicity and speed of installation, with support of the slope being provided within minutes of spinning the bolt. The disadvantages are the limited length and tension capacity of the bolt (400 kN) and the fact that only rigid bars can be used. Furthermore, the resin is not as effective as cement grout for corrosion protection of steel. Unlike cement grout, resin does not provide the high pH protective layer against corrosion, and it cannot be verified that the cartridges completely encapsulate the steel.

3.1.1.2 Mechanical Anchors

Mechanical anchors consist of a pair of steel wedges that are pressed against the walls of the drill hole. For the Williams anchor shown in Figure 18-8(*a*), the installation procedure is to first drill to the specified diameter so that when the bar is installed, the cone threaded on the bar is in con-

FIGURE 18-7 Resin cartridges for anchoring rock bolts.

FIGURE 18-8 Examples of rock bolts with (*a*) Williams hollow core mechanical wedge anchor and (*b*) Dywidag grouted anchor.
WILLIAMS FORM HARDWARE AND ROCK BOLT COMPANY AND DYWIDAG SYSTEMS INTERNATIONAL



tact with the walls of the hole. When the bar is torqued, the cone moves along the bar and expands the wedges against the walls of the hole to anchor the bar.

The advantages of mechanical anchors are that installation is rapid and tensioning can be carried out as soon as the anchor has been set. Grouting can then be done using a grout tube attached to the bar or through the center hole in the case of the Williams anchor. The disadvantages of mechanical anchors are that they can be used only in medium-to-strong rock in which the anchor will grip, and the maximum working tensile load is about 200 kN. Mechanical anchors for permanent installations must always be fully grouted because the wedge will creep and corrode, resulting in loss of support.

3.1.1.3 Cement-Grout Anchors

Cement grout is the most common method of anchoring long-service-life rock bolts because the materials are inexpensive and installation is simple. Cement-grout anchorage can be used in a wide range of rock and soil conditions, and the cement protects the steel from corrosion. Figure 18-8(*b*) shows a typical rock-bolt installation with cement-grout anchorage and centering sleeves to ensure complete encapsulation of the steel. The grout mix usually consists of nonshrink cement and water at a water:cement ratio in the range of 0.4 to 0.45. This ratio will produce a grout that can be readily pumped down a small-diameter grout tube, yet produce a high-strength, continuous grout column with minimal bleed of water from the mix. Admixtures are sometimes added to the grout to reduce shrinkage and bleeding and to increase the viscosity.

Tensioned rock bolts include a free-stressing length (l_f) and a bond length (l_b) [Figure 18-8(*b*)], with the full bond length being beneath the potential failure surface. Figure 15-6 shows the method of calculating the factor of safety of a planar failure in which tensioned bolts are installed. When the bolt is at a flatter angle than the normal to the failure surface, the bolt tension increases the normal force and decreases the shear-displacing force on the surface. The design of rock anchors to sustain the necessary tension load requires the selection of an appropriate hole and bar diameter and free-stressing and anchor lengths with respect to the geological conditions.

For cement-grout-anchored bolts, the stress distribution along the bond length is highly nonuniform; the highest stresses are concentrated in the proximal end of the anchor and ideally the distal end of the anchor is unstressed (Farmer 1975; Aydan 1989). However, it is found that the required length of the bond zone can be calculated with the simplifying assumption that the shear stress at the rock-grout interface is uniformly distributed along the anchor and is given as follows:

$$\tau_a = \frac{T}{\pi d_h l_b} \quad \text{or} \quad l_b = \frac{T}{\pi d_h l_b} \tag{18.2}$$

where

T = design tension force,
d_h = hole diameter,
τ_a = allowable bond stress, and
l_b = bond length.

Values of τ_a can be estimated from the uniaxial compressive strength (σ_u) of the rock in the anchor zone according to the following relationship (Littlejohn and Bruce 1975):

$$\tau_a = \frac{\sigma_u}{30} \tag{18.3}$$

Approximate ranges of allowable bond stress (τ_a) related to rock strength and rock type are presented in Table 18-3.

The diameter of the drill hole is partially determined by the available drilling equipment but must also meet certain design requirements. The hole diameter should be large enough to allow the anchor to be inserted without driving or hammering and be fully embedded in a continuous column of grout. A hole diameter significantly larger than the anchor will not materially improve the design and will result in unnecessary drilling costs and possibly excessive grout shrinkage. A guideline for a suitable ratio between the diameter of the hole (d_h) and the diameter of the anchor (d_a) is

$$0.4 \le \frac{d_a}{d_h} \le 0.6 \tag{18.4}$$

The working shear strength at the steel-grout interface of a grouted deformed bar is usually greater than the working strength at the rock-grout interface. For this reason, the required anchor length is typically determined from the stress level developed at the rock-grout interface.

Table 18-3
Allowable Bond Stresses in Cement-Grout Anchors (Wyllie 1991)

Rock Strength and Type	Allowable Bond Stress (MPa)	Compressive Strength Range (MPa)
Strong	1.05–1.40	>100
Medium	0.7–1.05	≈50–100
Weak	0.35–0.7	≈20–50
Granite, basalt	0.55–1.0	
Dolomitic limestone	0.45–0.70	
Soft limestone	0.35–0.50	
Slates, strong shales	0.30–0.45	
Weak shales	0.05–0.30	
Sandstone	0.30–0.60	
Concrete	0.45–0.90	

3.1.2 Tensioning

When tensioned rock anchors are to be installed, it is important that a procedure be carried out to check that the full design load is applied at the required depth and that there will be no loss of load with time. A suitable testing procedure has been drawn up by the Post Tensioning Institute (1985) that includes the following four types of test: performance, proof, creep, and lift-off.

The performance and proof tests consist of a cyclic testing sequence to a maximum load of 150 percent of the design load, in which the deflection of the head of the anchor is measured as the anchor is tensioned. The purpose of these tests is to check that the anchor can sustain a load greater than the design load and that the load in the anchor is transmitted into the rock at the location of the potential failure surface. The creep test checks that there will be no significant loss of load with time, and the lift-off test checks that the tension applied during the testing sequence has been permanently transferred to the anchor. The Post Tensioning Institute (1985) provides acceptance criteria for each of the four tests, and it is necessary that each anchor meet all the acceptance criteria.

The usual method of tensioning rock bolts is to use a hollow-core hydraulic jack that allows the applied load to be precisely measured as well as cycles the load and holds it constant for the creep test. It is important that the hydraulic jack be calibrated before each project to check that the indicated load is accurate. The deflection of the anchor head is usually measured with a dial gauge to an accuracy of about 0.05 mm; the dial gauge is mounted on a reference point that is independent of the anchor. Figure 18-9 shows a typical test arrangement for a tensioned cable anchor including a hydraulic jack and a dial gauge set up on a tripod.

3.1.3 Corrosion Protection

Corrosion protection is provided for almost all permanent anchors to ensure their longevity. Even if anchors are not subject to corrosion at the time of installation, conditions may change, which must be accounted for in design. The following is a list of conditions that will usually create a corrosive environment for steel anchors (Hanna 1982):

- Soils and rocks that contain chlorides,
- Seasonal changes in the groundwater table,
- Marine environments where sea water contains chlorides and sulfates,
- Fully saturated clays with high sulfate contents,
- Passage through ground types that possess different chemical characteristics, and
- Stray direct electrical current that develops galvanic action between the steel and the surrounding rock.

The corrosive environments described above can be quantified in terms of the pH value and the resistivity of the site. In highly acidic ground (pH < 4), corrosion by pitting is likely, whereas in slightly alkaline ground (pH just greater than 7), sulfate-reducing bacteria flourish, producing a corrosive environment. Corrosion potential is also related to the soil resistivity by the magnitude of current that can flow between the steel and the soil. In general, the degree of corrosiveness decreases as follows (King 1977):

FIGURE 18-9
Typical arrangement for measuring applied tension and deflection of head of multistrand anchor during performance and proof tests of grouted anchor.

Organic soil > clays > silts > sands > gravels

A number of rock-anchor manufacturers have proprietary corrosion protection systems, all of which meet the following requirements for long-term reliability:

- There will be no breakdown, cracking, or dissolution of the protection system during the service life of the anchor;
- The protection system can be fabricated either in the plant or on the site in such a manner that the quality of the system can be verified;
- The anchor can be installed and stressed without damage to the protection system; and
- The materials used in the protection system will be inert with respect to both the steel anchor and the surrounding environment.

Methods of protecting steel against corrosion include galvanizing, applying an epoxy coating, and encapsulating the steel in cement grout. Cement grout is commonly used for corrosion protection, primarily because it creates a high pH environment that protects the steel by forming a surface layer of hydrous ferrous oxide. In addition, cement grout is inexpensive and simple to install and has sufficient strength for most applications and a long service life. Because of the brittle nature of grout and its tendency to crack, particularly when loaded in tension or bending, the protection system is usually composed of a combination of grout and a plastic [high-density polyethylene (HDPE)] sleeve. In this way the grout produces the high pH environment around the steel, and the plastic sleeve provides protection against cracking. In order to minimize the formation of shrinkage cracks that reduce corrosion resistance of the grout, nonshrink grouts are usually used for all components of the installation. Figure 18-10 shows an example of a three-layer corrosion protection system for a rock anchor. The strands are encapsulated in a grout-filled HDPE sheath, and the outer annular space between the sheath and the rock is filled with a second grout layer.

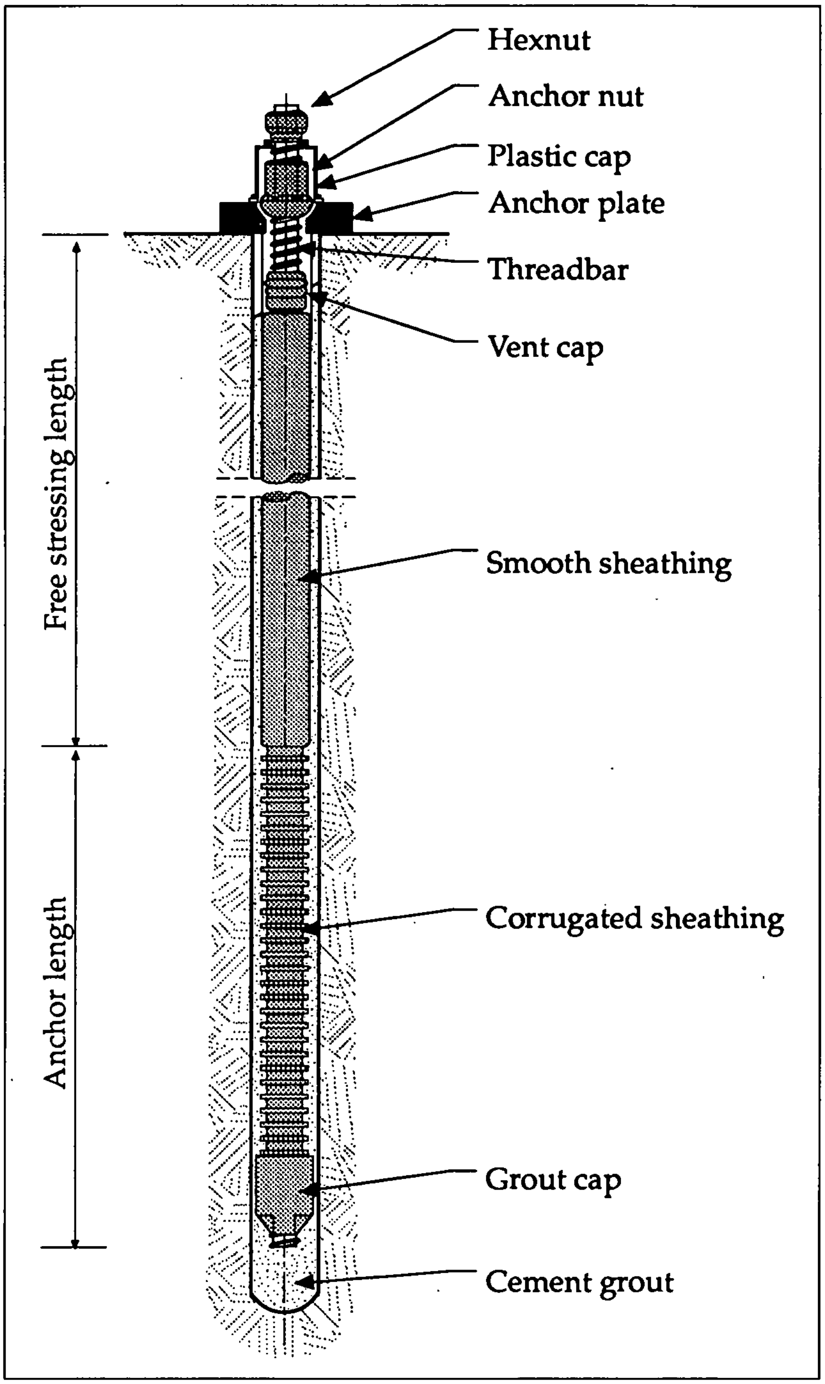


FIGURE 18-10 Corrosion protection system for rock anchor. DYWIDAG SYSTEMS INTERNATIONAL

3.2 Dowels

Loosening and failure of small blocks of rock on the slope face can be prevented by the installation of passive dowels at the toe of the block. Dowels are composed of lengths of reinforcing steel grouted into holes drilled in the underlying, stable rock, with a cap of reinforced concrete encasing the exposed steel (Figure 18-5, Method 1). It is important that the concrete be in intimate contact with the rock that it is supporting so that movement and loss of interlock on the potential rupture surface are minimized. The reinforcing steel dowels used to anchor the concrete to the rock are usually about 25 mm in diameter, embedded about 0.5 m into sound rock, and spaced about 0.5 to 0.8 m apart.

Because dowels provide only passive shear resistance to sliding, they are used to support slabs of rock with thicknesses up to 1 to 2 m. Dowels are most effective when there has been no prior movement of the rock so that there is interlock on the potential sliding surface. For slabs thicker than 1 to 2 m or where there has been movement, the required support may be provided more reliably by tensioned anchors.

3.3 Tieback Walls

Method 3 in Figure 18-5 is used where there is potential for a sliding failure in closely fractured rock. Tensioned rock bolts are required to support this portion of the slope, but the fractured rock may degrade and ravel from under the reaction plates of the anchors; thus, eventually the tension in the bolts will be lost. In these circumstances, a reinforced concrete wall can be constructed to cover the area of fractured rock, and then the holes for the rock anchors can be drilled through sleeves in the wall. Finally, the anchors are installed and tensioned against the face of the wall. The wall acts as both a protection against raveling of the rock and a large reaction plate for the rock anchors.

Since the purpose of the concrete wall is to distribute the anchor loads into the rock, the reinforcement for the tieback wall should be designed so that there is no cracking of the concrete from the concentrated loads of the anchor heads or from bending between anchors. It is also important that there be drain holes through the concrete to prevent buildup of water pressure behind the wall.

3.4 Shotcrete

Zones or beds of closely fractured or degradable rock can be protected by applying a layer of shotcrete to the rock face (Figure 18-5, Method 4). The shotcrete controls both the fall of small blocks of rock and progressive raveling that will produce large, unstable overhangs on the face. However, shotcrete provides little support against sliding of the overall slope; its primary function is surface protection. Shotcrete is a pneumatically applied, fine-aggregate mortar (less than 13 mm aggregate size) that is usually placed in a 75- to 100-mm layer (American Concrete Institute (1983).

The effectiveness of shotcrete depends to a large degree on the condition of the rock surface to which it is applied. The surface should be free of loose and broken rock, soil, vegetation, and ice and should be damp to improve the adhesion between the rock and the shotcrete. It is important that drain holes be drilled through the shotcrete to prevent buildup of water pressure behind the face; the drain holes are usually about 0.5 m deep and are located on 1- to 2-m centers. In massive rock it is important that the drain holes be drilled before the shotcrete is applied and that they be located so as to intersect fractures that carry water. The holes are temporarily plugged with wooden pegs or rags while the shotcrete is applied.

For all permanent applications, shotcrete should be reinforced to reduce the risk of cracking and spalling. The two common methods of reinforcement are welded-wire mesh and steel fibers. Welded-wire mesh is fabricated from light-gauge (3.5-mm-diameter) wire on 100-mm centers and is attached to the rock face on about 1- to 2-m centers with steel pins, complete with washers and nuts, grouted into the rock face. The mesh must be closely attached to the rock surface and fully encased in shotcrete, with care being taken to eliminate voids within the shotcrete. On irregular rock faces it can be difficult to attach the mesh closely to the rock surface. In these circumstances the mesh can be installed between two layers of shotcrete, with the first layer creating a smoother surface to which the mesh can be more readily attached.

An alternative to mesh reinforcement is to use steel fibers that are a component of the shotcrete mix and form a reinforcement mat throughout the shotcrete layer (Wood 1989). The fibers are manufactured from high-strength carbon steel with a length of 30 to 38 mm and diameter of 0.5 mm. To resist pullout, the fibers have deformed ends or are crimped. The principal function of steel fibers is to significantly increase the shear, tensile, and postcrack strength of the shotcrete compared with unreinforced shotcrete. These loading conditions tend to develop when the fractured rock loosens behind the face.

3.5 Buttresses

When a rock fall has occurred that forms a cavity in the slope face, it may be necessary to construct a concrete buttress in the cavity to prevent further falls (Figure 18-5, Method 6). The buttress serves two functions: to retain and protect areas of weak rock and to support the overhang. Buttresses should be designed so that the thrust from the rock to be supported loads the buttress in compression. In this way bending moments and overturning forces are eliminated, and there is no need for heavy reinforcement of the concrete or for tiebacks anchored in the rock.

For the buttress to prevent relaxation of the rock, it should be founded on a clean, sound rock surface. If this surface is not at right angles to the direction of thrust, it should be anchored to the base using steel pins to prevent sliding. Also, the top should be poured so that it is in contact with the underside of the overhang. In order to meet this second requirement, it may be necessary to place the last pour through a hole drilled downward into the cavity from the rock face and to use a nonshrink agent in the mix.

3.6 Drainage

As shown in Table 18-1, groundwater in rock slopes is often a contributory cause of instability. The usual method of reducing water pressures is to drill drain holes at the toe of the slope to create a series of outlets for the water.

The most important factor in the design of drain holes for rock slopes is to locate them so that they intersect the fractures that are carrying the water; little water is contained in the intact rock. For the conditions shown in Figure 18-5, the drain holes are drilled at a shallow angle to intersect the more continuous fractures that dip out of the face. If the holes were drilled at a steeper angle parallel to these fractures, the drainage would be less effective because the holes would only intersect the steeply dipping, less continuous fractures.

There are no formulas from which to calculate the required spacing of drill holes because it is more important that the holes be located to suit the geological conditions at the site. As a guideline, holes are usually drilled on a spacing of about 3 to 10 m to a depth of at least one-third of the slope height. The holes are often lined with a perforated casing, with the perforations sized to minimize infiltration of fines that are washed from fracture infillings. Another important aspect of the design of drain holes is the disposal of the seepage water. If this water is allowed to infiltrate the toe of the slope, it may result in degradation of low-strength materials or produce additional stability problems downstream of the drains. Depending on site conditions, it may be necessary to collect all the seepage water in a manifold and dispose of it some distance from the slope.

Other methods of reducing water pressures in rock slopes may be appropriate, depending on site conditions. For example, if surface runoff infiltrates open tension cracks, it is often worthwhile to construct diversion ditches behind the crest of the slope and to seal the cracks with clay or plastic sheeting. For large slides it may not be possible to significantly reduce the water pressure in the slope with small drain holes. In these circumstances a drainage tunnel can be driven into the base of the slide from which a series of drain holes is drilled up into the saturated rock. For example, the Downie Slide in British Columbia has an area of about 7 km^2 and a thickness of about 250 m. There was concern regarding the stability of the slope when the toe was flooded by the construction of a dam. A series of drainage tunnels with a total length of 2.5 km was driven at an elevation just above the high-water level of the reservoir. From these tunnels, 13 500 m of drain holes was drilled to reduce groundwater pressures within the slope. These drainage measures have been effective in reducing the water level in the slide by as much as 120 m and in reducing the rate of movement from 10 mm/year to about 2 mm/year (Forster 1986).

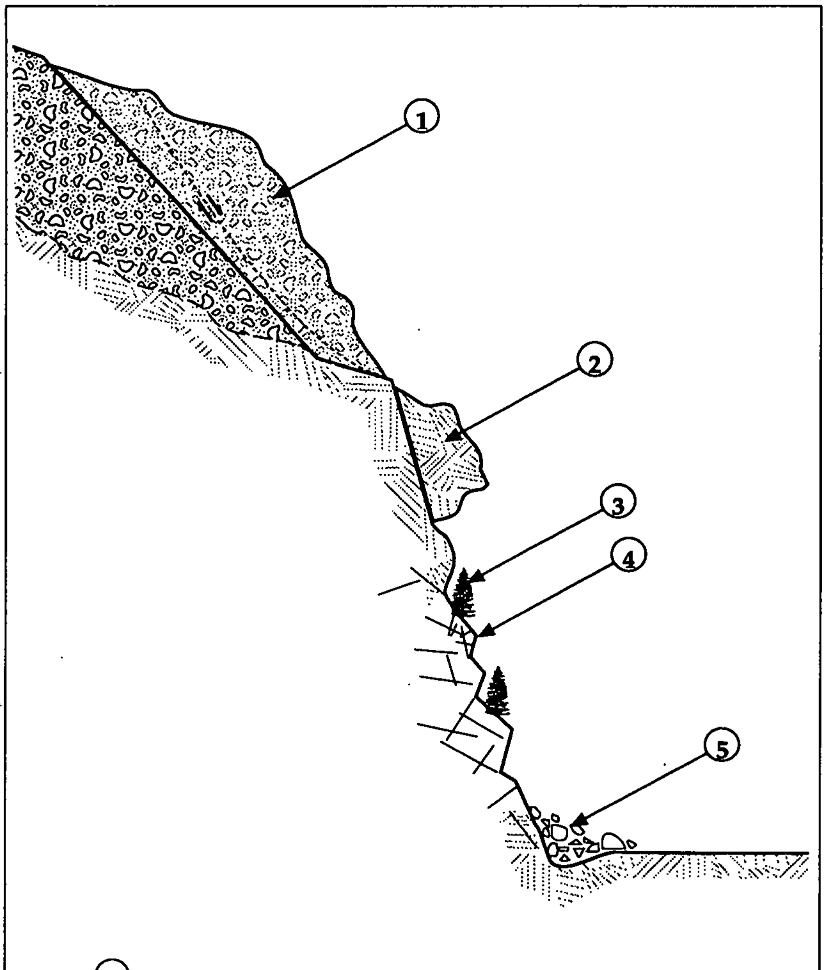
An important aspect of slope drainage is to install piezometers to monitor the effect of drainage measures on the water pressure in the slope. For example, one drain hole with a high flow may only be draining a small, permeable zone in the slope and monitoring would show that more holes would be required to lower the water table throughout the slope. Conversely, monitoring may show that in low-permeability rock, a small seepage quantity that evaporates as it reaches the surface may be sufficient to reduce the water pressure and significantly improve stability conditions.

3.7 "Shot-in-Place" Buttresses

On landslides where the surface of rupture is a well-defined geological feature such as a continuous bedding surface, stabilization may be achieved by blasting this surface to produce a "shot-in-place" buttress (Aycock 1981). The friction angle of the broken rock is greater than that of the smooth and planar bedding surface because of the greater effective roughness of the broken rock. If the total friction angle is greater than the dip of the surface of rupture, sliding is prevented. Fracturing of the rock also enhances drainage.

The method of blasting involves drilling a pattern of holes through the sliding surface and placing an explosive charge at this level that is just sufficient to break the rock. This technique requires that the drilling begin while it is still safe for the drills to reach the slope and before the rock becomes too broken for the drills to operate.

FIGURE 18-11 Rock-removal methods for slope stabilization.



4. ROCK REMOVAL

Stabilization of rock slopes can be accomplished by the removal of potentially unstable rock; Figure 18-11 shows typical removal methods including

- Resloping zones of unstable rock,
- Trim blasting overhangs, and
- Scaling individual blocks of rock.

Methods of rock removal and circumstances in which removal should and should not be used are described in this section. In general, rock removal is a preferred method of stabilization because the work eliminates the hazard and no future maintenance is required. However, removal should only be used where it is certain that the new face will be stable and where construction on active transportation routes can be carried out efficiently and safely.

4.1 Resloping and Unloading

When there is overburden or weathered rock in the upper portion of a cut, it is often necessary to cut this material at an angle flatter than that of the more competent rock below (Figure 18-11, Method 1). Careful attention should be given to investigating this condition because the thickness and properties of the overburden or weathered rock can vary considerably over short distances. Contract disputes may arise if there are major design changes during construction because of a variation in excavation quantities.

Another condition that should be considered during design is the rock weathering that takes place after the cut has been excavated. An initially stable, steep slope may become unstable some years after construction, at which time resloping may be difficult to carry out. Where a slide has developed, it may be necessary to unload the crest of the cut to reduce its height and diminish the driving force.

Resloping and unloading are usually carried out by equipment such as excavators and bulldozers. Consequently, the cut width must be designed to accommodate suitable equipment on the slope with no danger of collapse of the weak material; this width would usually be at least 6 m. Safety requirements for equipment access usually preclude the excavation of "sliver" cuts in which the toe of the new cut coincides with that of the old cut.

The design procedure for resloping and unloading starts with back analysis of the unstable slope. By setting the factor of safety of the unstable slope to 1.0, it is possible to calculate the rock-mass strength parameters (see Chapter 14). This information can then be used to calculate the required reduced slope angle or height, or both, that will produce the required factor of safety. For most slopes on highway and railway projects, an acceptable factor of safety for long-term stability is about 1.5.

4.2 Trimming

Failure of a portion of a rock slope may form an overhang on the face (Figure 18-11, Method 2), which may be a hazard to traffic if it also were to fail. The following is a brief discussion of methods of controlled blasting that can be used to excavate rock to narrow tolerances without damaging the rock behind the face.

Figure 18-12 shows the difference in stability conditions between heavily blasted rock (upper face) and controlled blasting in which there are no blast-induced fractures behind the face (lower face). Controlled blasting involves drilling a series of closely spaced, parallel holes on the required break line to evenly distribute the explosive on the face. The spacing between holes on the final line is generally in the range of 10 to 12 times the drill-hole diameter. Hole diameters on most stabilization projects range from about 40 mm for hand-held pneumatic drills to 80 mm for light, track-mounted drills. It is also important to limit the hole length to about 3 m for hand-held drills and to about 9 m for track drills, so that hole deviation is not excessive. However, rock with a pronounced structure can also cause excessive hole deviation even in short holes.

In controlled blasting a low-velocity explosive with a diameter about one-half that of the hole is used. The air gap around the explosive provides a cushion that minimizes shattering of the rock around the hole but generates sufficient energy to fracture the rock between the final line holes. The ratio between the hole diameter and the explosive diameter is termed the *decoupling ratio*. At a decoupling ratio of 2, the pressure generated in the sides of the borehole is about an order of magnitude less than that when the explosive is packed in the hole. As a guideline the explosive load in the final line holes should be in the range of 0.1 to 0.5 kg/m^2 of face area (Langefors and Kihlstrom 1967; Hemphill 1981; Federal Highway Administration 1985). It is normal practice to stem the holes to minimize venting of the explosive gases.

It is usual in controlled blasting to detonate each hole on a single delay, with the detonation sequence starting at a free face and progressing back to the final line. With a delay interval between holes of about 25 msec, there will be little risk of sympathetic detonation between holes and sufficient time for the broken rock to displace before the subsequent detonation. This methodology helps to limit the shock energy that is transmitted to the rock behind the face.

An alternative detonation sequence is to detonate the final line first in the sequence, sometimes on a single delay. This procedure is termed *preshear blasting*. Preshearing should only be used when the total burden is at least two to three times the hole depth to ensure that there is adequate weight of rock to confine the explosive energy. Instances of displacement of the entire burden by preshear blasting have been recorded. Also, preshearing is

FIGURE 18-12 Comparison of stability conditions of blasted rock face: *upper face,* heavy fracturing and overbreak; *lower face,* stable, presheared face.

generally not used in closely fractured rock because there is little relief for the explosive gases, which can cause damage to the rock behind the final line. Furthermore, ground vibrations produced by the instantaneous detonation of the preshear blasting can be greater than those produced by controlled blasting with a single hole per delay.

4.3 Scaling

Scaling describes the removal of loose rock, soil, and vegetation on the face of a slope using hand tools such as scaling bars, shovels, and chain saws. On steep slopes workers are usually supported by ropes anchored at the crest of the slope and tied to belts and carabiners around their waists (Figure 18-13). The most appropriate type of rope for these conditions is made of steel-core hemp that is highly resistant to cuts and abrasion. The scalers work their way down the face to ensure that there is no loose rock above them.

A staging suspended from a crane is an alternative to using ropes for the scalers to reach the face. The crane is located on the road at the toe of the slope if there is no access to the crest of the slope. The disadvantage of using a crane rather than ropes is the expense and the fact that with the outriggers extended, it can occupy several lanes of the road with consequent disruption to traffic. Also, scaling from a staging suspended from a crane can be less safe than using ropes because the scalers are not able to direct the crane operator to move quickly in the event of a rock fall from the face above them.

An important component of a scaling operation in wet climates is the removal of all trees and vegetation growing on the face and to a distance of several meters behind the crest of the slope. Tree roots growing in fractures on the rock face can force open the fractures and eventually cause rock falls. Also, movement of the trees by the wind produces leverage by the roots on loose blocks. The general loosening of the rock on the face by tree roots also permits increased infiltration of water, which in temperate climates will freeze and expand and cause further opening of the cracks. As shown in Table 18-1, approximately 0.6 percent of the rock falls on the California highway system can be attributed to root growth.

FIGURE 18-13 High scaler suspended on rope and belt while removing loose rock on steep rock slope.

4.4 Rock-Removal Operations

When rock-removal operations are carried out above active highways or railroads or in urban areas, particular care must be taken to prevent injury or damage from falling rock. Usually all traffic should be stopped while rock removal is in progress and until the slope has been made safe and the road cleared of debris. When pipelines or cables are buried at the base of the slope, it may be necessary to protect them, as well as pavement surfaces or rail track, from the impact of falling rock. Adequate protection can usually be provided by placing a cover of sand and gravel to a depth of about 1.5 to 2 m. For particularly sensitive structures, additional protection can be provided by a mat of tires or timber buried in sand.

During blasting it often is necessary to take precautions to control rock fragments thrown into the air by the explosions. These fragments, called *flyrock*, may damage buildings and utilities such as overhead transmission lines. Flyrock can be controlled by means of appropriate stemming lengths, burden distances, and detonation sequences and by the use of blasting mats. Blasting mats are fabricated from rubber tires or conveyor belts chained or wired together.

The advantage of removing unstable rock in comparison with stabilization by the installation of tensioned rock anchors, for example, is that removal can be a permanent stabilization measure. However, removal of loose rock on the face of a slope will be an effective stabilization measure only if there is no risk of undermining the upper part of the slope and if the rock forming the new face is sound. Method 4 in Figure 18-11 is an example of a case in which rock removal should be carried out with care. It would be safe to remove the outermost loose rock provided that the fracturing was caused by blasting and extended only to shallow depth. However, if the rock mass is deeply fractured, continued scaling will soon develop a cavity that will undermine the upper part of the slope.

Removal of loose rock on the face of a slope is not effective when the rock is highly degradable, such as shale. In these circumstances exposure of a new face will start a new cycle of weathering and instability. For this condition, a more appropriate stabilization method would be protection of the face with shotcrete and rock bolts or with a tieback wall. Design of the protection measure should consider its longevity compared with the design life of the facility and the stability of the slope, because shotcrete alone will not provide support against sliding.

5. PROTECTION MEASURES AGAINST ROCK FALLS

An effective method of minimizing the hazard of rock falls is to let the falls occur and to control their distance and direction of travel. Methods of rock-fall control and protection of facilities at the toe of the slope include catchment ditches and barriers, wire mesh fences, mesh hung on the face of the slope, and rock sheds. A common feature of all these protection structures is their energy-absorbing characteristics, which either stop the rock fall over some distance or deflect it away from the facility that is being protected. As described in this section, it is possible, by the use of appropriate techniques, to control rocks with diameters of as much as 2 to 3 m falling from heights of several hundred meters and striking with energies as high as 1 MJ. Rigid structures, such as reinforced concrete walls or fences with stiff attachments to fixed supports, are rarely appropriate for stopping rock falls.

5.1 Rock-Fall Modeling

Selection and design of effective protection measures require the ability to predict rock-fall behavior. An early study of rock falls was made by Ritchie (1963), who drew up empirical ditch design charts related to the slope dimensions, as described in Section 5.3. In the 1980s, the prediction of rock-fall behavior was enhanced by the development of a number of computer programs that simulate the behavior of rock falls as they roll and bounce down slope faces (Piteau 1980; Wu 1984; Descoeudres and Zimmerman 1987; Spang 1987; Hungr and Evans 1988; Pfeiffer and Bowen 1989; Pfeiffer et al. 1990).

Figure 18-14 shows an example of the output from the Colorado Rockfall Simulation Program (CRSP). The larger plot shows the path of a single falling rock, which lands and comes to rest on the outer slope of the ditch. The inset shows the distribution for 100 rock falls with bounce heights at each 1.7-m horizontal interval measured from the top of the slope. Similar histograms can be obtained for the bounce height and rock velocity at the analysis point (in the example, this point is located at a 21.6-m horizontal distance from the top of the slope). The input for the program includes definitions of the slope and ditch geometry, irregularity (roughness) of the face, attenuation characteristics of the slope materials, and the size and shape of the block. The degree of variation in the shape of the ground surface is modeled by randomly varying the surface roughness in relation to the block size between fixed limits for each of a large number of runs.

The results of analyses such as those shown in Figure 18-15, together with geological data on block sizes and shapes, can be used to estimate the dimensions of a ditch or its optimum position and the required height and strength of a fence or barrier. In some cases it may also be necessary to verify the design by constructing a test structure. In the following sections, types of ditches, fences, and barriers and the conditions in which they can be used are described.

5.2 Benched Slopes

Excavation of intermediate benches on rock cuts usually increases the rock-fall hazard. Benches can

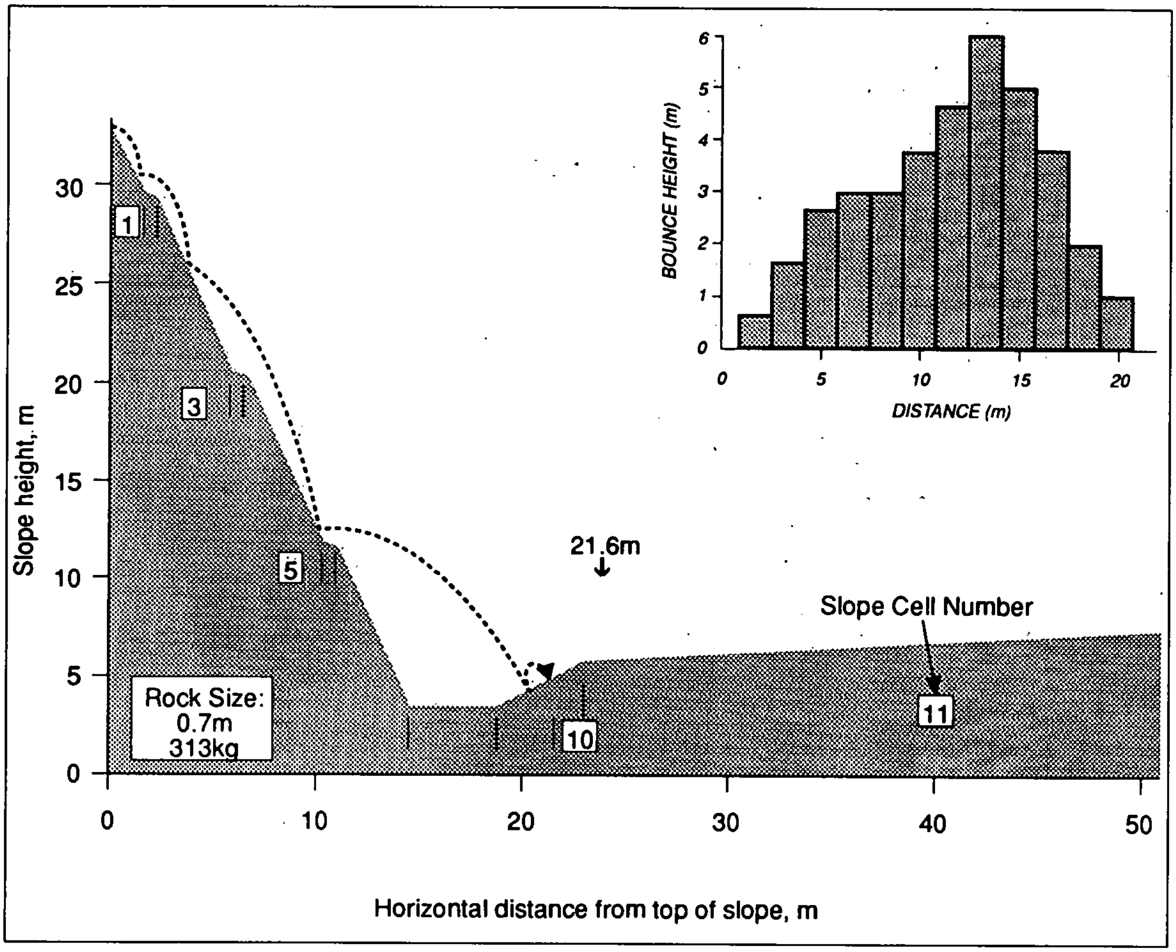


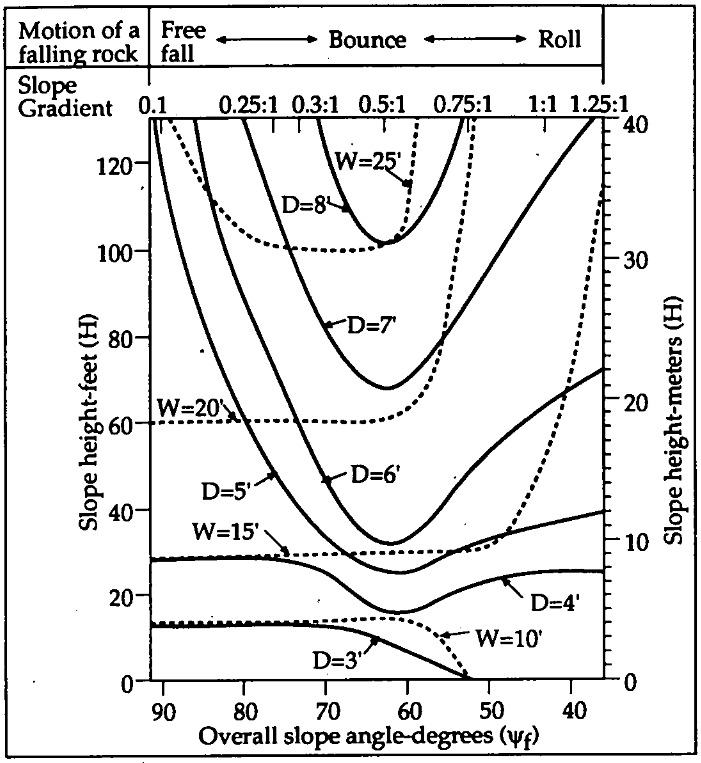
FIGURE 18-14 Example of analysis of rock-fall behavior using Colorado Rockfall Simulation Program (CRSP) with analysis point at horizontal distance of 21.6 m from crest (Pfeiffer and Bowen 1989).

be a hazard when the crests of the benches fail because of blast damage and the failed benches leave irregular protrusions on the face. Rock falls striking protrusions tend to bounce away from the face and land a considerable distance from the toe. When the narrow benches fill with debris, they will not be effective in catching rock falls. It is rarely possible to remove this debris because of the hazardous conditions of working on narrow, discontinuous benches.

5.3 Ditches

A catch ditch at the toe of the slope is often a cost-effective means of stopping rock falls when there is adequate space for it. The required dimensions of the ditch, as defined by the width of the base and the depth, are related to the height and face angle of the slope (Richie 1963). A ditch design chart developed from field tests shows the effect of the slope angle on the path that rock falls tend to follow and how this influences ditch design (Figure 18-15). For slope angles steeper than about 75 degrees, the rocks tend to stay close to the face and land near the toe of of the slope. For slope angles between about 55 and 75 degrees, rocks tend to bounce and spin, with the result that they can land a considerable distance from the toe and a wide ditch is required. For slope angles between about 40 and 55 degrees, rocks will tend to roll down the face and into the ditch, and a steep outer face is required to prevent them from rolling out. Where the slope is irregular witih protrusions on the face, the ditch dimensions should be increased from those shown on the chart to account for less predictable bounces of falling rock, or the slope can be modeled as shown in Figure 18-14 to determine required ditch dimensions.

Recent updates of Ritchie's ditch design chart incorporate highway geometric requirements as well as cut dimensions (Washington State Department of Transportation 1986), and the Oregon Department of Transportation has carried out an extensive series of tests of ditch require-



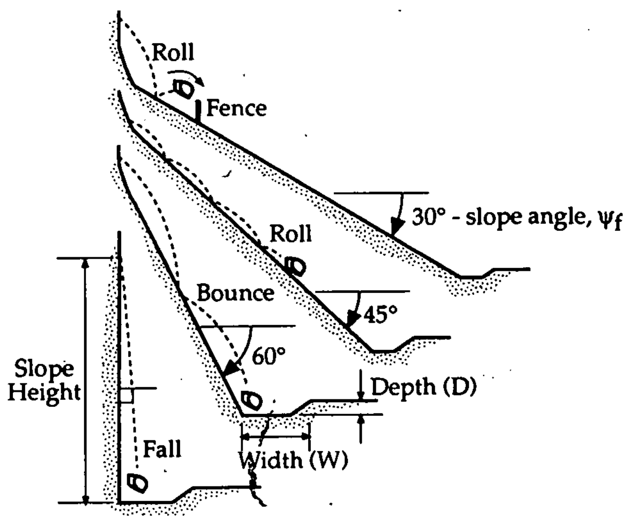


FIGURE 18-15 Design chart to determine required width and depth of rock catch ditches in relation to height and face angle of slope (Ritchie 1963).

ments for rock falls on 1/4:1 cut faces (Pierson et al. 1994).

5.4 Barriers

A variety of barriers can be constructed to either enhance the performance of excavated ditches or form catchment zones at the toes of slopes. The required type of barrier and its dimensions depend on the energy of the falling blocks, the slope dimensions, and the availability of construction materials. Commonly used examples are gabions and concrete blocks or geofabric-and-soil barriers.

The function of a barrier is to form a ditch with a vertical outer face that traps rolling rock. Barriers are particularly useful at the toes of flatter slopes where falling rock rolls and spins down the face but does not bounce significantly. Such rocks may land in a ditch at the toe of the slope but can roll up the sloping outer side onto the road; a vertical barrier helps to trap such falls.

5.4.1 Gabions and Concrete Blocks

Gabions are rock-filled baskets, typically measuring 1 m by 1 m in cross section, that are often constructed on site with local waste rock (Threadgold and McNichol 1985). Advantages of gabions are the ease of construction on steep hillsides and irregular foundations and their capacity to sustain considerable impact from falling rock. However, gabions are not immune to damage by impacts of rock and maintenance equipment, and repair costs can become significant. Concrete barriers are also used extensively on transportation systems for rock-fall containment because they are often readily available and can be placed quickly. However, concrete blocks are somewhat less resilient than gabions.

Gabions and concrete blocks or concrete Jersey barriers form effective protection from falling rock with diameters up to about 0.75 m. Figure 18-16 shows an example of a ditch with two layers of gabions along the outer edge forming a 1.5-m-high barrier (Wyllie and Wood 1981).

5.4.2 Geofabric-and-Soil Barriers

Various barriers have been constructed using geofabric and soil layers, each about 0.6 m thick, built up to form a barrier that may be as high as 4 m. By wrapping the fabric around each layer, it is possible to construct a barrier with vertical front and back faces; the impacted face can be strengthened with such materials as railway ties, gabions, and rubber tires (Figure 18-17). The capacity of a barrier of this type to stop rock falls depends on its mass in relation to the impact energy and on its capacity to deform without failing. The deformation may be both elastic deformation of the barrier components and shear displacement at the fabric layers or on the base.

FIGURE 18-16 Rock catch ditch with 1.5-m-high gabion wall along outer edge (Wyllie and Wood 1981).

Extensive testing of prototype barriers by the Colorado Department of Transportation has shown that limited shear displacement occurs at the fabric layers and that they can withstand high-energy impacts without significant damage (Barrett and White 1991). For example, a 1.8-m-wide geofabric barrier stopped rock impacts delivering 950 kJ of energy.

5.5 Rock Catch Fences and Attenuators

During the 1980s various fences and nets suitable for installation on steep rock faces, in ditches, and on talus runout zones were developed and thoroughly tested (Smith and Duffy 1990; Barrett and White 1991; Hearn 1991; Kane and Duffy 1993). A design suitable for a particular site depends on the topography, anticipated impact loads, bounce height, and local availability of materials. A common feature of these designs is that with the absence of any rigid components, they exhibit energy-absorbing characteristics. When a rock collides with a net, there is deformation of the mesh, which then engages energy-absorbing components over an extended time of collision. This significantly increases the capacity of these components to stop rolling rock and allows the use of lighter, lower-cost elements in construction. The total impact energy is the sum of the kinetic and rotational energies; the rotation of the block is also significant in determining the amount of damage at the point of impact.

5.5.1 Woven Wire-Rope Nets

Nets with high-energy-absorption capacity constructed with woven wire-rope mesh or ring mesh (submarine netting) have been developed. They can be located either in the ditch beside the road or railroad or on the steep slopes above it. The components of these nets are a series of steel I-beam posts on about 6-m centers anchored to the foundation with grouted bolts. Additional support for the posts can be provided by up-slope anchor ropes incorporating friction brakes that are activated during high-energy events to help dissipate the impact forces (Figure 18-18).

The mesh is a two-layer system composed of a 50-mm chain-link mesh and a woven wire-rope net constructed typically with 8-mm-diameter wire rope in a diagonal pattern on 100- to 200-mm centers. The wire rope and net dimensions will vary with expected impact energies and block sizes. An important feature of the wire-rope net is the method of fixing the intersection points of the wire rope with high-strength crimped fasteners. The mesh is attached to the posts by lacing it on a continuous-perimeter wire rope that is attached to brackets at the top and bottom of each post.

An extensive series of full-scale tests conducted on mesh nets has shown that they have the capacity to stop rocks with impact energies as high as 800 kJ without sustaining significant damage (Smith and Duffy 1990; Duffy and Haller 1993). This energy is equivalent to rock weighing 3000 kg traveling at a velocity of 23 m/sec. The tests also showed that rock falls could readily be cleared out from behind the net by unthreading the perimeter rope and lifting the mesh.

5.5.2 Flex-Post Rock-Fall Fence

The Flex-post fence (Hearn 1991) is composed of a series of posts constructed of bundles of seven-wire prestressing strands grouted into lengths of steel pipe (Figure 18-19). A portion of the strand bundle is left ungrouted, forming a flexible spring element that can withstand large rotations of the post without damage. The posts are spaced at about 5 m and are installed by being grouted into drill holes or pits to depths of about 1.0 m. Pairs of diagonal wire-rope stays installed between the tops and bottoms of adjacent posts help to dissipate impact energies over an extended length of the fence. The mesh is standard 50 mm square attached to the posts with intertwined steel wires. When small rocks strike the fence, the energy is absorbed by the mesh and wire stays; for larger rocks, the mesh is stretched taut and the posts bend. The fence then rebounds to its initial vertical position, sometimes throwing the rock back into the ditch.

The Flex-post fence is a low-cost, low-energy rock-fall barrier that should be designed preferably so that the highest impact energy does not result in a deflection of the posts of more than about 45 degrees. This deflection limit provides protection from rock-fall events in which a number of boulders strike the fence at approximately the same time.

5.5.3 Fences on Talus Slopes

Rock falls down talus slopes tend to roll and stay close to the slope surface. In these conditions a lightweight chain-link fence placed either on the slope or along the outside edge of the ditch will decelerate or catch the rock. The position of the fence depends on the size of the blocks and the height of

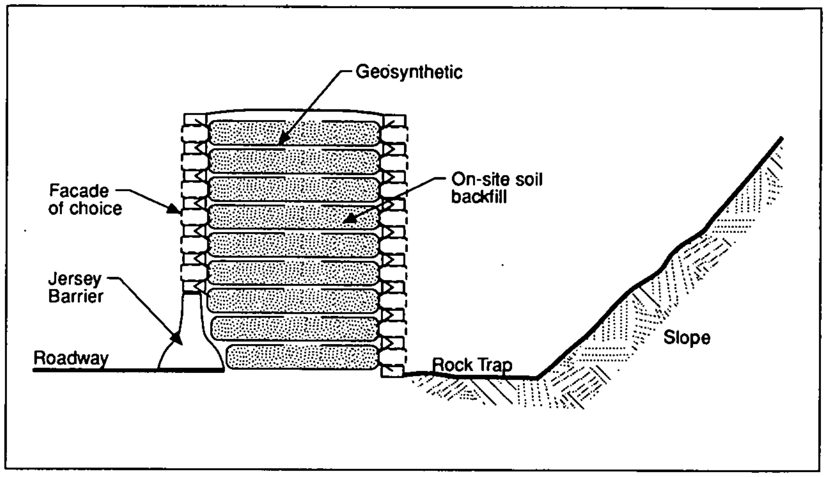


FIGURE 18-17 Rock-fall barrier constructed with soil wrapped in geofabric reinforcing strips and timber protective facade (Barrett and White 1991)

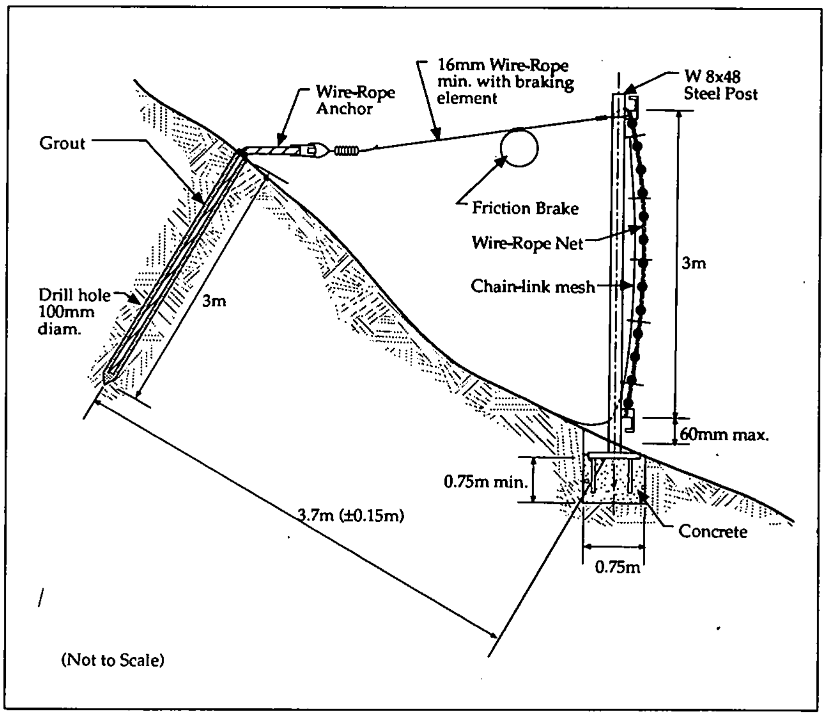


FIGURE 18-18 Side view of woven wire-rope rock catch fence (Smith and Duffy 1990).

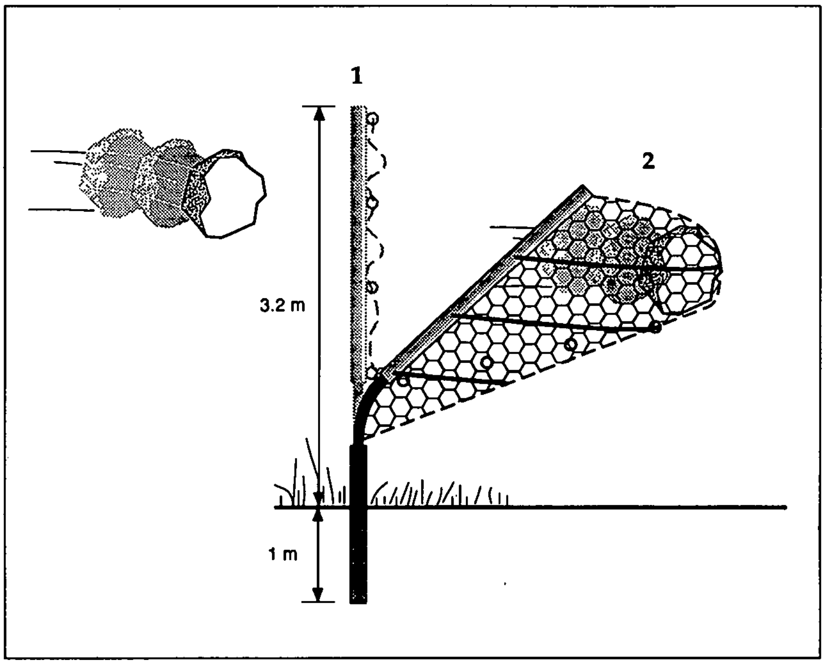


FIGURE 18-19 Colorado Flex-post rock-fall fence (Hearn 1991).

the fall. Such fences are used extensively in the states of Washington and Oregon (Lowell 1987).

5.5.4 Rock-Fall Attenuators

When rocks fall down a narrow gully or chute bounded by stable rock walls, it is possible to install a variety of relatively lightweight fences that slow down and absorb the rock-fall energy. The general method of construction is to install a pair of anchors to support a wire rope from which the fence, spanning the gully, is suspended. For rock falls with fragment dimensions up to about 200 mm, it is possible to use chain-link mesh draped down the chute from the anchor rope; wire-rope mesh can be used for larger blocks. Falling rocks are gradually brought to a halt as they bounce and roll under the mesh.

For blocks as large as 1 m, an attenuator fence utilizing waste automobile tires has proved to be successful in a number of installations in Colorado (Andrew 1992). The fence consists of a wire rope from which a number of steel rods are suspended each holding a stack of tires mounted on rims (Figure 18-20). The stacks of tires are arranged so that they form a continuous barrier across the chute. When the attenuator is struck by a falling rock, the kinetic energy is dissipated by a combination of the compression of the rubber tires and the movement of the stacks of tires, which swing on the suspension cable rope. In order to improve the aesthetics of the fence, a second fence constructed from hanging timbers can be located just downslope from the tire attenuator (Figure 18-20).

Maintenance requirements and worker safety should be considered in the design of fence systems. A properly designed system should not need frequent repairs if the actual impacts are within the tolerances of design energies. However, clearing out of accumulated rock is necessary for any system. Typically, fixed barriers such as geofabric walls require room behind them for clearing operations. In contrast, woven wire-rope fences do not have this requirement because of their modular design, which allows the nets to be cleared from the front by the removal or lifting of each panel in turn.

5.6 Draped Mesh

Wire mesh hung on the face of a rock slope can be an effective method of containing rock falls close to the face and preventing them from bouncing onto the road (Ciarla 1986). Figure 18-21 shows an example of mesh installed on a near-vertical face. Because the mesh absorbs some of the energy of the falling rock, the required dimensions of the ditch at the toe of the slope are considerably reduced from those shown in Figure 18-15. When there is no ditch, the lower end of the mesh should be no more than about 0.6 m above the toe to prevent rocks from rolling onto the road.

Chain-link mesh is suitable on steep faces for controlling rock falls with dimensions less than about 0.6 m, and woven wire rope may be suitable for rock with dimensions up to about 1 m. For larger blocks, ring nets can be used. For very high installations, chain-link mesh can be reinforced with lengths of wire rope. In all cases the upper edge of the mesh or net should be placed close to the source of the rock fall so that the blocks will have little momentum when they strike the mesh.

The features of a mesh installation are as follows. Anchors are installed along the slope above the anticipated rock-fall source. The anchors can consist of steel loop-eye rock bolts grouted into holes drilled in the rock or concreted into larger-diameter holes excavated in soil. Support cables consisting of an upper-perimeter cable and vertical reinforcement cables are attached to the anchors. The mesh is secured to the cables with lacing wire, hog rings, or other types of specialty fasteners. The mesh is not anchored at the bottom of the slope or at intermediate points. This permits rocks to work their way down to the ditch rather than accumulating behind the mesh; the weight of such accumulations can cause the mesh to fail.

5.7 Warning Fences

Fences and warning signals that are triggered by falling rock are sometimes used to protect railroads and highways (Figure 18-22). The warning fence consists of a series of timber posts and cantilever crossarms that support rows of wires spaced about 0.5 m apart. At least one wire will be broken by rocks rolling or bouncing down the face. The wires are connected to a signal system that displays a red light when a wire is broken. The signal light is located far enough from the rock slope that drivers have time to stop and then to proceed with caution before they reach the rock-fall location.

Warning fences are most applicable on transportation systems where traffic is light enough to accommodate occasional closures of the line. However, the use of warning fences as a protection measure has a number of disadvantages. The signal lights must be located a considerable distance from the slope, and falls may occur after the traffic has passed the light. False alarms can be caused by minor falls of rock or ice, and maintenance costs can be significant.

5.8 Rock Sheds and Tunnels

In areas of extreme rock-fall hazard where stabilization work would be very costly, construction of a rock shed or even relocation of the highway or railroad into tunnels may be justified. Figure 18-23 shows an example of a number of rock sheds constructed to protect a railroad. The sheds were built with timber roofs that are inclined at a slightly shallower angle than the chutes down which the rocks are falling. The function of the roof is to direct the rocks over the track rather than to withstand a direct impact.

Figure 18-24 shows a possible construction procedure for a rock shed where there are steep slopes above and below the highway. In this case the roof will sustain the direct impact of falling rock and must be designed to withstand impact loads. The critical feature of the design is the relationship between the energy of the falling rock and the energy-absorbing characteristics of the components of the shed—anchorage of the roof to the cliff face, the roof, and the footings in the slope below the road. A stable anchorage requires that the anchors themselves, and the rock face to which the roof is anchored, withstand both the

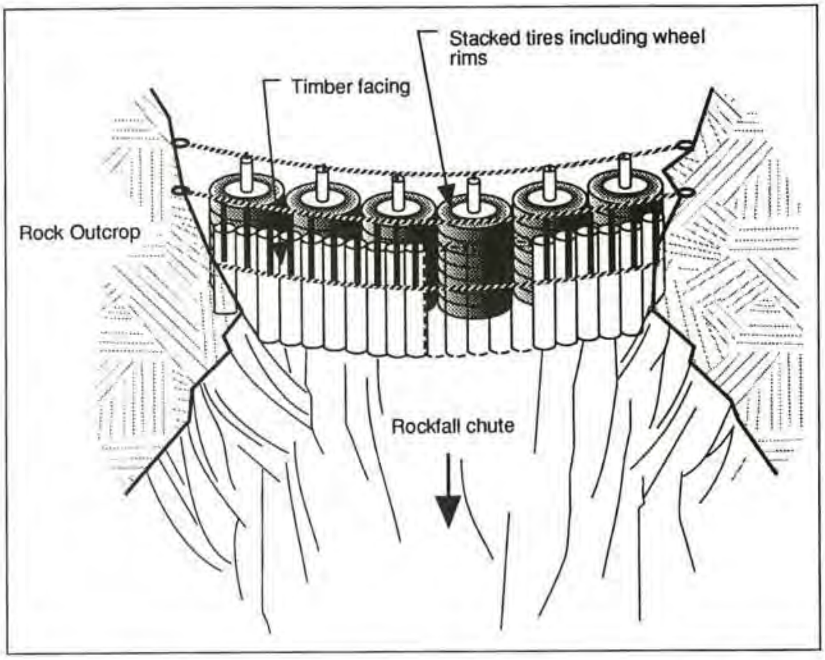


FIGURE 18-20 Rock-fall attenuator constructed in chute (Andrew 1992).

FIGURE 18-21 Wire mesh hung on vertical face to direct falling rock into ditch at toe of slope.

FIGURE 18-22
Typical warning fences at toe of steep rock slope to detect rock falls on railway.

FIGURE 18-23
Rock sheds constructed with steeply sloping roofs that deflect rock falls over railway tracks.
CANADIAN NATIONAL RAILWAY

dead load of the structure and the live load produced by the rock-fall impact. The impact load will include both a normal load and some tensile component. The gravel on the roof acts as an energy absorbent and protection for the concrete, but its thickness must be balanced against the weight that must be supported by the structure. Alternatively, a lighter-weight crushable element, such as a cellular metal panel, may be used. The columns supporting the lower end of the roof are founded on rock-socketed piers that extend through the fill into sound rock. This type of foundation provides the required support with minimal settlement.

In locations at which it is impractical to construct a rock shed, it may be necessary to drive a tunnel to bypass the hazard zone. For example, a railway in British Columbia drove a 1200-m-long tunnel to avoid a section of track located on a narrow bench between a steep, unstable rock cliff above a 400-m-deep lake. Major rock falls were a hazard to train operations and had caused track closures lasting as long as two weeks (Leighton 1990).

6. CONTRACTING PROCEDURES

Specifications and contract administration for projects involving rock slope stabilization have particular requirements that differ from those for most excavation projects. Some contractual requirements of these projects and how they may be addressed in the specifications are described in the following sections.

6.1 Flexibility

Contracts on most stabilization projects must be flexible with regard to both the scope of the work and the procedures required because these may only be determined as the work progresses. For example, scaling may be specified for an area of loose, broken rock, but the actual scaling time will depend on the extent of the loose rock that becomes evident once scaling begins. Furthermore, the number and length of bolts will be assessed from the dimensions and shear-strength characteristics of the potentially loose blocks remaining on the face after scaling is complete. It is particularly difficult to estimate the scope of the stabilization work when the slope is partially covered with soil and vegetation and when it is too dangerous to reach the slope during the investigation phase of the project.

One form of stabilization in which the scope of the work can be defined with some precision is resloping, in which an existing slope is cut back at a flatter angle. If a series of cross sections is prepared, the excavation volume can be specified.

6.2 Payment Methods

In order to prepare a flexible contract, it is necessary to have a method of payment that allows for possible changes in the quantities and methods of work. This requirement can usually be met by requesting bids on unit prices for items the quantities of which are uncertain. An estimate is given for the expected quantity of an item, and the product of this quantity and the unit bid price forms the basis for comparing bids and making payments. A formula should be included for increasing unit prices if actual quantities are less than those estimated or for decreasing unit prices if quantities are greater than those estimated. This adjustment takes into account the contractor's fixed overhead, which is independent of the quantity of work.

Work items often bid on unit prices include scaling hours for a crew of a specified size, length of rock bolts, volume of shotcrete, and standby time on transportation routes when work has to stop for traffic openings. Work items often bid as lump sums include mobilization and demobilization and concrete volume for buttresses of specified dimensions.

Cost-plus-fixed-fee is an appropriate type of contract for emergency work. In this type of contract the contractor is paid specified costs for labor, equipment, and materials related to the direct costs plus an additional fee. This fee is a profit-plus management fee as reimbursement to the contractor for overhead costs incurred from the performance of the work. Items making up the fee include, but are not limited to, salaries, rent, taxes, and interest on money borrowed to finance the project. For work on emergencies the fee may be negotiated on a percentage of the cost. However, in order to control costs, the scope of the work should be defined as soon as possible and either a fixed fee should be negotiated or the contract should be converted into a lump-sum contract.

6.3 Preselection

When the stabilization contract requires work on steep slopes and specialized skills such as trim blasting and installation of tensioned rock bolts, it is usually advantageous to have the work performed by experienced contractors. Therefore, if it is permitted by procurement regulations, only suitably qualified contractors should be invited to submit bids.

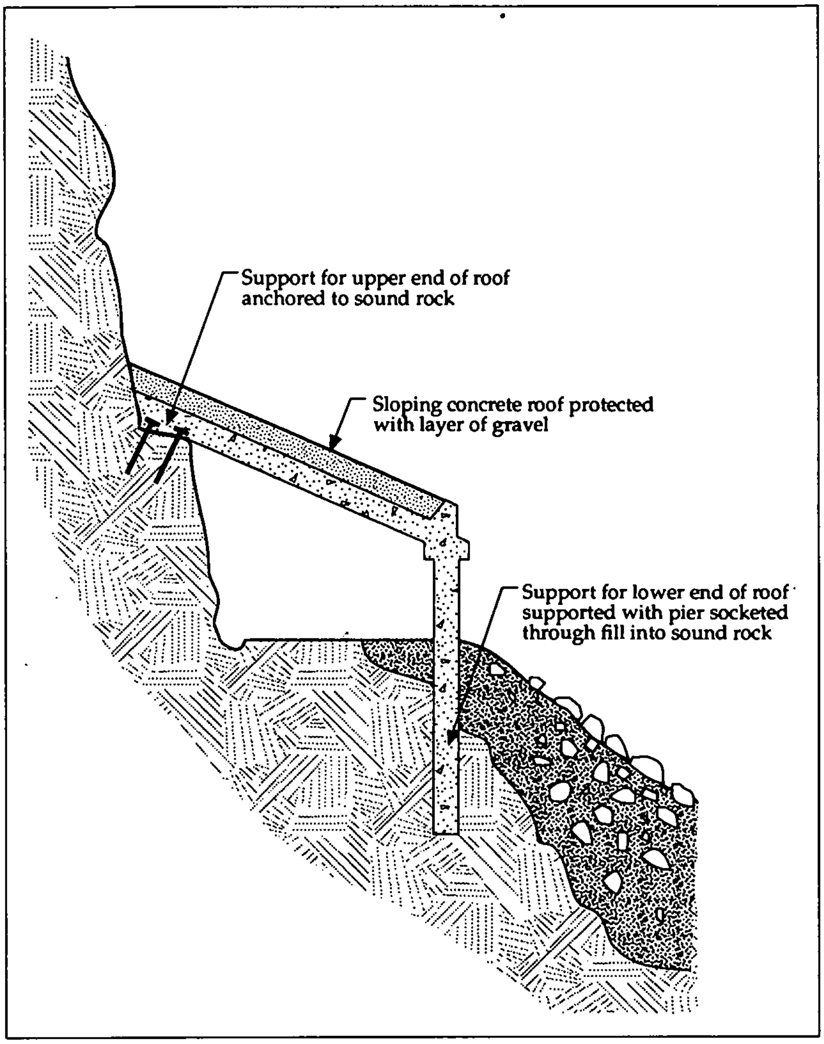


FIGURE 18-24
Rock shed showing possible anchoring and foundation construction methods.

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