Chapter 9

Engineering of Rock Slopes

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In transportation corridors, the objective of rock-slope engineering is to maintain slopes for maximum safety and efficiency. Minimizing rock excavation and predicting the safety and ultimate behavior of rock slopes, whether for highway, railway, spillway, quarry, dam site, or opencut mine, are common objectives of civil, geology, and mining engineers. The rational design of rock slopes is particularly important if slopes are steep, if safety is important, and if slope design significantly affects project costs.

The present empirically based, cut-and-try methods and techniques used to design rock slopes are inadequate. New principles and improved technological capability are needed, particularly in mining operations, in which the trend is toward open-pit mines, and in transportation facility construction, which may be increasingly required in steep terrain.

That rock slopes must be treated differently from soil slopes cannot be emphasized enough. In the analysis of rock slopes, one must recognize the differences in the basic characteristics and behavior of soil and rock. Unlike a soil mass, which is a relatively homogeneous and continuous medium composed of uncemented particles, a rock mass is a heterogeneous and discontinuous medium composed essentially of partitioned solid blocks that are separated by discontinuities. The geometry of the spaces between the solid materials and the interlocking properties of the components of soil and rock are totally different. Failure in soil tends to occur within the soil mass, and the direction of the surface of failure tends not to depend on variations of soil properties. The surface of failure in hard rock masses, however, tends to follow the preexisting discontinuities and not to occur through the intact rock to any great extent unless the rock is quite soft. The shear strength of a rock mass is determined largely by the presence of the discontinuities, and the result is that the rock mass is anisotropic in its strength and deformational properties.

The design of rock slopes involves both engineering and geology and, in addition, a combination of the knowledge of precedent with the art of estimation and judgment. The engineer-geologist must obtain quantitative information on those factors that are necessary for making calculations of the probable stability of the slope. Those factors include structural geology, local topography, drainage, hydrogeology, tectonic history, and other environmental features that may add to or detract from the stability of the slope.

Whether a slope will be stable or unstable will depend on how the forces that tend to resist failure compare with those that tend to cause failure. This concept defines the factor of safety for the slope as the ratio of the sum of the resisting forces that act to prevent failure to the sum of the driving forces that tend to cause failure. To a considerable extent, the problem of rock-slope design and related aspects is one of applied mechanics, and the necessary margin of safety to be provided in any particular case is a question of judgment.

The terms of reference and related slope-design problems of an open-pit excavation, for example, may be entirely different from those of highway and railway cuts. For that matter, slope-design problems and requirements for highways and for railways can also differ markedly. Highways can usually have a greater degree of slope instability on their rights-of-way than railways. Unlike automobiles, trains cannot steer or brake readily and can be derailed by rocks no larger than 30 cm (1 ft). Therefore, a remedial measure that may be used on a highway slope may be entirely unacceptable on a railway slope. No particular attempt has been made here, however, to separate railway and highway rock-slope engineering techniques.

Rock-slope engineering is concerned not with large landslides but with rock falls of individual blocks, translation of small rock masses, and occasional slides of accumulated debris from gullies, talus slopes, and postglacial slide areas. This chapter discusses the aspects of rock-slope engineering relevant to designing cut slopes and maintaining the long-term efficiency and safety of existing slopes. Discussed are the significant factors in rock-slope design, the procedures

Figure 9.1. Well-developed, steeply dipping set of joints along highway at Slocan Lake, British Columbia. Joints control slope stability almost entirely.

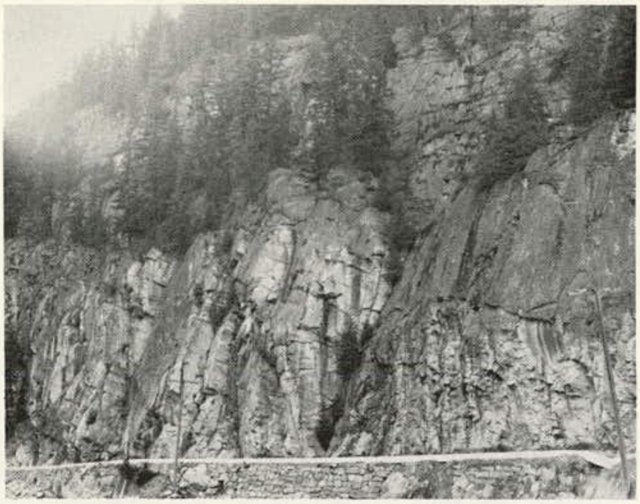


Figure 9.2. Well-developed discontinuities dipping toward highway at Porteau Bluffs, British Columbia. Two significant slides have occurred in this area.

for the analysis of rock-slope stability, the general planning involved in extensive rock-slope engineering along transportation corridors, and the stabilization, protection, and warning measures that can be used to remedy rock-slope problems.

SIGNIFICANT FACTORS IN DESIGN OF ROCK SLOPES

This section describes the basic factors that are significant to the stability of rock slopes. Additional information is given by Stacey (*9.105*), Piteau (*9.87*), Hoek and Bray (*9.42*), Duncan (*9.23*), Deere and others (*9.21*), Terzaghi (*9.108*), Jennings (*9.47*), Goodman (*9.29*), Coates (*9.16*), and the Canada Department of Energy, Mines and Resources (*9.94*).

Structural Discontinuities

The stability of rock slopes depends largely on the presence and nature of defective planes or discontinuities within the rock mass. For the greatest part, the significant physical and mechanical properties of the rock mass are a function of the attitude, geometry, and spatial distribution of these defective surfaces. The basic principles of rock-slope design are based on

1. The systems of joints and other discontinuities;
2. The relation of these systems to possible failure surfaces;
3. The strength parameters of the joints, which include the properties of both the joint surfaces and any joint infilling materials; and
4. The water pressure in the joints.

Examples of the significance of structural discontinuities in slope design are shown in Figures 9.1 and 9.2.

The stability of rock slopes, therefore, is assessed principally by analyzing structural discontinuities in the rock mass and not by assessing the strength of the intact rock itself. Observations relating to shear failure along discontinuities should be made insofar as the discontinuities affect the cohesion and friction developed with respect to shear along these features. The relations of discontinuities to the direction and inclination of the rock slope and to any factors that might influence any potential surface failure must receive special attention. These statements apply even moreso to faults, regardless of how great or small displacements have been.

Field studies show that rocks are usually jointed in preferential directions. Depending on their modes of origin, however, rocks have joint sets whose characteristics can vary greatly. Wide variations can occur in the average spacing between joints, the nature and degree of joint infilling materials, the physical characteristics of their surfaces, and the degree of their development. One joint set can, therefore, have effects on shear characteristics quite different from those of another set, and the various properties of each of the joint sets must be considered individually in rock-slope design. (Strength evaluations are explained in Chapter 6). The main properties that are associated with structural discontinuities and that require quantitative and qualitative evaluation follow.

Orientation or Position in Space

Orientation or position in space is the most important property. If the orientation of joints favors potential slope failure, the effects of other properties are generally unimportant. Discontinuities dipping out of the slope must be carefully considered. Their potential instability increases proportionally as the strike of the discontinuities approaches that of the slope.

Continuity or Size

Continuity or size is the most difficult property to assess. The strength reduction on a failure surface that contains one or more discontinuities is a function of their size. The average continuity of a particular joint set partly indicates the extent to which the rock material and the discontinuities will separately affect the mechanical properties of the mass (*9.23*, *9.47*); in addition, the average continuity affects the magnitude of possible failures involving these features.

Infilling Materials and Openness

Infilling materials are those materials that occur between the walls of the discontinuities in the mass. The three most important characteristics of infilling materials are thickness, type, and hardness. Jaeger (*9.45*) notes that, if infilling is sufficiently thick, the walls of the discontinuity will not touch and the strength properties will be those of the infilling material.

Spacing

The spacing of discontinuities partly indicates the extent to which the intact rock and the discontinuities will separately affect the mechanical properties of the rock mass. A rock mass is inherently weaker if spacing is close. Also, greater joint frequency increases the potential for dilatancy.

Asperities

Two orders of asperities are recognized. First-order asperities (waviness) are unlikely to shear off; they affect the shear-movement characteristics along the discontinuity and effectively modify the direction of movement during slope failure (*9.21*). Undulations or waves on the discontinuity reduce the effective apparent dip of the plane, and this dip angle and not the mean dip angle should be used for calculating the disturbing forces on a potential failure plane (*9.68*). Second-order asperities (roughness) are much smaller and are likely to shear off during movement; these produce an apparent increase in frictional strength along the discontinuity (*9.99*).

Previous Shear Movement

Shear displacement on a discontinuity results in breaking through asperities and thus reducing the shear strength from initial peak values to values that approach residual shear strength (*9.54*).

Rock Type

When a slope consists of several rock types, their combined mechanical behavior may differ considerably from that of the constituent units themselves. Hence, each particular rock type may require individual assessment for its behavior in the slope. Different rock types and the products that result from their alteration have inherently different weaknesses and strengths as a result of their origins and compositions. Hence, the properties of different rock and infilling materials can vary within wide limits (*9.107*). The characteristics of each rock type significantly influence the friction angle, nature of asperities, and hardness of the walls of the discontinuities.

Rock Hardness

There is a relation between rock hardness and unconfined compressive strength. According to Jennings (*9.46*), an increase in hardness has a corresponding increase in the shear strength. Infilling material and second-order asperities are sheared through during shear failure along a discontinuity; the shear strength of the discontinuity is a function of the shear strength of the infilling materials and the rock materials that form the asperities.

Origins

The origins of discontinuities will affect their engineering significance in the slope. Faults, as compared to joints, for example, have different origins and accordingly different geometry, spatial distribution, weathering and infilling characteristics, and seepage characteristics.

Groundwater

The presence of water in joints has probably been responsible for more rock slides than all other causes combined. Hence, a thorough knowledge of the character and influence of the hydrogeologic regime is necessary, and a knowledge of the water pressure distribution and the factors that influence it is the most essential. One must consider the controlling influences of texture, stratigraphy, and structure on factors such as flow, permeability, recharge, and storage capacity. Consideration should also be given to environmental factors, such as variations in climatic conditions, that result in periods of either high or low recharge and other variations in groundwater conditions. Further discussions concerning climatic influences are given later.

According to Terzaghi (*9.108*), Serafim (*9.103*), and Müller (*9.77*), water in the slope can affect stability by

1. Physically and chemically affecting the pore water and its pressure in joint infilling materials, thus altering the strength parameters of the materials;
2. Exerting hydrostatic pressure on joint surfaces, thus reducing the shearing resistance along potential failure surfaces by reducing the effective normal stresses acting on them; and
3. Affecting intergranular shearing resistance, thus causing a decrease in compressive strength.

Lithology, Weathering, and Alteration

Before one can completely comprehend the particular problems of stability, one must understand the lithology of the physical properties not only of the rock mass itself but of all the materials in the mass. Usually a slope is made up of a complex of rocks of diverse geologic origins. It may have markedly different sequences of sediments, may be intruded by bodies of igneous rocks, or may be partly metamorphosed. The mass represents an association of several lithologic units whose mechanical behavior is that of an integral whole, which may differ considerably from the individual lithologic units themselves.

A sedimentary rock sequence, for example, is markedly different from an igneous series or a metamorphic complex. Each particular type is characterized by a certain texture, fabric, bonding strength, and macro and micro structures. The most important rock properties are the nature of the mineral assemblage and the strength of the constituent minerals; a rock material cannot be strong if its mineral constituents are weak or if the strength of the bonds between the minerals is weak.

The body of rock or the host rock in which the discontinuities occur directly influences the strength characteristics of the discontinuities, particularly if joint infilling materials are absent. The wall rock of the discontinuities affects the joint strength in two ways (*9.5*): It affects the frictional properties of the material forming the joint, and it affects the intact strength of the asperities of the joint surfaces. The frictional properties of joints are highly dependent on the proportions of the various minerals that are exposed along their surfaces.

The properties of rock can be altered by weathering, i.e., action by atmospheric elements and conditions. Weathering can adversely affect the deformation properties of rocks and can reduce their ultimate bearing resistance and other strength properties. The effects of weathering are usually estimated qualitatively, but in a thorough slope analysis should be estimated quantitatively.

Moisture can also cause alteration of rocks. An increase of moisture content can cause high swelling pressures in montmorillonite, which occurs in joints either as infilling or as a product of alteration. These high swelling pressures and the low shear strength of montmorillonite can lead to rock falls and, in some instances, rock slides. Changes in the water table can also affect rocks containing soluble minerals, such as rock salt, gypsum, limestone, and dolomite, which are especially susceptible to dissolution and physical alteration. Many slope failures have been attributed to the low strength of moist graphite, talc, chlorite, and other layer-lattice minerals occurring in fault zones.

A fluctuating water table can also contribute markedly to the alteration and periodic changes in the mechanical properties of rocks.

In some rocks, changes in moisture content lead to slaking, a crumbling or disintegration of the rock. Usually shales with higher percentages of clay-size material and mudstones soften on exposure to the atmosphere and revert to a muddy condition when submerged in water.

Climatic Conditions

The effects of climate on the stability of rock slopes in transportation corridors and the various remedial measures that must be taken to accommodate these conditions are important in rock-slope engineering. Daily temperature variations, precipitation, snow, and freeze-thaw conditions, acting either independently or in combination, often cause significant stability problems.

Terzaghi (*9.108*) noted that the groundwater conditions and hence the effective hydrostatic pressures can vary within wide limits, depending on the climatic conditions and geologic environment. Variations in the position of the groundwater table as a result of seasonal rainfall, sudden heavy storms, and ice on the face of the slope are shown in Figure 9.3 (*9.108*). Because of the yield of the excavation face, a zone of higher permeability will be close to the face, but the extent of this zone is still largely undefined. If the rock at the toe of the slope is already loaded nearly to failure, the additional water pressure may be sufficient to cause the slope to fail.

From correlations made from several hundred slope failures in Norway, Bjerrum and Jørstad (*9.9*) show the significance of periods of high rainfall infiltration. They note

Figure 9.3. Hypothetical possible positions of groundwater table in jointed rock slope (*9.108*).

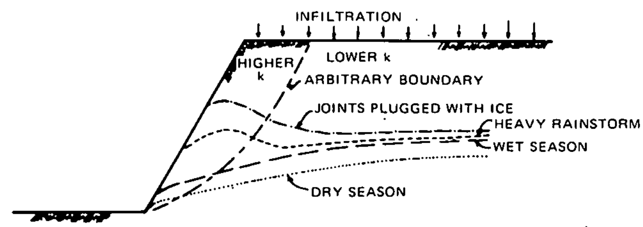


Figure 9.4 Rock falls in eastern Norway in relation to altitude, time of year, and temperature (*9.9*). Dots indicate rock falls occurring during different seasons below and above a 100-m (328-ft) elevation.

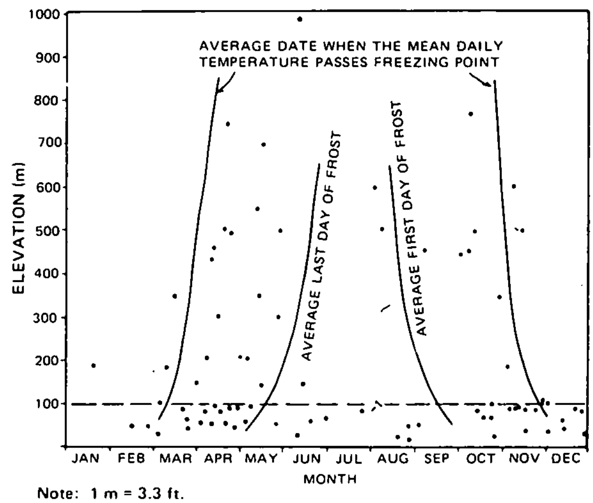
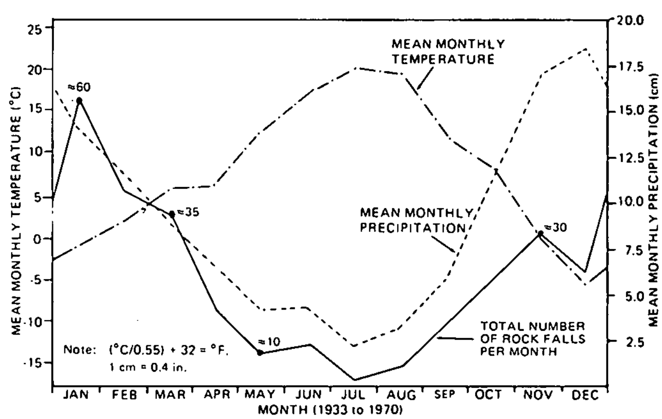


Figure 9.5. Correlation of number of rock falls with temperature and precipitation on railway line in Fraser Canyon, British Columbia (*9.84*).



(Figure 9.4) that failures are most prevalent when the water table is high in the spring because of snowmelt and in the fall during heavy rainfall. Some of these failures are also attributed to ice forming on the slope face and causing water pressure to build up in joints.

Figure 9.5 shows comparative analyses by Peckover (*9.84*) of rock-fall occurrence, rainfall, and temperature in the Fraser Canyon in British Columbia. Maximum rock falls occur in the spring and fall when the mean temperature is about 0°C (32°F) and frequent freeze-thaw cycles are occurring. When temperatures are above freezing, the frequency of rock falls is a function of degree of rainfall. The number of rock falls originating on the steep rock faces in the Fraser Canyon is probably high because of the absence of snow, vegetation, and soil, the presence of which would insulate the rock from unusually low temperatures and from changes in temperature.

Frost action probably directly or indirectly accounts for more rock falls than all other factors combined. Water undergoes about a 9 percent volume increase when it freezes and exerts tremendous pressure when it freezes in a confined space. According to Reiche (*9.97*), "Water-filled cracks or joints which terminate downward and which are narrow and perhaps irregular may be converted into essentially closed systems by preliminary freezing of the water in the superficial parts. In such cases the combination of expansion and low compressibility may exert a disruptive force which, if the temperature continued to fall and rock pressure permitted, would approach 30,000 pounds to the square inch at -22°C (i.e., -7.6°F)." Freezing temperatures of this order are not uncommon in temperate zones.

Piteau and others (*9.93*) correlated mean annual movements of an overturning failure with average monthly rainfall, snowfall, and temperature. Although they found that a direct relation existed between movement and rainfall (i.e., groundwater pressure buildup), they found that snowfall and freezing retarded movement. The average monthly rainfall peaked in January, but movement decreased in December, when freezing and snowfall conditions started. The snow blanket prevented rainfall from infiltrating into the slope, and freezing temperatures reduced the availability of free water on the slope.

Slope Geometry in Plan and Section

Most current theories of slope stability consider the slope to be two dimensional (i.e., a unit length of an infinitely long slope is considered to be in plane strain) and the plan radii of the crest and toe of the slope to be infinite. However, this latter condition is not normally encountered in practice; slopes that are concave in plan tend to be more stable than those that are convex.

Highway or railway cut slopes in mountainous terrain are often convex and therefore have a greater tendency toward instability. In open-pit mines, on the other hand, slopes are usually concave and therefore more stable. In a study of slopes of diamond pipes in South Africa, Piteau and Jennings (*9.92*) and Piteau (*9.89*) found that the plan radius of curvature of the slopes has a marked effect on slope stability. They applied this finding to the DeBeers Mine to predict the final breakback position, subject to its slope geometry. Lorente de No (*9.65*) and Förster (*9.27*) made similar findings.

Horizontal tangential stress concentrations in such a slope can be either compressive or tensile, depending on the slope geometry (*9.64*). Horizontal stresses tangent to the slope are beneficial in a concave slope, for they create an archlike effect whereby the blocks forming the partitioned rock mass tend to be squeezed together. Compressive stresses substantially improve the shearing strength. All three principal stresses are compressive, and the maximum and minimum principal stresses act in the vertical and radial directions respectively. For the convex slope, the converse is the case, and horizontal tangential stresses are tensile. The maximum principal stress is still vertical, but the minimum principal stress acts in a horizontal direction tangent to the slope, and the slope material is in tension. Cohesion that would normally have occurred is reduced. Since a rock mass is relatively weak in tension, tensile stress concentrations in the slope induce instability, causing unrestrained blocks to slide out.

For an infinite slope in rock (normally specified in rock mechanics when rock strength includes cohesion), a slope can be steeper than that indicated by the angle of friction. Normal stress increases downslope, and the effect of cohesion relative to that of friction decreases. The slope thus flattens to the angle of internal friction. The profile of such a slope, therefore, is theoretically concave from top to bottom. The variation of slope angle with slope height has a small but significant influence on the stress distribution in the slope according to Yu and Coates (*9.111*) and, therefore, can be expected to have some influence on the stability.

Time Factor and Progressive Failure

Natural rock slopes undergo progressive failure in time by the processes of creep and flow. Hence, it is of considerable importance that one recognize whether the analysis and ultimate design of the slope meet the requirements of short-term or long-term stability. Also, because of progressive failure, one must consider potential maintenance problems and design accordingly. For practical purposes, allowances must be made for some reduction in the strength properties of the rock mass with time (*9.76*).

Murrell and Misra (*9.78*) note that time-dependent strains occur when rock material is subjected to relatively high stresses for long periods. Most of the forces involved in such deformations are indeterminate functions of time, being dependent on the effects of the excavation, regional stresses, alteration processes in the mass, physical and chemical action of groundwater, and seasonal variations of temperature and rainfall. These lead to fatigue and opening of cracks with irreversible deformations and progressive weakening of the mass. Therefore, continued movement of a rock slope is cause for concern. Movements are important, for relative displacements along defects in the rock mass tend to reduce the resistance along these defects and may bring about failure.

A slope may be stable when first excavated but, because of gradual deterioration and adjustments toward equilibrium, may become unstable with the passage of time. The time required for deep-seated failure in hard rocks is almost impossible to evaluate. Near-surface failure, such as raveling or detaching of rock segments, however, may develop only a few years after the excavation has been completed. This kind of failure should be considered in the slope design and remedial measures proposed. Soft rocks, such as shale, mudstone, and other types of argillaceous materials, can undergo magnitudes of deformation that may lead to failure within significantly shorter periods, sometimes within several days.

Residual and Induced Stress

The cut slope created by an excavation affects the stresses in a rock mass at the boundary of the excavation. However, predictions of the magnitude of these stress concentrations and their effects on the stability of the slope are complex. To date, results of studies of stress distribution in slopes and the manner in which stresses affect the stability are largely hypothetical. No mathematical or physical models are available to predict the effects of varying the slope excavation geometry and the ultimate variation in stress concentration (*9.73*). In this regard, probably the most significant advances have been made in using finite element procedures to consider the stress-strain compatibility of the slope.

Contrary to earlier views, bedrock is not a predictable platform in which the only acting forces are vertical due to the weight of the rock. Rocks may also be subject to significant horizontal residual stresses that under certain circumstances could have important influences on the behavior of rocks at excavations. Hast (*9.32*) notes that this influence has been proved to be quite substantial in deep excavations. Regional stresses, denudation, tectonic uplift, glacioisostatic rebound, and other conditions might also affect the stability of surface excavations in rock, even though less dramatically.

Existing Natural and Excavated Slopes

Slope design should take into account past experience with both stable and unstable slopes. Kley and Lutton (*9.51*), Lutton (*9.66*), and Shuk (*9.104*) show that analyses of both natural and excavated slopes will provide valuable background information for proposed excavation design, particularly those for mountainous terrain.

Usually the angles of both natural and excavated slopes provide conservative estimates of slope angles that can be achieved in slope design. Humans can invariably improve on the slope angles provided in nature by giving careful attention to drainage, artificial stabilization methods, and control of natural slope-forming processes. Whether one is evaluating profiles from surface excavations or natural slopes, the problem is in estimating the degree of the conservatism inherent in the population of slope profiles being considered. This problem is properly resolved by the engineer and the geologist (primarily geomorphologist) complementing each other.

The safety factor of natural slopes is commonly not much greater than unity (*9.76*). The value of unity, however, applies specifically to that situation in which the most adverse groundwater conditions develop naturally. A hillside will normally have a safety factor greater than one most of the time, but it may become one when the water conditions or the natural disturbing forces, such as recurring earthquakes, are as severe as are likely to occur.

Slopes should not be compared if the general modes of their formation (i.e., types of excavation and slope-forming processes) differ. When slope processes are similar, stable slope case histories can be relied on to predict a lower bound to the design slope angle. The use of slope case histories requires that factors such as slope and failure geometry, geology, and material properties be obtained. Slope monitoring can also be helpful when case histories are used in slope design.

In highway and railway slope problems, the most important factor relating to case history analyses is probably the incidence of failure. This is shown by Piteau (*9.90*) in a regional slope stability study of the Fraser Canyon. Comparative analyses of incidence of slope failures (rock falls, landslides, debris slides) and of geological factors (geomorphology, structure, lithology, groundwater, river hydraulics) were made to assess those factors controlling slope stability. More than two-thirds of all incidents occurred where the river had been deflected into the bank and had undermined the slope. Severe lateral erosion by the river was the result of either general directional changes in the river or the development of alluvial fans that forced the river into the opposite bank.

Until failure mechanisms of rock slopes are better understood, consistently reliable predictions of rock-slope behavior are not possible. Because natural slopes may provide some clues, as much attention as possible should be given to examining the way in which local slides and deformations develop. Analysis of rock slides in an area can provide excellent information on the mechanics and patterns of their formation.

Coates and Gyenge (*9.17*) make use of information from the performance and characteristics of existing or previously existing slopes and have developed the principle of incremental design for slopes. This is the "process of extrapolating from the known to the new, or predicting the conditions that result from a change in the present operations." Although this work is basically applied to open pits and the incremental predictions may not be of high accuracy, some of the basic concepts developed can be applied to rock-slope design for transportation routes.

Dynamic Forces

The significance of earthquake vibrations is well documented elsewhere; effects of blasting are considered later in this chapter. The brief discussion here is of the effects of vibrations along transportation corridors due to vehicles.

There is some belief that traffic vibrations, particularly from trains, may be a significant factor with respect to slope stability. However, a comparison of relative masses shows that traffic-induced vibrations are extremely small, even for slopes close to the right-of-way. Therefore, such vibrations can be considered insignificant. According to Peckover (*9.84*), the incidence of slope failures along railways when the train is passing, for example, is no greater than would be expected from the proportion of total time that the railway is occupied.

FUNDAMENTAL PROCEDURES IN ANALYSIS OF ROCK SLOPES

This section describes the basic approach to analysis of rock slopes as well as the theoretical and analytical process. The process can be summarized as follows: Discontinuities in the rock mass are systematically measured and statistically analyzed to determine their nature and distribution. Estimates are made of the strength properties of the discontinuities. These factors are quantitatively described and theoretically applied to determine the strength along any potential failure plane. Shear strength parameters are assessed, thus allowing the factor of safety of the slope to be calculated and slope design to be considered.

Determination of Structural and Other Relevant Geologic Characteristics

In the analysis of a high rock slope, structural discontinuities of the rock mass are mapped in detail and each feature is quantitatively characterized. The geologic survey is aimed at measuring a sufficient number of joints to allow the data to be analyzed statistically. The statistical analyses and judgment indicate whether the best estimate has been made for the whole population. The entire geologic survey must be conducted so that, from the joint characteristics recorded, the shear strength in the direction of the joints can be numerically assessed by comparison with similar features tested in the laboratory.

Physical access to all discontinuities in a rock mass is not possible. Therefore, maximum information must be extracted from all locations where access is possible. For other locations, information is obtained by a variety of means including exposure mapping, tunnels, trenches, drilling (core logging), terrestrial photogrammetry, aerial photograph interpretation, and various geophysical methods. In exposure mapping, either some variation of detail line mapping or fracture set mapping is generally best to use.

Line mapping methods are discussed by Jennings (*9.46*), Piteau (*9.87*), and Halstead, Call, and Rippere (*9.30*), and fracture set mapping by Call (*9.14*), Mahtab, Bolstad, an l Kendorski (*9.67*), Da Silveira and others (*9.20*), and Herget (*9.36*).

Sources of errors in joint surveys are discussed by Terzaghi (*9.109*), Robertson (*9.99*), and Piteau (*9.88*).

General collection and processing of geologic data are discussed by Knill (*9.52*), the Canada Department of Energy, Mines and Resources (*9.94*), and the International Society of Rock Mechanics (*9.44*).

Geologic conditions vary from project to project, and thus a geologic survey at one site may be entirely different from that at another. Features that should be considered in the survey are coordinates; elevation; rock type and hardness; type of geologic structure; strike (direction of structure surface) and dip (angle of structure surface); dip and strike continuity; thickness, type, and hardness of infilling materials and the proportion of voids and presence of water in the materials; roughness; and waviness (wave shape or interlimb angle). Coordinates, elevation, strike, and dip are used to define position and orientation of the discontinuity in space. These features, plus dip and strike continuity and bearing of the sample line, serve to define intensity. Rock type and hardness, characteristics of infilling material, presence of voids and water, roughness, and waviness are used for assessing frictional and cohesive strength and deformation properties.

The survey data are processed, and the attitude, geometry, and spatial distribution of the jointing are determined by computer analysis to yield structural domains, joint sets and their average properties, and major discontinuities.

After the regional and local geology is assessed, a geologic map is constructed to show the major and minor structural features and the general lithologic distribution. Stereographic projections are easy to use and are of great benefit in evaluations of geometric relations and in showing structural populations. These projections can also be shown on the general geologic map of the area.

Stereographic projection principles and the use of equal-area, equal-angle, and polar projections are documented by Donn and Shimer (*9.22*), Phillips (*9.86*), Terzaghi (*9.109*), and John (*9.48*).

Determination of Structural Domains and Design Sectors

Because both the geologic conditions and the bearing of the proposed cut face can vary from one location to another along a cut, different slope designs and remedial measures may be required at different locations. Therefore, one should analyze individually those parts of the proposed cut slope that are similar in terms of both orientation and physical and mechanical characteristics. Areas of similar geologic characteristics are designated structural domains. Samples of the geologic structural properties within a structural domain will not differ significantly from one part to another. Therefore, the slope stability characteristics and design parameters of the entire structural domain can be determined from sampling only part of it.

Boundaries of straight slope segments that have similar orientations are determined and superimposed on structural domain boundaries to form design sectors. Characteristics of the various design sectors are selected for stability analysis and slope design. Within each design sector, typical joints or design joints, which represent the mean characteristics of the relevant joint sets, are selected for use in the design calculations (*9.94*). Discussions relating to definition of properties of design joints are given by Steffen and Jennings (*9.106*) and Piteau (*9.88*).

Boundaries of structural domains usually coincide with major geologic features, such as faults, shear zones, dikes, sills, geologic contacts, and unconformities (*9.88*). The analyst's ability and experience in assessing the structural geology and the variations in structural characteristics will determine the accuracy and usefulness of the structural domain determinations. Comparative analyses of joint populations within structural domains or between structural domains should be subsequently performed.

Development of a Rock Mass Model Depicting Geologic Structure

After the characteristics of the geologic structural population in each structural domain are defined, a model of the rock mass is developed to depict the three-dimensional relations of the geologic structure. Some workers refer to the rock mass model as a schematic concept or structural picture of the rock mass. Since each structural domain is similar in a statistical sense, a rock mass model is developed for each structural domain. The model can be of a graphical, physical, or mathematical nature or a combination of these.

An essential requirement of the model is that it accurately represent the actual geologic structural population in a statistical sense and that it apply to the entire design sector. A graphical model using the stereographic projection (*9.48*) is usually used, and extensions are often made to mathematical or physical models to determine whether the first boundaries selected are adequate or should be changed. A typical rock mass model using a stereographic net depicting the angular relations between faults that form a potential wedge failure and a proposed cut slope is shown in Figure 9.6. A spatial diagram of these relations is shown in Figure 9.7.

Once the boundaries of the various structural domains are defined, attention is given to delineating joint sets within each domain and determining the characteristics of each joint

Figure 9.6. Typical graphical model or schematic concept of mass on Wulff stereographic net. Angular relations between faults, which form potential wedge failure, and proposed cut slope are shown.

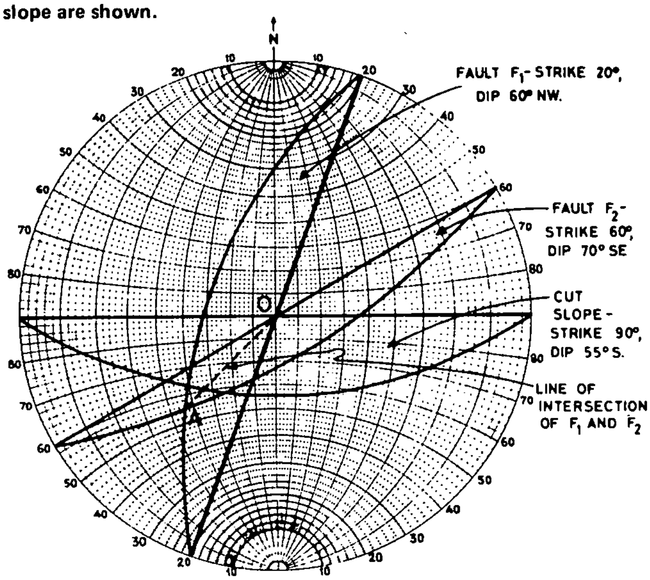
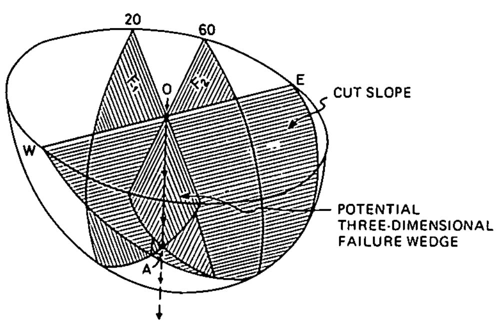


Figure 9.7. Three-dimensional diagram of spatial relations of salient features of rock model shown in Figure 9.6.



set. The discontinuities are analyzed by using some form of graphical projection or plot (e.g., stereographic projection or rectangular plot). The structural data and joint sets are defined on the basis of the specified criteria, and the properties of the joint sets lead to the design joint. The percentage of joints in any particular joint set having any one property is determined by relating the number of joints in the set with that property to the total number of joints in the set. Various statistical techniques can be used for this operation (*9.94*, *9.99*).

Determination of Kinematically Possible Failure Modes and Performance of Slope-Stability Analysis

The assumption is made that the surface of failure in the slope consists of a plane or combination of planes, as discussed earlier. The model is investigated to determine what failure modes on planes or combinations of planes (i.e., design joints) are kinematically possible. Four of the basic failure modes usually investigated are plane, wedge, stepped, and circular. (Discussions of different failure modes are presented in Chapters 2 and 7.) A tension crack for each case can be assumed to exist at the surface. Methods of slope-stability analysis for various failure modes are discussed by Jennings (*9.47*), Hamel (*9.31*), Heuze and Goodman (*9.37*), Hoek and Bray (*9.42*), Hendron (*9.34*), and Goodman (*9.29*).

The analyses consist of the following operations:

1. Estimations of continuity of jointing on potential failure planes;
2. Assessment of the strength of the intact rock;
3. Determination of the effects of the joint characteristics on the strength along joints;
4. Development of the necessary equations of limit equilibrium for the possible modes of failure;
5. Use of various potential failure planes singly or in combination to test for these failure modes; and
6. Determination of the factor of safety of the slope.

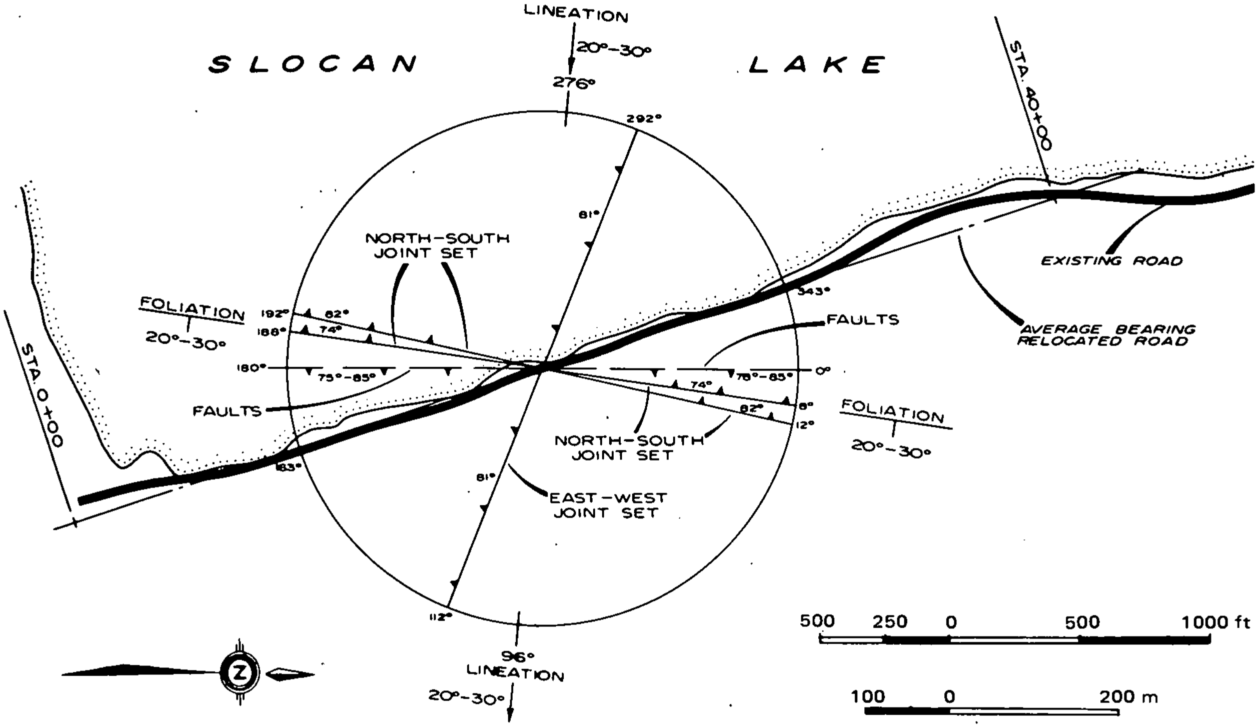
An analytical treatment of the relevant data in the stability analysis is described in Chapter 7 and the brief discussion that follows.

Synthesis of Basic Data

The method of analysis is considerably more detailed and vigorous for high rock cuts than for shallow rock cuts. The following procedures may be used.

1. Coefficients of continuity are determined from the lengths of the joints in relation to the probable length of the potential failure surface in the slope. Of all the assessed factors, these figures for continuity are probably the most difficult to determine and accordingly are subject to some doubt.
2. The effects of waviness are assessed, and an angle of waviness is defined from measurements of the wave shape of the joint; this shape is dependent on the length and amplitude of the wave on the joint plane. The angle of waviness is used to modify the apparent dip of the joint with respect to the direction of the slope.
3. The hardness of the intact rock (assessed in the joint survey) is used with an empirical curve to determine a conservative value of the compressive strength of the rock. Cohesive and tensile strength values of the intact rock are estimated on the basis of the compressive strength, and the friction angle of the intact rock is estimated from the rock type.
4. A factor of safety is then determined based on knowledge of the strength parameters applying to failure along the potential surface of failure and the assumption that the Mohr-Coulomb relations apply to shear failure through the intact rock and along joint surfaces. The analysis for failure is based on the apparent dip of the potential failure surface with respect to the strike of the slope.
5. Water pressures are used in the analysis in the same way that they are normally used in soils, but with the further assumption that apparent cohesion and apparent friction parameters are based on effective stresses (*9.47*).
6. Stability calculations are made to test the different failure modes. For each particular case, the joint sets are ex-

Figure 9.8. Graphical representation of average orientation of main geologic structural features along highway location in Slocan Lake Bluffs area, British Columbia.



amined in their various combinations until the worst situation is found (i.e., the case that gives the minimum factor of safety for the particular slope angle examined). The procedure is repeated for different slope angles, and the respective factor of safety is determined for each case.

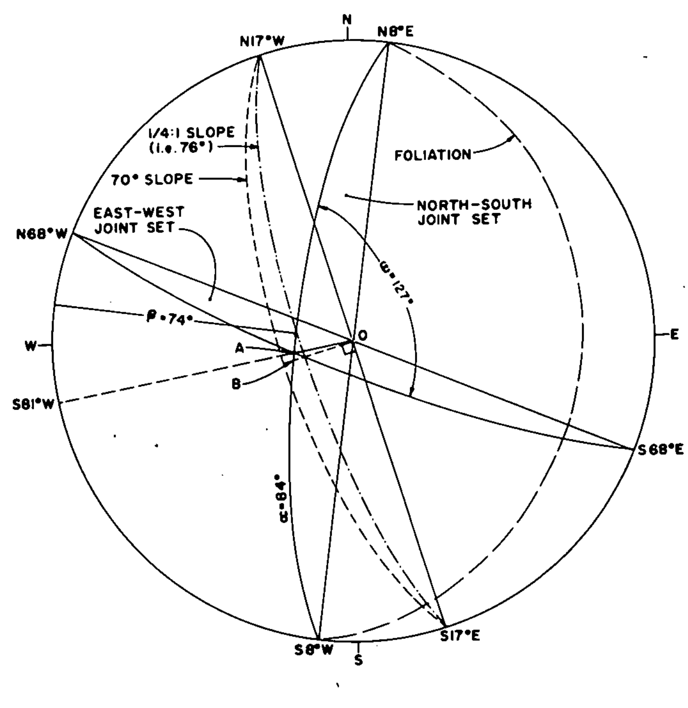
Most rock-slope analysis procedures are not so complicated as these and generally require only an evaluation of the orientation of geologic structure with respect to the bearing and alternative slope angles of the proposed excavation. In such cases the rock mass model can be depicted most suitably by means of stereographic projection techniques shown in Figures 9.6 and 9.7.

Case History of a Typical Rock-Slope Stability Problem

A rock-slope design was required for a proposed highway cut in basically hard gneissic rock along the edge of Slocan Lake, British Columbia. Two dominant steeply dipping joint sets occur in the area: One set is designated the east-west joint set and the other the north-south joint set. Both of these joint sets are shown in Figure 9.1. The orientation of these joint sets with respect to the proposed highway location in plan is shown in Figure 9.8.

The rock mass model in stereographic projection for design is shown in Figure 9.9, including great circles of the average orientation of the east-west joint set and north-south joint set and two alternative cut slopes. One of these slopes is 70°, which is the slope that is to be recommended, and the other slope is 76° (i.e., 1/4:1), which is the slope angle tentatively proposed for preliminary design before the study was initiated. Figure 9.9 shows that the two joint

Figure 9.9. Graphical model in stereographic projection showing angular relations between joint sets, which form potential wedge failure, and proposed cut slope location shown in Figure 9.8.



sets form potential wedge failures that have a plane of intersection trending N81°E and plunging 73° (toward the highway). Since this wedge is flatter than the 76° slope, a 76° slope would clearly be unstable.

A mechanical stability analysis in this case is somewhat redundant because of the steepness of the potential wedges. For illustrative purposes, however, a simple three-dimensional analysis (*9.13*) can be carried out to show the minimum angle of residual joint friction (ϕ_r), which would be required to hold up such a wedge if undercut. Consider the following values determined from the rock mass model shown in Figure 9.9:

$\beta = 74^\circ$, average dip of the north-south joint set,
$\alpha = 84^\circ$, angle measured in the plane of the north-south joint set between the strike of this plane and the line of intersection, and
$\omega = 127^\circ$, angle measured in a plane perpendicular to the line of intersection of the two joint sets.

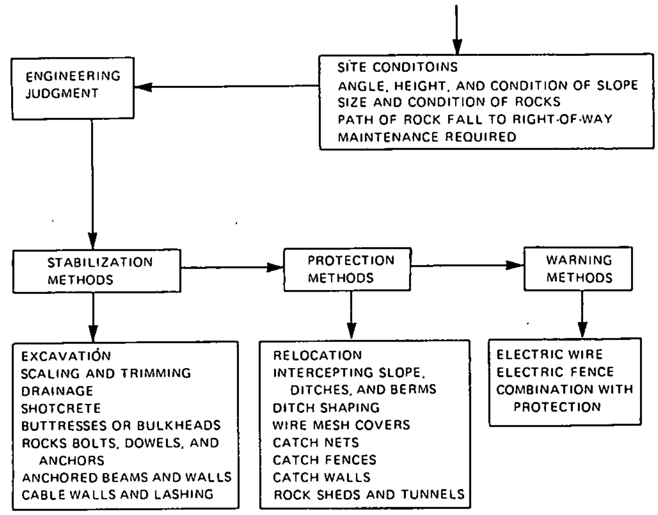
ϕ_r would have to be a minimum of 57.2° in order to achieve a factor of safety of unity. Since the value of ϕ_r was estimated to be on the order of 35°, a 76° slope would obviously lead to serious wedge failures. Cable anchors, bolts, or other artificial reinforcement, which would be required in order to prevent failure of a 76° slope, would also be uneconomical. Hence, it was recommended that the slopes be cut at an angle of 70°, which is 3° flatter than the line of intersection 0A (Figure 9.9) of the potential wedge failures.

SLOPE DESIGN AND REMEDIAL MEASURES

Remedial measures for rock slopes can consist of stabilization, protection, or warning methods or a combination of these basic methods. These remedial measures and the suggested order in which they should be considered are shown in Figure 9.10 and discussed below.

1. Stabilization methods give a positive solution to the problem in that either the driving forces are reduced or the resisting forces are increased. Because of the complex nature of the rock mass, the effectiveness of these methods is often difficult to assess quantitatively. Stabilization measures reduce the likelihood of rocks moving out of place and generally should be considered first in the remedial treatment of rock slopes. Stabilization also reduces the rate of deterioration, a process that leads ultimately to failure.
2. Protection methods prevent rock materials that have moved out of place on the slope from reaching the roadway and thus offer an additional positive solution to slope-stability problems. The initial cost is usually considerably less than that of the stabilization measures, but the slopes usually require considerably more maintenance (in some cases almost continual maintenance). Protection measures can sometimes be combined effectively with warning measures.
3. Warning methods warn that movements have taken place or that failure has occurred and that a hazard may be imminent in the vicinity of the roadway. These methods have no effect on the source of the hazard and may lead to increased maintenance costs and unnecessary delays when insignificant events accidentally trigger the warning system. Warning methods also tend to provide undue confidence, and sometimes result in relaxation of advisable precautions. Although they may seem economical in some instances, warning measures are not an effective way to remedy rock-slope problems and generally should be considered only as a last resort. Except for railways in North America, warning methods are seldom used extensively on a permanent, long-term basis by those experienced in rock-slope engineering.

Figure 9.10. Order in which slope treatment methods should be considered for selection.



During the planning and construction phases of a project, the considerable advantages in recognizing rock-slope problems that exist or that may develop and in applying the appropriate rock-slope remedial measures at the time cannot be overemphasized. Sound rock-slope engineering at the outset of the project and during the excavation phase will obviate both hazards and much more costly work at a later date. The cost of stabilization and protection measures applied during the excavation phase also can be largely absorbed in the initial construction cost; this is especially true if a contingency item is included in the contract budget.

The following discussions do not include costs of remedial measures, other than in a general way, because costs in one environment or location may be entirely different from those in another. Time, access, and space available for remedial work on highway rock slopes, for example, are often much greater than for similar work on narrow, busy railway lines in mountainous terrain. For this reason, both methods and costs of highway rock-slope engineering work are not always applicable to similar railway problems. Unit costs and other items can also vary widely, depending on local requirements and the skill of the planner.

Remedial measures for rock slopes are discussed by Záruba and Mencl (*9.112*), Baker and Marshall (*9.3*), Fookes and Sweeney (*9.26*), Root (*9.100*), Mehra and Natarajan (*9.72*), and Behr and Klengel (*9.7*). Landslide measures in urban developments are discussed by Legget (*9.61*) and Leighton (*9.62*). Crimmins, Samuels, and Monahan (*9.19*) provide practical information with regard to construction in rock although the reference is mainly to foundations.

Planning and Related Procedures

The stability of a rock slope will change significantly with time because of strength changes, rock deterioration, fluctuating groundwater, and other environmental factors, which generally produce less stable slopes. The most economical overall design, therefore, accounts for this decrease in stability with time and includes consideration of a maintenance and remedial program. Remedial measures for rock slopes along transportation corridors in steep terrain must be integrated with other maintenance tasks. If the route runs for some distance through mountainous terrain, work will probably be sufficient to keep personnel on rock-slope maintenance work full time. Unlike most soil slopes, on which weathering is not an important factor, rock slopes require continual attention if serious problems are to be avoided. A consistent long-term remedial program is required that contains carefully selected priorities. Experienced people are needed for both engineering and construction. Judgment must be based on a detailed knowledge of occurrences at each dangerous location.

Personnel

In setting up a rock-slope remedial and maintenance program for a major transportation route, a project engineer should be assigned full time to the work. The project engineer should have available an experienced engineering geologist and geotechnical engineer and the capability to supervise contract work. If necessary, consultants should be used for highly specialized parts of the work and to provide some personnel on a temporary basis, depending on the magnitude and frequency of the problems. Continuity of personnel should be encouraged, for experience in rock-slope engineering is a most important attribute. An engineering geologist on the team should be able to translate information clearly into engineering terms and integrate information with engineering requirements.

The project engineer must keep in touch with all developments relating to the overall project. The geotechnical engineer and engineering geologist should make detailed inspections, perform slope-stability analysis and design, exercise judgment on specific rock conditions, and help in planning and supervising remedial work. It is particularly important that the staff of the highway, railway, or other transportation agency concerned with rock-slope problems develop sufficient expertise within the organization to critically evaluate work performed by private companies or other agencies. Some of the principal functions of the project engineer and engineering geologist in planning, design, and maintenance are described by McCauley (*9.70*).

Work Arrangements

To estimate the cost of remedial measures in advance is difficult, for unforeseen conditions are often revealed as the work proceeds. For this reason, decisions on an overall budget required for the improvement of rock-fall hazards are often based on judgment as well as analysis. All concerned should understand that some flexibility is needed in the total budget approved for an annual program so that adjustments can be made in expenditures required at different locations as well as in contract provisions to deal with changed conditions during the work. Because major revisions often are required during the work, rock work should be done on the basis of proposals or bids invited from contractors known to have up-to-date knowledge of the equipment and techniques required. The proposal or bid should be for a unit price or cost-plus-fixed-fee contract and not for a lump sum. The U.S. National Committee on Tunneling Technology (*9.79*), although addressing mainly underground work, is a good source of information on sound contracting practices in construction engineering and contract law for rock construction work.

Selection of Priorities

The condition of all rock slopes along transportation routes in steep terrain should be thoroughly inspected once a year or more frequently depending on the severity and implications involved. Inspection can be done by (a) a helicopter survey of inaccessible locations and of the overall conditions of the slopes and (b) a ground survey of hazards at specific locations to determine the need for detailed studies. Based on annual surveys and case histories of the slopes, detailed studies and alternative treatments are planned. Factors to be considered in deciding on locations for remedial work are

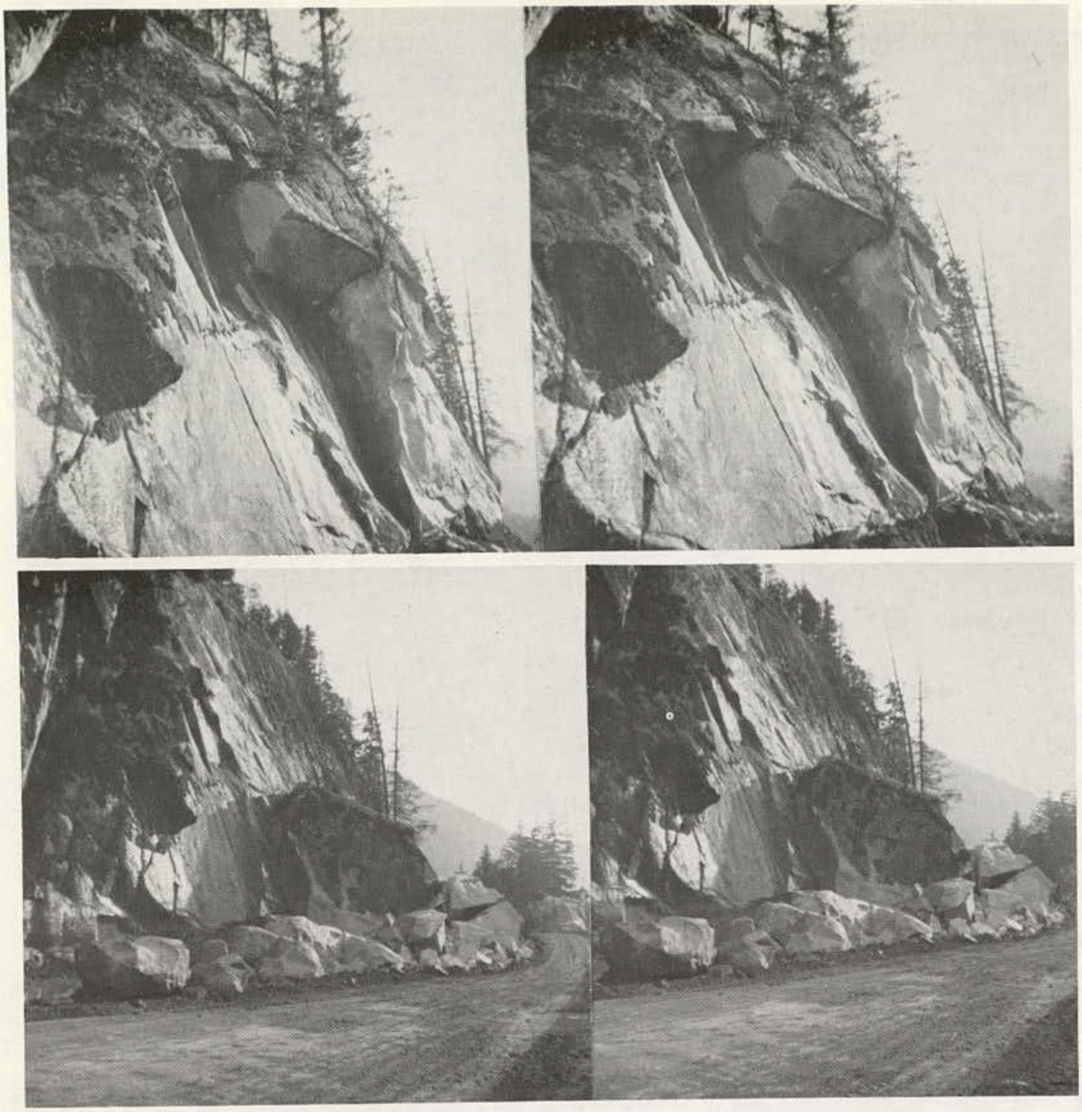
1. Maintenance costs, including patrols required;
2. Costs of remedial measures and expected benefits to be gained;
3. Degree of risk to route, determined by considering the amount of traffic, records of past events, measured rock movements, and frequency of clearing rocks from the vicinity;
4. Occurrence of accidents, rock falls, washouts, landslides, and the like;
5. Conditions downhill from the right-of-way, which would indicate how serious an accident or derailment might be; and
6. Views of maintenance and patrol personnel.

The annual program should emphasize stabilizing or protecting the greatest number of locations where rock-fall hazards exist in order to obtain the maximum economical improvement in safety. Large problem areas can often be divided into smaller areas for treatment. The tendency to overprotect the route at locations of recent accidents should be resisted in the interest of obtaining a balanced overall program. Conditions at each particular location may vary significantly, and the application of different remedial measures should be considered accordingly. Details of remedial work must be developed section by section along the route and be based on detailed inspections, existing records, experience, and sound judgment. However, the most careful choosing of hazardous locations and the most thorough analysis in selecting remedial measures will never entirely prevent unexpected rock falls from occurring.

Records

For the efficient planning of rock-slope engineering work, a data storage and retrieval system should be organized and maintained for permanent reference. The following types of records should be maintained:

Figure 9.11. Rock slope before (top) and after (bottom) removal of overhang.



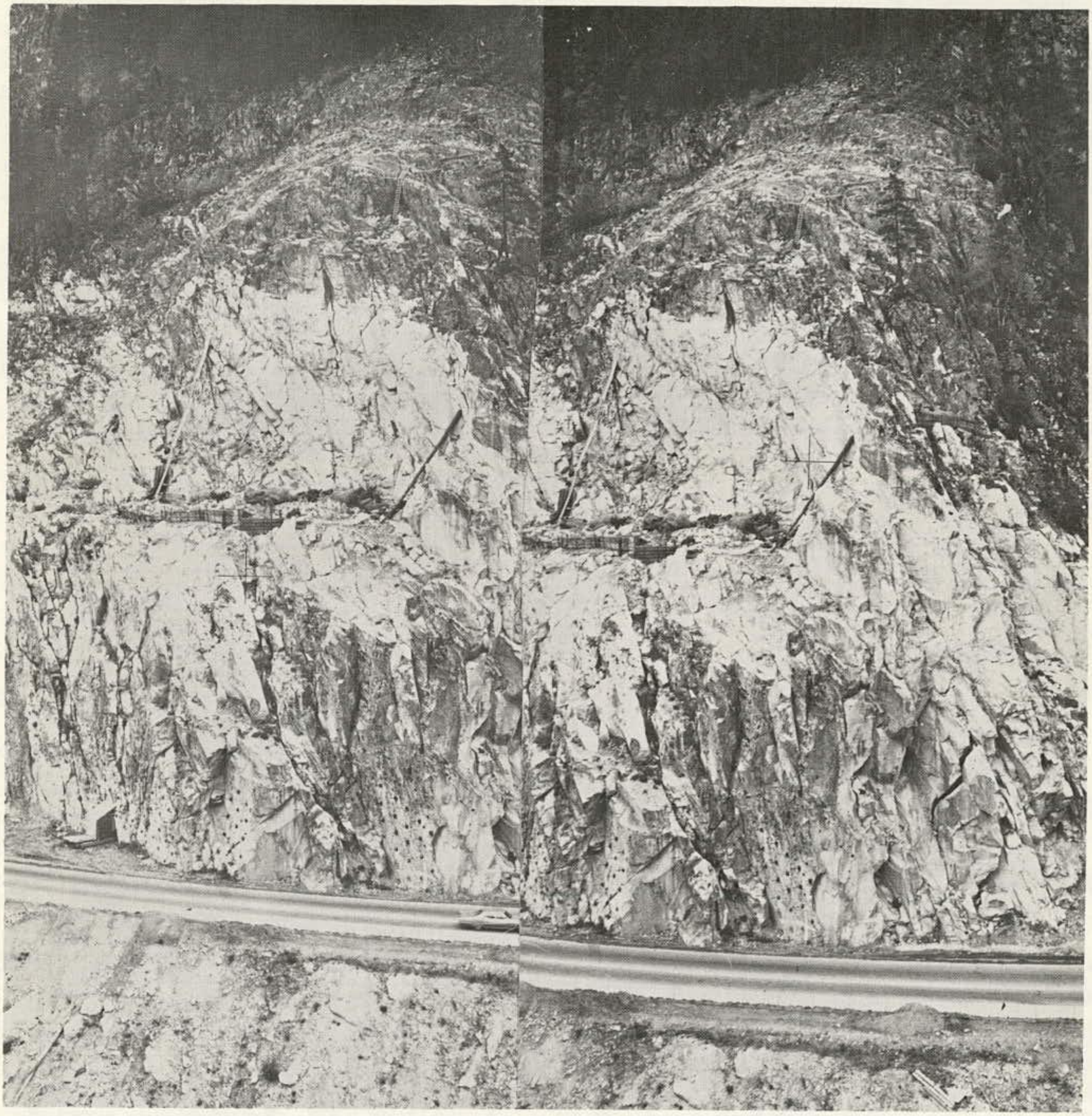
Item	Description
Photographs	Aerial photographs (standard, close-range vertical and oblique stereopairs) and ground photographs
Weather	Daily records of precipitation and temperature from local weather stations and from recorders at selected locations
Traffic delays or highway closure	Time, number of hours, and cause
Rock falls or slides on the right-of-way	Time, location, size of average and largest rocks, volume of material, height and length, source of material, and distance up slope
Removal of fallen material from roadway, ditches, behind walls	Date, location, size of average and largest rocks, volume removed, whether routine or emergency
Stabilization and protection measures	As-constructed plans and continuing inspection records including bolt tensions
Repairs to warning installations	Time, location, maintenance required, effect on installation, size and final location of rocks
Rock movements	From distance-reading and direct-reading instruments
Costs	All remedial and maintenance work done

In addition, a continuing record should be kept of the maintenance performance, the installations, and the measurements conducted at each location requiring attention. For accurate identification, all locations should be recorded to within about 8 m (25 ft).

Photographs

Combined with maintenance records, photographs are essential tools in evaluations of slope conditions and decisions on priorities and types of remedial work required. Photographs should be taken regularly to record the details of rock conditions on slopes both before and after remedial work has been carried out (Figure 9.11). Telephoto lenses can be used for those locations where access is difficult.

Figure 9.12. Oblique photographs of cut bench at Hell's Gate Bluffs, British Columbia, above which excavation was proposed to remedy overturning failure.



Aerial photographs can be useful for studying the condition and configuration of slopes in steep terrain and for giving important clues to the causes and potential sources of slope instability. High-level photographs are generally of minimal use for local slope-design purposes. If possible, low-level photographs at a scale of 1 cm equals 50 to 120 m (1 in equals 400 to 1000 ft) should be obtained. Uses of aerial photographs for engineering purposes are described in Chapter 3 and by Norman (*9.81*) and Mollard (*9.74*).

Aerial oblique or terrestrial oblique single photographs or stereoscopic photographs are frequently used for rock-slope engineering work (Figure 9.11). Stereoscopic photographs can be used to examine slopes three-dimensionally for purposes of tentative slope-excavation design. Aerial oblique photographs can be taken with cameras mounted on the ends of a 5-m (15-ft) boom carried by a helicopter. Photographs can be taken at any angle from vertical to horizontal and contours can be plotted to a fixed datum. Cracks and features on the order of 5 to 8 cm (2 to 3 in) wide can be detected in this way. Photographs are particularly useful in evaluations of rock slopes that are in steep terrain and not easily accessible from the ground (Figure 9.12). Use of terrestrial photogrammetry in rock-slope engineering is discussed by Ross-Brown and Atkinson (*9.101*).

Monitoring and Inspection

Planning for rock-slope engineering work should include a program of monitoring slope movements and identifying characteristics that indicate changing stability. Detection of general creep or slow translation of highly fractured or soft slope-forming material is important in decisions regarding remedial work. Horizontal and vertical movements of points plotted against time or depth or both provide important information concerning the behavior of a rock slope. Graphs provide a clear indication of the onset of slope failure when plots representing the change in position do not remain linear. When such accelerated movements are evident, the slope must be approaching failure, and measures should be taken to analyze and remedy the situation.

For studies of complex stability problems, the advice of a specialist is needed to determine the most suitable slope-monitoring program. A wide variety of commercial instruments is available for measuring both surface and subsurface movements and for recognizing the characteristics indicative of potential instability. However, the simplest methods and mechanical equipment are not only the most practical but also invariably the most reliable. The various methods of instrumentation are discussed in Chapter 5, in the June 1972 issue of Highway Focus (*9.39*), and by Franklin and Denton (*9.28*), Benson (*9.8*), and Hedley (*9.33*).

The simplest and generally most efficient method of monitoring recently excavated slopes is to measure both horizontal and vertical displacements of protected metal survey pins or hubs set in concrete or grout along a straight line at the crest and toe of the slope. Main control reference points should be located outside the area where movement may occur. If, in the first year, movements appear negligible (other than what might be expected from normal elastic rebound due to unloading), the number of surveys in succeeding years may be decreased until the slope becomes stable. In this work, high precision is essential so that the onset of nonlinear movement can be determined as early as possible and remedial work can be planned accordingly.

Individual cracks, which may be indicative of general instability in an area, can be monitored simply by bridging the crack with a wad of grout or shotcrete or by inserting a wedge in the crack. A regular part of the work program for evaluating and monitoring should involve inspection and maintenance of remedial installations, such as rock bolts, walls, nets, and drains, already in place. Regular inspection is an essential precaution in protecting a substantial capital investment.

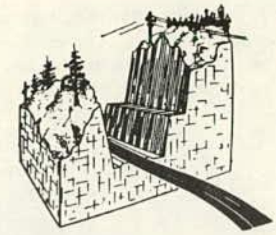
Methods of Stabilization

Excavation and Related Design Aspects

For purposes of improving the stability of rock slopes, excavation is used either to reduce the driving forces contributing to failure or to remove unstable or potentially unstable sections of the slope that may lead to failure (i.e., rock falls and slides). Stability, therefore, can be achieved by

1. Removing unstable or potentially unstable material,
2. Flattening the slope,
3. Removing weight from the upper part of the slope,

Figure 9.13. Forty-three-meter (140-ft) presheared through cut involving 130 000 m³ (170 000 yd³) near Britannia, British Columbia, and including cut bench to protect highway from rock falls. Photo is toward south, but sketch is toward north.



4. Incorporating benches in the slope, and
5. Excavating in a manner that minimizes damage to the rock mass.

Excavations made for remedial purposes should provide a permanent solution to the slope-stability problem so that additional excavation work in the future need not be necessary. The principal problems and disadvantages associated with excavation methods lie in their cost. Accessibility after construction may be difficult and, since the slope usually must be excavated from the top downward, mobilization and setup costs can be prohibitive. Also, disposal sites often are limited with the result that waste rock may have to be transported for some distance, unless it can be used for local construction.

Benches

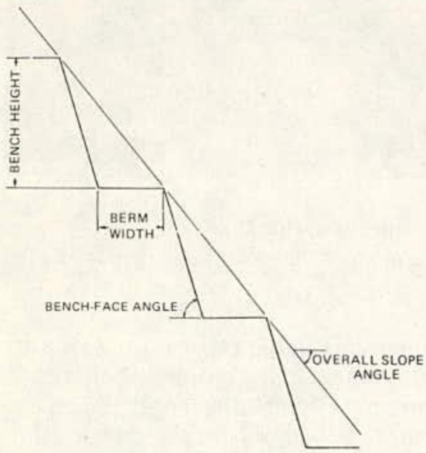
The overall slope angle and different slope angles that may be designed for different parts of the slope are determined by slope-stability analysis methods (Chapter 7). Each cut should be designed to suit the prevailing rock conditions. Slopes containing materials with strength properties that vary significantly in section may require that some parts be flatter than other parts if rock falls and maintenance costs are to be minimized. The use of variable slopes, benches below layers of rapidly weathering rock, and structural support and correct blasting techniques can practically eliminate rock falls in new cuts.

Benches are used in slopes to minimize rock falls onto the roadway (Figures 9.13 and 9.14). For soft rocks (Figure 9.14), such as shale, mudstone, and other argillaceous rocks, benches tend to reduce excessive weathering and erosion and provide rock-fall catchments. Erosion due to groundwater runoff is also controlled since the energy of surface flows is

Figure 9.14. Benches that follow stratigraphy in Niagara Escarpment sedimentary rocks on Highway 403 near Hamilton, Ontario. Design also includes overburden being stripped back 3 m (10 ft) from top of rock and light fence barrier on edge of highway.

Figure 9.16. Use of multiple benches and controlled blasting techniques at Mica Dam, British Columbia, to maintain rock slope in soundest possible condition and to prevent rock falls.

Figure 9.15. Slope design parameters that are used when benches are incorporated in slopes.



dissipated. Construction safety on benched slopes is usually increased since the hazard of rock falls above workers is reduced. In general, the faces of benches can be considerably steeper than the overall slope angle; hence, any rocks that do fall remain on the benches.

Benches appear to have no effect on the slope with respect to deep-seated failure. Although shear stresses increase with slope height, the direction of the maximum shear stress is supposedly independent of slope height and bench geometry. Benches may require the top of the slope to be moved back and, therefore, considerable additional excavation. However, although they may add to the initial cost of construction, benches may significantly reduce subsequent maintenance costs and thus sometimes more than offset the increased cost of construction. The Colorado Department of Highways, for example, found that the unit cost of cleanup and maintenance work after rock falls is about ten times the unit cost of the original excavation.

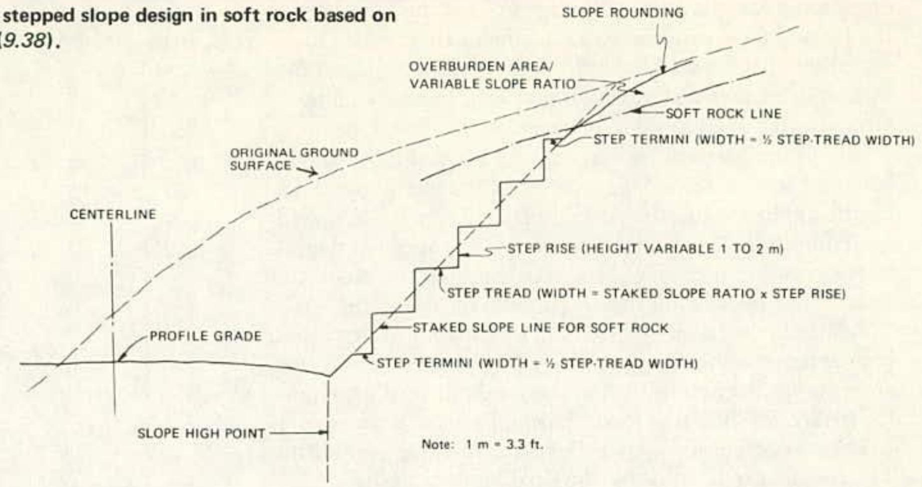
Figure 9.15 shows the parameters of a slope incorporating benches, including bench height, berm width, and bench-face angle. These parameters are governed by the physical and mechanical characteristics of the rock mass. Bench height should provide a safe and efficient slope and an optimum overall slope angle. Bench height can be greater in stronger rock, and the bench face can be terminated at the base of weaker horizons and water-bearing zones. Without affecting the overall slope angle, higher benches generally will allow for wider berms, giving better protection and more reliable and easier access for the regular cleaning of debris. The width of berms should be governed by the size of the equipment working on the bench and by the nature of the slope-forming material, but should generally be no less than 7 m (20 ft).

If the bench faces are inclined, high tensile stresses are less likely to develop near bench crests and thus tension cracks and overhangs are minimized. Avoiding these problems reduces the amount of rock-fall material and increases the safety of the slope. Tension zones in slopes involving benches are discussed by Bukovansky and Piercy (*9.11*). Design of the bench-face angle should be governed to a large extent by the attitude of unfavorable structures in the slope to prevent excessive rock falls onto the berms. Figure 9.16 shows a high rock cut at Mica Dam in British Columbia, where all three bench design parameters change in different parts of the slope.

If benches are included in the slope design, berms should be equipped with drainage ditches to intercept surface runoff and water from drain holes and other drainage facilities and divert it off the slope and away from problem areas. These ditches should be kept open and free of all debris and ice to ensure adequate performance. Ditch lining (e.g., clay, slush grout, asphalt, polyethylene sheeting) may be required if ditch leakages are anticipated or develop afterward. Berm surfaces should be graded to assist the collection of water in ditches and also to facilitate general drainage in a direction away from potential areas of instability. Care must be taken so that ditches do not create problems by channeling water from one area to another.

Stepped benches may be used on slopes cut in highly weathered rock material to control erosion and to establish vegetation. Figure 9.17 (*9.38*) shows stepped cut slopes that consist of 0.6 to 1.2-m (2 to 4-ft) high benches with approximately similar berm width and an overall slope angle based on stability analysis. The design objective is that the material weathering from each rise will fill up the step of the bench and finally create a practically uniform overall slope. The steps are constructed horizontally to avoid the longitudinal movement of water, which could cause considerable

Figure 9.17. Idealized cross section showing stepped slope design in soft rock based on recommendations in several research papers (*9.38*).



erosion. Seeding and mulching or other suitable methods of slope stabilization can be readily applied; however, for rapidly raveling slopes, about half of the bench or step width should be filled before seeding is done to prevent smothering of the seed.

Scaling and Trimming

Scaling of loose, overhanging, or protruding blocks is a basic maintenance operation on rock slopes of all sizes along transportation routes in steep terrain. Scaling on the upper reaches of high faces is usually carried out by workers on ropes with hand pry bars, hydraulic splitters or jacks, and explosives (Figure 9.18). Of necessity, this work is slow and intermittent. Mechanical scaling equipment is more efficient and safer, but may have limitations because of severe access problems. Trimming involves drilling, blasting, and scaling to remove small ragged or protruding rock in overhang areas where repetitive scaling would otherwise be required.

Postconstruction scaling and trimming should be carried out on a regular basis. In temperate regions, this is usually started in the spring after the frost leaves the rock. If thorough scaling is performed during excavation, subsequent maintenance and remedial work can be greatly reduced. Before work begins, an engineering geologist and rock foreman should thoroughly inspect each location and make decisions on the rocks to be removed. Scaling and trimming work requires specialized experience, and the performances of different contractors should be compared when additional work is considered. Depending on site conditions, potentially useful tools are bencher drills, gas jackhammers, air-operated scaling tools, suspended powered platforms (spiders), and hydraulic boom cranes (giraffes) for access to low and intermediate slopes. Equipment for scaling and other related activities in lower and higher reaches of slopes is shown in Figures 9.19 and 9.23 respectively.

Blasting Procedures

Rock should be preserved beyond excavation lines and grades in the soundest possible condition. The effects of tension, compression, and shear stresses developed in a rock mass as a result of blasting damage are documented by Bauer (*9.6*), Lang and Favreau (*9.57*), Langefors and Kihlstrom (*9.59*), the U.S. Bureau of Public Roads (*9.12*), Larocque (*9.60*), and Lambooy and Espley-Jones (*9.55*). Uncontrolled blasting results in rough uneven contours, overbreak, overhangs, excessive shattering, and extensive tension cracks in the crest of the slope. Blasting damage, therefore, can lead to significantly higher scaling, excavation, remedial treatment, and maintenance costs. The results of blast shock waves and gases along faults, joints, bed-

Figure 9.18. Typical rock-scaling operation high above highway.

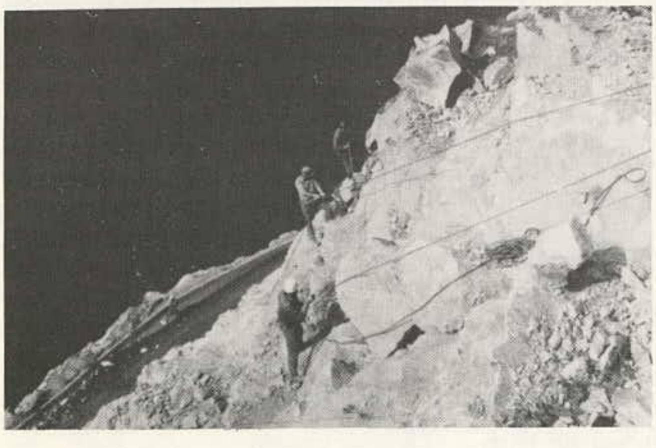


Figure 9.19. Typical hydraulic crane (giraffe) and basket used for scaling lower reaches of slopes.

ding, and discontinuities, although not readily apparent on the blasted face, can lead to loosening of the rock. This sometimes occurs well behind the face, allowing easier infiltration of surface water, which may lead to unfavorable groundwater pressures and unnecessary frost action.

Blast-hole patterns and powder loads must be properly balanced so that advantage is taken of the energy released by the explosive and the desired blast effects are obtained with minimum damage to the rock. Control of the degree of fragmentation can also facilitate handling the muck and ensure that the blasted rock is suitable for use as fill. Although guidelines can be specified, the blasting design should be based on practical experience with the rock in question and can best be determined in the field. Special provisions for trial blasts should be made, particularly for large projects, to ensure optimum results for existing operating conditions. The engineer must approve the final blasting design.

On projects involving blasting, claims are commonly submitted by nearby property owners for damages allegedly caused by undesirable detonation by-products such as fly rock, air concussion, and vibration. Evidence is required to assess the validity of claims for blasting damage. A concise outline of the current status for the law concerning damages resulting from blasting (also from slides, runoff, and drainage) is given by Lewis and others (*9.63*). The project engineer must apply specialized knowledge and expertise on the project to reduce the possibility of valid damage claims and to negate any invalid claims.

In heavily populated areas, fly rock and concussion can be controlled by common sense and good blasting practice. Production blast holes should be adequately stemmed, sufficient collar should be left, and overloading should be avoided. The exposed area to be blasted should be covered with blasting mats or some other suitable blanketing material. Blast vibration measuring equipment is required, however, to assess the potential damage due to ground vibration.

According to McAnuff (*9.69*):

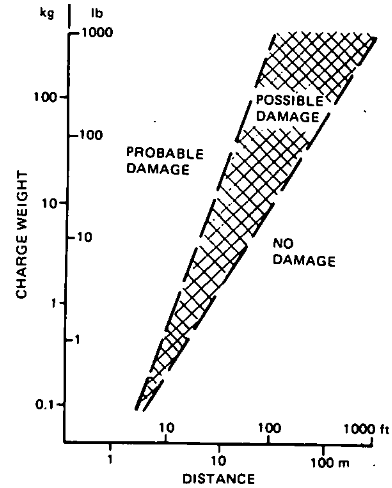
The particle velocity of earthborne vibration is now generally accepted to be the best measure of damage potential. . . . A particle velocity between 2.8 and 3.2 inches per second is required to reopen or extend old plaster cracks. . . . A peak particle velocity of 2.0 inches per second is safe with regard to plaster cracks. . . . Ground motion particle velocities below 4.5 inches per second are well within the safe range for most engineered structures.

In Figure 9.20, Northwood and Crawford (*9.82*) show a direct relation between charge and distance and the probability of damage. They indicate that "a simple relationship defining a conservative safe limit is $E^{2/3} = d/10$, where E is the weight of a single charge in pounds and d is the distance in feet."

Before blasting is carried out in populated areas, Ehrlich, Scharon, and Mateker (*9.25*) recommend as general practice the following procedures:

In addition to the usual examination and description of all structures within a reasonable distance of the blast area in advance of blasting, an effort should be made to inform the public about such things as the characteristics of blasting vibrations, the differences in structural damage caused by blasting from that produced by settling phenomena, the response of loose objects, such as wall pictures or mirrors and shelf knick-knacks to vibration, and the human response to vibration. This can be achieved by distributing literature, showing films, conducting lectures at civic meetings, and broadcasting on radio and television.

Figure 9.20. Probability of damage versus charge and distance (*9.82*).



Ehrlich and others also suggest carrying out a reasonable study of the area to determine the nature and extent of the overburden for purposes of predicting where blast vibrations may be critical and designing the blast accordingly. They also recommend that a "contractor should detonate only at predetermined times and alert the surrounding public by blowing horns or whistles."

Excavation Lifts and Related Procedures

For quality control, each lift of rock excavation generally should not exceed about 10 m (30 ft) in height. Benches this high or lower generally prove to be the best for achieving effective scaling and rock bolting. Also, the accuracy of drilling and, hence, the quality of controlled blasting tend to decrease with increased height of excavation lifts.

Rock bolting, scaling, and similar work, if possible, should be carried out as each successive lift is excavated and completed: This will ensure that the slope above the working area is safe. Careful supervision by qualified personnel is essential to minimize both excavation and future maintenance costs and to maintain the safety of the working area. Unfavorable slope stability, groundwater, and other conditions, though not necessarily of major proportions, may arise during the excavation phase. These should be recognized by the site engineer and appropriate remedial measures provided.

When heavy equipment, such as hydraulic backhoes and tractors with rippers, is used in areas of soft, weathered, or highly broken rock, care should be taken to avoid loosening the final cut face of the slope by equipment operations. Procedures such as line drilling and careful backhoe cutting with special tools are generally advisable.

Surface and Subsurface Drainage

Methods that can be applied to improve either the surface or subsurface drainage conditions and, hence, increase the stability of the slope should be given high priority in the proposed work. Drainage measures, as compared with other possible measures, frequently result in substantial benefits at significantly lower cost. Often large failures, involving several thousand cubic meters of material, cannot be controlled within practical limits by any means other than some form of drainage. The application of surface and subsurface drainage as part of the general slope design or for stabilization purposes should be considered at the outset of the project because substantial benefits may result at relatively low costs.

Surface Drainage Control

Adequate surface drainage facilities, particularly if the rocks are relatively soft or susceptible to erosion, can substantially improve the stability of a slope where unfavorable groundwater conditions exist. Areas behind the upper portions of unstable slopes should be thoroughly inspected to determine whether surface water is flowing toward unstable areas or into the ground or both. The following methods have been used successfully to control surface drainage (*9.40*, *9.41*):

1. Drain sag ponds, water-filled depressions, and kettles that occur above the working area from which water could seep into unstable zones;
2. Reshape the surface of the area to provide controlled flow and surface runoff;
3. Above the crest of the slope, use concrete, slush grout, asphalt, or polyethylene (Figure 9.21) to temporarily or permanently seal or plug tension cracks and other obviously highly permeable areas that appear to provide avenues for excessive water infiltration (sealing cracks will also prevent frost action in the cracks);
4. Provide lined (e.g., paved, slush-grouted) or unlined surface ditches, culverts, surface drains, flumes, or conduits to divert undesirable surface flows into nonproblem areas; and
5. Minimize removal of vegetative cover and establish vegetative growth.

Drain Holes

Methods for subsurface drainage of slopes include drain holes, pumped wells, drainage galleries, shafts, and trenches. Noteworthy discussions concerning this subject have been presented by Záruba and Mencl (*9.112*), Baker and Marshall (*9.3*), Hoek and Bray (*9.42*), and Cedergren (*9.15*). Only under special circumstances are subsurface drainage methods other than subhorizontal drain holes used for highway and railway cut slopes; therefore, only a discussion of drain holes is given here.

The purpose of subsurface drainage facilities is to lower the water table and, hence, the water pressure to a level below that of potential failure surfaces. A practical approach that appears to be best suited for most rock-slope problems encountered in highway and railway cuts is to incorporate a system of drain holes to depress the water level below the zones in which failure would theoretically take place in the slope. The drain holes should be designed to extend behind the critical failure zone. Determining the drain-hole design with respect to the geometry of the potential failure mode

Figure 9.21. Temporary cover of polyethylene sandwiched between two layers of mesh to prevent precipitation infiltration into slope at Hell's Gate Bluffs, British Columbia.

provides a reasonable guideline for the preliminary design of the drain-hole system. If for design purposes one assumes a circular failure mode, for example, a reasonably conservative drain-hole design results. In this case, the length of the drain holes is about half the height of the immediate slope needing removal of water. The direction of the drain holes may depend to a large degree on the orientation of the significant discontinuities. The optimum drain-hole design is to intersect the maximum number of significant discontinuities for each meter of hole drilled.

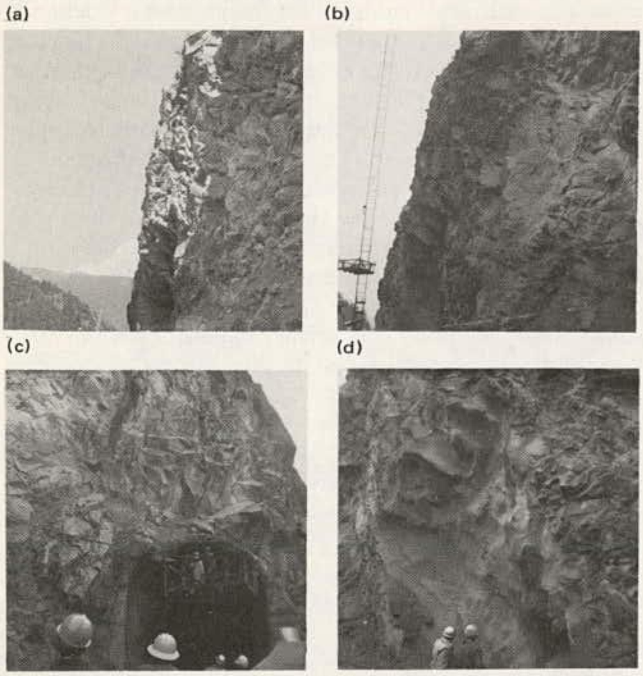
The effectiveness of drains depends on the size, permeability, transmissibility, and orientation of the discontinuities. A drain does not have to produce any noticeable flow of water to be effective; it may have flow only under extreme conditions. Furthermore, the absence of damp spots on the rock face does not necessarily mean that unfavorable groundwater conditions do not exist. Groundwater may evaporate before it becomes readily apparent on the face, particularly in dry climates.

Drain holes usually are inclined upward from the horizontal about 5°. In erodible materials, however, the holes may have to be inclined slightly downward to prevent erosion at the drain-hole outlet due to water flowing out of the drain hole. In this case, a small pipe can be left in the mouth of the drain hole to retard erosion. Spacing of drain holes can range from 7 to 30 m (20 to 100 ft), but 10 to 15-m (30 to 50-ft) spacing generally is used. For high rock cuts, installing drain holes at different levels on the slope may be advantageous to increase the effectiveness of the overall drainage system. For certain conditions on a slope, a series of drain holes may best be installed in a fan pattern so that drill machine setup and moving time is minimized.

Drain holes should be thoroughly cleaned of drill cuttings, mud, clay, and other materials; drain holes not properly cleaned may have their effectiveness reduced by 75 percent. High-pressure air, water, and in some instances a detergent should be used to clean drain holes. In highly fractured ground, care should be taken to ensure that caving does not block drain holes. If caving is significant, perforated linings should be installed so that drain holes remain open. If freezing conditions exist, drain-hole outlets should be protected from ice buildup that could cause blockage. Insulating ma-

Figure 9.22. Shotcrete applied to steep high slope that is prone to minor slides and rock falls.

Figure 9.23. Shotcreting dangerous overhang in extremely steep terrain along tracks of Canadian National Railways in Fraser Canyon, British Columbia: (a) steep highly fractured rock face with serious overhang; (b) shotcrete equipment and buildup of shotcrete at top of picture and under overhang; (c) start of shotcreting operation at tunnel portal; and (d) close-up of shotcrete buildup in overhang shown above.

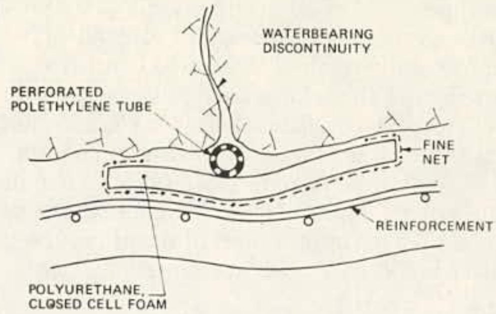


terials, such as straw, sawdust, gravel, or crushed rock, have been used for this purpose. Electric current has also been used to keep the drain-hole pipe warm enough to prevent ice buildup.

Shotcrete

Shotcrete is a concrete that consists of mortar with aggregate as large as 2 cm (¾ in) in size and that is projected by air jet directly onto the surface to be treated. It is one of the basic methods for treating unstable sections of rock slopes. It is used to prevent weathering and spalling of rock surfaces and to provide surface reinforcement between blocks. The force of the jet compacts the mortar in place. Shotcrete is usually applied in 8 to 10-cm (3 to 4-in) layers, and each layer is allowed to set before successive layers are applied. Shotcrete on rock slopes generally appears to have

Figure 9.24. Example of drainage behind shotcrete (*9.1*).



replaced gunite, a similar material that contains smaller aggregate. Rock slopes should be thoroughly scaled to provide the soundest rock condition before shotcrete is applied. An extensive shotcrete application in an area prone to slides and rockfalls is shown in Figure 9.22. Figure 9.23 shows the use of shotcrete for an overhang in extremely steep terrain.

Specifications and discussions of the application of shotcrete are given by the American Concrete Institute (*9.2*). When shotcrete is applied to an irregular rock surface, the resulting surface configuration is smoother. The shotcrete helps to maintain the adjacent rock blocks in place by means of its bond to the rock and its initial shear and tensile strength acting as a membrane. The result is that a composite rock-shotcrete structure is developed on the surface of the rock. There is no transfer of load from the rock mass to the shotcrete. In that the interlocking quality of the surface blocks is improved (*9.10*), shotcrete acts as reinforcement and not as support. The more quickly the shotcrete is applied after excavation, the more effective the results are.

Deterioration of shotcrete can result from frost action, groundwater seepage, or rock spalling due to lack of shotcrete bond. Therefore, unfavorable groundwater flows should be drained for long-term stability of the shotcrete cover. Weep holes should be drilled or installed through the hardened shotcrete and into the rock to prevent building of water pressure behind the shotcrete. In Norway and Sweden, semirounded plastic pipes are glued to the rock surface to form surface drainage channels. Small volumes of water are protected against freezing by rock wool, plastic foil, or even heating cables. More universally used are short flexible plastic pipes, which are placed in cracks or holes drilled into water-bearing broken rock; an example of this technique, as described by Alberts (*9.1*), is shown in Figure 9.24.

Initially dry rock surfaces are preferred in the shotcrete process, although careful control of setting admixtures and nozzle water can give successful applications on wet surfaces. Where alteration products, such as clay or mud, exist on joint or fault planes, care should be taken to clean such surfaces by air or water jet to ensure a good bond between shotcrete and rock. As a rule of thumb, weak material should be removed to a depth at least equal to the width of the weak zone before shotcrete is applied. These areas are where the shotcrete will do the most good and where extra attention will be required in its application.

Shotcrete can be used in combination with steel wire mesh and rock bolts to give structural support and also to form buttresses for small loads. Where shotcrete is applied to mesh, all loose material should be removed from the rock surface and the mesh fabric tightened. Shotcrete can also be used behind anchor beams to provide a uniform contact with the rough or uneven rock surface and, if applied across cracks, may provide a simple means of indicating where movement is occurring. An extensive application of steel fibrous shotcrete to stabilize potential rock falls in basaltic rocks along a railway is described by Kaden (*9.49*).

The most important advantage of shotcrete in treating rock slopes is that it offers a rapid, mechanized, and often uncomplicated solution to rock-fall problems. Various other materials, such as polymers, fiber glass, epoxies, plastic, and rubberized and asphalt compounds, have been either tried or considered to protect rock slopes from the effects of climate and infiltration of groundwater. However, none of these has yet proved entirely successful on a general basis. Polyvinyl chloride, butyl, and neoprene sheeting have been used with limited success in isolated cases, and spray-applied rubberized bitumen coatings have been used with success in retarding slaking of shales (*9.50*).

Figure 9.25. Cast-in-place reinforced concrete buttress approximately 15 m (50 ft) high providing stabilization protection of large overhanging slope.

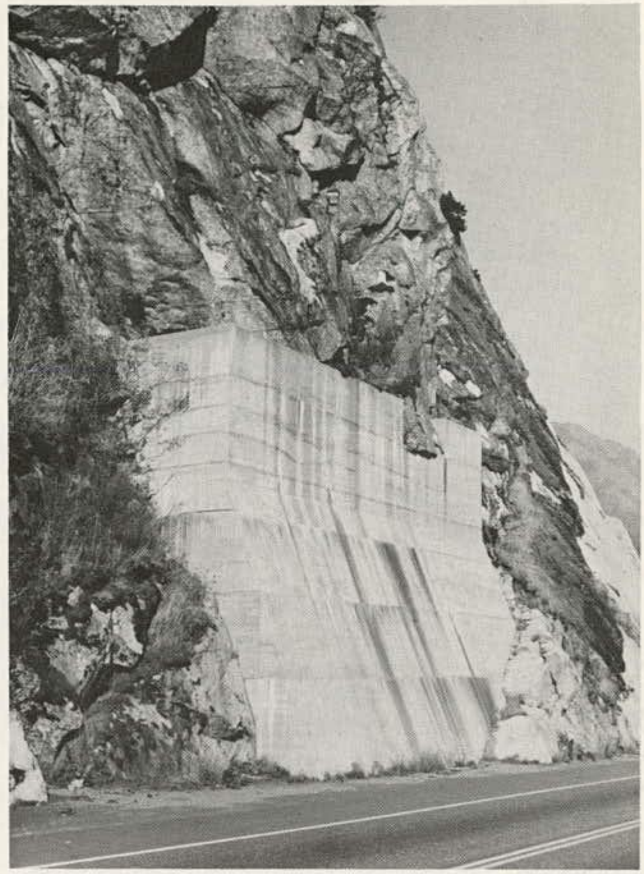


Figure 9.26. Cast-in-place reinforced concrete buttress approximately 15 m (50 ft) high providing stabilization support against possible movement of large block above overhang (note unfavorable discontinuities in the slope).

Support and Reinforcement Systems

Buttresses, bulkheads, and retaining walls are classified as external support systems in that they offer passive resistance to loads imposed by the slope-forming materials that undergo deformation in stages of slope failure and general elastic rebound. Rock bolts, rock anchors, anchored beams, anchored cable nets, and cable lashing are classified as reinforcement systems because they add strength to the rock mass by increasing the general tensile strength and by improving its resistance to shear along discontinuities.

Buttresses and Bulkheads

Buttresses, bulkheads, and other support structures are used for stabilization where failure of overhangs appears to be imminent or where slight cracking or vertical displacement appears to be occurring. Buttresses are designed to take part of the weight of the slope, thus inducing stable conditions and preventing rock falls.

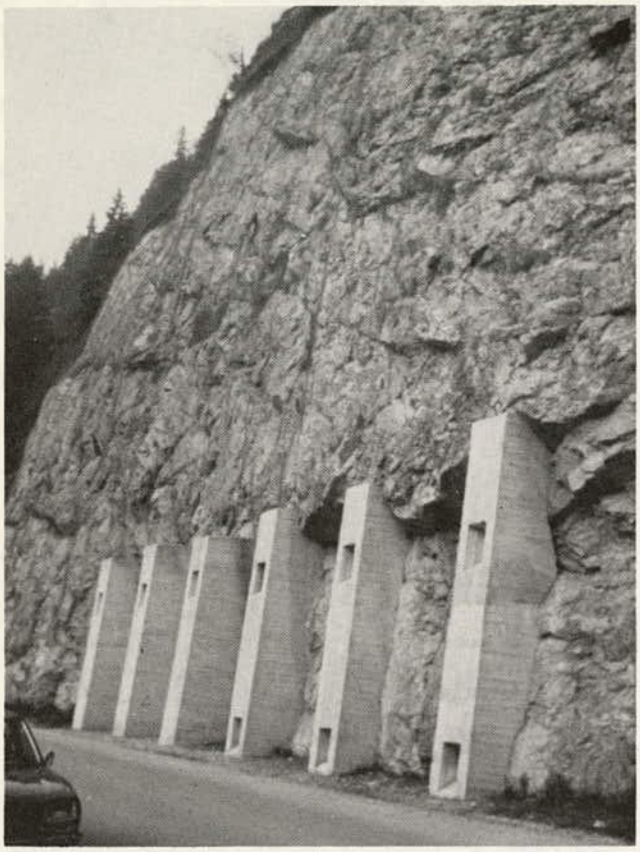
Buttresses, although often costly, are simple, effective, and permanent. They are most effective where overhangs have developed and where excavation to remove the overhangs would be costly because of quantities involved or problems of accessibility. Buttresses are usually constructed at highway or railway level, but they can be just as effective in the upper reaches of the slope. In steep terrain, especially where considerable lateral river erosion has taken place, buttresses or bulkheads are particularly useful. Typical rock buttresses that have been used in such areas are shown in Figures 9.25 and 9.26. Buttresses combined with rock anchors are shown in Figure 9.27.

Retaining Walls

For purposes of rock-slope engineering, retaining walls are used to prevent large blocks in the slope from failing and to control or correct failures by increasing the resistance to slope movement. Retaining walls have the advantage of lessening weathering of the rock slope and of, thus, offering permanent protection. Basic design and use of retaining walls are documented by Peck, Hanson, and Thornburn (*9.83*).

The space along railways and highways is often too narrow for normal gravity or cantilever types of walls, but anchors or rock-bolt tiebacks may be used to overcome this problem. Tied-back walls need only have the strength required for bending and shear resistance between rock bolts. The use

Figure 9.27. Cast-in-place reinforced concrete pillar buttresses supporting overhang on road in Austria. Each buttress has two 9-m (30-ft) long, 356-kN (40-tf) capacity rock bolts.



of rock bolts in combination with a concrete facing wall as a gravity section to stabilize a vertical rock face in sedimentary rock is described by Redlinger and Dodson (*9.96*).

Various types of free-standing and tieback retaining walls include solid cast-in-place concrete walls (*9.35*), prefabricated concrete slabs with anchored waling, vertical concrete ribs to hold precast concrete panels (*9.53*), and steel sheet piling with anchored waling. Figure 9.28 shows an interesting application of a tieback retaining wall formed of galvanized steel members for protection on a high sedimentary rock cut requiring face protection. Anchor bolts were grouted in place at 1.5-m (5-ft) vertical intervals to a minimum depth in the rock of 1.5 m (5 ft). Vertical U-channels of 8-gauge galvanized steel were then mounted on the bolts at 3-m (10-ft) intervals, and horizontal lagging was placed between them. The void behind the wall was backfilled with free-draining material; the water was collected and conducted to a storm sewer.

Steel Reinforcement

Steel reinforcement such as rock bolts and rock anchors reinforce or tie together the rock mass so that the stability of a rock cut or slope is maintained. Rock bolts are commonly used to reinforce the surface or near-surface rock of the excavation or natural slope, and rock anchors are used for supporting large masses of unstable rock. Short reinforcing bars fully grouted into the rock mass are commonly called dowels. Their action, however, is somewhat similar to fully grouted, untensioned rock bolts.

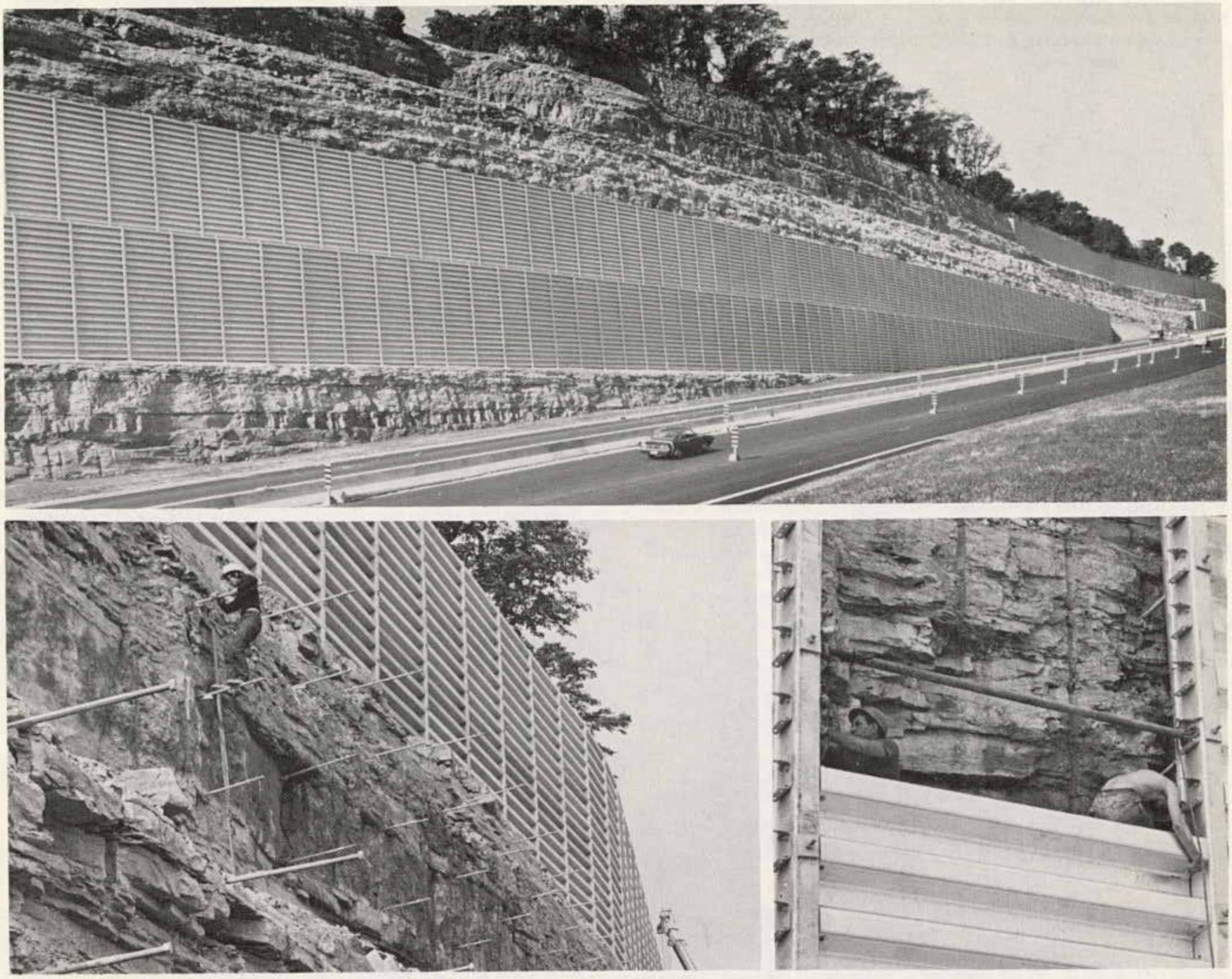
Steel reinforcement of the rock mass, whether considered as rock bolts or rock anchors, has been carried out with many types of bars, bar bundles, cables, and cable tendons. Both active and passive steel reinforcement systems have been successfully used for stabilizing rock slopes. The active system involves tensioned or prestressed bars or cables that have been anchored at one end within the rock mass by mechanical means or by cement or chemical grouts or by both. The passive system involves untensioned bars that have been fully grouted throughout their length by cement or chemical grouts. The active and passive systems are often compared to prestressed and reinforced concrete, but, since the rock mass is a discontinuous medium, the mechanical properties of the rock mass are different from controlled, man-made concrete.

The main advantage of the active or prestressed anchor system over the passive system is that no movement has to take place before the prestressed anchor develops its full capacity; thus, deformation and possible tension cracking of the slope are minimized. A further advantage of the active system is that a known anchor force is applied, and proof loading can be accomplished during installation of each anchor. Also, the tensioned rock-bolt member can be rechecked, if necessary, at any subsequent time to determine whether the load is being maintained. A greater degree of confidence in the anchor design is thus provided.

General considerations regarding rock-bolt and rock-anchor reinforcement of rock masses are given by Hoek and Londe (*9.43*) and the U.S. Army Corps of Engineers (*9.110*). The reinforcement is usually designed by using the limit equilibrium method of analysis. This method considers only the ultimate failure condition of the potentially unstable rock mass, which is assumed to fail along a probable failure surface or surfaces that have been defined by geologic investigations. For the active or prestressed system, the anchor force required for stability is applied partly as an increased compressive normal stress on the failure plane, thus increasing frictional resistance, and partly as a force resisting the driving force that is causing instability of the slope. The value of the component forces will depend on both the design geometry and orientation of the anchors and the characteristics of the assumed failure surface. For the passive system, the maximum available anchor force is determined from the tensile and shear resistance of the steel cross section at the assumed failure surface crossed by the anchor.

Both systems require adequate anchorage of the steel reinforcing member beyond the assumed failure surface; adequate anchorage involves good bond of the steel to the grout and good bond of the grout to the rock. Long-term corrosion protection of the anchors must also be provided; this sometimes requires special sheathing and grouting for the prestressed anchors. Grout mixes, installation procedures, and testing procedures must be carefully considered by the engineer in design and specifications. In this regard the ability to test load each anchor after installation is a valuable field control that is available when prestressed anchors are used. The various types of slope-failure modes that are judged to be kinematically possible should be considered in the design of the artificial support system. Actual slope-failure conditions in the field may be complex and difficult to analyze; therefore, a considerable degree of engineering

Figure 9.28. Galvanized sheet steel retaining wall that is anchored by shallow rock bolts to prevent failures of high cut slope in sedimentary rock above highway in Hamilton, Ontario.



judgment and experience will continue to be necessary for anchor design.

Analysis of rock-slope deformation is not generally required for transportation routes, except for those involving support of adjacent structures. Theoretical procedures using the finite element method are dependent on the assumed boundary conditions and properties of the rock mass. At the present time, only cases involving small deformations can be analyzed with any reliability. Otherwise, the only procedure is to use as high a factor of safety as possible against ultimate shear failure to minimize undesirable deformation. This problem is discussed by Morgenstern and Eisenstein (*9.75*).

Rock Bolts

The use of rock bolts is discussed by Lang (*9.58*) and Lancaster-Jones (*9.56*). Rock bolts are basically used to reinforce the surface and near-surface rock of the excavated or natural slope. In that it is essentially a tension member, the rock bolt exerts a compressive force, which tends to prevent elastic rebound, frost action phenomena, and general relaxation or exfoliation by keeping the rock in its original position. The rock-bolt system should be designed in such a manner that shear resistance along discontinuities is improved; the rock mass will then acquire significant tensile strength as a result of the operation.

To minimize the decompression or loosening effects associated with recently excavated rock slopes, rock bolts should be installed and tensioned as soon as possible after each lift of an excavation and preferably before the next lift is blasted to inhibit dilation effects of the near-surface section of the excavation. The sooner the rock bolts are installed, the less the subsequent postexcavation movements of the rock mass will be. Figure 9.29 shows a group of large-diameter rock bolts (*9.80*) concentrated in one area at the base of a slope to maintain the integrity of large fault blocks. Figure 9.30 shows numerous small-diameter rock bolts that have been applied over the entire slope in an attempt to secure smaller individual blocks in the slope.

The method used for transferring load from the head of the rock bolt to the rock depends largely on the condition of the rock. Broken rock may weather away from the bolt head and cause the bolt to lose tension. A choice must be

Figure 9.29 Concentration on bank of 3.5-cm ($1\frac{3}{8}$-in) diameter 14-m (45-ft) long rock bolts at base of section at Hell's Gate Bluffs, British Columbia. Each rod was cold stretched and stress relieved to provide minimum ultimate strength of 1.1 MPa (160 000 lbf/in^2). They were tensioned to 712 kN (80 tf) and dropped to 623 kN (70 tf) and then grouted.

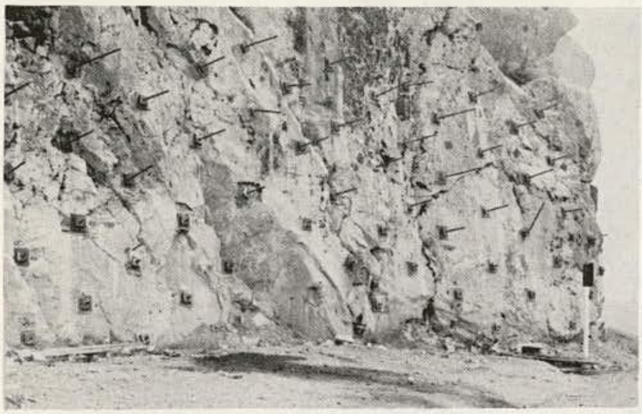


Figure 9.30. Broadly distributed 1.6-cm ($\frac{5}{8}$-in) diameter by 1.2 to 3-m (4 to 10-ft) long rock bolts at Hell's Gate Bluffs, British Columbia. These rock bolts were posttensioned to about 53.4 kN (6 tf) to maintain integrity of small individual blocks in slope.

made between protecting the rock face around the head with wire mesh and shotcrete, retaining the rock in place by steel strapping, or distributing the point load by use of concrete pads. Usually the tentative design load does not exceed approximately 60 percent of the ultimate strength of the bolt.

If grouted bolts are used, extreme care should be taken to ensure that the grout does not spread. Excessive grout spread will reduce the permeability of the rock mass and retard drainage by plugging joints through which groundwater would normally have free access. The grout must be pumped into the borehole under low pressure, possibly not exceeding 103 kPa (15 lbf/in^2), and other measures must be taken to retard grout spread. If grouted bolts are used in the lower reaches of the slope, free drainage must be maintained.

Rock bolts should be installed by a carefully controlled procedure. The design of the rock-bolt system should be checked and tested during installation for bolt tension and adequate anchorage to ensure reliability and performance of the rock bolts. However, the most important responsibility of the site engineer is to regularly inspect the slope after the rock-bolt installation to guarantee that areas that may become unstable in the future have been sufficiently supported.

Dowels and Perfobolts

Dowels consist essentially of steel reinforcing bars that are cemented into boreholes; they are not subjected to any posttensioning. Unlike rock bolts that provide a form of reinforcement, dowels are in the strict sense a form of rock-slope support. Whereas rock bolts increase both shearing resistance and normal stress, dowels basically increase the shearing resistance across potential failure surfaces. A typical use of dowels for rock slope stabilization is shown in Figure 9.31. Dowels are also used to provide anchor keys and tiebacks for shearing resistance at the toe and flanks of retaining walls, as shown in Figure 9.32. Dowels, together with rock bolts, can fasten small blocky rocks when applied with strapping or bearing pads, stabilize broken rock zones when applied with wire mesh and shotcrete, and anchor buttresses or beams placed below sloping blocks of rock. They can also anchor restraining nets and cables, catch nets, catch fences, cable catch walls, and cantilever rock sheds.

The perfobolt is a form of steel dowel encased within a thin sheet-metal liner filled with low-slump slush grout and perforated. The liner is inserted into a drilled hole in the rock, and a steel reinforcement bar is then inserted into the liner, displacing the grout and forcing it through the perforations. A bond is thus effected with the rock. Perfobolts are particularly applicable in sections of the rock mass in which grouting of standard rock bolts would prove difficult because of open cracks and in which fully grouted artificial support is required. Perfobolts as long as 6 m (20 ft) have been installed; for greater lengths, inserting the reinforcing bar in the liner by hand is difficult.

Tendon and Cable Anchors

Tendon and cable anchors are used to control large failure blocks and are accordingly longer than rock bolts. Anchors usually affect a much larger volume of rock than rock bolts. Although the basic principles that apply to rock bolts are

Figure 9.31. Rock dowels used to provide support in sedimentary strata that dip toward Rhaetische Railway line in Switzerland (note fence in upper part of slope) (*9.84*).

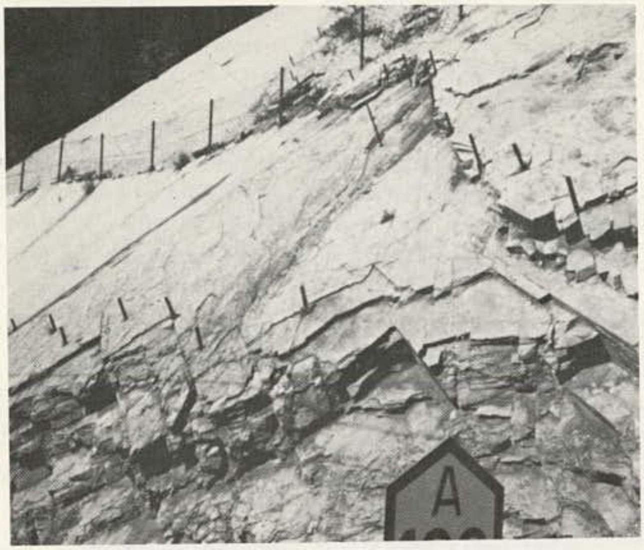


Figure 9.32. Retaining wall (top) that is partly constructed and being cast in place on sound bedrock and finished retaining wall (bottom) with stone facing. In top photo, note broad base of rock dowels that provide for added shear strength at toe of retaining wall as well as excellent bond between concrete and rock.

also applied to rock anchors, rock or cable anchors are generally used more for support than for reinforcement. Anchors can be used to replace retaining walls and other support structures that are normally required for surface excavations. They have also been used effectively to provide intermediate points of support for retaining walls, thus reducing bending moments in the structure and achieving the benefit of requiring a smaller retaining structure.

Compared to their use in mining and hydroelectric developments, large tendon and cable anchors on high rock cuts have had relatively little use on transportation routes. An example of an exception is the considerable success of rock anchors in combination with drain holes in stabilizing progressive failure caused by undercutting the slope on a highway at Windy Point, Australia (*9.95*). The rock is highly jointed sandstone with silty clay beds that dip out of the slope at about 27°. About 181 440 Mg (200 000 t) of rock were moving, and the anchor design involved 45 anchors, each with an average working load of 166.9 kN (375 000 lbf). The anchor design and drain holes are shown in Figure 9.33 (*9.18*).

Figure 9.33. Geological section of Windy Point Slide, Australia, showing location of 166.9-kN (375 000-lbf) anchors (average working load) and drain holes (*9.18*).

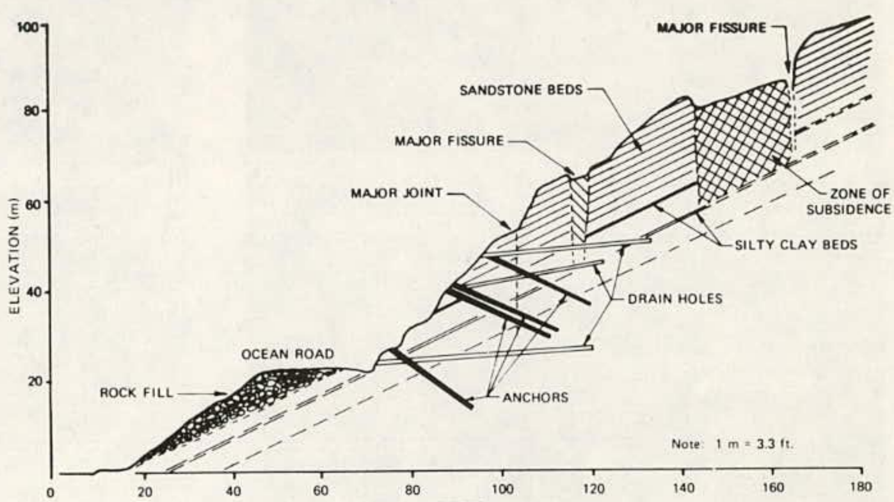
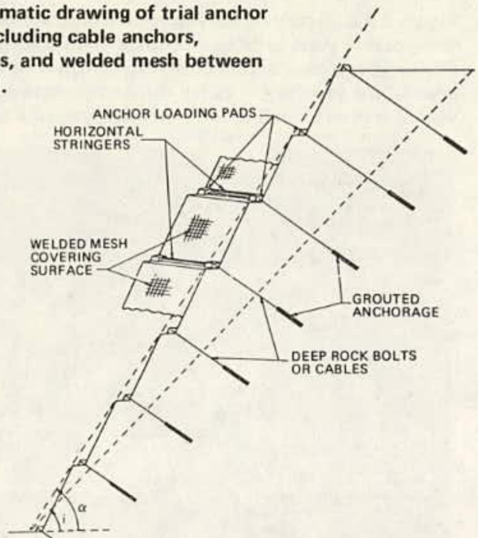


Figure 9.34. Schematic drawing of trial anchor support system including cable anchors, horizontal stringers, and welded mesh between anchor heads (*9.4*).



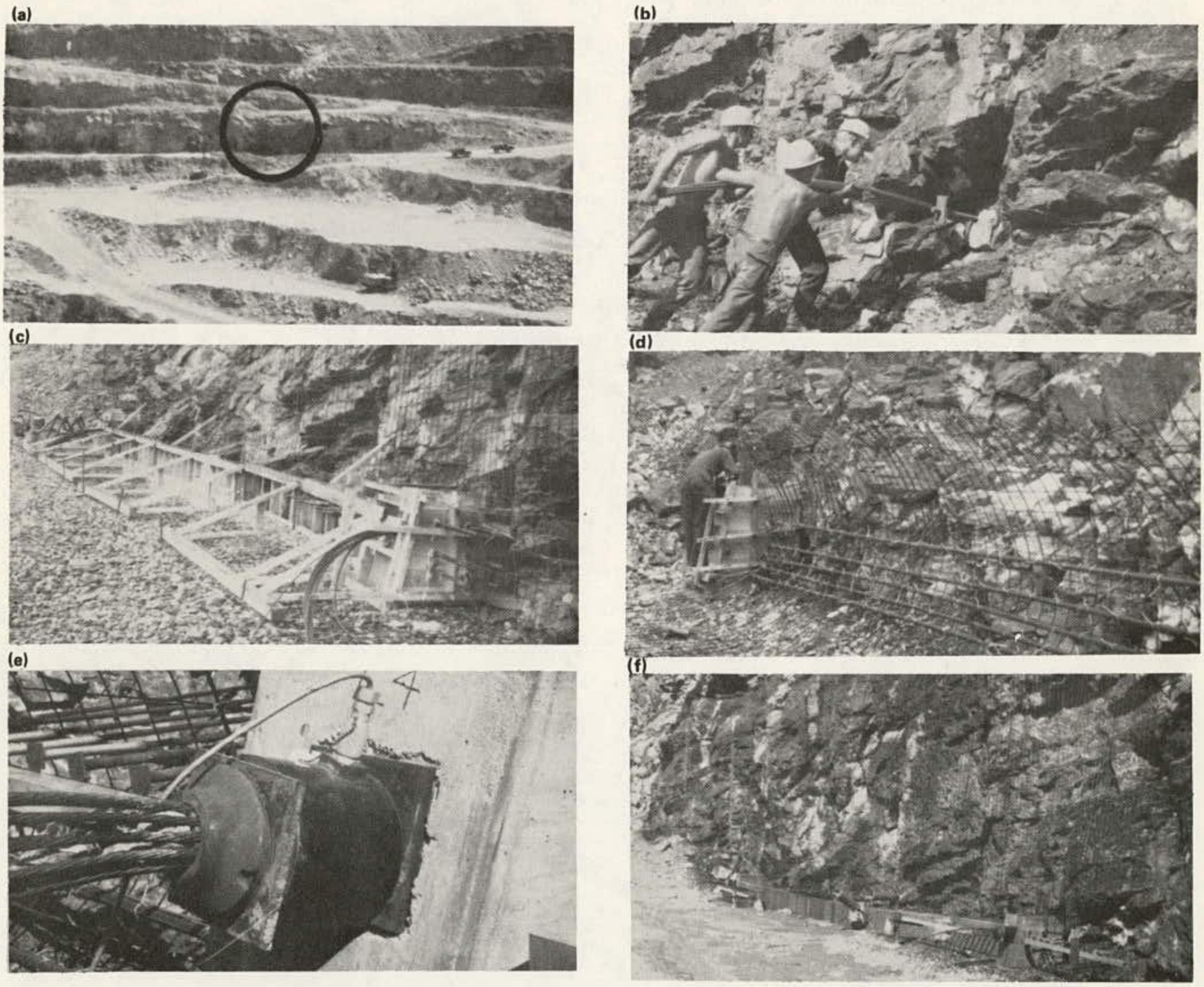
An interesting anchorage system, shown in Figures 9.34 and 9.35, was carried out by Barron, Coates, and Gyenge (*9.4*) at a Canadian mine. The system consisted of deep preloaded cable anchors to support the tentatively unstable ground between the slope face at angle α and some predetermined potential failure plane at angle i. Welded wire mesh was placed over the surface, and horizontal stringers were installed between the cable terminations to prevent local rock falls and to provide bench support.

A practical guide for the use and installation of anchors is given by Coates and Sage (*9.18*), and other useful aspects relating to design and quality control of rock anchors are described by Seegmiller (*9.102*).

Anchored Beams

Anchored beams can be used to distribute the support of rock bolts over a wide area of the slope and, hence, minimize the number of bolts required. The beams can be made of concrete or steel and applied in any direction across a rock face. Anchored beams are particularly useful where

Figure 9.35. Installation of system shown in Figure 9.34: (a) location of two benches where the trial anchor installation is conducted, (b) inserting the cable in the borehole, (c) formwork for horizontal concrete stringer on lower bench and 66-44 welded wire mesh, (d) formwork for anchor pads on top bench and steel bar horizontal stringer, (f) cable anchor tensioned and locked, and (f) completed trial anchor installation on lower bench (*9.4*).



(a) (b) (c) (d) (e) (f)

Figure 9.36. Cast-in-place reinforced concrete beams used to distribute point loads from rock bolts seated in the beams (*9.84*). Location is above main line of Austrian Federal Railways.

heavy slabs of rock are unstable and where a relatively uniform distribution of point load forces of the anchors or bolts is required. Typical anchored beams installed in Austria are shown in Figure 9.36. Several variations of anchored beams have been used: In Czechoslovakia, for example, anchored concrete frames are used rather than beams. Anchored concrete aprons have been used on heavily jointed rock around bridge piers in Washington State.

Anchored Cable Nets

Anchored cable nets can be used to restrain masses of small loose rocks or individual loose blocks as large as 1.5 to 2.5 m (5 to 8 ft) in diameter that protrude from a rock face. In principle, an anchored cable net performs like a sling or reinforcement net, which extends around the surface of the unstable broken rock to be supported. The cable net strands are gathered on each side by main cables leading to rock anchors, as shown in Figure 9.37.

Beam and Cable Walls

Beam and cable walls can also be used to prevent smaller

Figure 9.37. Application of anchored cables and cable nets.

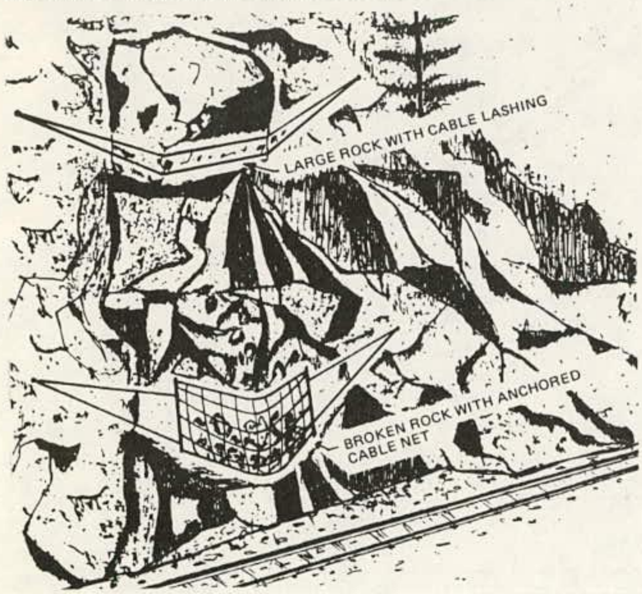
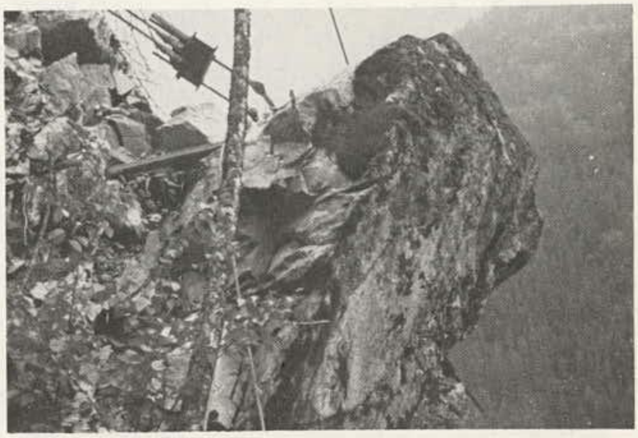


Figure 9.38. Cable lashing of a large, potentially hazardous, 2-m (6-ft) diameter block about 90 m (300 ft) above highway in Fraser Canyon, British Columbia.

Figure 9.39. Large, potentially hazardous rock block supporting considerable volume of smaller material secured by two cable strands that were inserted in boreholes through the block and anchored on other side.



blocks of rock from falling out of the slope and may be used on slopes as high as 20 m (60 ft). Beam and cable walls consist of cable nets fastened to vertical ribs formed of steel beams laid against the slope at 3 to 6-m (10 to 20-ft) intervals. The beams are held by anchored cables at the top of the slope and by concrete footings at ditch level.

Cable Lashing

Cable lashing is a simple, economical type of installation for restraining large rocks. As shown in Figure 9.37, this method merely involves tying or wrapping unstable blocks with individual cable strands anchored to the slope. Eckert (*9.24*) describes a case in France in which cables with a capacity of 227 Mg (250 t) and a length of more than 90 m (300 ft) were extended around a mass of broken rock and anchored into sound rock at each side. Lashing using both cables and chains is described by Bjerrum and Jørstad (*9.9*). If rock is too broken to be restrained by individual cables, vertical concrete or steel ribs can be used to spread the points of contact of the cables with the rock. Typical cable lashing of a large block is shown in Figure 9.38. Figure 9.39 shows a unique application of cables in that two cables pass entirely through the block.

Methods of Protection

Rock-Fall Characteristics Affecting the Design of Protection Measures

Loose rocks can be kept from reaching the transportation route right-of-way by retaining the rocks on the slope, intercepting falling rocks above the roadway, or directing falling rocks to pass over or under the transportation route without causing harm. Before the best methods can be chosen for a particular location, the characteristics of the rock falls and the general geometry of the slope should be evaluated. The rate at which fallen rock accumulates is also an important factor. Without encroaching on the right-of-way, storage must be provided for rock falls that are likely to arrive during a reasonable maintenance interval. Protection measures must be designed to protect against the various ways in which rock falls reach the right-of-way (*9.85*).

A rock can arrive at the base of a slope by free falling, bouncing, rolling, or sliding down the slope. For free-falling rocks, the only means of protection are to move the roadway away from the slope and prevent the rocks from bouncing and rolling after landing or to protect the roadway with a rock shed or tunnel. The paths of bouncing rocks are difficult to predict, and interception measures above the right-of-way require high structures, such as a wall or net. Rolling or sliding rocks are easier to intercept since they are in constant contact with the slope; in gullies, their paths are even more predictable.

A rock-fall model developed by Piteau and Clayton (*9.91*) for a computer program can simulate several hundred rock falls for a slope of specified geometry in a matter of seconds. A coefficient of restitution is used for the rock fall in the bouncing mode, and increased friction is used for the rolling mode to determine the paths of the rock falls for different input velocities and heights. The slope profile is divided into straight segments or slope cells (Figure 9.40), which are numbered consecutively from the top to the bottom of the slope. Rock falls are introduced at different locations above the slope as desired. Probability factors can be assigned to these locations to recognize areas that are either more likely or less likely to be the source of the rock fall. Catch walls at various positions and heights can be assumed in the model to evaluate their effectiveness. In the same manner, the effect of varying the slope geometry by including ditches, benches, and berms can be evaluated.

This rock-fall model was used to relocate a concrete catch wall at the base of an active rock-fall area (Figure 9.51). It was also used to determine whether large blocks located at the crest of a major slide area would reach facilities at the base of the mountainside located some 1500 to 1800 m (5000 to 6000 ft) away from the crest of the slide.

Figure 9.40. Rock-fall paths traced by calcomp plotter based on computer model for typical rock falls at Porteau Bluffs, British Columbia (*9.91*).

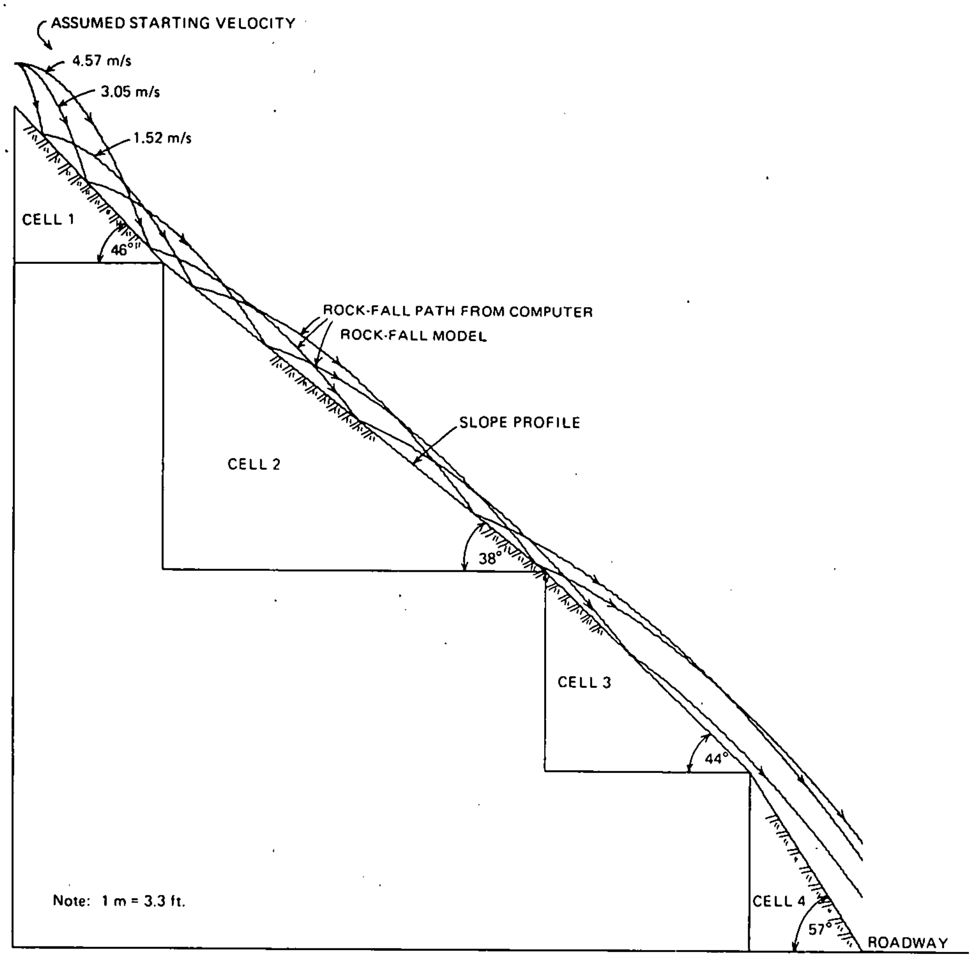


Figure 9.41. Intercepting slope ditch or cross ditch, north of Hope in Fraser Canyon, British Columbia, to catch rolling rocks originating up slope on steep rock faces (note that ditch is shaped partly in talus and makes use of blocks of rock).

Figure 9.42. Shaped berm that is at toe of high rock face and cut in top of high talus slope above Hope-Agassiz highway in British Columbia.

Relocation

Relocation is most effective where rocks are free falling from steep rock faces in proximity to the roadway or where stabilization or some other protection measure is not feasible. Relocation is feasible when space is available and the design criteria are not affected. When combined with proper ditch shaping and possibly with other ditch-level protection, relocation can be the most economical solution and, in some cases, one of the few solutions to the problem. Unless relocation practically eliminates the possibility of accidents, other measures also should be included along with relocation.

Intercepting Slope Ditches and Shaped Berms

Slope ditches are used to intercept rock falls partway up the slope, and shaped berms are used at the top of the slope. These methods ensure that rock falls get caught either before they start to roll or while they are on their way down the slope. In some locations, ditches can be made to intercept rocks and guide them laterally into disposal areas. A typical intercepting slope ditch is shown in Figure 9.41, and a shaped berm at the base of a steep slope is shown in Figure 9.42.

Depending on the nature of the terrain, these methods are generally inexpensive, simple to construct, and easy to maintain. An intercepting slope ditch should only be installed on a slope that can accept the introduction of a ditch without the stability of the entire slope being impaired. The ditch must be designed and built carefully so that the upper slope is not at any time steeper than planned. The ditch should be suitably located to facilitate periodic cleaning by mechanical equipment.

Anchored Wire Mesh

Wire mesh is a versatile and economical material for use in protecting the right-of-way from small rocks. Layers of mesh are often pinned onto the rock surface to prevent small loose rocks from becoming dislodged (Figure 9.43). Figure 9.44 shows that mesh can also be used essentially as a blanket draped over the rock surface to guide falling rock into the ditch at the base of the slope. The same arrangement can be used on stony overburden slopes to prevent dislodged stones from rolling down the slope. This practice is commonly used on talus slopes in steep mountainous terrain. Mesh can be combined with long rock bolts to provide a generally deeper reinforcement. Mesh in combination with both shotcrete and rock bolts provides general reinforcement and surficial support and retards the deleterious effects of weathering.

Conditions for the use of mesh are particularly suitable if no individual rocks are larger than 0.6 to 1 m (2 to 3 ft) and if the slope is uniform enough for the mesh to be in almost continuous contact with the slope. If large falling blocks in certain areas are likely to dislodge or tear the mesh and present a hazard, rock bolts should be used to reinforce these particular areas. For anchoring mesh at the top of overburden slopes, posts are cast in concrete blocks in hand-dug holes. For bedrock slopes, grouted rock bolts or dowels are used. A cable is slung between the anchorages; the mesh is then fastened to the cable and rolled down the slope, and the vertical seams of the mesh are wired together. For wire mesh blankets, the bottom end of the mesh is usually left a meter or so above ditch level (Figure 9.44) and only a narrow ditch is required. The mesh normally used is 9 or 11-gauge galvanized, standard chain-link or gabion wire mesh. Gabion mesh appears to have an advantage over the standard chain-link materials in that the gabion mesh has a double-twist hexagonal weave that does not unravel when broken.

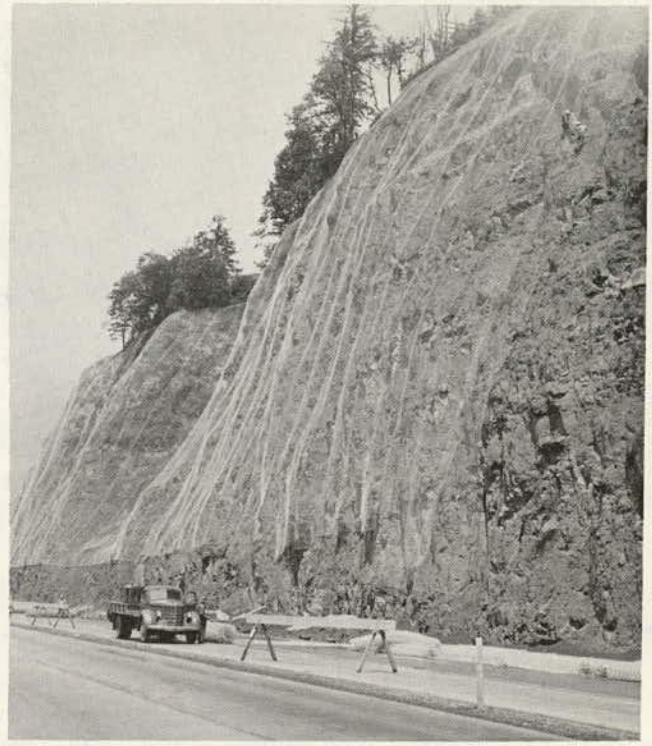
Protection Methods at Ditch Level

Shaped Ditches

Depth, width, and steepness of the inside slope and storage volume of the ditch are important factors in the design of ditches to contain rock falls. The choice of ditch geometry should take into consideration the angle of the slope that influences the behavior of falling rocks. Ritchie (*9.98*) evaluated the mechanics of rock falls from cliffs and talus slopes and developed design criteria for ditches. The criteria, which

Figure 9.43. Anchored double-twist, hexagonal wire mesh being fastened on a high rock face to prevent rock falls onto highway.

Figure 9.44. Wire (No. 9) mesh blanket used to control rock falls near Kelso, Washington (*9.3*). Falling rocks roll down surface of slope under mesh and drop into ditch.



involve the height and angle of slope, depth of ditch, and width of fallout area, are given below (where 1 m = 3.3 ft) and in Figure 9.45, which also shows the nature of rock trajectories for different slope angles. An example of a typical shaped ditch is shown in Figure 9.46.

Rock Slope Angle	Height (m)	Fallout Area Width (m)	Ditch Depth (m)
Near vertical	5 to 10	3.7	1.0
	10 to 20	4.6	1.2
	>20	6.1	1.2
0.25 or 0.3:1	5 to 10	3.7	1.0
	10 to 20	4.6	1.2
	20 to 30	6.1	1.8[a]
	>30	7.6	1.8[a]
0.5:1	5 to 10	3.7	1.2
	10 to 20	4.6	1.8[a]
	20 to 30	6.1	1.8[a]
	>30	7.6	2.7[a]
0.75:1	0 to 10	3.7	1.0
	10 to 20	4.6	1.2
	>20	4.6	1.8[a]
1:1	0 to 10	3.7	1.0
	10 to 20	3.7	1.5[a]
	>20	4.6	1.8[a]

[a]May be 1.2 m if catch fence is used.

If lack of space prevents the shaped ditch from being as wide as indicated, limited protection against small rolling rocks can still be provided inexpensively by excavating a

Figure 9.45. Path of rock trajectory for various slope angles and design criteria for shaped ditches (*9.98*).

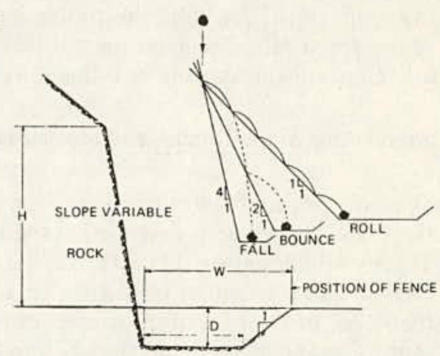


Figure 9.46. Shaped ditch to retain falling rock from nearby vertical slope on highway in Washington State (note steep slope and barrier fence on roadway side).

Figure 9.47. Shaped ditches, wire mesh catch fences, and wire mesh catch nets (*9.84*).

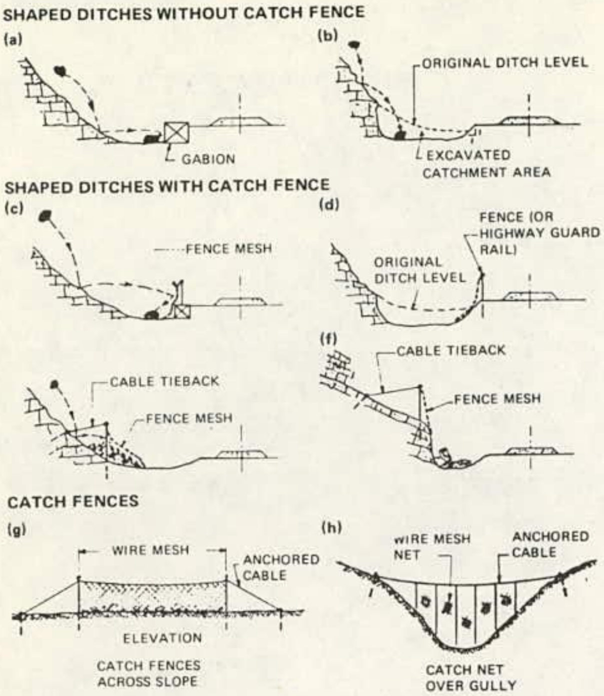


Figure 9.48. Special mobile catch fence mounted on flat deck to protect motorists from rock falls during excavation work and scaling in extremely steep terrain.

catchment area, as shown in Figure 9.47b (*9.84*), by forming a windrow of material or by installing a low barrier formed of standard highway guardrail, precast concrete elements, or gabions at the shoulder of the roadway, as shown in Figure 9.47a (*9.84*). Bedrock should not remain exposed in the bottom of ditches, but should be covered with small broken rock or loose earth to keep falling rocks from bouncing or shattering. Mearns (*9.71*) describes how a ditch at grade was filled with a 0.3-m (1-ft) layer of sand, which acted as an energy absorber to prevent rolling rock falls from reaching the road. Several workers have used a similar technique to successfully dissipate energy of rock falls.

Wire-Mesh Catch Nets and Fences

Wire mesh can be effective in intercepting or effectively slowing bouncing rocks as large as 0.6 to 1 m (2 to 3 ft) when the mesh is mounted as a flexible catch net rather than as a standard, fixed wire fence. If suspended on a cable, the mesh will absorb the energy of flying rocks with a minimum of damage to the wire catch net. Catch nets can consist of standard chain-link wire mesh or gabion mesh. This type of barrier has economic potential in some troublesome locations, as shown in Figure 9.47h (*9.84*). It is best to locate catch nets at the lower end of steep gullies where rocks tend to bounce down to the right-of-way. The cable supporting the catch net is anchored to sound rock on either side of the gully with the result that rocks hit the catch net and fall harmlessly at the base of the structure. If there is a steady accumulation of rock, a catch wall on the shoulder of the roadway also may be required.

The principle of the catch fence is similar to that of a catch net. Its purpose is to form a flexible barrier to dissipate the energy of rapidly moving rocks. Various arrangements are shown in Figure 9.47 (*9.84*) for catch fences located at or near ditch level. The wire mesh (chain-link or gabion) forming the catch fences is hung on cables supported on posts or strung between posts or trees. Wire-mesh catch fences are usually located on the roadway side of the ditch or at the base of the slope and can be used with or without a shaped ditch. The fence should be suitably situated so that accumulated rocks can be removed easily. Figure 9.31 shows a catch fence, and Figure 9.48 shows a special application of a mobile catch fence for use in scaling operations.

Catch Walls

Catch walls can be used to form a rigid barrier to stop rolling or bouncing rocks as large as 1.5 to 2 m (5 to 6 ft) from reaching the right-of-way. If effective, they usually increase the storage capacity of the ditch so that maintenance intervals can be extended. In many locations, large ditches themselves are not effective for intercepting large rolling rocks and the use of catch walls is advised.

To achieve maximum protection and storage capacity, the catch walls should be located on the side of the ditch closest to the road. In steep terrain, catch walls are commonly used where the right-of-way cuts across postglacial slide areas. Many postglacial slide areas that have been disturbed by excavation are in a constant state of sloughing and readjustment. If broad areas require wall protection, gaps should be left in the catch wall to allow access of maintenance equipment required to remove rock-fall debris.

Concrete Catch Walls

Concrete catch walls are the most widely used type of catch wall in steep mountainous terrain. Concrete walls may be cast in place or precast in short sections and assembled on the site. An efficient precast concrete wall installation is shown in Figure 9.49 (*9.84*). Figures 9.50 and 9.51 show

Figure 9.49. Precast concrete wall that could be used for track protection (*9.84*).

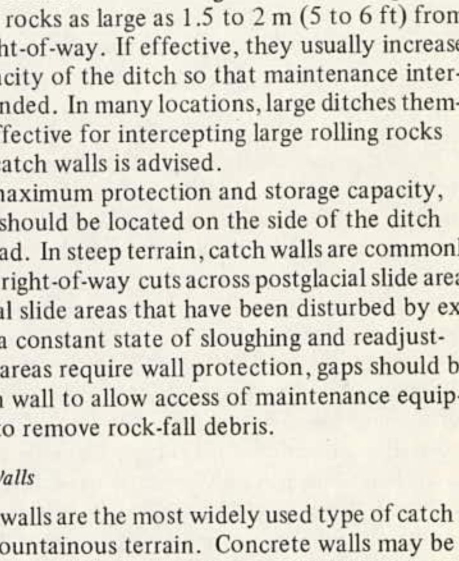


Figure 9.50. Reinforced cast-in-place concrete wall approximately 2 m (7 ft) high and 1 m (3 ft) wide across a major postglacial slide area along edge of Trans-Canada Highway in Fraser Canyon, British Columbia (note break in retaining wall where large blocks have hit wall).

Figure 9.51. Close-up of concrete catch wall shown in Figure 9.50 (note breaks in wall due to large blocks from postglacial slide debris rolling down slope).

the use of a cast-in-place concrete catch wall in a postglacial slide area, where large rock falls resulted in breaking the wall.

Box Gabion Catch Wall

The box gabion catch wall is a rectangular basket divided by diaphragms into smaller rectangles that are filled with stones. The basket is formed of woven hexagonal steel galvanized wire mesh. Baskets can be placed to be filled individually or wired together in groups and filled accordingly. The wire mesh tends to reinforce the stone in tension. The gabion is a flexible structure that, upon settling or being hit by impact, tends to deflect and deform instead of break. Because gabion walls are highly deformable, differential settlement is not important. Like the gabion mesh, the gabion does not unravel if broken. Gabions make good catch-wall structures, but only recently have they been recognized in North America as a feasible alternative to more rigid concrete walls for protecting the right-of-way from rolling rocks. They prove particularly useful where the right-of-way crosses postglacial slide areas. They can be used efficiently to stop rolling stones as large as 0.6 to 1 m (2 to 3 ft). When adequate filter materials are used as backfill, they provide long-term, free-draining walls. A gabion catch wall is shown in Figure 9.52.

Rail Walls

Rail walls or rail-and-tie walls consist of vertical posts and horizontal members that are extended between the vertical posts. The vertical posts are usually scrap steel rails set in holes, which are hand dug or blasted and backfilled with concrete. The horizontal members are either ties, as shown in Figure 9.53, or scrap rails cut into 3 to 4-m (10 to 13-ft) lengths, as shown in Figure 9.54. If feasible, cables can be anchored back from the top of the wall to provide resistance to overturning.

Cable Walls

Cable walls consist of steel posts set in concrete, cables strung horizontally between, and a smaller cable or coarse wire woven between the cables to form a crude net or mesh. This type of installation is sometimes used in Europe.

Rock Sheds and Tunnels

Rock sheds and tunnels can be used for protection against rock falls and slides when warranted and when other forms of stabilization and protection are not effective. Although expensive, they can give complete protection and should be considered in areas with serious problems. Maintenance costs are normally negligible. The methods of design and construction of tunnels are dealt with thoroughly in the literature, but the design of rock sheds is not so adequately covered, and experience is required to decide on the most suitable type of structure and the loads to be carried. A rock shed should be able to resist the energy transmitted by the largest rock mass likely to pass over it during its life; therefore, probability analysis should be involved. The energy transmitted will depend on whether rocks are falling free, bouncing, or rolling. High stress concentrations in the structure can be reduced by the provision of a thick cover of loose sand.

If foundation conditions on the outer side of the roadbed are not suitable for footings, heavy rock anchors extending into the upper slope may be used to support a cantilevered shed of the type shown in Figure 9.55. Tunnel portals can also act as sheds to protect the roadway or track, as shown in Figure 9.56. When a rock shed is to be located at the lower end of a gully, wing walls are usually used above the structure to channel material onto it (Figure 9.57). Wing walls should be sufficiently high because the debris will tend to clog and build up when its path is restricted by the structure. For this reason, the slope angle of the roof of the shed should be steeper than the angle of repose of the material to be conveyed over the roadway.

Methods of Warning

Although warning systems do not prevent rock falls, they are necessary on transportation routes where other measures are too expensive or impractical or where a new hazard has developed. In North America, warning methods have been used on railways in mountains to detect rock falls on tracks so that trains can stop before hitting the material.

Patrols

The simplest type of warning method is provided by human patrol. Patrols have the advantages of being reliable and flexible, and their frequency can be adjusted to the demands of traffic and weather conditions. The disadvantages are that they incur continuing costs and require personnel who are willing to work in uncomfortable and often hazardous conditions.

Electric Fences and Wires

There are several variations of electric warning methods. Electric fences are based on the principle that a falling rock large enough to endanger traffic will break or pull out one of the wires and thus actuate a signal to warn approaching traffic. This principle is particularly adaptable to railways on which a signal system to control traffic is already in use. The standard electric warning fence used on railways consists of a row of poles spaced along the uphill ditch line and wires strung between them at a vertical spacing of 25 cm (10 in). Overhead wires, which are supported on members cantilevered out from the top of the poles, as shown in the installation in Figure 9.58, are often required where rock faces are steep and close to the right-of-way.

Figure 9.52. Gabion catch wall along edge of high slope in unconsolidated material on main highway.

Figure 9.53. Rail and tie wall founded on masonry wall to prevent rolling rock from reaching track of Austrian Railway.

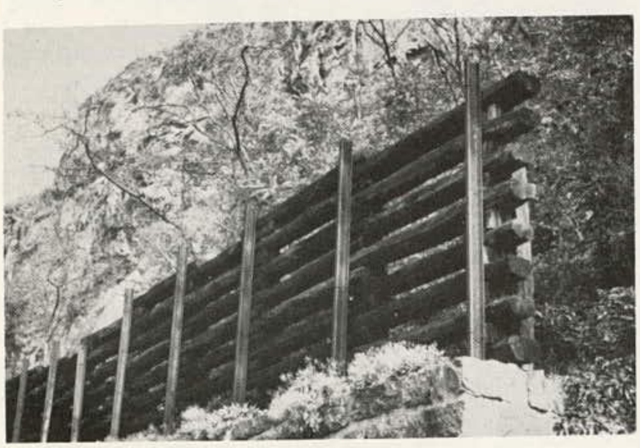


Figure 9.54. Rail wall with vertical posts seated in cast-in-place footings (note top of wall anchored by cables to bedrock for support).

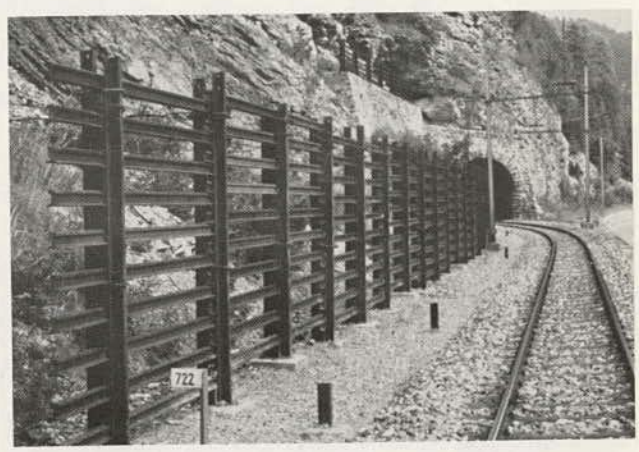


Figure 9.55. Cantilevered rock shed supported by deep anchors inserted in upper slope protecting rail line in Switzerland (*9.84*).

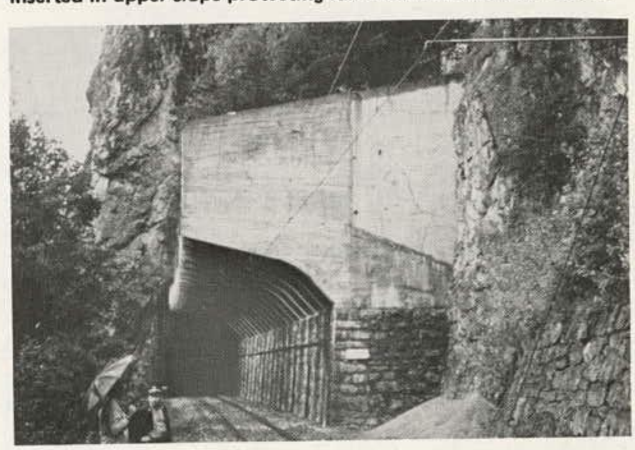
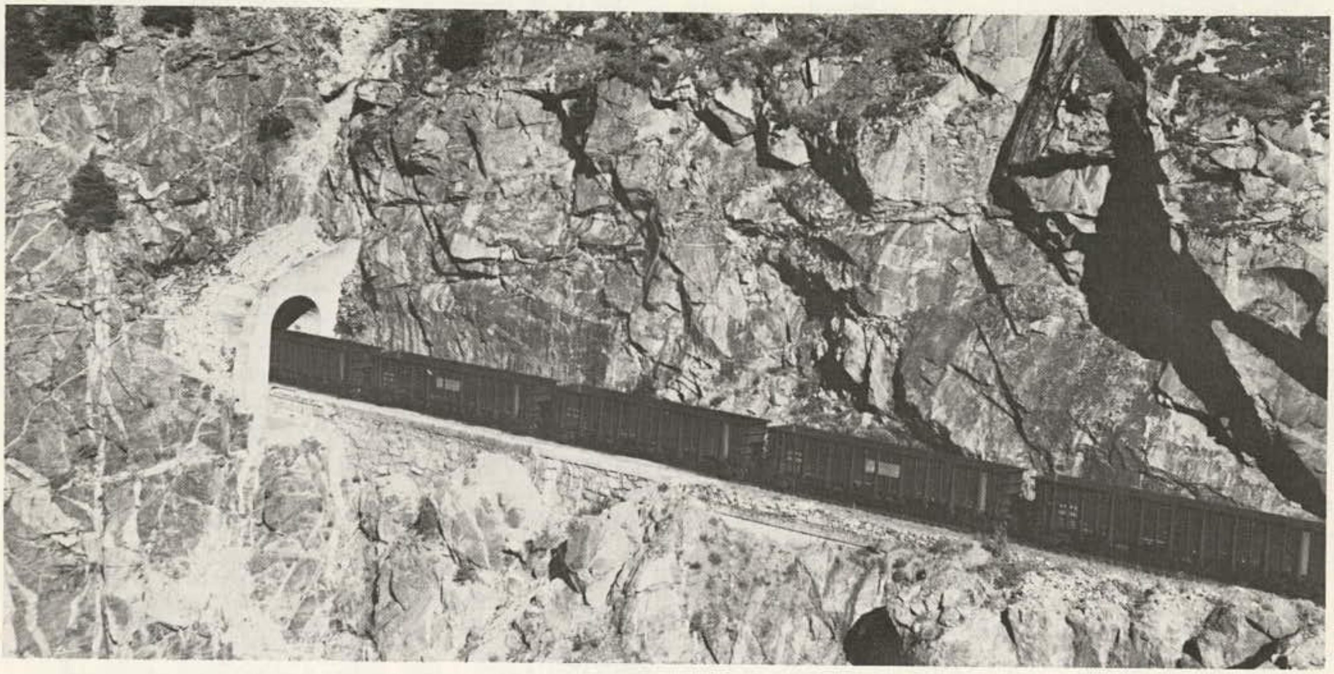


Figure 9.56. Tunnel in steep terrain with extended portal shedlike structure to protect Canadian National Railway line from rock falls generated in a slide area (note use of retaining walls of rock blocks below track).



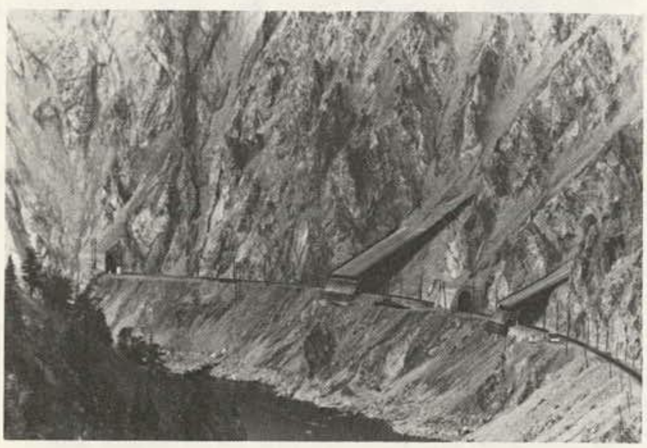


Figure 9.57. Rock sheds to carry rock debris originating in deep gullies over main line of Canadian National Railway in White Canyon in Thompson River Valley, British Columbia.

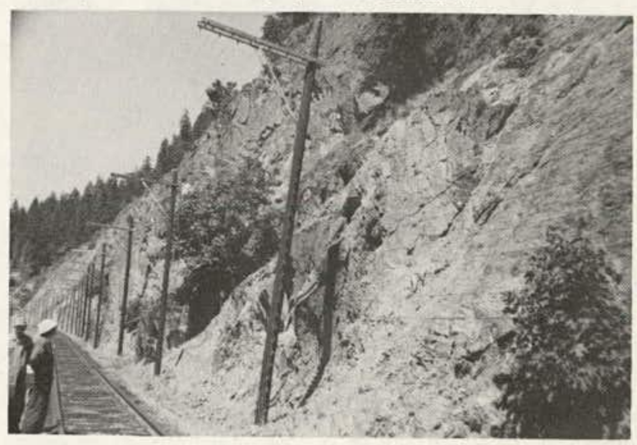


Figure 9.58. Standard type of railway electric warning fence with wires strung between upright poles and horizontal members that are cantilevered out from top of poles along Canadian National Railway line in Fraser Canyon, British Columbia.

Electric warning fences have some advantages, and the design of each facility can be made to suit the individual sites. One disadvantage is low efficiency; in some instances 80 percent of the alarms have been found to be false. Another disadvantage is that snow-clearing operations, if required, are often impeded and maintenance of the fences proves difficult. These difficulties may be reduced or eliminated by

1. Choosing the spacing of wires according to previous experience at a site,
2. Providing a catchment ditch behind the warning fence,
3. Eliminating the lower wires where the lower slope can be scaled and stabilized, and
4. Supporting overhead wires on a canopy type of frame bolted to the rock face on the uphill side and supported on poles on the downhill side.

A particularly effective type of electric warning system consists of a single wire, anchored at both ends and linked to a warning signal (*9.85*). Such a wire may be fastened around a large unstable rock or across a rock slope above the right-of-way, across a gully where large rocks roll down, or on top of a protective catch wall, as shown in Figure 9.49. The installation is simple, economical, and efficient.

Other Methods

Warning methods that are dependable under all conditions are undergoing continued study. At this writing, the following are being tried: (a) geophones or vibration meters buried at intervals along the roadway shoulder to pick up vibrations from falling rocks by the Canadian National Railways and Swedish State Railways; (b) television monitoring by the Federal Highway Administration; (c) guided radar by the Canadian Institute of Guided Ground Transport and the Japanese National Railways; and (d) laser beams by the Radio Corporation of America. However, none is known to have been sufficiently developed to be recommended for general use.

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