International Society
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6

Rock Reinforcement and Rock Support

Ömer Aydan

CRC Press
Taylor & Francis Group
A BALKEMA BOOK

Rock Reinforcement and Rock Support

ISRM Book Series
Series editor: Xia-Ting Feng
Institute of Rock and Soil Mechanics, Chinese Academy of Sciences, Wuhan, China

ISSN : 2326-6872
eISSN: 2326-778X

Volume 6

International Society for Rock Mechanics

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Ömer Aydan

Department of Civil Engineering, University of the Ryukyus, Nishihara, Okinawa, Japan

CRC Press
Taylor & Francis Group
Boca Raton London New York Leiden

CRC Press is an imprint of the
Taylor & Francis Group, an **informa** business
A BALKEMA BOOK

Cover illustration: Okumino Underground Power House. Courtesy of Chubu Electric Power Company.

CRC Press/Balkema is an imprint of the Taylor & Francis Group, an informa business

Typeset by Apex CoVantage, LLC
Printed and Bound by CPI Group (UK) Ltd, Croydon, CR0 4YY

Library of Congress Cataloging-in-Publication Data
Names: Aydan, Ömer, author.
Title: Rock reinforcement and rock support / Ömer Aydan, University of Ryukyus Dept. of Civil Engineering.
Description: Boca Raton : CRC Press/Balkema, 2017. | Series: ISRM book series, ISSN 2326-6872 ; volume 6 | Includes bibliographical references and index.
Identifiers: LCCN 2017045063 (print) | LCCN 2017045840 (ebook) | ISBN 9781315391304 (ebook) | ISBN 9781138095830 (hardcover : alk. paper)
Subjects: LCSH: Rock mechanics. | Engineering geology.
Classification: LCC TA706 (ebook) | LCC TA706 .A9295 2017 (print) | DDC 624.1/5132—dc23
LC record available at https://lccn.loc.gov/2017045063

ISBN: 978-1-138-09583-0 (Hbk)
ISBN: 978-1-315-10420-1 (eBook)

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About the author

Born in 1955, Professor Aydan studied Mining Engineering at the Technical University of Istanbul, Turkey (B.Sc., 1979), Rock Mechanics and Excavation Engineering at the University of Newcastle upon Tyne, UK (M.Sc., 1982), and finally received his Ph.D. in Geotechnical Engineering from Nagoya University, Japan, in 1989. Prof. Aydan worked at Nagoya University as a research associate (1987–1991), and then at the Department of Marine Civil Engineering at Tokai University, first as Assistant Professor (1991–1993), then as Associate Professor (1993–2001), and finally as Professor (2001–2010). He then became Professor of the Institute of Oceanic Research and Development at Tokai University and is currently Professor at the University of Ryukyus, Department of Civil Engineering & Architecture, Nishihara, Okinawa, Japan. He has furthermore played an active role on numerous ISRM, JSCE, JGS, SRI, and Rock Mech. National Group of Japan committees and has organized several national and international symposia and conferences. Professor Aydan has received the 1998 Matsumae Scientific Contribution Award, the 2007 Erguvanlı Engineering Geology Best Paper Award, the 2011 Excellent Contributions Award from the International Association for Computer Methods in Geomechanics and Advances, and the 2011 Best Paper Award from the Indian Society for Rock Mechanics and Tunneling Technology, and he was awarded the 2013 Best Paper Award at the 13th Japan Symposium on Rock Mechanics and the 6th Japan-Korea Joint Symposium on Rock Engineering. He was also made Honorary Professor in Earth Science by Pamukkale University in 2008 and received the 2005 Technology Award, the 2012 Frontier Award, and the 2015 Best Paper Award from the Japan National Group for Rock Mechanics.

Acknowledgments

The author sincerely acknowledges Prof. Xia-Ting Feng for inviting the author to contribute to the ISRM Book Series on "Rock Reinforcement and Rock Support". The content of this book is an outcome of the studies carried out by the author at Nagoya University, Tokai University, and University of the Ryukyus in Japan over more than three decades. The author would like to thank Emeritus Prof. Dr Toshikazu Kawamoto of Nagoya University, Prof. Dr Yasuaki Ichikawa of Okayama University, Prof. Dr Yuzo Obara of Kumamoto University, Prof. Dr Takashi Kyoya of Tohoku University, Prof. Dr Tomoyuki Akagi and Prof. Dr Takashi Ito of Toyota National College of Technology, Assoc. Prof. Dr Naohiko Tokashiki of University of the Ryukyus, Dr Seiji Ebisu of Okumura Corporation, and his deceased friends Dr Mitsuhiro Sezaki of Miyazaki University and Mr Jun Itoh of Oriental Consultants Global Co. Ltd for their guidance, help, and suggestions at various stages of his studies quoted in this book.

The author also acknowledges particularly Dr Y. Miyaike, Mr S. Tsuchiyama, Mr H. Okuda, Dr S. Komura and Mr F. Uehara of Chubu Electric Power Company (Nagoya, Japan) for collaboration with the author in accessing many sites and providing financial funds to develop a dynamic direct shear testing device used in conventional, cyclic, creep, and dynamic behavior of interfaces of rockbolts and rockanchors and non-destructive equipment for the inspection of rockbolts/rockanchors utilized in the studies quoted in this book. He also acknowledges his former students, Mr Mitsuo Daido and Dr Yoshimi Ohta of Tokai University, for their help during experiments and computations reported in this study.

The author would also like to thank Alistair Bright, acquisitions editor at CRC Press/Balkema, for his patience and collaboration during the preparations for this book and Ms José van der Veer, the production editor, for her great efforts in producing this book.

Finally, I want to thank my wife, Reiko, my daughter, Ay, my son, Turan Miray, and my parents for their continuous help and understanding, without which this book could not have been completed.

Chapter 1

Introduction

The stability of underground and surface geotechnical structures during and after excavation is of great concern to designers, as any kind of instability may result in damage to the environment, as well as high repair costs and time consumption (Figs. 1.1–1.4). The rock in nature is not always continuous and may have numerous discontinuities that vary in scale. As a result, the safety evaluation of a structure under consideration is a highly complex problem and requires very careful investigation. Accordingly, it is always necessary to examine the most likely forms of instability in relation to the physical nature of the rock mass and the geometry of the structure and its site, as well as the pre-existing state of stress. The forms of instability and their mechanism and the factors and conditions associated with them must be clearly understood to correctly stabilize the structure.

Figure 1.1 Various underground structures in rock.

Figure 1.2 Tunnels in rock.

Figure 1.3 Foundations on rock.

Figure 1.4 Rock slopes.

In addition to the stability problems, the environmental requirements and functional duties of structures may need to be carefully evaluated. All these factors together with those related to the stabilization procedure will result in setting the conditions for the selection of support members that satisfy mechanical as well as environmental and functional requirements.

The design of support members and the evaluation of the stability of structures are not possible unless one understands what rock mass really is. Most of the available approaches are either mechanically orientated without proper consideration of rock mass or geologically orientated without paying proper attention to the mechanics. In this respect, the present volume attempts to bridge the two approaches and bring a unified approach for the design of support and reinforcement systems for rock engineering structures, from not only the mechanical engineering but also the geological engineering point of view.

Rockbolts of various types (i.e. mechanically anchored, grouted, etc.) have recently become one of the principal support members in the civil and mining engineering fields. This probably results from the ease of their transportation, storage, and installation and their rapidly developing reinforcement effects as compared with other support members, such as steel sets and concrete liners. Their superior reinforcement effects in securing the stability of geotechnical engineering structures excavated in various types of ground and states of stress are very well known qualitatively in engineering practice. However, the first fundamental study for quantifying the reinforcement effects of rockbolts has been carried out by

Aydan (1989) in his doctorate study. Subsequent studies by Pellet (1994); Moosavi (1997); Marence and Swoboda (1995) and Ebisu *et al*. (1994a, 1994b) have made further contributions on the behavior of rockbolts under different conditions. The studies on rockbolts, cable rockbolts, and rockanchors are now orientated towards their response under dynamic conditions (e.g. Aydan *et al*., 2012; Owada *et al*., 2004; Owada and Aydan, 2005; Li, 2010).

In the last decade, the use of shotcrete has rapidly increased, particularly in tunnel construction, and shotcrete has become an important element of modern tunnel-support techniques. The development of the early age strength of shotcrete is a decisive factor, because the excavation cycle and attainable excavation speeds are significantly influenced by it. The first fundamental study on the characteristics of shotcrete and its representation in numerical simulations was undertaken by Sezaki (1990) and his colleagues (Sezaki *et al*., 1989, 1992; Aydan *et al*., 1992).

Steel ribs or steel sets have long been used in many rock excavations. Their design concept is based as a moment-resisting structure under uniform or concentrated loads, and their load-bearing capacity is evaluated by assuming moment resistance capacity or buckling failure.

Despite decades of use of concrete liners in rock excavations, the supporting effects of concrete liners is not well understood. This is due to a poor understanding of how they interact with the surrounding rock mass, together with the incorporation of other support and reinforcement members and in relation to the installation stage in the overall construction scheme. The concrete liners are auxiliary support members rather than main load-bearing structures. Therefore, there is a strong debate whether they are necessary support members. In this book, various aspects of concrete liners are also presented and discussed.

The present book has been undertaken to highlight the reinforcement functions of rockbolts/rockanchors and support systems consisting of shotcrete, steel ribs, and concrete liners under various conditions and to evaluate their reinforcement and supporting effects, both qualitatively and quantitatively.

The book consists of 12 chapters. The contents of 10 chapters out of 12 are described briefly as follows:

Chapter 2 is devoted to the mechanism and influencing factors of failure phenomena in rock engineering structures. The rock and types of discontinuities encountered in natural rock are briefly described, and their combined effects on the mechanical response of rock mass as a structure are discussed together with the implications on real rock structures. Then, classifications on the forms of instability in underground openings, slopes, and foundations, under both compressive and tensile stress fields, are described in relation with the structure of rock mass.

Chapter 3 is concerned with the present design philosophy of support and reinforcement for rock engineering structures. A brief description of available design approaches, such as empirical, analytical, and numerical methods, are given and discussed. The approaches, which are used independently of each other, are presented in a unified manner. The presently available support members and their functions are briefly described and discussed, with an emphasis on rockbolts and rockbolting.

Chapter 4 describes experimental studies undertaken on the mechanical behavior of the rockbolt system. First, the behavior of the bolt material used in practice is given, then the experimental study undertaken for the anchorage performance of rockbolts in push-out and pull-out tests and subsequent shear tests on the mechanical behavior of interfaces within the system and grouting material are described. In this chapter, the constitutive laws for the rockbolt system are described. A constitutive law for the bar is derived based on the classical incremental elasto-plasticity theory, as bar materials such as steel exhibit a non-dilatant plastic behavior. On the other hand, the constitutive law for the grout annulus and interfaces is derived based on the multi-response theory proposed by Ichikawa (Ichikawa, 1985; Ichikawa *et al.*, 1988), as the grout annulus and interfaces exhibit a dilatant plastic behavior. Then, procedures to determine the parameters for the constitutive laws from the experimental data are described and several examples are given. Evaluation of the contribution of rockbolts/rockanchors for improving the properties of rock mass is described and the shear reinforcement effect of rockbolts on rock discontinuities is presented in view of some theoretical and experimental findings. A detailed presentation of estimation of pull-out capacity of rockbolts/rockanchors under various conditions are described. Furthermore, the evaluation of reinforcement effect of mesh bolting on rock masses subjected to tensile stresses are presented.

Chapter 5 describes the characteristics of various support elements, such as shotcrete, concrete liner, and steel ribs/sets. The constitutive laws of each support member and various experimental studies on their characteristics are presented. Furthermore, the concepts for their mechanical modeling are also explained.

Chapter 6 describes the models representing reinforcement and support systems in numerical analyses, particularly in finite element studies. Details of rockbolt elements, shotcrete, and beam elements are presented.

Chapter 7 is concerned with the analytical and numerical methods for evaluating support and reinforcement systems and their effects in underground excavations. Analytical methods for evaluating the ground-response-support reaction, which incorporates various support members, rockbolts, and rockanchors, and the face effect are presented, and several examples of applications are given. Furthermore, a theoretical formulation of the effect that mesh bolting has for compressed air energy storage schemes is given, and several examples of excavations are presented. A series of finite element simulations are presented to show the effects of various conditions for the effective utilization of reinforcement and support systems for underground structures. The effect of rockbolting with other support members is investigated in relation to some practical situations. Several examples are analyzed on the response of rockbolts in discontinuum, and their implications for interpreting field measurements of rockbolt performances are discussed. Furthermore, the presently available proposals on the suspension effect, the beam building effect, and the arch formation effect of rockbolts are re-examined and more generalized solutions are presented. In addition to covering the reinforcement effect of rockbolts against the sliding type of failure, solutions for the reinforcement effect of bolts against the flexural and columnar type of toppling failure are given.

Chapter 8 describes the effect of support and reinforcement systems for the stabilization of rock slopes. Procedures for stabilizing the rock slopes against some typical failure modes are presented, along with several examples of applications. Furthermore, the chapter presents applications of the discrete finite element method, incorporating the effect of rockbolts to rock slope stability problems. In addition, model experiments on the effect of rockbolting against planar sliding and block-toppling modes are given and compared with estimations from the limit equilibrium technique.

Chapter 9 is concerned with the stabilization of the foundations of bridges, pylons, and dams subjected to tension or compressive forces. Examples of applications include the potential use of rockanchors as foundations of pylons and of tunnel-type anchorage for suspension bridges. The use of rockanchors for the stabilization of bridge and dam foundations under compression is also presented and discussed.

Chapter 10 deals with dynamic issues such as rockburst, earthquakes, and blasting, which cause dynamic loads on rock support and rock reinforcement. Theoretical, numerical, and experimental studies on rockbolts and rockanchors under shaking are presented, along with several examples of applications.

Chapter 11 describes the mechanisms and techniques for evaluating corrosion in steel and iron materials in relation to the long-term performance and degradation of reinforcement and support systems and provides site examples. Furthermore, some procedures are presented for non-destructive evaluation of support and reinforcement systems.

Chapter 2

Mechanism of failure in rock engineering structures and its influencing factors

This chapter deals with natural rock, the types of discontinuities encountered in it, rock mass, and the mechanism of the modes of instability in underground and surface structures and associated factors and conditions.

The first part of this chapter is devoted to the geological description of rocks and of the formation and types of discontinuities in rocks and rock mass. Then, the mechanical behavior of rock mass is discussed, considering the behaviors of intact rock, discontinuities, and the structure of the rock mass.

In the second part of the chapter, the discussion of various modes of instability of rock engineering structures and the factors associated with the modes of instability are presented. Then, Aydan's classifications for the modes of instability in rock engineering structures are presented in relation to the elements associated with the modes of instability (Aydan, 1989).

2.1 ROCK, DISCONTINUITIES, AND ROCK MASS

2.1.1 Rocks

Rocks in nature can be geologically classified into three main groups: igneous, sedimentary, and metamorphic, and each of these groups may be further subdivided into several classes. For example, igneous rocks are subdivided into three classes: extrusive, intrusive, and semi-intrusive, although the chemical composition of the three types may be same (Fig. 2.1). The order of minerals and the internal structure of rocks is a result of the chemical composition of rising magma, its velocity, and the environmental conditions during the cooling process, which greatly affects the discontinuity formation in such rocks.

Sedimentary rocks, on the other hand, result from the accumulation of particles differing in size, shape, and chemical composition in some certain geographical locations and a rebonding through certain physical or chemical agents or processes under various thermo-hydro environmental physical conditions (Fig. 2.2). The rocks belonging to this group are usually found in the form of layers, and the orientation of grains or minerals have some regularity in relation to the sedimentation process.

Metamorphic rocks are the result of the restructuring of existing rocks, which may be sedimentary, igneous, or even metamorphic under high pressures and/or high temperatures (Fig. 2.3). Because of high pressures and temperatures, the internal structure of rocks becomes highly anisotropic.

Figure 2.1 Views of some igneous rocks.

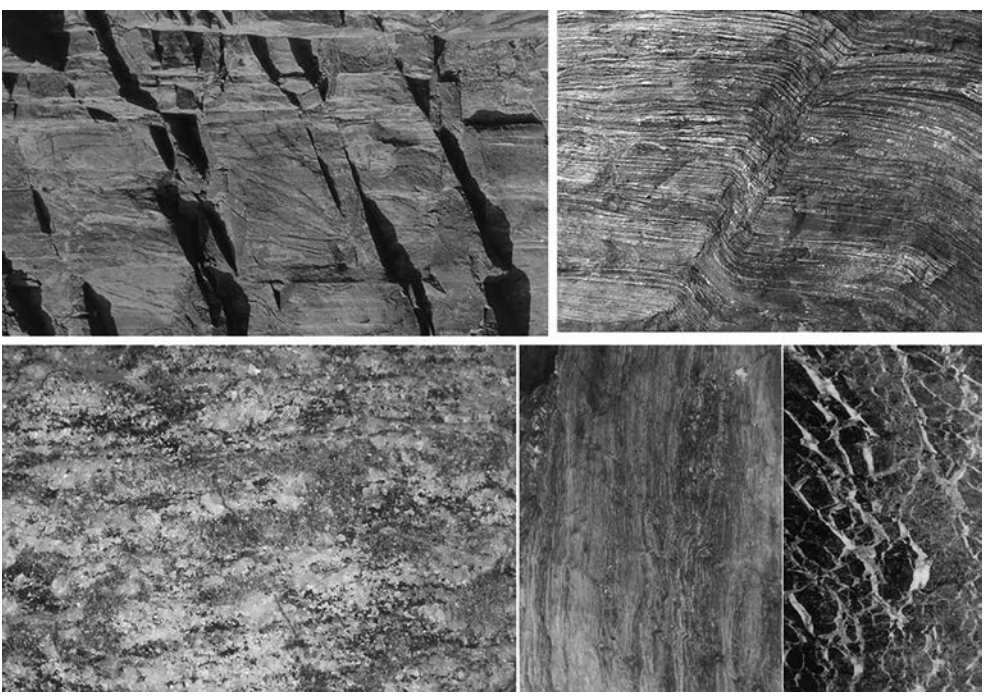


Figure 2.2 Views of some metamorphic rocks.

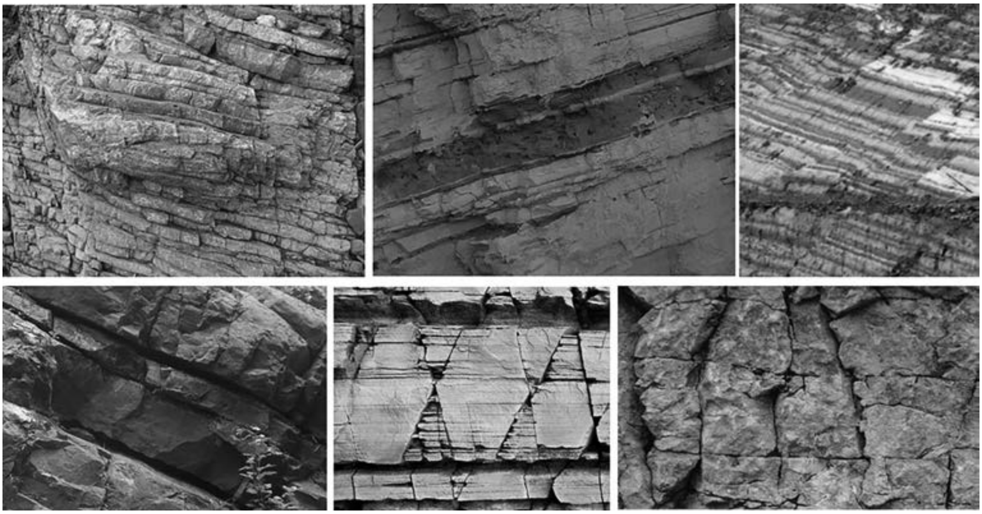


Figure 2.3 Views of some sedimentary rocks.

All rocks are an assemblage of a single mineral or several minerals of regular or irregular shapes differing in size and arranged in certain patterns, depending on the chemical and thermal phase changes and physical conditions at the time of their occurrence. The mechanical behavior of rocks is an apparent behavior of the mechanical response of minerals or grains and the interaction taking place among the grains due their shape and spatial distributions in relation to the applied constraint and force conditions.

2.1.2 Origin of discontinuities in rock and their mechanical behavior

Discontinuities in rocks are termed cracks, fractures, joints, bedding planes, schistosity, or foliation planes and faults. Discontinuities are products of certain phenomena the rocks were exposed to in their geological past and are expected to be regularly distributed within a rock mass. They can be classified into the four groups outlined below according to the mechanical or environmental process they underwent (Erguvanlı, 1973; Yüzer and Vardar, 1983; Miki, 1986; Ramsay and Huber, 1987; Aydan *et al*., 1988b, etc.) (Fig. 2.4).

i) Tension discontinuities due to

- Cooling
- Drying
- Freezing
- Bending
- Flexural slip
- Uplifting
- Faulting
- Stress relaxation due to erosion, glacier retreat, or human-made excavation

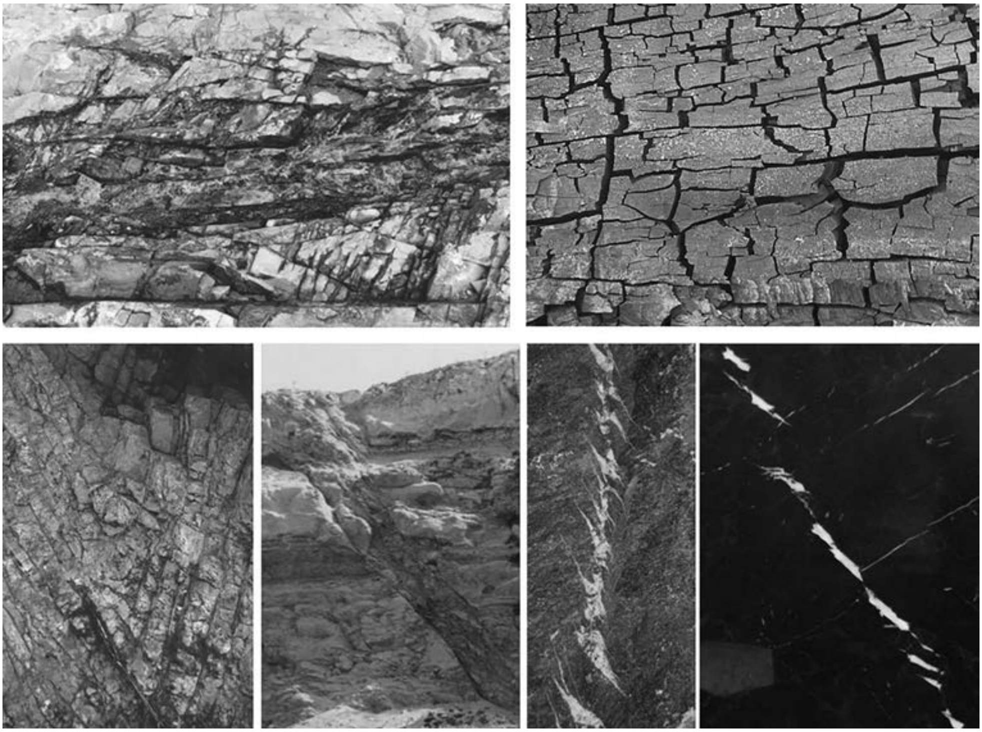


Figure 2.4 Views of discontinuities *in situ*.

ii) Shear discontinuities due to
   - Folding
   - Faulting

iii) Discontinuities due to periodic sedimentation
iv) Discontinuities due to metamorphism

Because of the discontinuities resulting from one or more of the combined actions of the abovementioned processes, the structure of rock mass in nature may look like an assemblage of blocks of typical shapes (Figs. 2.5 and 2.6). The most common block shapes are rectangular, rhombohedral, hexagonal, or pentagonal prisms. While hexagonal and/or pentagonal prismatic blocks are commonly observed in extrusive basic igneous rocks, such as andesite or basalt, and some fine-grained sedimentary rocks underwent cooling or drying processes, the most common block shapes are between a rectangular prism and a rhombohedral prism. The lower and upper bases of the blocks are usually limited by planes called flow planes, bedding planes, and schistosity or foliation planes in igneous, sedimentary, and metamorphic rocks, respectively. These discontinuities can be regarded very continuous for most of the rock structures concerned. Other discontinuities are usually found in, at least, two or three sets, crossing these planes orthogonally or obliquely. These secondary sets, if present, may

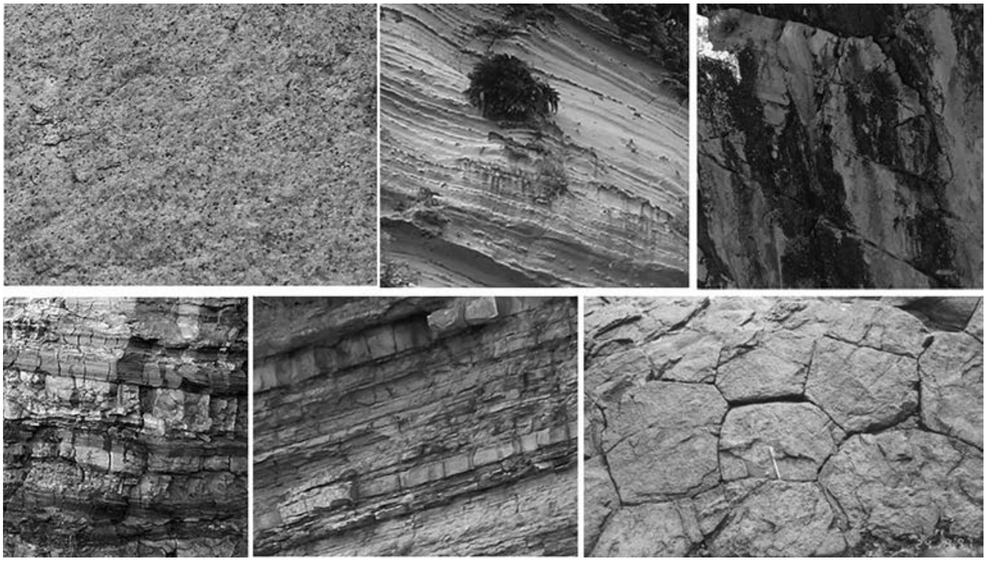


Figure 2.5 Views of rock mass in nature.

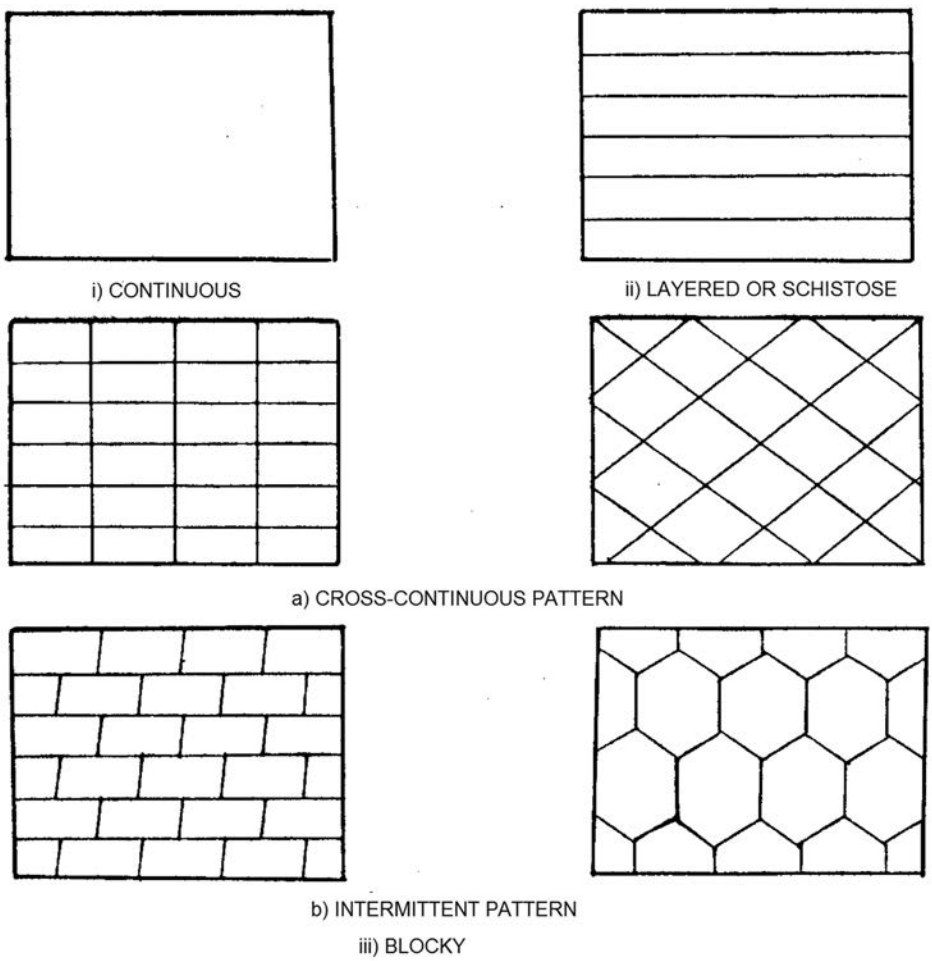


Figure 2.6 Geometrical modeling of rock mass.

be very continuous or intermittent. As a result, the rock mass may be viewed as (Fig. 2.6) (Goodman, 1976; Aydan *et al.*, 1988b):

- Continuous medium
- Tabular (layered) medium
- Blocky medium

Blocky medium can be further subdivided into two groups, depending upon the continuity of secondary sets as follows (Aydan and Kawamoto, 1987; Shimizu *et al.*, 1988):

- Cross-continuously arranged blocky medium
- Intermittently arranged blocky medium

Discontinuities, although they may be viewed as planes in large scale, have undulating surfaces varying in irregularity. As a result, they may be regarded as bands with a certain thickness associated with the amplitude of the undulations. The discontinuities may be filled with material, such as calcite, quartzite, or weathering products of host rock or transported materials, or they may exist from the beginning as thin films of clay deposits in sedimentary rocks along bedding planes.

The mechanical behavior of discontinuities is mostly associated with the inclination and amplitude of undulations, mechanical response of discontinuity wall rock, the level of normal stress, and the presence and the thickness of infilling materials. The typical shear and normal responses of various types of discontinuities are illustrated in Figure 2.7.

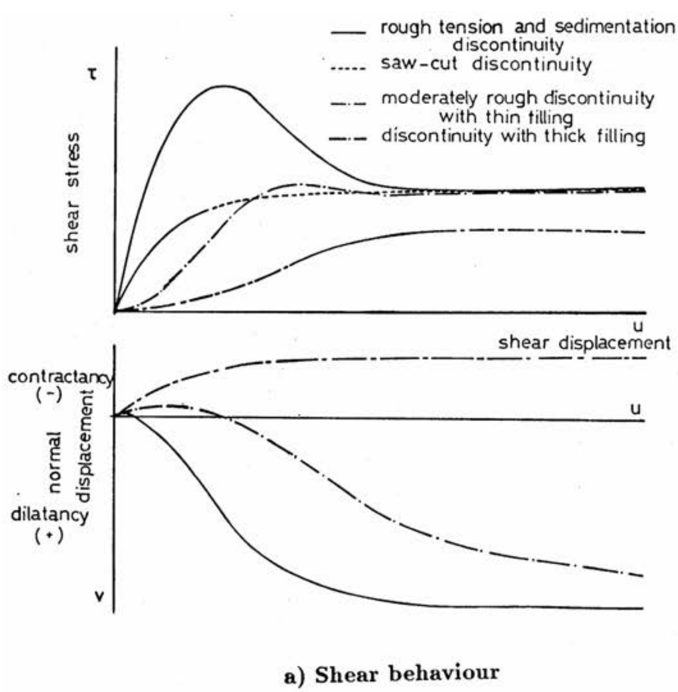
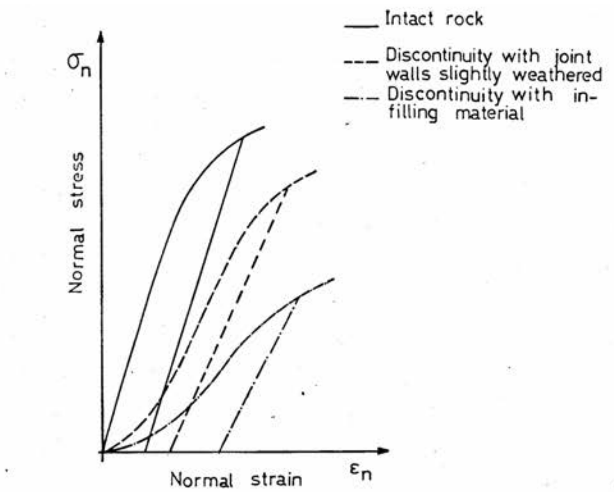


Figure 2.7 Mechanical behavior of discontinuities.



b) Normal behaviour

Figure 2.7 (Continued)

2.1.3 Rock mass and its mechanical behavior

Rock mass generally consists of blocks or layers of rock bounded by discontinuities, which look like a masonry wall with or without cementation (Figs. 2.4 and 2.5). As a result, its mechanical behavior depends on the mechanical behaviors of the rock element and of discontinuities and their orientations with respect to the applied load and constraint conditions. Although rock mass is modeled as an equivalent continuum in many studies and projects, the rock mass should be regarded as a structure and its mechanical response as a structural response rather than a material response. It is always pointed out that the strength of rock samples and discontinuities measured in the laboratory are not of much use for evaluating the stability of rock engineering structures. Let us consider a sample with a continuous discontinuity set subjected to a triaxial state of stress and assume that the failure is only governed by shearing. The triaxial strength of such a sample can be shown to be (Jaeger, 1962; Aydan *et al.*, 1987b):

$$\sigma_1^d = \frac{2c_d + \sigma_3(1+\cos 2\alpha)\tan\phi}{\sin 2\alpha - (1-\cos 2\alpha)\tan\phi} \tag{2.1}$$

where
c_d = cohesion of discontinuity set
α = inclination of discontinuity set from horizontal
ϕ_d = friction angle of discontinuity
σ_3 = least lateral principal stress
σ_1^d = strength of rock mass involving only the failure at a discontinuity plane)

When $\sigma_1^d \le \sigma_1^i$, ($\sigma_1^i$ is strength of rock mass involving only the failure of intact rock), the strength of the mass is equal to the strength offered by the discontinuity set. On the other hand, if $\sigma_1^d \ge \sigma_1^i$, the strength of the mass is governed by the intact rock element (Fig. 2.8), except at some transition zones where the failure by tensile splitting, bending, or buckling

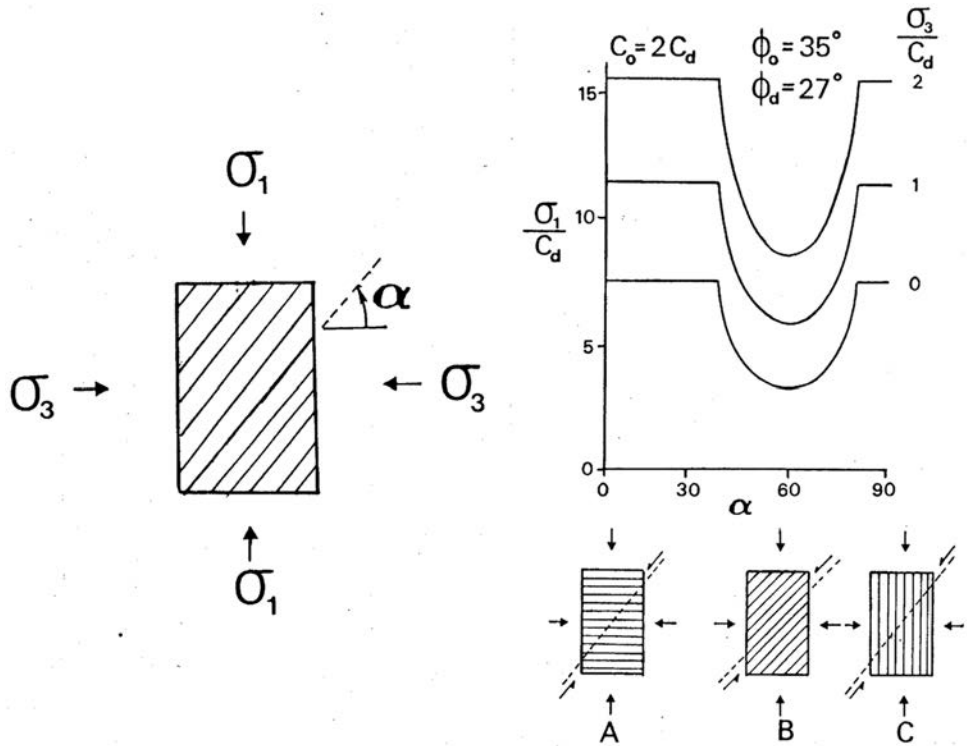


Figure 2.8 Strength of layered rock mass.

may be prevailing. The next problem is what the relation between the behavior of such samples with the situations in actual rock engineering structures is. Let us consider three specific cases in which rock mass is layered (Fig. 2.9):

- Slope
- Foundation (a dam abutment)
- Underground opening
    - Shallow underground opening
    - Deep underground opening

and assume that failure takes place by shearing. The states corresponding to the states denoted by A, B, and C in Figure 2.8 for the sample are indicated in each Figure for three specific cases in Figure 2.9. These simple illustrations clearly show that the important elements are the strength of rock elements and discontinuities in association with the specific loading condition and the geometry of the structure. Therefore, the stability of any rock engineering structure in a rock mass should be evaluated in terms of the mechanical response of the rock element and the discontinuity sets and the structure of rock mass, although it may be quite cumbersome due to the input of geometrical and material parameters in analyses.

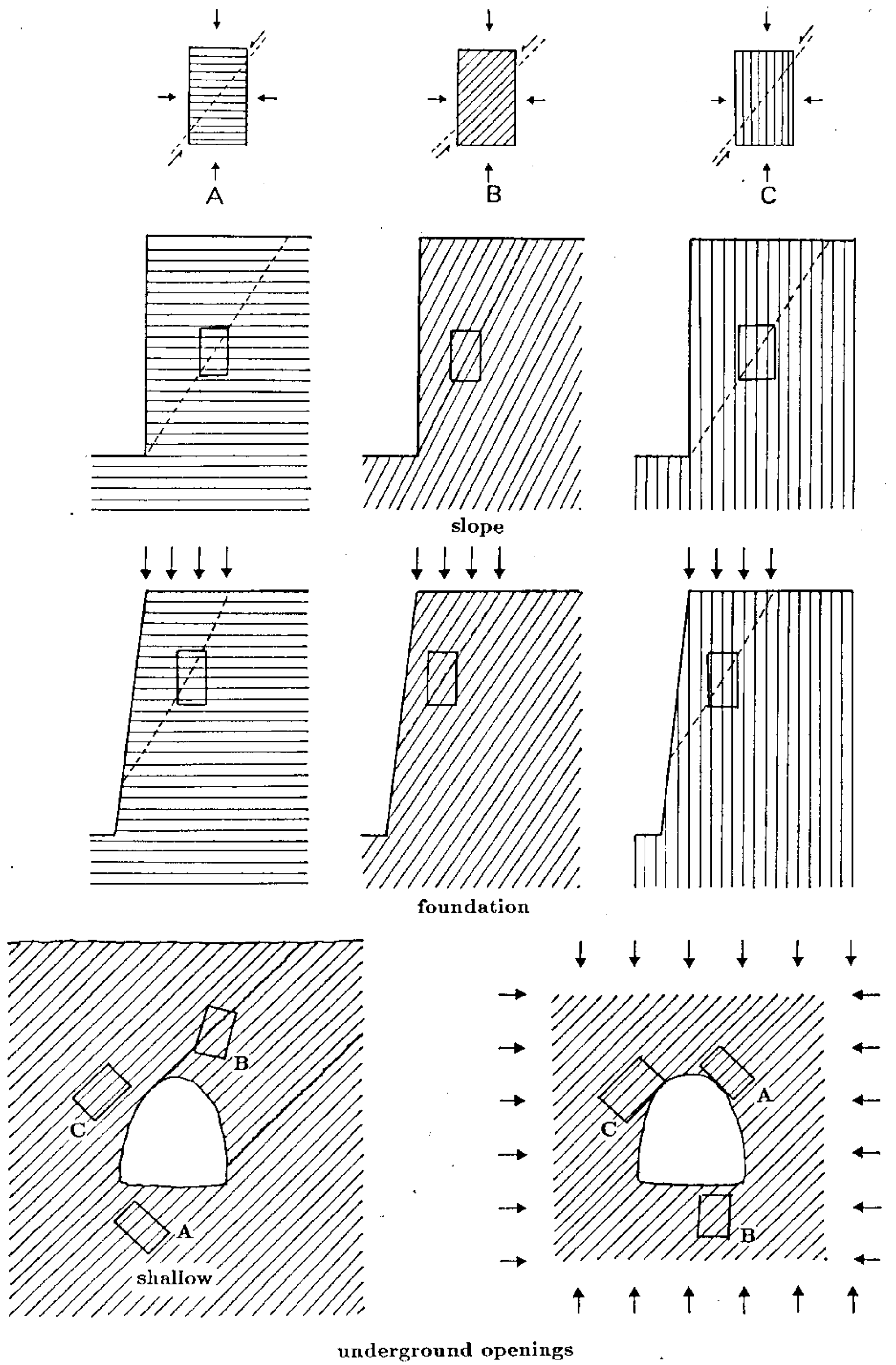


Figure 2.9 Situations in structures in layered rock mass corresponding to the situations in laboratory tests.

2.2 MODES OF INSTABILITY ABOUT UNDERGROUND OPENINGS

In the light of previous discussion on rock mass, the modes of instability likely to take place in the vicinity of underground openings may be classified as below, depending upon the structure of rock mass as shown in Figures 2.10, 2.11, and 2.12 (Aydan *et al*., 1987c; Kawamoto and Aydan, 1988):

i) Failure modes involving only intact rock

Rockbursting: This type of instability results from the combined action of initial shearing and the subsequent splitting, resulting in sudden detachment of rock slabs with a high velocity. This type of failure is usually observed in brittle hard rocks, such as unweathered igneous rocks and siliceous sedimentary rocks (Panet, 1969; Bieniawski and van Tonder, 1969; Hoek and Brown, 1980; Aydan, 1989, etc.). As the rock becomes less brittle, the rockbursts become less severe. Figure 2.13 shows plots of some compiled data on underground excavations in which rockbursts were observed.

Squeezing: This type of instability is results from the complete shearing of rock surrounding an excavation. This type of failure can be observed in ductile materials, such as rock salt, thickly bedded mudstone, halite, chalk, etc. (Terzaghi, 1946; Sperry and Heuer, 1979, etc.). It should be noted that σ_c denotes the uniaxial strength of the rock element, not that of the rock mass in Figure 2.13. These plots confirm that the critical parameter controlling the stability in rockburst and squeezing phenomena is the strength of rock element.

ii) Failure modes involving discontinuities and intact rock

Bending: This type of instability is usually observed in sedimentary rocks due to gravitational forces, when layers are generally parallel to the roof and *in situ* stresses parallel to layering is relatively low. Figure 2.11 shows a typical example of a bending failure observed in a model test. This type of failure is associated with the tensile strength of layers at the early stages of failure (Birön and Arıoğlu, 1983; Hoek and Brown, 1980; Whittaker and Reddish, 1989, etc.). This is confirmed by the plots of some failed excavations due to bending (Fig. 2.14).

Buckling: Contrary to bending failure, this type of instability is observed when high *in situ* stresses parallel to layering are present and the thickness of layers in comparison with the span is relatively small. Figures 2.10 and 2.11 show some field examples and examples of model openings failed through buckling (Everling, 1964; Detzlhofer, 1970; Amberg, 1983, etc.). It is usually observed in metamorphic rocks and thinly layered sedimentary rocks. The plots of some data on excavations where buckling was observed confirm this conclusion (Fig. 2.14).

Punching and sliding: This highly localized form of instability is observed when the rock is relatively thinly layered. Some field examples are reported by Arnold *et al*. (1972).

Flexural toppling: This type of failure is also a localized form of instability, and it can be observed particularly in roofs and sidewalls of openings excavated in sedimentary and metamorphic rocks. Some examples of this type of instability are shown in Figures 2.10 and 2.11 (Goodman, 1977; Aydan *et al*., 1988c). Layers of rock bend and fail like interacting cantilevers that fail in flexure.

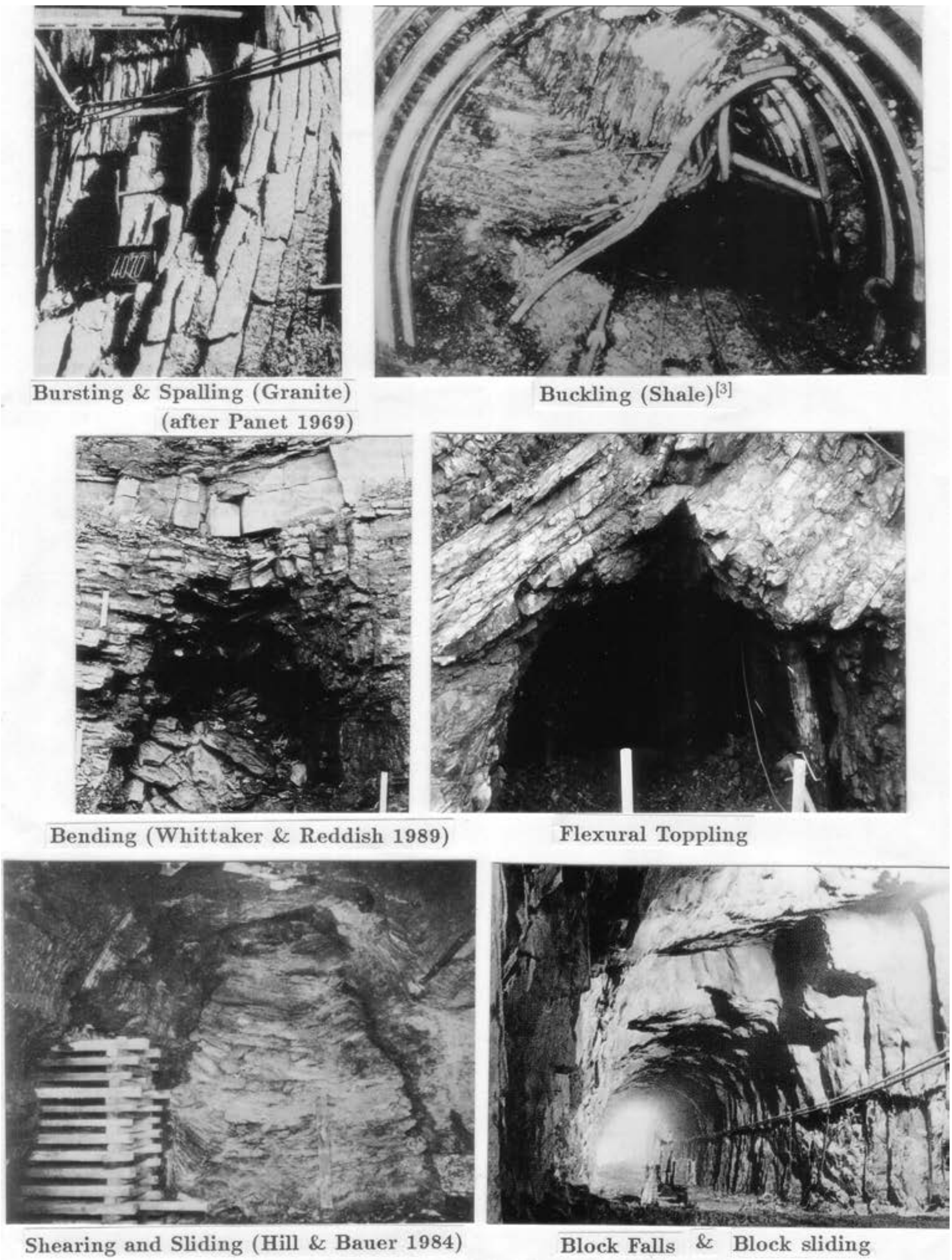


Figure 2.10 Pictures of failures observed in underground openings in the field.

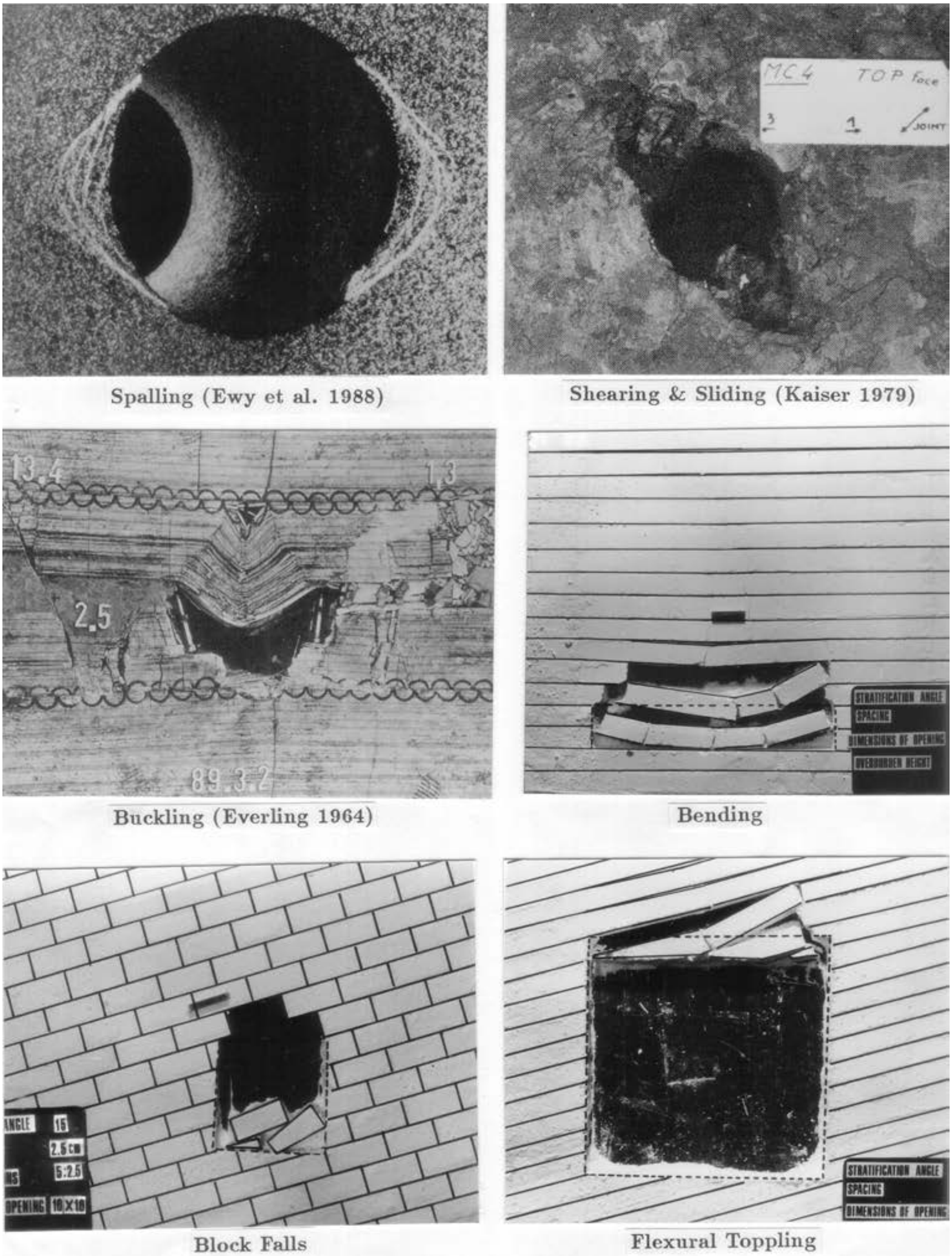


Figure 2.11 Pictures of failures observed in underground openings in model tests.

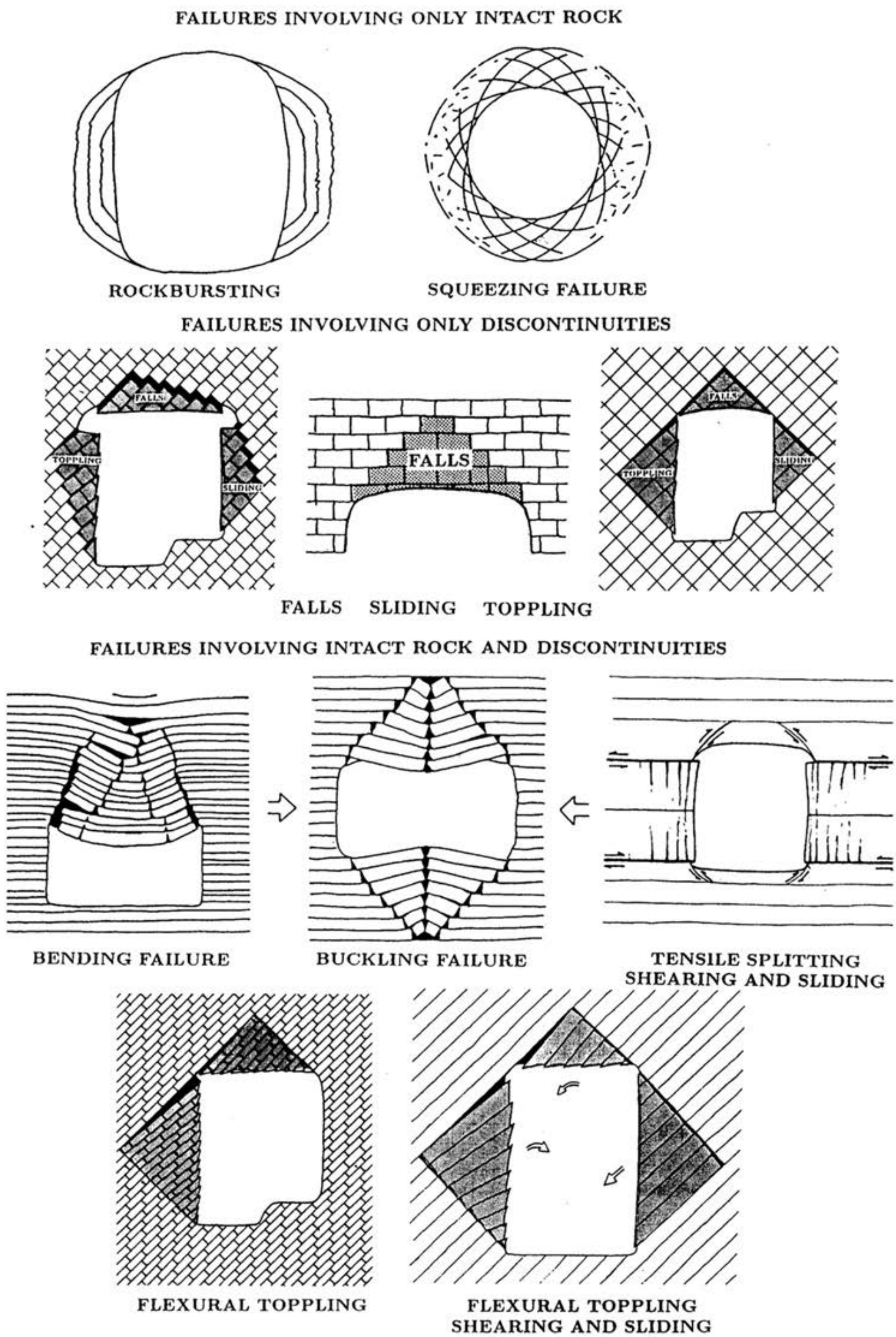


Figure 2.12 Classifications of modes of instability in underground openings.

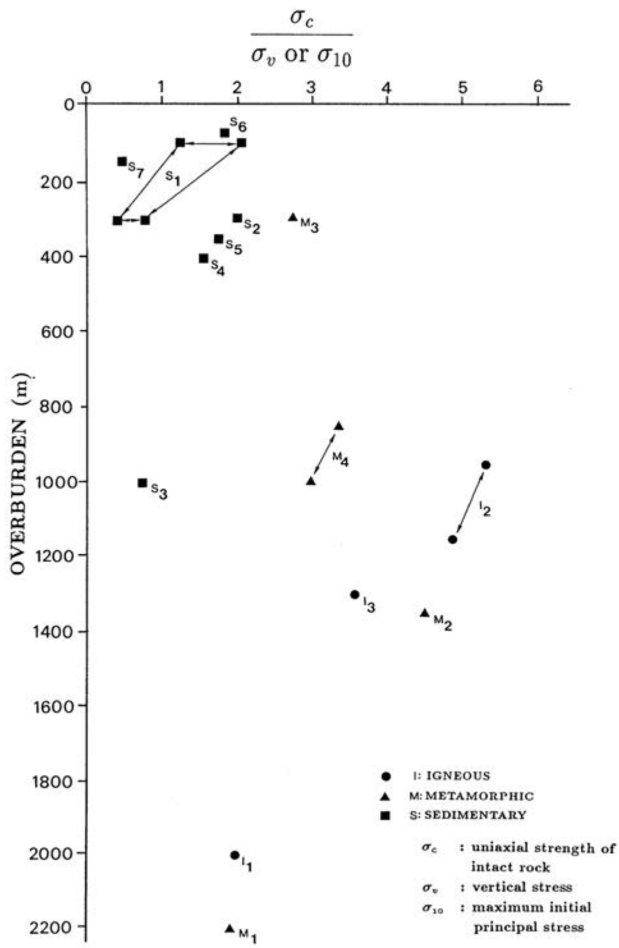


Figure 2.13 Plots of failed case studies involving only intact rock. Note that the strength σ_c is the strength of rock element.

Shearing and sliding: This type of failure involves combined sliding of unstable part along discontinuities and shearing through intact rock. It is most likely to be seen when *in situ* stresses are higher than the compressive strength of rock, making buckling failure impossible. Some severe field examples are reported by Sperry and Heuer (1979), who observed in Navajo irrigation tunnels in shale and sandstone and by Hill and Bauer (1984), who observed in mine openings in shale. In the model tests of circular openings in jointed coal carried out by Kaiser (1979), this type of failure was observed dominantly, even though the samples were loaded hydrostatically.

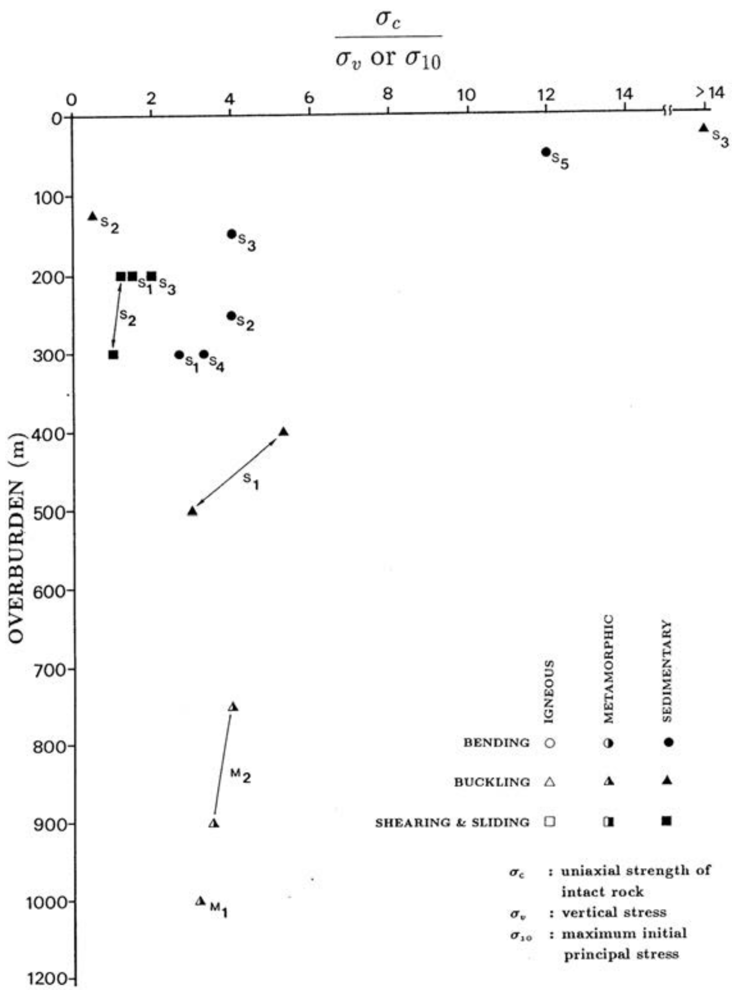


Figure 2.14 Plots of failed case studies involving intact rock and discontinuities.

iii) Failure modes involving only discontinuities (blocky medium only)

These types of failure can occur at any depth, as long as the rock mass has discontinuity sets of two or more (Fig. 2.15):

Block falls: This type of failure is observed in the roofs of openings due to gravitational forces. Some examples were observed in the field and model tests were done in the laboratory, shown in Figure 2.10 (Isaac and Bubb, 1981; Dezhen and Sijing, 1982; Weiss-Malik and Kuhn, 1979; Pistone and del Rio, 1982; Detzlhofer, 1968, etc.).

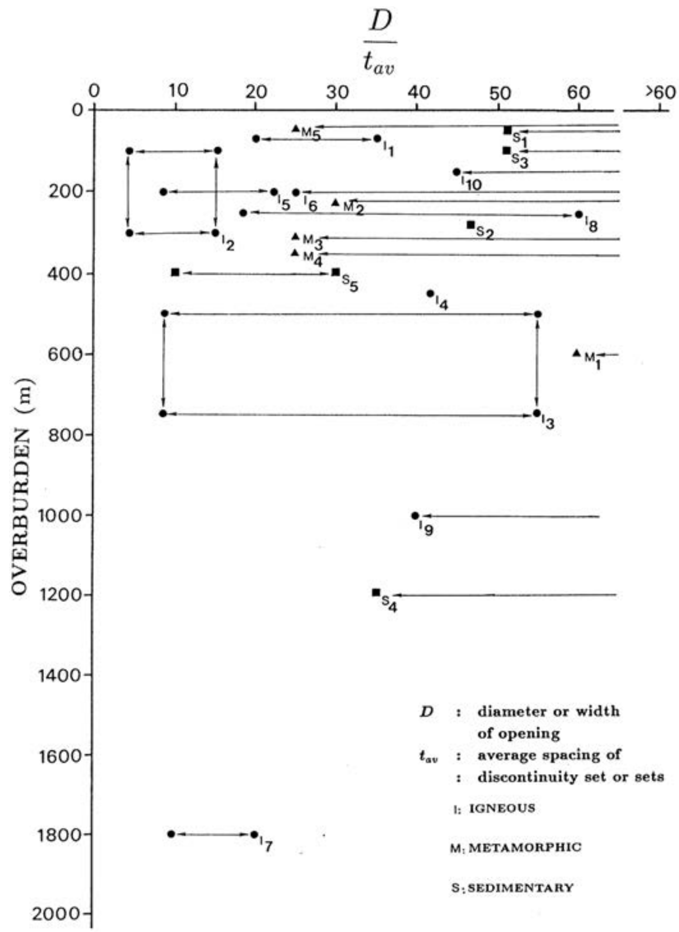


Figure 2.15 Plots of failed case studies involving discontinuities only.

Sliding: This type of failure is observed when one of the discontinuity sets daylights near the toe of sidewalls and the disturbing forces are greater than its shear resistance. Some examples of such failures in field and model tests are shown in Figure 2.11 (Pistone and del Rio, 1982; Kamemura *et al.*, 1986; Reik and Soetomo, 1986, etc.).

Toppling: The inclination of the critical discontinuity set, on which toppling will occur, should be such that no sliding failure is possible. Some examples of such failures in field and model tests are shown in Figure 2.11 (Pistone and del Rio, 1982; Isaac and Bubb, 1981, etc.).

Sliding and toppling: This type of failure is observed when the conditions for the two types of failures are satisfied. Some examples for such failures are shown in Figure 2.11.

2.3 MODES OF INSTABILITY OF SLOPES

As in the case of underground openings, a similar type of classification can be made for rock slopes (Fig. 2.16) (Aydan *et al*., 1988b). Pictures of some slope failures observed *in situ* and in laboratory tests are shown in Figures 2.17 and 2.18.

i) Failure modes involving only intact rock

Shear failure: This type failure is observed in cases such that the slope angle and height are sufficient to cause shearing of the intact medium in continuous, tabular, or blocky medium. In tabular or blocky medium, the internal structure and slope geometry should be such that no other forms of instabilities are possible. Some examples observed in field and laboratory model tests are shown in Figures 2.17 and 2.18 (Hutchinson, 1971; Hoek and Bray, 1977; Tokashiki and Aydan, 2010). Depending upon the slope angle, tensile cracks at the top of slopes may appear, and the failure of slopes, therefore, can be due to a combination of shearing and tensile stresses.

Bending failure: This type of failure is likely to be seen in the case of slopes with a toe eroded. The mode of failure is similar to that of cantilevers. Some examples for such failure observed in model tests are shown in Figure 2.18. The failure is often observed in cliffs near sea sides or river embankments (Skudrzyk *et al*., 1986; Tharp, 1983; Okagbue and Abam, 1986, etc.). For this type of failure, the ratio of the erosion depth to the slope height should be sufficient to cause bending failure rather than shear failure.

ii) Failure modes involving discontinuities and intact rock

Combined shear and sliding failure: This type of failure can occur when one of the discontinuity sets has an inclination equal to the slope angle and no other forms of failure is possible. This failure manifests itself as sliding along a critical plane and the shearing of intact rock near the toe of the slope (Fig. 2.16) (Brawner *et al*., 1971; Aydan *et al*., 1992).

Buckling: This type of failure occurs when the slope angle is equal to that of the discontinuity set and the ratio of discontinuity spacing to the slope height is relatively small. It is a recently recognized form of instability and reported case studies are rare (Walton and Coates, 1980; Cavers, 1981, etc.). A field example for such a failure at the Elbistan open-pit mine is shown in Figure 2.17 (Aydan *et al*., 1996).

Flexural toppling: This type of failure occurs in the case of slopes excavated in sedimentary or metamorphic rocks. Although this type of failure is a local one in the case of underground openings, it is a global form of failure in the case of slopes. Flexural toppling was first recognized by Erguvanlı and Goodman (1972) and Hoffmann (1974), and some fundamental studies on this failure form were undertaken by Aydan and Kawamoto (1987, 1992) and Aydan *et al*. (1988c). Some *in situ* and laboratory examples for such a failure are shown in Figures 2.17 and 2.18.

ii) Failure modes involving only discontinuities

Sliding failure: There are two types of sliding failure (Fig. 2.16). These are:

Planar sliding: This involves only one set, the strike of which is parallel or nearly parallel to the slope axis, and occurs along a critical plane, daylighting near the toe

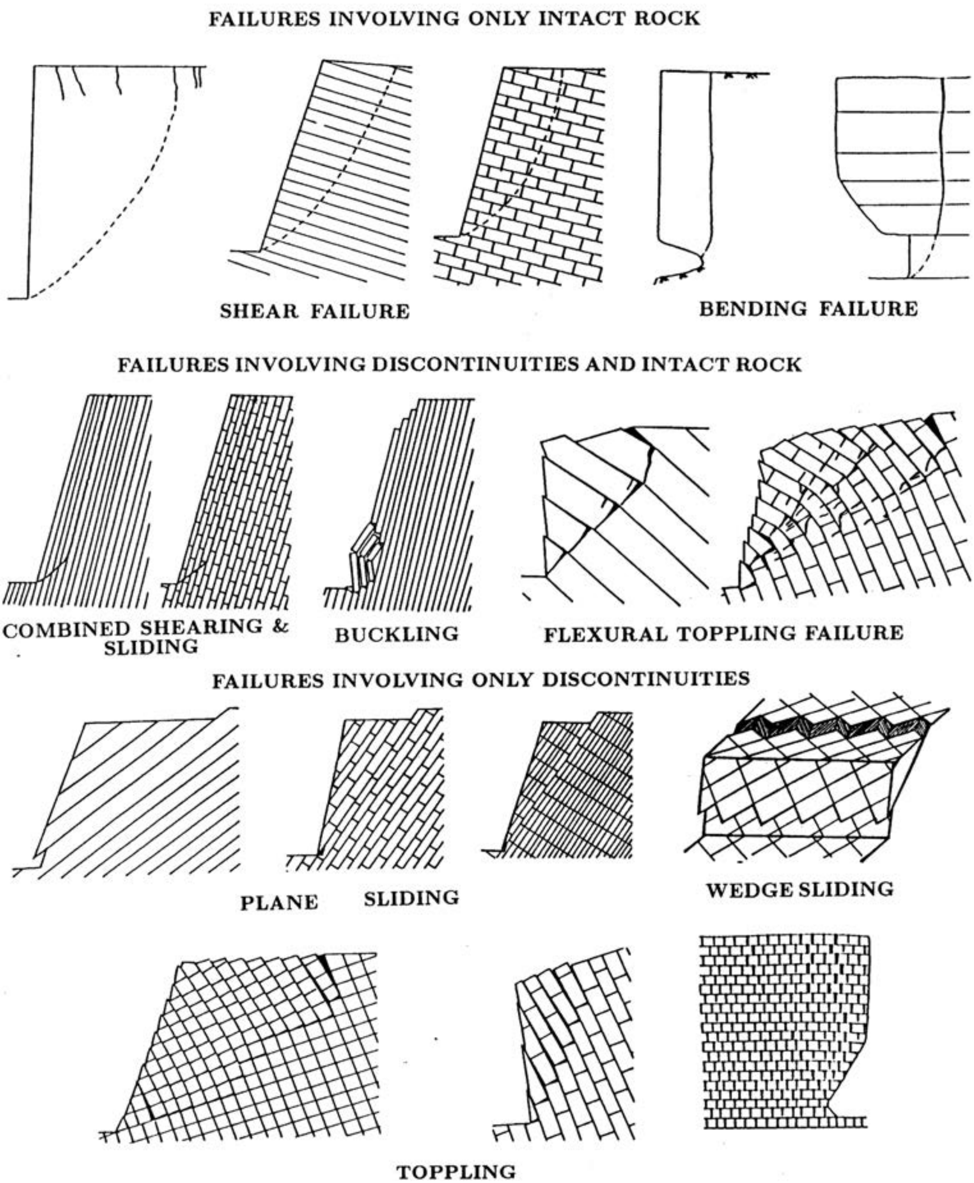


Figure 2.16 Classifications of modes of instability in slopes.

of the slope (Hoek and Bray, 1977; Aydan *et al*., 1989). Some examples of failed slopes in field and model tests are shown in Figures 2.17 and 2.18.

Wedge sliding: This involves two throughgoing discontinuity sets and occurs when the intersections of two sets daylight near the toe of the slope (Wittke, 1964; Shimizu *et al*., 1988; Kumsar *et al*., 2000; Aydan and Kumsar, 2010). An example of failed slopes in the field is shown in Figure 2.17.

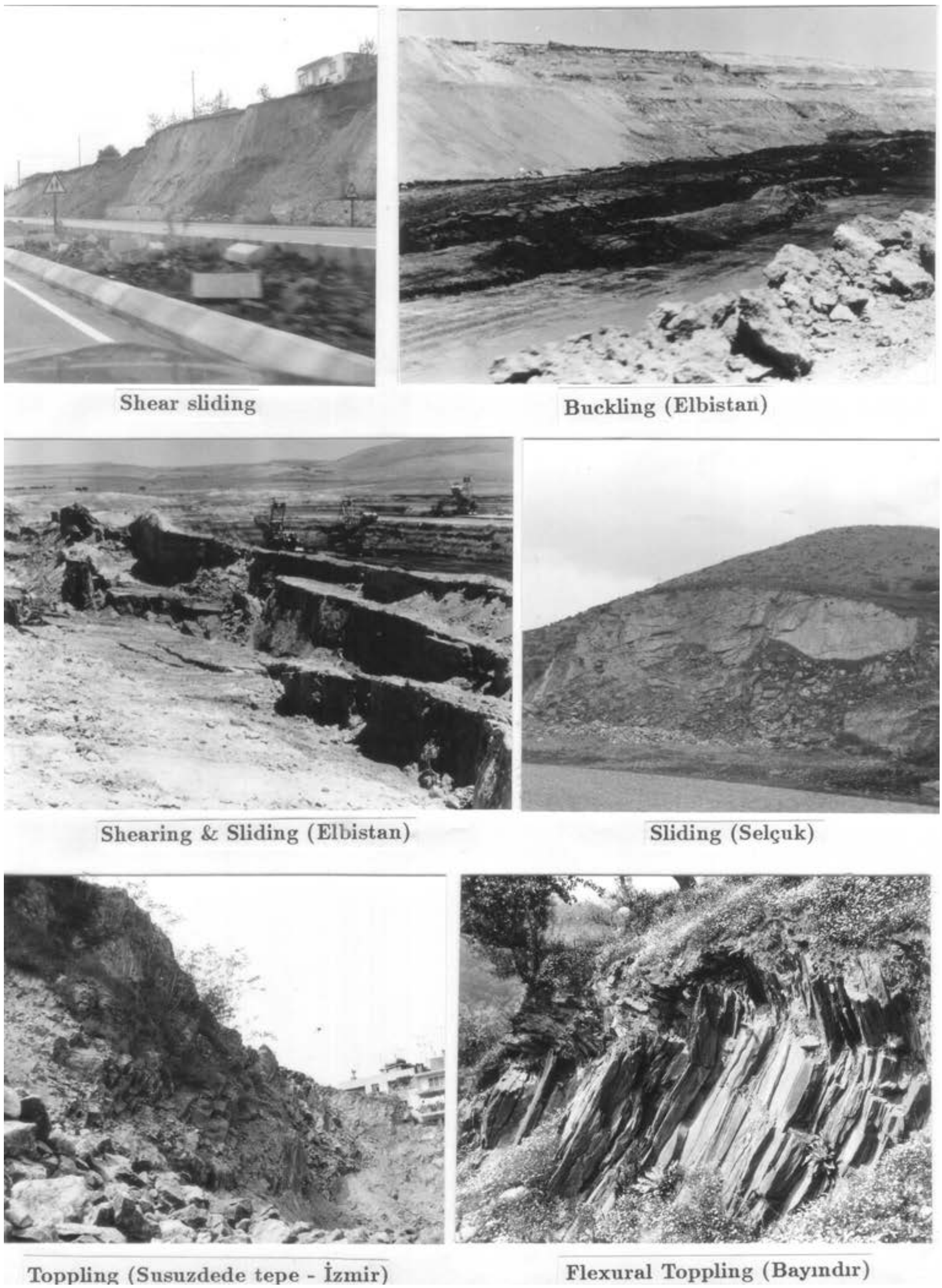


Figure 2.17 Pictures of failures observed in slopes in field.

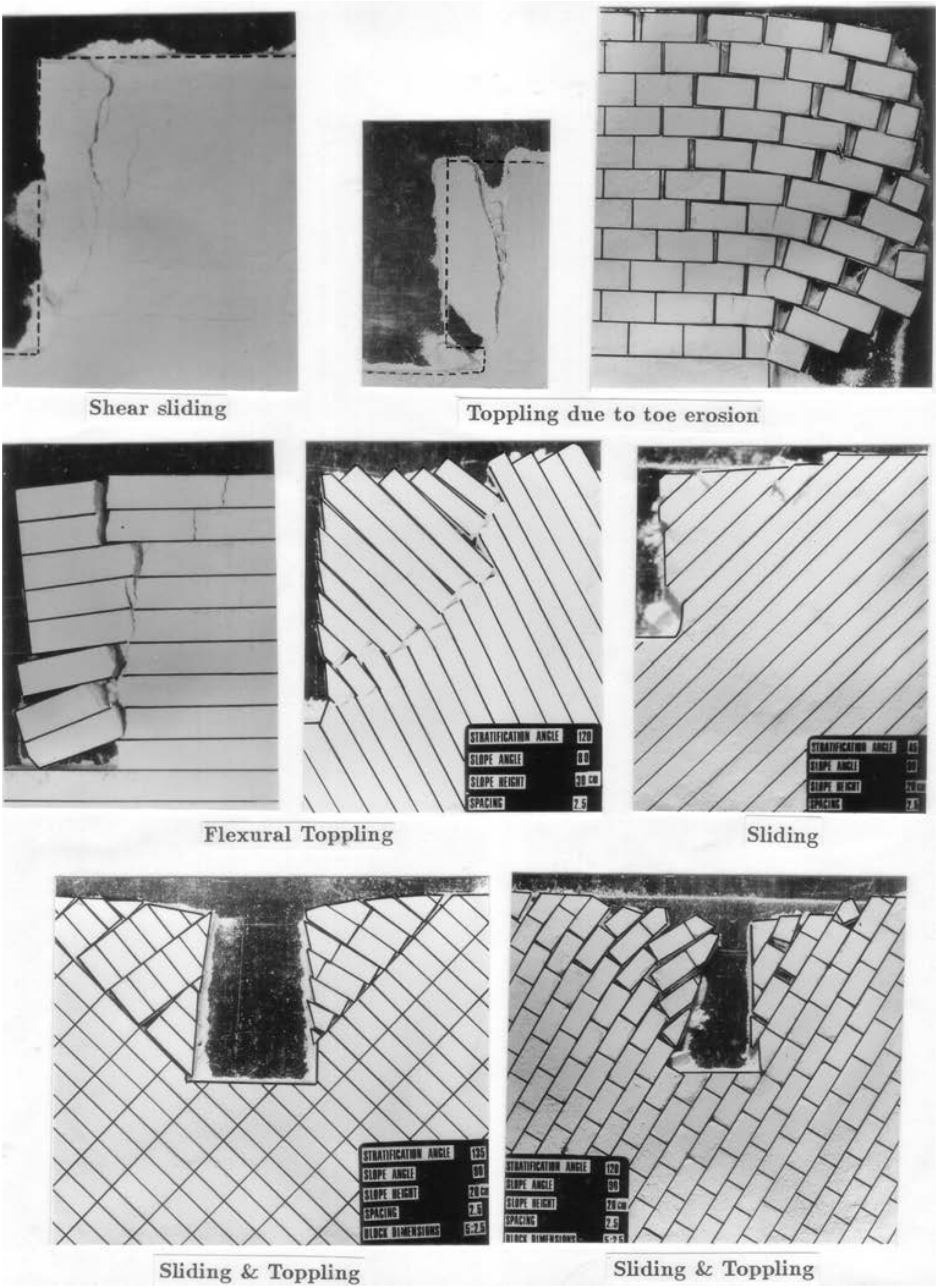


Figure 2.18 Pictures of failures observed in slopes in laboratory tests.

Toppling failure: This occurs when one of the discontinuity sets, the strike of which is parallel or nearly parallel to the axis of slope, has an inclination such that no sliding is possible (Goodman and Bray, 1976; Aydan and Kawamoto, 1987; Aydan *et al.*, 1989). Some field and laboratory examples are shown in Figures 2.17 and 2.18.

Combined toppling and sliding failure: This type of failure is observed when both conditions for toppling and sliding are satisfied (Aydan *et al.*, 1989; Aydan *et al.*, 1992). An example of failed slopes in model tests in the laboratory is shown in Figure 2.17.

Failure modes of sliding and toppling are also global forms of failure, as compared to the local character in the case of underground openings.

2.4 MODES OF INSTABILITY OF FOUNDATIONS

The modes of failure and the classification for foundations would be similar to those of slopes. Therefore, the repetition is avoided, but some pictures and illustrations of foundation failure together with their classifications are shown in Figures 2.19, 2.20, and 2.21 under

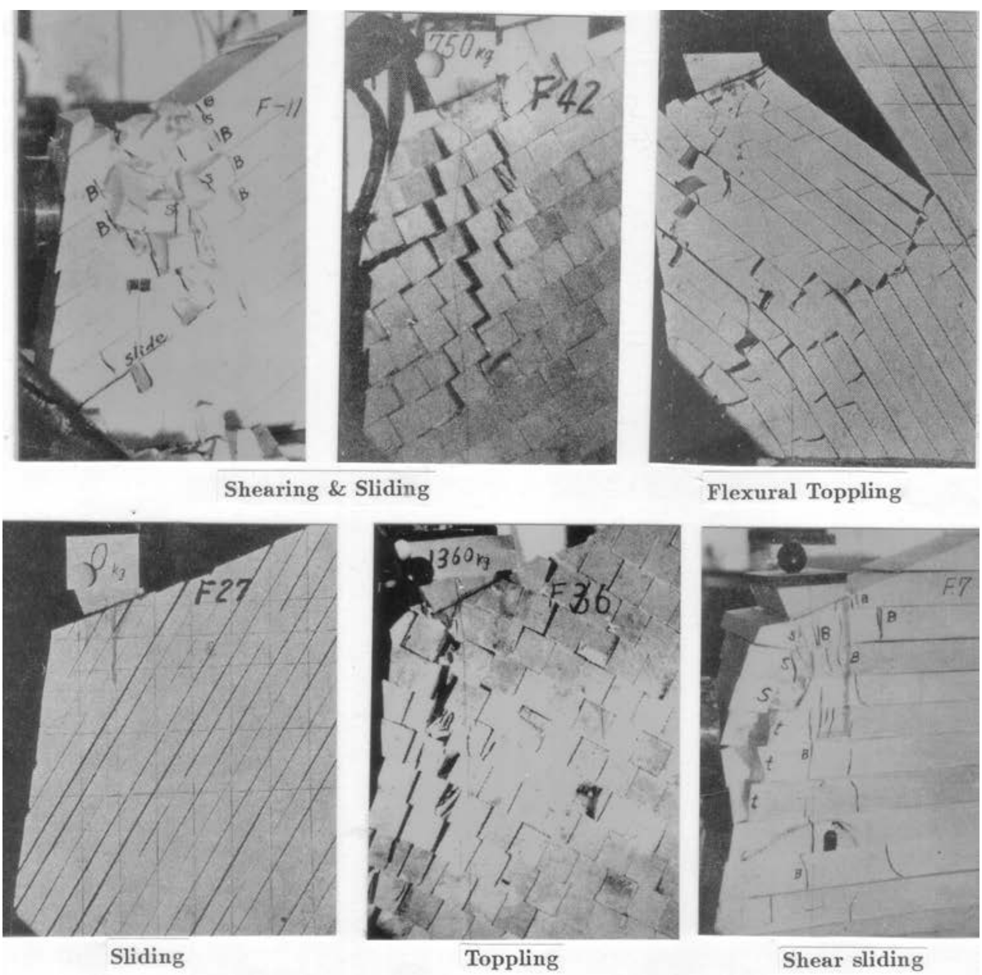


Figure 2.19 Pictures of modes of instability in foundations under compressive and tensile stress fields.

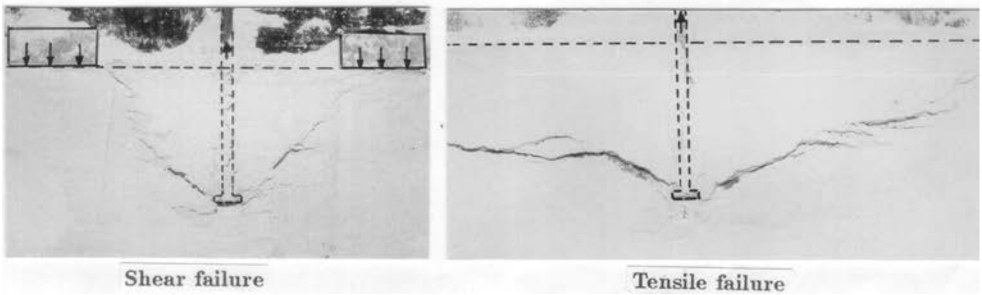


Figure 2.19 (Continued)

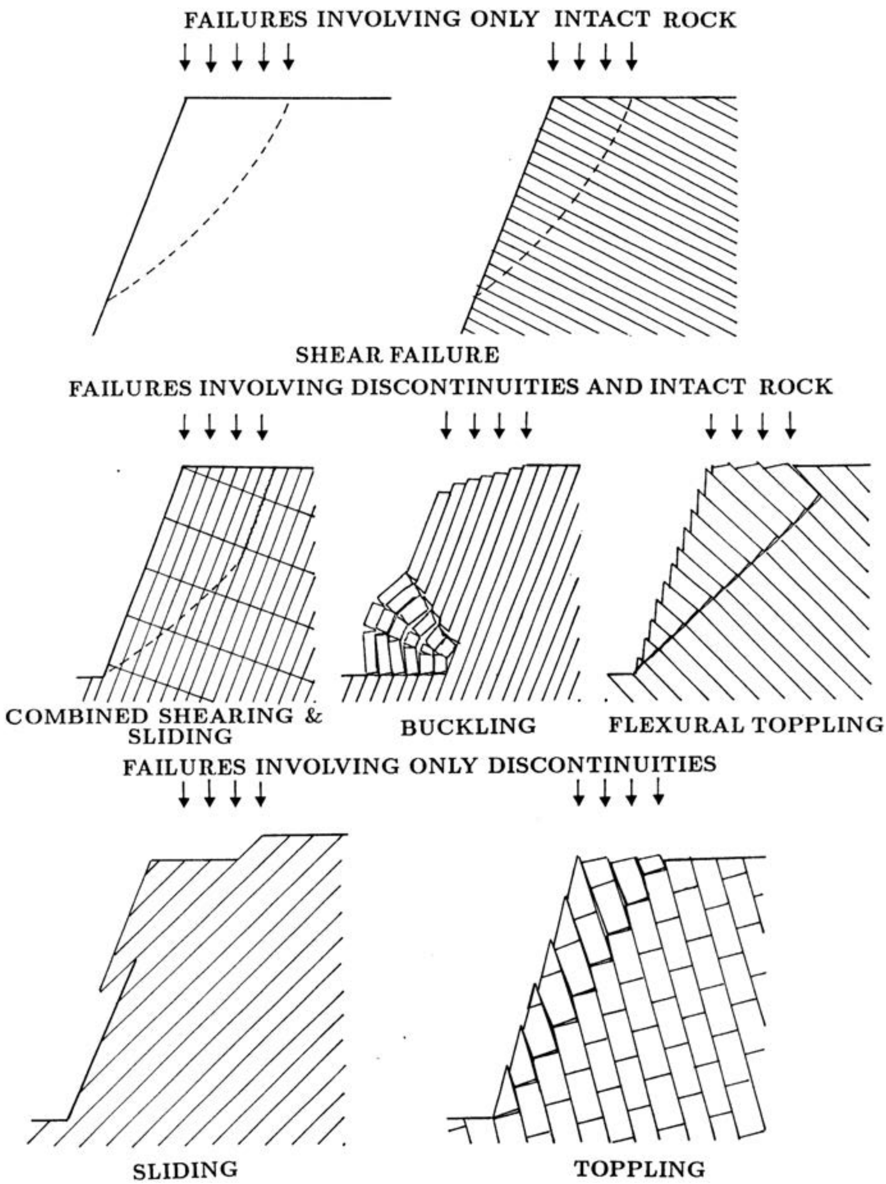


Figure 2.20 Classifications of modes of instability in foundations under compressive stress field.

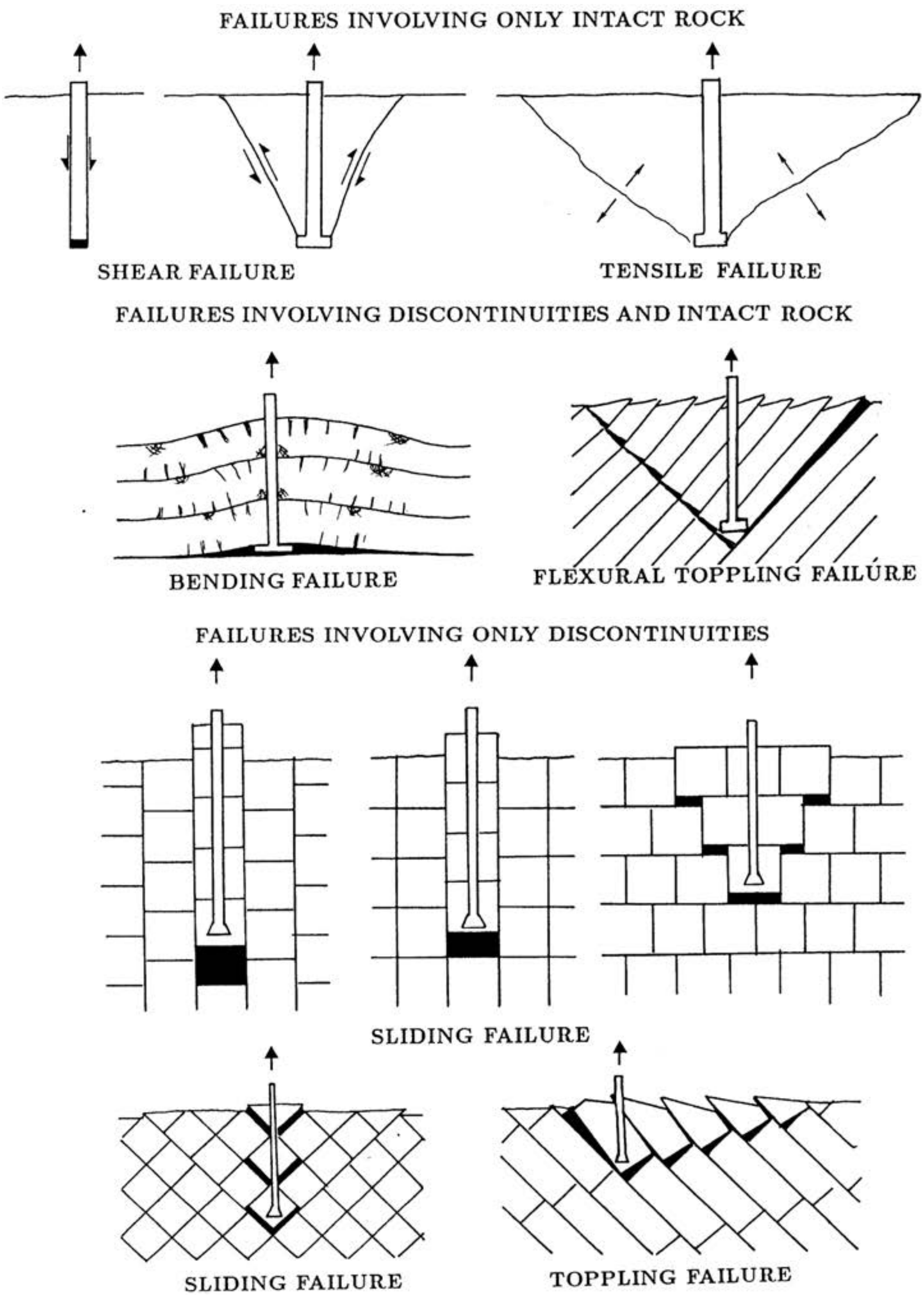


Figure 2.21 Classifications of modes of instability in foundations under tensile stress in the field.

compressive and tensile stress fields. The reported examples of failures of foundations in the field and the laboratory are presently few, and most of the tests are associated with model tests (Bernaix, 1966; Krsmanovic *et al*., 1965; Hayashi and Fujiwara, 1963; Goodman, 1976; Ebisu *et al*., 1994a, 1994b).

Chapter 3

Design philosophy of rock support and rock reinforcement

3.1 INTRODUCTION

The primary concerns regarding the support and reinforcement design of rock engineering structures, as apparent from the term, is whether the structure under consideration is self-supporting, and if it is not, what kind of strategy must be followed for the overall stability of the structure during and after excavation. The selection of support/reinforcement members is not only closely associated with their mechanical functions but also with their advantages and disadvantages related to environmental, constructional, and economic conditions. Nevertheless, as this book is more concerned with the mechanical functions of the support/reinforcement members, the discussions are herein restricted mainly to the mechanics of support/reinforcement members and supporting procedures, with occasional references made to the environmental, constructional, and economic aspects. The discussions are mainly concerned with the supporting/reinforcement philosophy used in the design of underground openings, as they are more generalizable than surface structures. Nevertheless, considerations are given to other structures from time to time.

The present support/reinforcement philosophy mainly consists of two fundamental steps:

Step 1: Determination of the magnitude of unbalanced loads to be resisted by the chosen single or combination of support/reinforcement members

Step 2: Selection of the support/reinforcement members suitable not only from the mechanical point of view but also from the constructional, economic, and environmental points of view

In rock engineering, the design approaches can be categorized into three groups:

- Empirical
- Analytical
- Numerical

In the empirical methods, rock mass classification systems are extensively used for feasibility and pre-design studies, and often also for the final design.

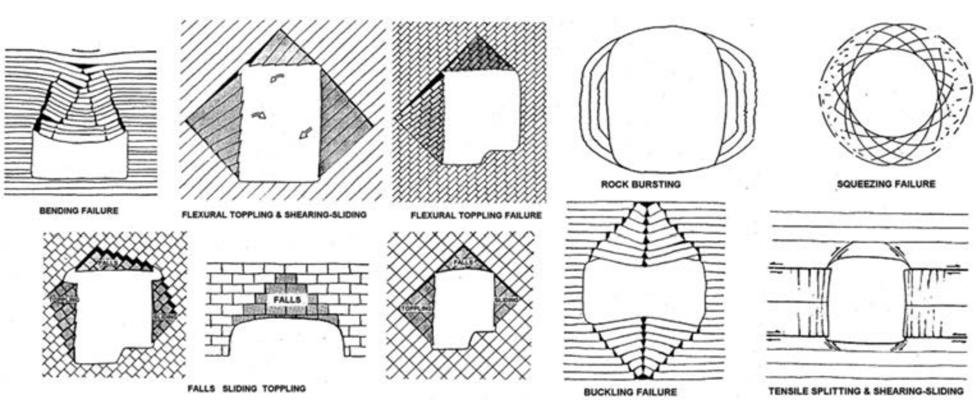
In the design of rock engineering structures, the anticipated form of instability is of great importance. The instabilities around underground openings in rock may be categorized as global instability and local instability, defined as (Aydan, 1989, 2016):

Global instability: This is defined as when the excavated space cannot be kept open and the failure of the surrounding mass continues to take place indefinitely unless any supportive and/or reinforcement measure is undertaken. The global instability would be as a result of exceeding the strength of surrounding rocks due to the redistribution of initial ground stresses.

Local instability: After clearance of the failed zone and without taking any supportive measures, if the remaining space can be kept open, the form of instability is termed local instability. The main cause of failure is the dead weight of rock in a particular zone about the cavity, defined by the geometry of underground openings and the spatial distribution of discontinuities.

The design of support/reinforcement systems of large underground openings and tunnels in rock engineering is of great importance, as these structures are required to be stable during their service lifetime (Aydan, 1989). Provided that the elements of support/reinforcement systems are resistant against chemical actions due to environmental conditions and their long-term behavior is satisfactory, the support systems must be designed against anticipated load conditions. As rock masses have many geological discontinuities and weakness zones, the load acting on support systems may be due to the dead weight of potential unstable blocks formed by rock discontinuities, which may be designated as structurally controlled or local instability modes and independent of *in situ* stress state or inward displacement of rock mass due to elasto-plastic or elasto-visco-plastic behavior induced by *in situ* stresses (Fig. 3.1). Therefore, the main purpose of the design of support/reinforcement systems must be well established with due considerations of these situations.

Rock mass classifications are commonly used for various engineering design and stability assessments and they are initially proposed for the design of a given rock structure. However, this trend has been changing, and the main objectives of rock mass classifications have become to identify the most significant parameters influencing the behavior of rock masses, to divide a particular rock mass formulation into groups of similar behavior, to provide the characterizations of each rock mass class, to derive quantitative data and guidelines for engineering design, and also to provide a common



(a) Local form of instabilities (b) Global form of instabilities

Figure 3.1 Instability modes of underground openings (re-arranged from Aydan, 1989).

basis for engineers and engineering geologists. These are based on empirical relations between rock mass parameters and engineering applications, such as tunnels and other underground caverns.

Although the history of rock classifications for a given specific structure is old, the rock mass classification system proposed by Terzaghi in 1946 for tunnels with steel set support has become the basis for the follow-up quantitative rock mass classifications. Currently, there are many rock classification systems in rock engineering, particularly in the tunneling area, such as Rock Mass Rating (RMR) (Bieniawski (1973, 1989), Q-system (Barton *et al*., 1974), RSR (Wickham *et al*., 1972), and Rock Mass Quality Rating (RMQR) by Aydan *et al*., 2014. In addition, rock mass classifications of NEXCO (known as DORO-KODAN) and JR (KYU-KOKUTETSU) are commonly used to design tunnels in Japan. Nevertheless, utilizing these systems to characterize complex rock mass conditions is a challenge for engineers.

In this chapter, several classification systems have been briefly explained, and quantitative assessments have been done based on RMQR. Because it is required for an engineer to select the most appropriate method for determining design parameters in rock engineering, a brief overview of the kinds of rock loads and the procedures to determine their magnitude, empirical, analytical, and numerical techniques has been provided and discussed, with the objective of unifying the present methods of design.

3.2 EMPIRICAL DESIGN METHODS

As mentioned in the introduction, Terzaghi (1946) considered steel ribs as the main support member and visualized a loosened region of rock mass in the roof and sidewalls, as illustrated in Figure 3.2. His main idea originates from his trapdoor experiments with soils, and he visualized that the support load (pressure) on steel ribs as a fraction of the weight of the potentially unstable ground, which is given as:

$$p_i^r = \gamma B \tag{3.1}$$

where B is tunnel width and γ is the unit weight of potentially unstable ground.

Since then, this concept has been utilized in visualizing and calculating the pressure on support members in geotechnical engineering, including both soil and rock tunnels. As noted from Figure 3.2, the load on tunnel support results from the surrounding ground (which may be soil or rock mass), which is a function of assumed ground properties and loosening zone around the tunnel. Although this concept is quite simple to use, major issues arise how to relate this pressure in rock mass to the true *in situ* stresses in rock mass. Although their main formulations differ, Protodyakonov and Terzaghi proposed independently the following relationship:

$$\frac{p_i^r}{\gamma B} = \frac{1}{\tan\phi} \tag{3.2}$$

where ϕ is the friction angle of potentially unstable ground.

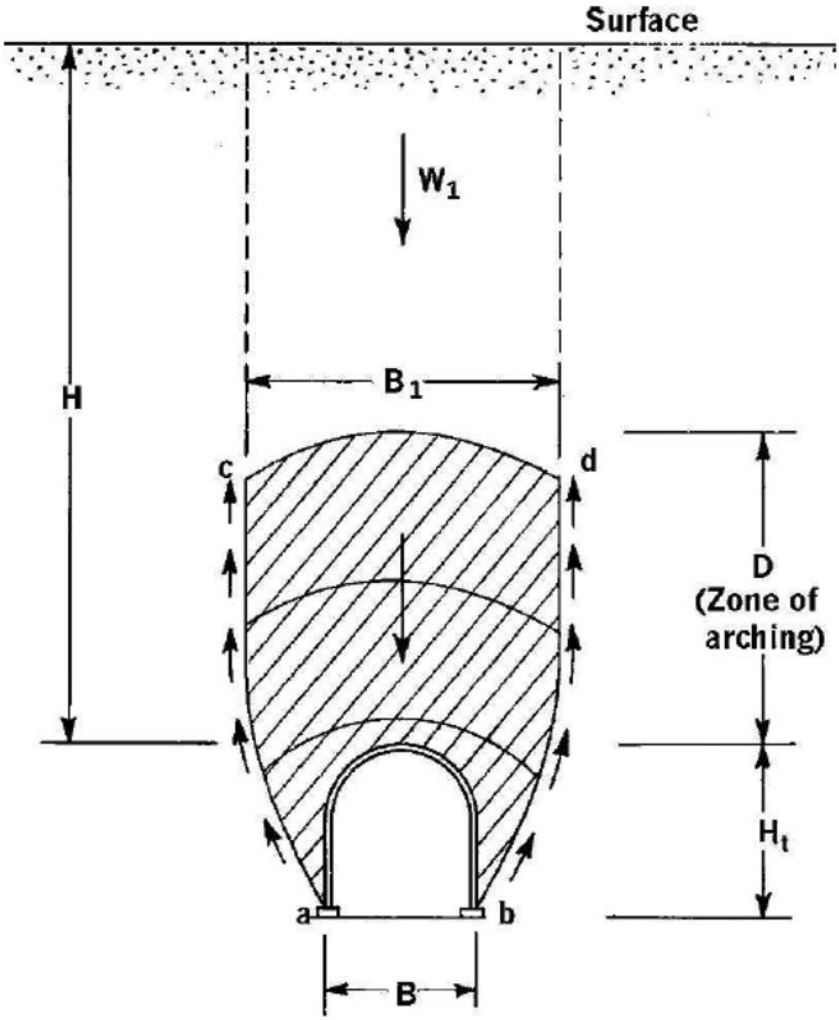


Figure 3.2 Terzaghi's rock load concept (from Terzaghi, 1946).

As rock mass always has discontinuities, the rock layers and/or blocks of rocks may become detached or loosened due to gravity, blasting, or groundwater seepage and act on support members as rock load. Such failures in rock mass may be classified as local failures (Aydan, 1989; Kawamoto *et al.*, 1991). They may loosen more if the ground is shaken further, such as by earthquakes (Fig. 3.3). The original concept of Terzaghi is utilized in many rock classification systems, which may be categorized as an empirical approach.

In the following subsection, the empirical approaches are briefly explained.

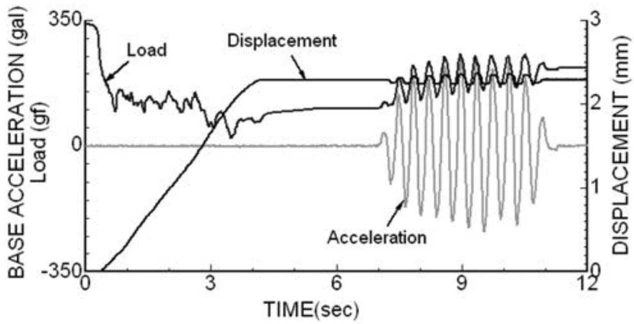


Figure 3.3 Load and displacement response of a trapdoor experiment subjected to shaking (note that the pressure on the trapdoor is increased after shaking despite no further downward displacement of the trapdoor).

3.2.1 Rock Quality Designation (RQD) method

Deere *et al*. (1969) suggested the following relationship between the roof pressure and the Rock Quality Designation (RQD), which is a percentage of rock cores whose length is greater than 10 cm for a given 1 m length of cores.

$$\frac{p_i^r}{\gamma B} = 0.2 + 0.025RQD \tag{3.3}$$

The length and number of rockbolts and rockanchors and the thickness of shotcrete are computed using their load-bearing capacity, loosened load height, and required anchorage length. This concept is followed in other rock classification systems, such as RMR and Q-system.

3.2.2 Rock Mass Rating (RMR)

Bieniawski (1973, 1976) published the details of a rock mass classification called the Geomechanics rock classification or the Rock Mass Rating (RMR) system. Over the years, this system has been refined as more case records have been examined, and the reader should be aware that Bieniawski (1989) has made significant changes in the ratings assigned to different parameters and he suggests that the 1989 version be used. In this section, support design according to RMR has been briefly described.

Bieniawski (1989) published a set of guidelines for the selection of support in tunnels in rock using the value of RMR for rock mass. These guidelines are reproduced in Table 3.1. Note that these guidelines have been published for a 10-m-span horseshoe-shaped tunnel, constructed using drill and blast methods, in a rock mass subjected to a vertical stress < 25 MPa (equivalent to a depth below surface of < 900 m). It should be noted that Table 3.1 has not had a major revision since 1973. In many mining and civil engineering applications, steel-fiber-reinforced shotcrete may be considered in place of wire mesh and shotcrete.