DESIGN OF ROCK SLOPES

by

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ABSTRACT

This paper outlines the basic engineering geology and stability considerations essential to the design of rock slopes. Significant geologic factors in design of rock slopes are presented and discussed. A systematic procedure consisting of a series of steps involving data collection, analysis and design is proposed as an approach to design of rock slopes. By following these steps a logical approach to design is developed. Design choices are simplified and a rational design is achieved.

A case history of a throughcut and sidehill cut for a railway in northern British Columbia is presented to illustrate the basic steps involved in rock slope design.

INTRODUCTION

Design of rock slopes requires a clear understanding of rock type, geological structure, physical and mechanical properties of the rock mass, groundwater flow, weathering and other natural conditions of the geological environment. The geological information is essential to define the mechanics of slope failure and related controls on slope stability. Engineering principles and calculations are required to analyze stability and prepare a rational design of the slope and remedial measures. Hence, as stated by Piteau and Peckover (1978), the design of rock slopes is a problem in engineering geology.

In transportation corridors, the objectives of rock slope design are to develop and maintain safe and efficient slopes. Minimizing rock excavation and predicting the stability and ultimate behaviour of rock slopes whether for highway, railway, spillway, quarry, damsite or open pit mines are common objectives of civil, geological and mining engineers. Rational design of rock slopes is particularly important if slopes are steep, if safety is important and if slope design significantly affects project costs. An assessment of both the long term performance as well as the immediate behaviour of the slope is required.

Many slopes have been designed on the basis of empirical techniques or "cut and try" methods based on previous experience. New principles and improved technology are needed as higher and steeper rock slopes are developed in steeper terrain or under different engineering geology conditions.

PRELIMINARY CONSIDERATIONS

COMPARISON OF ROCK AND SOIL

In the analysis of rock slopes, one must first recognize the differences in the basic characteristics and behaviour between soil and rock. Unlike a soil mass, which is a relatively homogenous 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 physical characteristics and interaction of particles within a soil mass are substantially different than those within a rock mass. Because particles within the soil mass are very small with respect to the size of the slope, failure tends to occur through the soil mass. In a rock mass the surface of failure tends to follow pre-existing discontinuities and does not generally occur through intact rock unless the rock is very weak or does not contain structural discontinuities. The shear strength of a rock mass is determined largely by the shear strength of the discontinuities with the result that the rock mass is anisotropic in its strength and deformation properties. Soils may also exhibit discontinuities and strength anisotropy due to their geological history, however, because the soil material is much lower strength than most intact rock masses, failure can be expected to occur within the soil mass.

GENERAL BASIS FOR ASSESSMENT OF STABILITY

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. Based on this concept a factor of safety, F for the slope may be defined as:

$$F = \frac{\text{sum of resisting forces which act to prevent failures}}{\text{sum of the driving forces which tend to cause failure}}$$

The necessary factor of safety to be provided in a particular case depends on the accuracy to which the engineering geology, strength properties, failure mechanics and other parameters are known and the degree of safety required for the slope in question. In this regard, slope stability considerations and factor of safety may be much greater for railway slopes than for highways which in turn are considerably different for forestry roads or open pit mines.

SCALE OF SLOPE FAILURE

When considering the design of a rock slope, the scale of potential slope failure is important. The designer must be aware of and assess the potential for:

i) Large scale instability of the overall slope
ii) Instability of large sections of the slope
iii) Small failures, rockfalls and ravelling which may be of prime importance to safety of persons and structures near the toe of the slope

The optimum slope design must be determined based on the analysis results and relative importance of each failure type with respect to the particular slope application being considered.

SIGNIFICANT FACTORS IN DESIGN OF ROCK SLOPES

The basic factors significant to the stability of rock slopes are discussed briefly in the following. More detailed discussions of each factor and its importance for design are given by Piteau (1971), Herget et al (1977), Hoek and Bray (1977), Piteau et al (1979) and others.

STRUCTURAL DISCONTINUITIES

Structural discontinuities such as faults, joints, bedding, foliation, etc., are universally present in rock masses. Discontinuities have appreciably lower strength than the intact rock and may control the physical and mechanical characteristics of the rock mass. Hence, the stability of rock slopes is assessed principally by analyzing structural discontinuities. Fig. 1 clearly illustrates the importance of structural discontinuities for slope stability.

Properties of structural discontinuities which are important to slope design are:

i) Nature of occurrence
ii) Orientation
iii) Location or position in space
iv) Continuity or size
v) Spacing
vi) Surface asperties: roughness and waviness
vii) Previous shear movement
viii) Infilling materials and strength of infilling
ix) Wall rock type and strength
x) Genetic type

Most discontinuities occur in sets which have a mean orientation and defineable physical properties. Identification and description of discontinuity sets is important to clarify the analyses and enable prediction of rock mass characteristics in all parts of the rock mass. Individual discontinuities which do not occur in sets must also be identified and described.

GROUNDWATER

The presence of water in discontinuities has probably been responsible for more rock slope failures than all other causes combined. Water in a slope can affect stability by:

i) Physically and chemically affecting the joint infilling materials, thus altering the strength parameters of the materials.

ii) Exerting hydrostatic pressure on joint surfaces, reducing the effective normal stresses and, hence, the shearing resistance along potential failure surfaces.

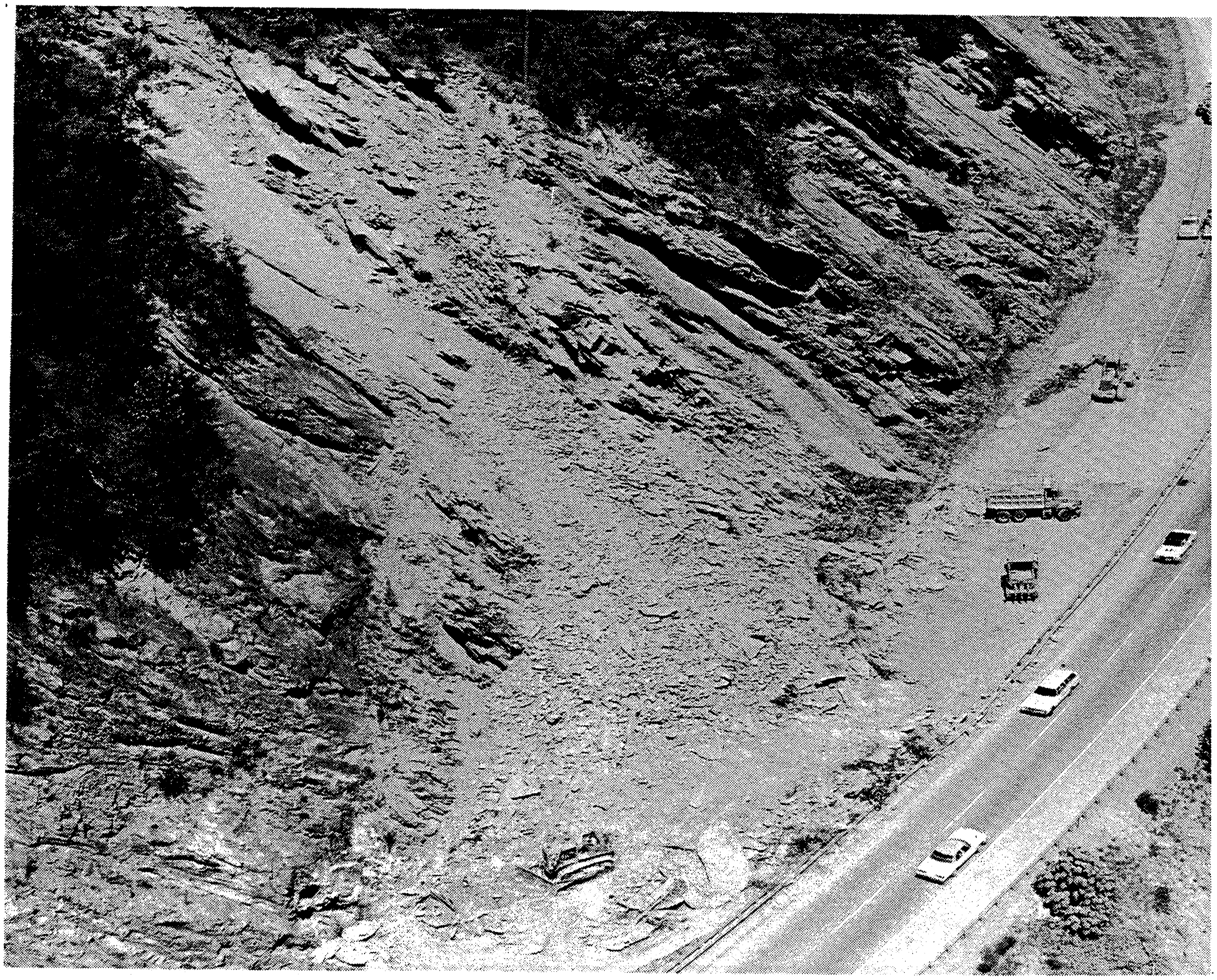


FIG. 1 Well developed discontinuities which control stability of rock slopes along Interstate Highway 40 (Photo by George Hornal, Tennessee Department of Transportation).

iii) Contributing to lateral driving forces due to presence of water or ice in tension cracks or vertical joints.

iv) Affecting intergranular shearing resistance, thus causing a decrease in compressive strength.

v) Increasing the total unit weight of the rock mass thereby increasing driving forces contributing to instability.

A thorough knowledge of the character and influence of the hydrogeologic regime is required. The controlling influences of rock type, stratigraphy and structure on the flow, permeability, transmissivity and storage capacity must be defined. Effects of climate or other environmental factors on recharge, discharge, etc. should also be considered.

The hydrogeological parameters are used to develop a groundwater flow model from which the groundwater pressures in the slope may be determined. The groundwater flow model will also be used to define the phreatic surface and estimate likely discharges of groundwater on the slope. These parameters are essential for input into the stability analysis.

ROCK TYPE, WEATHERING AND ALTERATION

Before one can completely comprehend the particular problems of stability, one must understand the lithology and the physical properties of all the materials in the rock mass. Slopes commonly consist of a complex of rocks of different geologic origins. Each particular rock 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. These basic properties determine the strength of the rock material as well as the contribution to frictional strength provided by asperities along discontinuity surfaces.

Physical and mechanical properties may be significantly changed by natural processes such as weathering, fluctuation of groundwater table, changes in moisture content, chemical alteration or other processes. If these processes are occurring, they must be defined, and both short term and long term effects on the rock properties must be described.

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 factors in rock slope engineering. Daily temperature variations, precipitation, snow and freeze-thaw conditions, acting either independently or in combination, often cause significant stability problems. Groundwater conditions, and hence, effective hydrostatic pressures can vary widely, depending on the climatic conditions and geologic environment. Work by several authors has shown that slope movements are more prevalent during periods of high precipitation, or when the groundwater table in the slope is higher.

Increased incidences of rockfalls have been documented to depend on seasonal precipitation (Bjerrum and Jorstad, 1968) or the occurrence of freeze thaw cycles i.e. frost action (Peckover, 1975).

SLOPE GEOMETRY IN PLAN AND SECTION

Many current theories of slope stability consider the slope to be two dimensional. Analyses are conducted for a unit length of an infinitely long slope which is considered to be in plane strain. In practice, slopes can be concave or convex in plan. Horizontal tangential stresses which develop in a concave slope are compressive. These stresses create an archlike effect whereby the blocks forming the partitioned rock mass tend to be squeezed together which substantially improves the overall shear strength. Horizontal tangential stresses in convex slopes are tensile. Since a rock mass is relatively weak in tension, tensile stress concentrations in the slope induce instability, causing unrestrained blocks to slide out. Similar principles apply to slopes which are concave or convex in section.

TIME FACTOR AND PROGRESSIVE FAILURE

Rock slopes have been documented to deform under applied stress. Most of the forces involved in such deformations are indeterminate functions of time, being dependent on the effects of the excavation, regional stresses, weathering and seasonal variations of groundwater and climatic conditions. These lead to fatigue, opening of cracks with irreversible deformations and progressive weakening of the rock mass. A rock slope which is stable when first excavated may become unstable with the passage of time because of gradual deterioration or adjustments in response to induced stresses.

The time required for development of deep seated failures in hard rocks is almost impossible to evaluate. Near surface failure, such as ravelling or rockfalls may develop shortly after excavation. Because local and overall stability may vary widely with the passage of time, it is of considerable importance that one recognize whether the analysis and ultimate design of the slope meet the requirements of short term and long term stability.

RESIDUAL AND INDUCED STRESS

The cut slope created by an excavation affects the stresses in a rock mass at the boundary of the excavation. Prediction of the magnitude of stresses and their effects on the stability of the slope are complex. Much of the theory relating to stress distribution in slopes and the manner in which stresses affect the stability has yet to be clearly demonstrated from observed field behaviour of actual slopes. Significant advances have been made by applying finite element and other numerical analysis techniques to predict the stress-strain characteristics of the rock mass in the slope. Application of these methods must consider the natural in situ stress in the rock mass as well as the stresses induced as a result of excavation.

EXISTING NATURAL AND EXCAVATED SLOPES

Slope design should take into account past experience with both stable and unstable slopes. Analyses of both natural and excavated slopes will provide clues to the way in which slopes may deform or instability may develop. Hence, they can provide valuable background information for excavation design, particularly in mountainous terrain. Many natural slopes are marginally stable having been subjected to erosion, fluctuation in groundwater conditions, seismic forces and adjustments of stress over a long period of time. Back analysis of such slopes can provide excellent information on the failure mechanics and strength parameters of the discontinuities as well as the rock mass.

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 documented. Slope monitoring can also be helpful when case histories are used in slope design.

DYNAMIC FORCES

The significance of earthquake induced vibrations are well documented. Consideration of these forces is important in areas susceptible to seismic activity. Consideration should also be given to dynamic forces due to blasting and the necessity for control of blasting during excavation to minimize disturbance to the rock.

BASIC STEPS AND APPROACH TO ANALYSIS AND DESIGN OF ROCK SLOPES

Analysis and design of rock slopes may be approached as a series of steps. Procedures are developed to obtain the necessary geological data, determine the required design parameters and prepare a rational design. By using a systematic and logical approach, the design of any structure in rock can be prepared based on reliable data and sound engineering principles. The basic steps and procedures in the analysis and design of rock slopes are listed in Table I and described in the following:

GEOLOGICAL DATA COLLECTION

The first step in analysis and design of rock slopes is to fully describe the rock units, the structural geology and the rock mass characteristics. The regional and local geology is assessed using available reports, government publications, air photos and other information. The mineralogy, lithology, physical characteristics and distribution of the various rock units are described. Major structural discontinuities such as faults, shears, contacts, etc. are mapped in detail and each feature is quantitatively characterized. Minor discontinuities such as joints, shears bedding, foliation, etc. are systematically measured over the study area to enable statistical assessment of discontinuity populations and the geotechnical parameters of each population. Statistical analyses and judgement indicate whether the best estimate has been made for the whole population.

TABLE I

BASIC STEPS IN THE ANALYSIS AND DESIGN OF ROCK SLOPES

1. GEOLOGICAL DATA COLLECTION
2. DETERMINATION OF STRUCTURAL DOMAINS
3. DETERMINATION OF DESIGN SECTORS
4. DEVELOPMENT OF A ROCK MASS MODEL
5. DETERMINATION OF MECHANICS OF SLOPE FAILURE
6. DETERMINATION OF STRENGTH AND HYDROGEOLOGY PROPERTIES
7. SLOPE STABILITY ANALYSES
8. PREPARATION OF SLOPE DESIGN

Geologic conditions vary from project to project, and thus a geology survey at one site may be entirely different from that at another. The best method of systematic collection of geological data is by detail line mapping methods described by Piteau (1971) and Herget et al (1977) or fracture set mapping as described by Call (1972). These methods involve systematic coverage of available exposures, slopes, etc.

Features that should be considered in the survey have been described by Piteau et al (1979). These consist of location, orientation, type, size and intensity of structural discontinuities which are essential to evaluate the mechanics of slope failure. Rock type, rock hardness, characteristics of infilling material, presence of voids and water, roughness and waviness are essential for assessing the strength and deformation properties and conducting stability analyses.

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: mapping of exposures, underground openings and trenches; drill core logging; terrestrial photogrammetry and aerial photograph interpretation; and various geophysical methods. Details of the various data collection methods are presented by and Hoek and Bray (1977) and Piteau et al (1979).

After the geological survey is completed, a detailed geologic map is constructed to show the major and minor structural features, general distribution of rock types and other relevant engineering geology features of the study area. The survey data are compiled, processed and the orientation, geometry and spatial distribution of the discontinuity populations are determined using a variety of statistical analysis techniques. Lower hemisphere equal area projections, stereographic projections or polar projections are most commonly used to plot, display and assess discontinuity data. These projections are easy to use and are of great benefit in evaluating and describing populations of structural discontinuities as described by Phillips (1971) and Priest (1985).

The geological survey can also be used to prepare assessments of the basic rock mass quality and competency using simple empirical techniques. A number of rock mass classification systems are available for this type of assessment as described by Rutledge (1977). In addition, simple empirical studies of rock mass quality based on documentation of existing natural or excavated slopes may be useful.

DETERMINATION OF STRUCTURAL DOMAINS

Rational geotechnical and rock mechanics assessments require that the rock mass be divided into areas with similar engineering geology, geological structure, strength and groundwater characteristics. The engineering behaviour of the rock mass can be expected to differ in areas which have different characteristics. Extrapolation of stability analysis results and slope design criteria is only valid within parts of the rock mass which have similar characteristics. Such areas are designated **structural domains.**

Structural domains are most often delineated based on the main rock units and structural discontinuity populations present in the rock mass. A typical example illustrating the selection of structural domains is shown in Fig. 2(a). Lower hemisphere projections are used to define the peak or average orientation and possible range of orientations of discontinuity populations within structural domains and identify structural domain boundaries. Orientation (strike and dip) of geological structures is the most important consideration in determining whether discontinuity populations within a structural domain or between structural domains are similar or dissimilar. Other parameters such as continuity (length), infilling, waviness, roughness, etc. are also considered in evaluating the engineering properties and nature of joint sets, but are less often used as a basis for designation of structural domains.

Boundaries of structural domains usually coincide with major geologic features, such as faults, shear zones, dykes, sills, geologic contacts, and unconformities. Once the boundaries of the various structural domains are defined, the discontinuity sets within each domain are delineated. The characteristics and properties of each joint set are defined.

DETERMINATION OF DESIGN SECTORS

Slope stability analysis and design requires prediction of the engineering geology and structural geology conditions on the excavated rock slope. Delineation of structural domain boundaries on the excavated rock slope is essential for detailed analysis and design.

It is also important to consider the orientation (dip direction and dip) of the excavated slope. Different slope orientations within a particular structural domain may require different design considerations. Hence, it is necessary to define **design** sectors which contain one structural domain and one general slope orientation.

If the orientation of the slope changes within a structural domain, two or more design sectors may result. Similarily, if two or more structural domains occur where the orientation of the pit wall is constant, two or more design sectors will result.

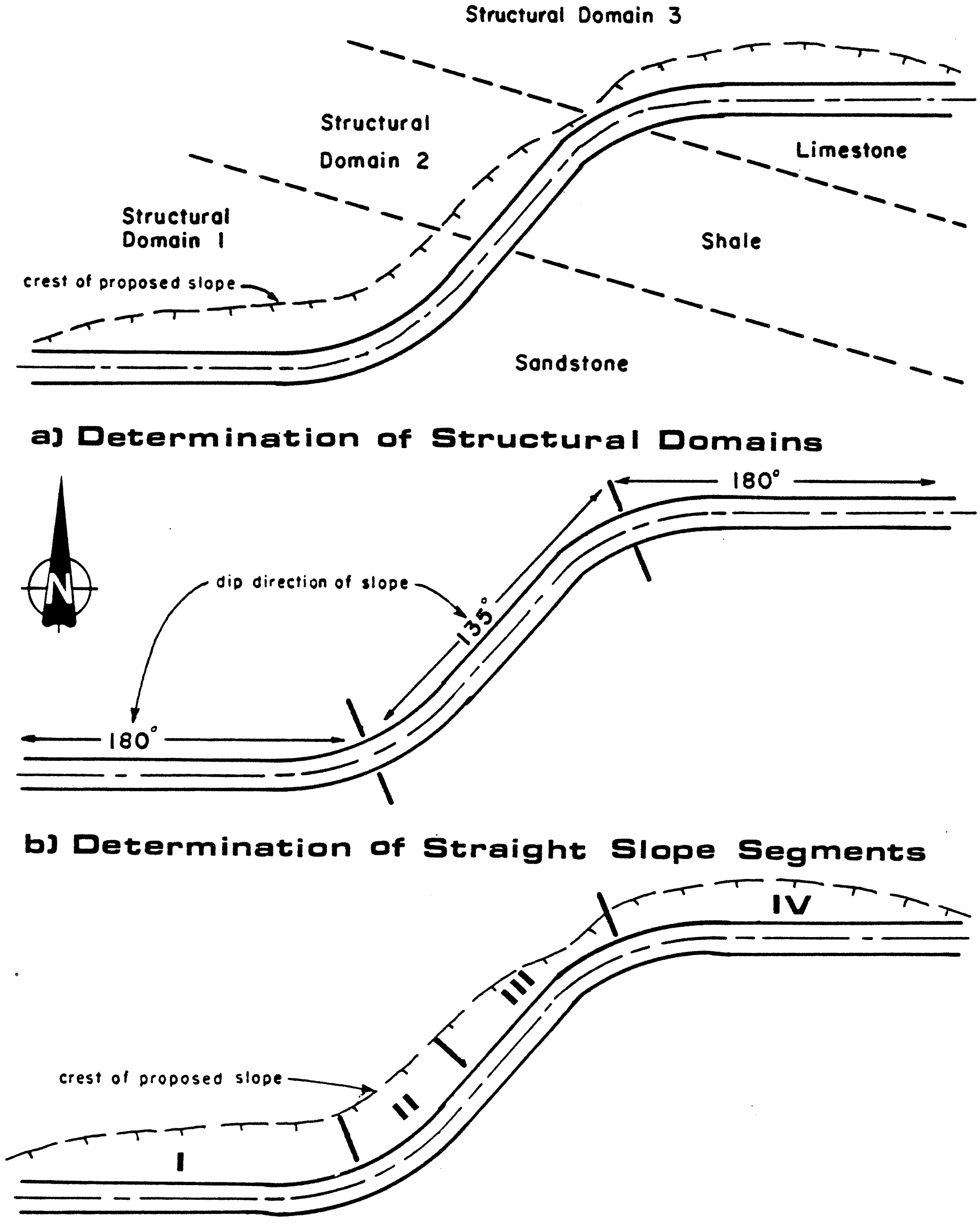


FIG. 2 Procedure for determination of design sectors for a hypothetical highway with south and southwest facing slopes in sedimentary rocks

Design sectors are determined by first dividing the proposed slope into straight slope segments (Fig. 2(b)). Boundaries of straight slope segments are superimposed on a plan of the structural domain boundaries and design sectors are determined as shown in Fig. 2(c)).

DEVELOPMENT OF A ROCK MASS MODEL

After the characteristics of the geologic structural population in each structural domain are defined, and design sectors have been determined, a model of the rock mass is developed to depict the three dimensional relations of the slope and geologic structure for each design sector. Some workers refer to the rock mass model as a schematic concept or structural picture of the rock mass. An essential requirement of the model is that it accurately represents the actual geologic structural populations and spatial relationships in a statistical sense. A graphical model using a lower hemisphere projection is often used. Extensions are often made to mathematical or physical models to determine whether the design sector boundaries selected are adequate or should be changed. Typical rock mass models are shown in Fig. 3.

DETERMINATION OF MECHANICS OF SLOPE FAILURE

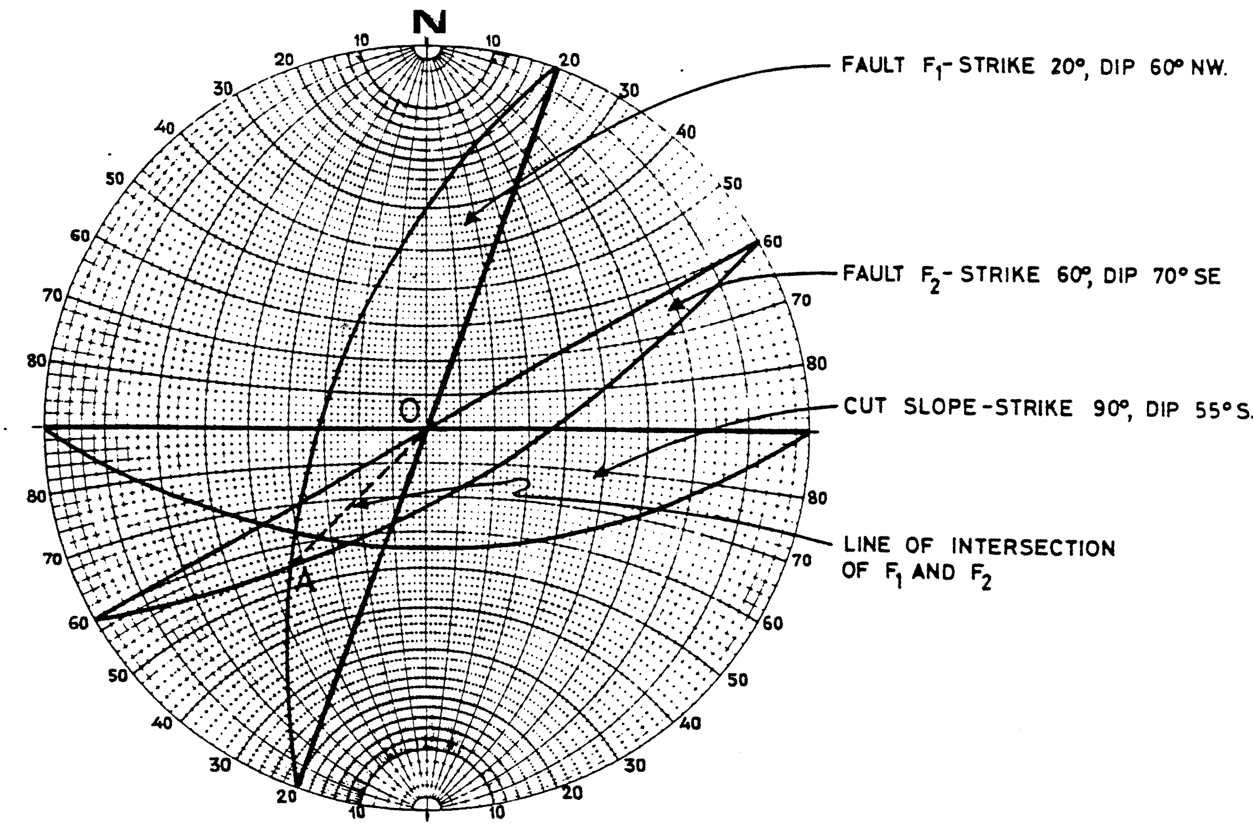
The rock mass model for each design sector and used as a basis to determine the mechanics of slope failure. The main mechanisms of rock slope failure as shown in Fig. 4 have been presented by Hoek and Bray (1977), Piteau and Martin (1981) and others.

Because discontinuities are considered to be inherently weaker than the rock material, assessments are carried out to define all kinematically possible failure modes involving sliding or separation along discontinuities. The main mechanics of failures involving discontinuities are planar sliding, stepped planar sliding, toppling and wedge failure as summarized in Fig. 4(a) to (h).

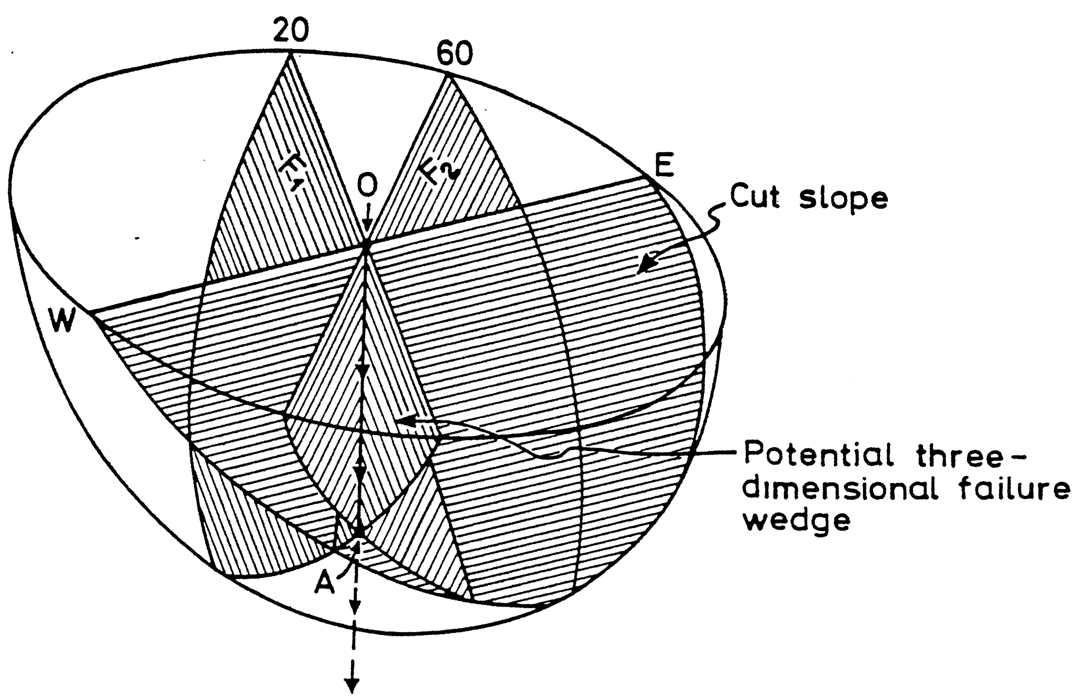
Kinematically possible failures involving major discontinuities which could affect the entire slope are most important in terms of overall slope stability. Also of importance are failures involving various combinations of major discontinuities and/or minor discontinuities which could lead to smaller failures involving a portion of the slope or which could significantly influence the local stability and safety of the slope.

It is also important to assess the potential for small failures, rockfalls and general ravelling (Fig. 4(i)). These types of failures may develop as a result of a highly fractured rock mass, deterioration (weathering or alteration) of the rock, effects of freeze-thaw, blasting damage, etc. In many cases the overall slope design is controlled by the necessity to control rockfalls and ravelling to ensure adequate safety of the highway or railway at the toe of the slope.

In addition to failures involving discontinuities, it may also be important to assess the stability of the slope with respect to "deep seated" failure of the rock mass. Such failures may occur along a complex failure surface formed in part along structural discontinuities and in part through intact rock (Fig. 4(j) and (k)). In such cases, an estimate of the rock mass shear strength and deformational properties are required to adequately model the slope and failure



a) Graphical model on a lower hemisphere projection showing orientation and angular relationships of discontinuities.



b) Three dimensional isometric model

FIG. 3 Typical rock mass models (after Piteau and Peckover, 1978).

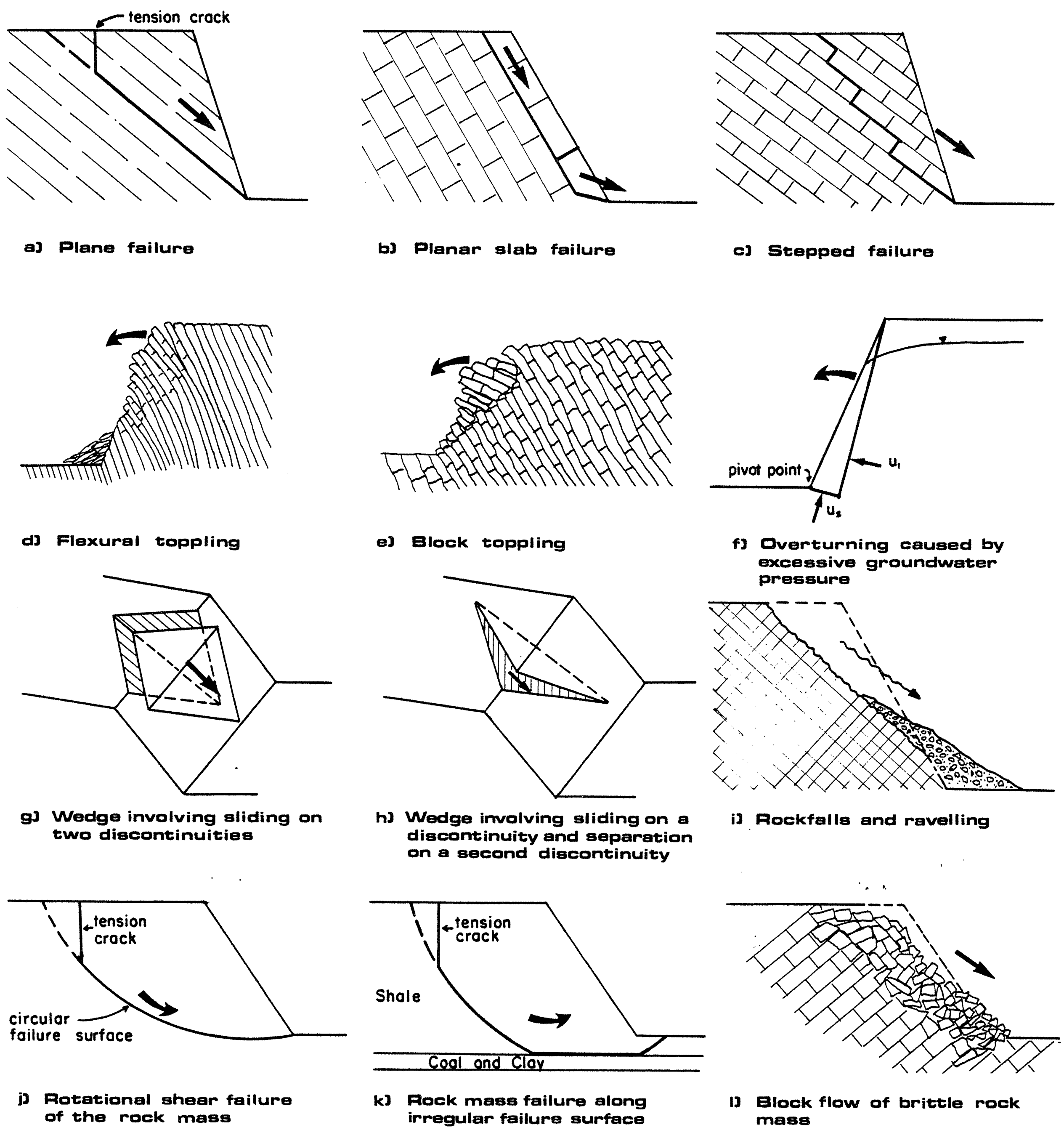


Fig. 4 Typical Mechanisms of Rock Slope Failure

mechanism. This type of failure mechanism is generally considered only in the case of very high steep slopes, slopes in weak or highly fractured rock masses or slopes where soft rock may be squeezed or removed in the toe of the slope allowing subsequent deformation of the entire slope. Possible block flow failure mechanisms (Fig. 4(l)) should also be considered for high slopes in relatively "brittle" rock masses.

DETERMINATION OF STRENGTH AND HYDROGEOLOGY PROPERTIES

Once the failure mechanisms which are considered to control the slope design in each design sector have been defined, the necessary shear strength and hydrogeological conditions considered to influence the stability should be determined. Appropriate laboratory testing of strength and field assessments of groundwater conditions and hydrogeological parameters should be conducted for the specific failure mechanisms or discontinuities considered to control failure.

In practice, it is often not practical to conduct a separate investigation of the strength and hydrogeological properties of the specific features after the bulk of the field data collection has been conducted. Hence, laboratory testing and hydrogeological investigations are often conducted in an earlier stage of the project. The designer must be aware of the problems with this approach. If these investigations are conducted before the failure mechanisms are fully understood, additional investigations and testing may be required before rational stability analyses can be attempted. In many instances, unnecessary testing is carried out on structures or in areas where information is not needed for the failure mechanism which controls the slope.

SLOPE STABILITY ANALYSES

Detailed stability analyses are conducted for the failure mechanisms determined for each design sector using the basic rock mass model, failure geometry and strength and hydrogeological parameters defined from the procedures described above. Limit equilibrium analysis techniques are commonly applied to failures involving discontinuities or rock mass failure. A factor of safety is calculated by a number of possible methods and the sensitivity of the factor of safety to variation in slope parameters is assessed. Analysis methods may be considerably more detailed for high rock slopes than for shallow rock slopes.

Analysis methods have been presented by numerous workers and have been summarized by Goodman (1976), Hoek and Bray (1977) and Piteau and Martin (1981) and others. The reader is referred to these or other specific publications for details of analysis methods for individual failure mechanisms.

Analysis of many rock slopes may require only an evaluation of the orientation of geologic structure with respect to the geometry and alternative slope angles of the proposed excavation. Lower hemisphere projections and simple analysis techniques are used to identify the failure modes and indicate the degree of stability. Design requirements are often determined from these basic analyses. If detailed rigorous analyses are required, these can be performed for specific failure mechansims, as required.

PREPARTION OF SLOPE DESIGN

Slopes are designed based on the results of the stability analyses and assessment of failure mechanisms for each design sector with due consideration of the sensitivity of the various parameters and possible other influences. The geotechnical engineer must input a large degree of judgement and engineering experience when evaluating the various failure mechanisms. If analyses have been carried out systematically, the design choices will be straight forward and follow logically from the results of the study. In the majority of cases, several possible failure mechanisms will be assessed and the optimum design selected.

For example, a particular slope may be subject to large wedge failures involving major faults as well as smaller steeper wedges which will affect local stability. The major wedges may be controlled by comprehensive depressurization using drainholes and diversion of surface water away from the slope. Hence, the slope design would be based on the requirements for the smaller steeper wedges, provided adequate surface water and groundwater controls are installed.

Designs for individual design sectors are expected to vary. Design of final slopes must include zones of transition between the slope designs of adjacent design sectors. Blending of designs for the entire slope will require engineering judgement and experience to achieve optimum results.

CASE HISTORY OF RAILWAY SLOPE DESIGN

The following case history provides an illustration of the steps and analyses discussed above. The project site consists of a throughcut and sidehill cut for a railway in mountaineous terrain along a river in northern British Columbia as shown in Fig. 5. The study extends from station 10+00 to 36+00. Maximum height of cuts is about 27m.

The investigation consisted of geological mapping of available bedrock and soil exposures at the site followed by structural analyses, stability analyses and slope design.

ENGINEERING GEOLOGY

Bedrock in outcrops and road cuts in the study area consists almost entirely of dark green porphyritic andesite of volcanic origin. The rock consists of a very fine grained ground mass with phenocrysts of greenish feldspars up to 2mm in diameter. Occasional inclusions of soft white zeolites were also noted. The rock is uniformly hard, having an estimated unconfined compressive strength greater than 70 MPa. Very little surface weathering was noted although the rock in many outcrops has a network of closely spaced, healed fractures. Andesite adjacent to fault zones is broken, altered and generally low strength.

Several major faults have been mapped or inferred along the grade (see Fig. 5). In many cases bedrock depressions are indicated to contain fault zones which may be composed of weaker, altered andesite.

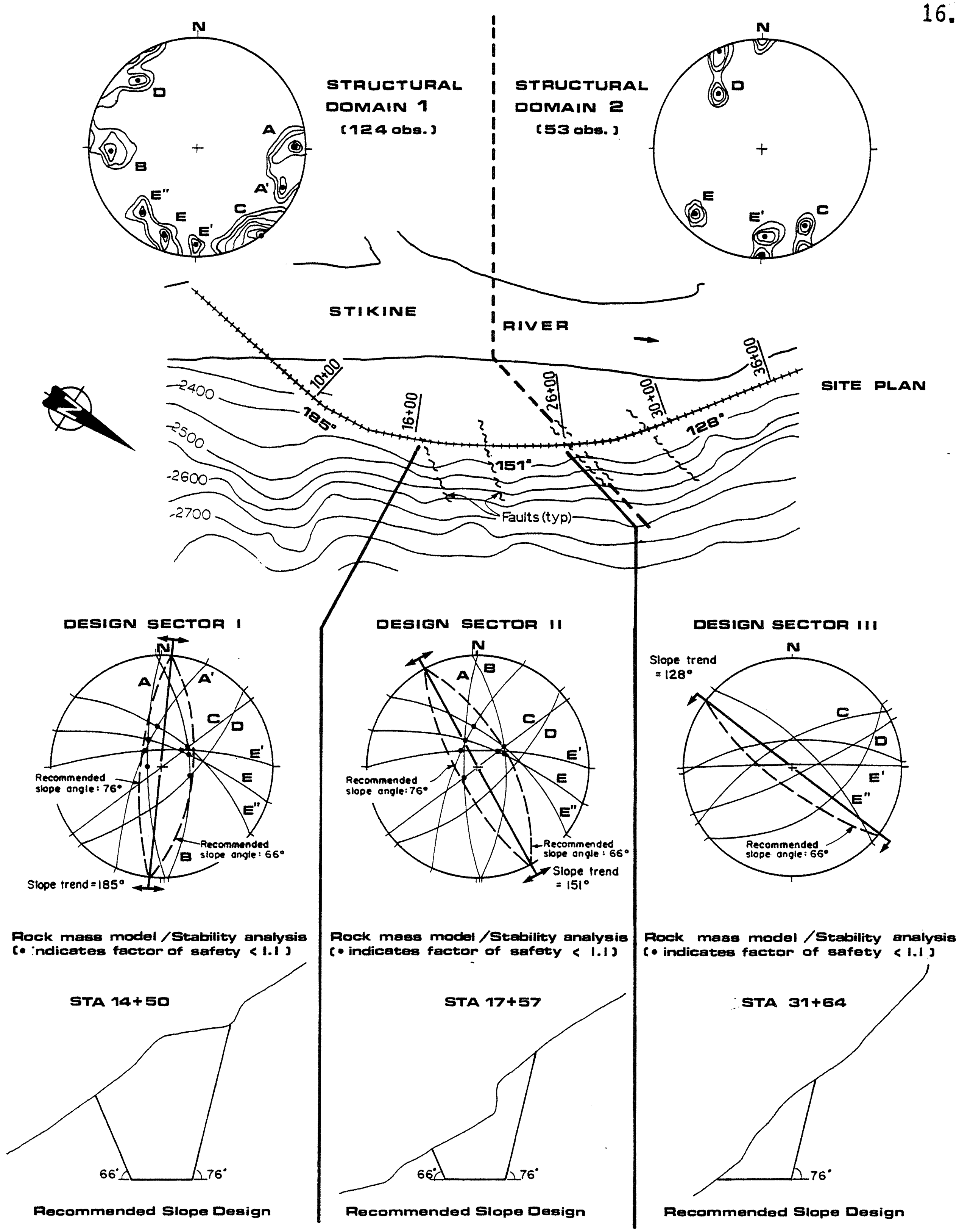


Fig. 5 Summary of engineering geology, structural geology, stability analysis results and slope design recommendations for throughcut and side hill cut in andesite for a railway in northern British Columbia

Examination of the discontinuity populations was conducted by preparing lower hemisphere equal area projections of the mapping data. Based on these analyses, the rock mass was divided into two structural domains (i.e. Structural Domains 1 and 2 as shown in Fig. 5). Lower hemisphere projections of discontinuity data for each structural domain are shown. The number, orientation and intensity of the joint sets are clearly different between each structural domain.

DESIGN SECTORS, ROCK MASS MODELS AND STABILITY ANALYSES

The proposed alignment is on a continuous curve. Slopes have been divided into three straight slope segments with slope trends of 185°, 151° and 128°, respectively (see Fig. 5). The combination of straight slope segments and structural domains results in three design sectors (Design Sectors I, II and III). Slopes on either side of the throughcut were considered separately within each design sector.

Lower hemisphere equal area projections of planes representing the peak orientation of each discontinuity set were used to prepare a rock mass model for each design sector (see Fig. 5). These projections were used to identify the modes of failure considered to be kinematically possible with regard to plane and wedge failure. In this assessment, each discontinuity set was considered individually and in combination with all other discontinuity sets.

Simple limit equilibrium stability analyses were conducted to determine the factor of safety against failure for each possible failure mode. The analyses were conducted assuming a drained (dry) slope. The friction angle and cohesion along discontinuities were assumed to be 50° and 0 kPa, respectively. The relatively high friction angle applied is considered appropriate due to the observed rough and tight nature of the bulk of the discontinuities mapped, and provided adequate blasting control measures are used to ensure minimal disturbance of the discontinuity surfaces.

The factor of safety for each failure mode considered to be kinematically possible is summarized in Table II. For the cases where F is less than 1.1, the slope angle which is required to prevent failure is determined from the lower hemisphere projections and recorded as shown in Table II.

SLOPE DESIGN

The slope design for each design sector is controlled by those failure modes which have a factor of safety less than 1.1. The failure mode which requires the shallowest slope angle to avoid undercutting the failure on the slope is considered to control slope stability. This case and the safe slope angle required to avoid undercutting the failure are shown on the equal area projections in Fig. 5 and summarized in Table II.

The stability analyses results indicate that the optimum slopes on the uphill side of the throughcut in Design Sectors I and II vary from 75° to 79° depending on the location and slope orientation. Based on these results, it appears that slopes of 76° (1/4:1) are appropriate for design. Optimum slopes on the downhill side of the throughcut vary from 66° to 68°. Hence, 66° slope angles are appropriate for design.

TABLE II

SUMMARY OF STABILITY ANALYSIS AND SLOPE DESIGN RECOMMENDATIONS FOR THROUGHCUT AND SIDEHILL CUT

DESIGN SECTOR (LOCATION)	SLOPE PARAMETERS			KINEMATICALLY POSSIBLE FAILURE MODES			STABILITY ANALYSIS		RECOMMENDED SLOPE ANGLE (°)
	SLOPE DESCRIPTION	DIP DIRECTION (°) (TREND IN BRACKETS)	MAXIMUM SLOPE HEIGHT (m)	FAILURE TYPE	DISCONTINUITY SETS INVOLVED	AVERAGE DIP OF FAILURE PLANE OR PLUNGE OF THE WEDGE (°)	FACTOR OF SAFETY	SLOPE ANGLE REQUIRED TO AVOID UNDERCUTTING UNSAFE PLANES OR WEDGES (°)	
I Station 10+00 to 16+00	West facing (Uphill) side of throughcut	275 (185)	27	Plane	A (C)	79	<1.0	79	76 (¼:1)
				Plane	A'(C)	75	<1.0	75*	
				Wedge	A and D	56	1.79	-	
				Wedge	A' and D	33	2.86	-	
				Wedge	A and E, E', E''	54 - 74	.48-1.31	76	
				Wedge	A' and E, E', E''	60 - 73	.48-.77	78	
	East facing (downhill) side of throughcut	95 (185)	11	Wedge	A' and B	36	4.53	-	66
				Plane	B (C)	69	<1.0	69	
				Wedge	B and D	68	.60	68*	
				Wedge	B and E, E', E''	66 - 68	.60-.72	70	
				Wedge	C and E, E', E''	66 - 71	.77	71	
				Wedge	D and E, E', E''	53 - 59	1.13-1.91	-	
II Station 16+00 to 26+00	West facing (uphill) side of throughcut	241 (151)	20	Wedge	A and C	78	.54	78*	76 (¼:1)
				Wedge	A and D	56	1.79	-	
				Wedge	A and E'	74	.48	86	
				Wedge	A' and C	61	2.09	-	
				Wedge	A' and D	33	2.86	-	
				Wedge	A' and E'	74	0.48	84	
	East facing (downhill) side of throughcut	061 (151)	7	Wedge	A and E''	54	1.31	-	66
				Wedge	A'and B	36	4.53	-	
				Wedge	A' and E, E''	60 - 70	.48-.77	76	
				Wedge	B and C	65	.77	66*	
				Wedge	B and D	68	0.60	74	
				Wedge	B and E, E', E''	66 - 68	.77	66	
				Wedge	C and E, E', E''	66 - 71	.77	66	
				Wedge	D and E, E', E''	53 - 59	1.13-1.91	-	
III Station 26+00 to 36+00	Southwest facing sidehill cut	218 (128)	17	None	Average dip of random south-west dipping joints is 78°		Slope angle of 76° recommended to minimize ravelling and rockfalls		76 (¼:1)

NOTE: Slope angles indicated by an asterisk (*) are the maximum safe slope angles indicated from the stability analyses. Recommended slope angles are based on these results and designs which are considered feasible for construction.

The stability analyses for Design Sector III indicates no kinematically possible failure modes. However, general ravelling, small rockfalls, etc. could develop. The average dip of random southwest dipping joints which could combine to small failures was determined to be 78°. Based on these considerations and the slope angles determined for the other design sectors, 76° (1/4:1) slopes are considered appropriate in Design Sector III.

The recommended design slope angles for each design sector are shown in Fig. 5. These slope angles apply to hard, unaltered bedrock. Soil slopes should be trimmed to safe angles based on experience in the area. In some areas the bedrock/overburden profile may require complete removal of soil in bedrock depressions above the rock cut. Fault zones may require specialized treatment to control rockfalls ravelling and small failures.

Controlled blasting is mandatory to maintain the maximum shear strength along discontinuities and to minimize future development of small rockfalls, ravelling etc. In addition, careful assessment of groundwater conditions and possible installation of drainholes may be required. Proper ditching to divert surface flows away from the slopes is also mandatory. Thorough scaling of loose, overhanging or protruding blocks would also be required as excavation proceeds. With thorough scaling, subsequent maintenance and remedial work can be greatly reduced.

CONCLUSIONS

Analysis and design of rock slopes requires a detailed knowledge of geology, engineering and rock mechanics. By following a systematic procedure and steps involving data collection, analysis and design a logical approach to slope design is developed. Hence, design choices are simplified, problems are identified and a rational design can be achieved.

Any rock slope design must account for possible unforseen variation in rock conditions and behaviour. As such, the design should have the flexibility for modification as required in the field during construction. In this regard, close field supervision, construction documentation and field control are essential to evaluate the design and modify construction procedures, as required.

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