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Rock Classifications and Horizontal Drilling and Drainage

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Part 1
Rock Classifications

Rock Classifications: State of the Art and Prospects for Standardization

Z. T. BIENIAWSKI

The purpose of this paper is to present a state-of-the-art review of rock classifications and to consider the need and prospects for attaining a standard classification. Recent developments concerning both intact-rock classifications and rock-mass classifications are described. Such engineering applications of the current rock-classification systems as tunnels and chambers, slopes, and foundations are discussed, and it is demonstrated how rock classifications enable estimation of the strength and the deformability of rock masses. It is shown that rock-mass classifications are already in existence that include both intact-rock and rock-mass properties and that correlations have been developed among the main classification systems. It is found that there is a distinct need for limited standardization specifications but that these should be in the form of suggested methods, one for each classification system, which would achieve some degree of standardization without inhibiting the development or improvement of techniques. There does not seem to be a need for one standard classification that has universal application since the various engineering applications have different classification requirements.

Rock classifications have received increasing attention in recent years and have been applied in many countries to different engineering problems. Although the first major rock-classification system was proposed more than 34 years ago by Terzaghi (1), the recent interest in this subject was prompted, on the one hand, by the construction of more-complex engineering structures such as large tunnels and chambers at greater depths and, on the other hand, by the potential of rock classifications as an aid in the design of those projects.

As a result, the original Terzaghi classification for rock tunneling that has steel supports was modified (2) and new rock-classification systems were proposed. These systems accommodated the new advances in rock-support technology, namely, rock bolts and shotcrete, and also addressed specifically different engineering projects such as tunnels and chambers, slopes and foundations, mines, and others. Both intact-rock and rock-mass characteristics were included. Today, there are so many different rock-classification systems that it is necessary to tabulate the more-common ones (Table 1).

Rock classifications have been successfully used in the United States (3-5), Canada (6), Europe (7-13), South Africa (14,15), Australia (16), New Zealand (17), Japan (18), the USSR (19), and China (20). The success of rock classifications stems from the recognition of their potential as a means of correlating the rock conditions at one site with the experience of rock conditions and support requirements gained at other sites (21). On many projects, the classification approach served as the only practical basis for the design of complex underground structures. The most significant recognition of the importance of rock classifications is found in Austria, in which tunnel-construction contracts incorporate a rock-mass classification as a basis for payment in accordance with standard contract documents (22).

However, the widespread use and development of so many rock classifications have also produced some problems. Questions had to be answered such as, Which rock-classification system is the best? Which system should be applied to a given type of engineering project? Are there any correlations among the systems? Can the rock-classification approach adequately replace other design approaches? Some of these questions were considered in the textbooks by Goodman (23) and by Hoek and Brown (21). Studies were also conducted aimed at comparing the various systems (5,24), and correlations were proposed (25,17). In addition, special committees were appointed to study rock-mass classifications. On the international scene, the International Society for Rock Mechanics (ISRM) and the International Association of Engineering Geology (IAEG) have each established a Commission on Rock Classification. In the United States, the Transportation Research Board (TRB) Committee on Exploration and Classification of Earth Materials has the responsibility of application, evaluation, and correlation of all existing and proposed earth-material classifications, and the American Society for Testing and Materials (ASTM) Committee D-18 has been charged with developing a set of rock-classification standards.

The purpose of this paper is to present a state-of-the-art review of rock classifications and to consider the need and prospects of attaining a standard classification. The paper identifies a number of issues and aims at providing a lead for a discussion of this subject in a way that could be of service to both TRB and ASTM.

AIMS OF ROCK CLASSIFICATIONS

Generally, a rock classification has the following aims in an engineering application:

1. To divide a particular rock mass into groups on the basis of similar behavior,
2. To provide a basis for understanding the characteristics of each group,
3. To yield quantitative data for engineering design, and
4. To provide a common basis for communication.

These aims should be fulfilled by ensuring that a classification system has the following attributes:

1. It is simple, easily remembered, and understandable;
2. Each term is clear and the terminology used is widely acceptable;
3. The most significant properties of rock masses are included;
4. It is based on measurable parameters that can be determined by relevant tests quickly and cheaply in the field; and
5. It is based on a rating system that can weigh the relative importance of classification parameters.

CLASSIFICATION PARAMETERS

The greatest problem in rock classifications is the selection of the parameters of greatest significance. There appears to be no single parameter or index that can fully and quantitatively describe a jointed rock mass for engineering purposes. Various parameters have different significance and only if taken together can they describe a rock mass satisfactorily.

In considering the rock-classification systems, one should first distinguish between classifications of intact-rock materials and those of rock masses. A rock mass (also referred to as a rock system or a

Table 1. Major rock classifications in use.

Name of Classification	Originator and Date	Country of Origin	Application
Rock load	Terzaghi, 1946 (1)	United States	Steel-supported tunnels
Strength coefficient	Protodyakonov, 1951 (19)	USSR	Material friction and cohesion
Stand-up time	Lauffer, 1958 (7)	Austria	Tunneling
Rock-quality designation	Deere, 1963 (2)	United States	Core logging and tunneling
Intact strength	Deere and Miller, 1966 (27)	United States	Communication
Tunnel class	Rabcewicz, Pacher, and Müller, 1970 (8)	Austria	Tunneling
Ground support	U.S. Bureau of Mines, 1971 (38)	United States	Mining
Rock-structure-rating concept	Wickham, Tiedeman, and Skinner, 1972 (3)	United States	Tunneling
Geomechanics classification	Bieniawski, 1973 (14, 29)	South Africa, United States	Tunnels, mines, slopes, and foundations
Q-system	Barton, Lien, and Lunde, 1974 (9)	Norway	Tunneling and large chambers
Geotechnical index	Louis, 1974 (10)	France	Tunneling
Strength and block size	Franklin 1975 (6)	Canada	Tunneling
Mine roof	Kidybinski, 1975 (11)	Poland	Coal mining
Weathering	Dearman, 1976 (39)	Great Britain	Granite
Basic geotechnical classification	ISRM, 1977 (28)	International	General
Rock structure	Hwong, 1978 (20)	People's Republic of China	Tunneling and mining
Rock discontinuity	ISRM, 1979 (26)	International	General

rock body) consists of blocks of rock material (also referred to as the intact rock or rock element) separated by various types of discontinuities such as joints, faults, and bedding planes. For engineering purposes, it is such a heterogeneous and anisotropic rock-mass assemblage that is of primary concern. Thus, the rock material--although a part of the rock mass--is not the most significant factor. Nevertheless, it is a necessary factor because the strength of the rock material constitutes the highest strength limit of the rock mass. A sample of the rock material sometimes represents a small-scale model of the rock mass since they have both been subjected to the same geological processes. In some instances, the rock material may be particularly important, as in the case of tunnel-boring machines.

The strength of the rock material is included as a classification parameter in the majority of rock-mass classification systems. This parameter can be determined in the field by means of the index of point-load strength (26). When only the rock-material properties are included, the classification is termed an intact-rock classification.

The second parameter most commonly employed is the rock-quality designation (RQD) (2). This is a quantitative index based on a modified core-recovery procedure that incorporates only those pieces of core 100 mm (4 in) or more in length. The RQD is a measure of drill-core quality, and it disregards the influence of joint tightness, orientation, continuity, and gouge (infilling). Consequently, the RQD cannot serve as the only parameter for the full description of a rock mass.

Other classification parameters used in the current rock-mass classifications are spacing of discontinuities, condition of discontinuities (roughness, continuity, separation, joint-wall weathering, infilling), orientation of discontinuities, groundwater conditions (inflow, pressure), and stress field.

An excellent discussion of the methods for quantitative description of discontinuities in rock masses can be found in a recent ISRM document (26).

I believe that, in the case of surface excavations and those near-surface underground rock excavations controlled by the structural geological features, the following classification parameters will be important: strength of intact-rock material; spacing, condition, and orientation of discontinuities; and groundwater conditions. In the case of deep underground excavations in which the behavior of rock masses can be stress controlled, knowledge of the virgin stress field, the changes in stress, or both can be of greater significance than the geological parameters. Most civil engineering projects, such as tunnels and subway chambers, will fall into the first category of geologically controlled rock-mass structures.

CURRENT ROCK CLASSIFICATIONS

Rock classifications may be conveniently divided into two groups: intact-rock classifications and rock-mass classifications.

Intact-Rock Classifications

The engineering classification of intact rock proposed by Deere and Miller (27) has been widely recognized as particularly realistic and convenient for use in the field of rock mechanics. It has subsequently been slightly modified to conform to the rounded values of the International System of Units (SI) (14). Recently, the ISRM Commission on Rock Classification recommended different ranges of values for intact-rock strength (28), and these are listed in Table 2 with the original Deere-Miller classification. The main reason for the new ISRM ranges was the opinion that the Deere-Miller classification did not include differentiation in the strength in the range below 25 MPa (<4000 lbf/in^2). It should also be noted that Table 2 leads to a recommendation that the convenient value of 1 MPa (145 lbf/in^2) for the uniaxial compressive strength may be taken as the lowest strength limit for rock materials. Hence, the materials with a strength lower than 1 MPa should be considered soils and described in accordance with the practice for soil mechanics.

As stated earlier, the uniaxial compressive strength of intact-rock materials can be determined in the field by means of the well-known point-load strength index. This involves testing on site of unprepared rock cores by using simple portable equipment. A piece of drill core is compressed between two points and the core fails as a result of fracture across its diameter. The point-load strength index is calculated as the ratio of the applied load to the square of the core diameter. A close correlation exists between the uniaxial compressive strength and the point-load strength index, namely, strength = 24 x point-load index. Standard testing procedures are available for point-load testing (26). The appropriate ranges of point-load strength index are included in Table 2.

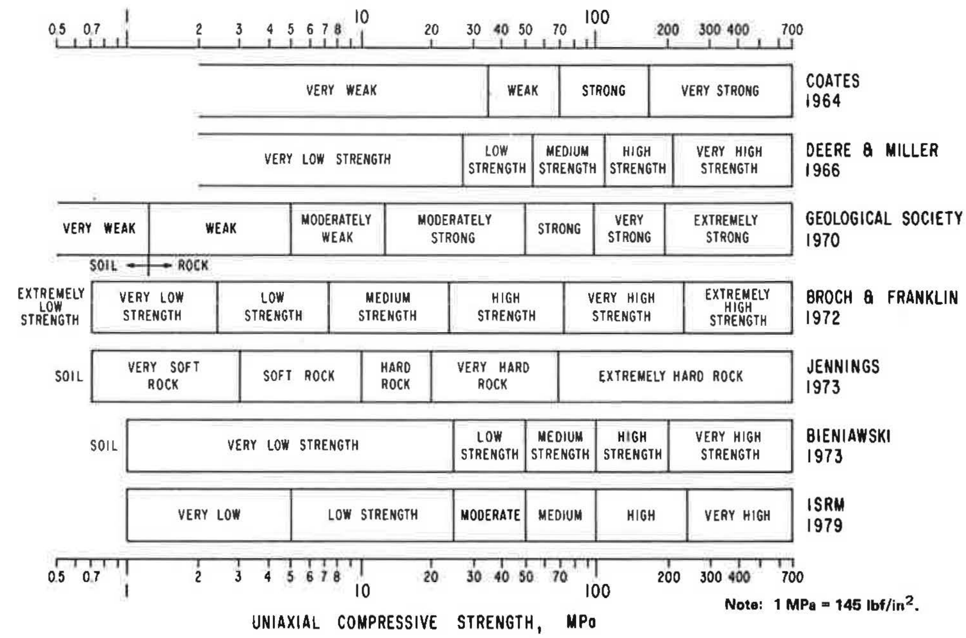
It should be noted that the whole subject of intact-strength classification is a fairly controversial topic since a number of classifications for strength of rock material have been proposed. For the sake of completeness, they are compared in Figure 1.

Table 2. Uniaxial-compressive-strength classifications for intact rock.

ISRM		Deere and Miller (27)		
Commission on Standardization (26) (MPa)	Commission on Rock Classification (28) (MPa)	Strength (MPa)	Strength (lbf/in^2)	Point-Load Strength Index (MPa)
>250 (very high)	>200 (very high)	>200 (very high)	>32 000	>10
100-250 (high)		100-200 (high)	16 000-32 000	4-10
	60-200 (high)	50-100 (medium)	8 000-16 000	2-4
50-100 (medium)		25-50 (low)	4000-8000	1-2
25-50 (moderate)	20-60 (moderate)	<25 (very low)	<4000	<1
5-25 (low)	6-20 (low)			
1-5 (very low)	<6 (very low)			NA

Note: 1 MPa = 145 lbf/in^2.

Figure 1. Classifications for strength of intact rock.



Some intact-rock classifications also include consideration of the modulus of elasticity of rock materials. The Deere-Miller classification, for example, features a diagram of intact-rock strength versus modulus ratio (strength/modulus) (27).

The major limitation of the intact-rock classifications is that they do not provide quantitative data for engineering design purposes. Therefore, their main value lies in enabling better communication in discussions of intact-rock properties.

Rock-Mass Classifications

Although it is not the function of this paper to review in detail all the rock-mass classification systems, it is appropriate to consider the main features of the more-important systems. From the point of view of the transportation engineer, six rock-classification systems may be selected from Table 1. These are as follows: Terzaghi's rock-load classification (1), Lauffer's stand-up time classification (7), the RQD classification (2), the rock-structure-rating (RSR) concept (3), the geomechanics classification (14), and the Q-system (9).

The United States has been particularly active in the field of rock-mass classifications. The Terzaghi classification and the RQD and RSR concepts were all developed in the United States and the geomechanics classification is being extended in the U.S. field of mining.

The first major classification system, proposed by Terzaghi in 1946, was dominant in the United States for some 30 years. The system is excellent for the purpose for which it was evolved, namely, to select steel supports for rock tunnels. It is not suitable for modern tunneling methods that use rock bolts and shotcrete. It provides no quantitative information on the properties of rock masses.

The Lauffer classification (1958) was a considerable step forward in the art of tunneling since it introduced the concept of an active unsupported span and the corresponding stand-up time as a function of rock-mass quality. The disadvantage of this classification is that the stand-up time of an unsupported span is difficult to establish, and the system depends on practical experience.

The Deere classification was introduced in the United States in 1970 and related his RQD to tunnel support. Although this method is simple and practical, it disregards the influence of joint orientations, continuity, and gouge infilling, which are of great importance in many cases (2).

In 1972, Wickham, Tiedeman, and Skinner proposed a classification called the RSR concept. It had the advantage of using numerical ratings for weighing the relative importance of classification parameters. Its disadvantage was that the concept was evolved primarily for steel supports (3).

Finally, in 1973, Bieniawski proposed the geomechanics classification (14,25) and, after working independently, Barton, Lien, and Lunde proposed the Q-system in 1974 (9). Both these

Figure 2. Relationships between stand-up time and unsupported span for various rock-mass classes.

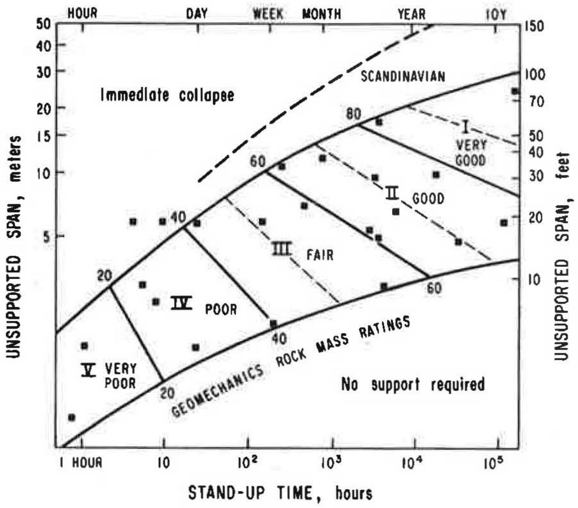
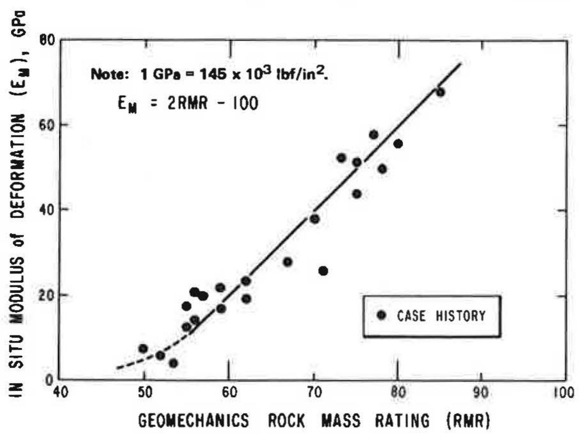


Figure 3. Correlation between the in situ deformation modulus of rock masses and the rock-mass rating from the geomechanics classification.



classifications provide a sound basis for engineering assessment of rock masses and are suitable for rock-bolt and shotcrete support systems. The geomechanics classification is somewhat easier to use and has been widely used in both tunneling and mining. The Q-system is particularly suitable for hard rock tunnels and large chambers (9).

RANGE OF APPLICATIONS

The rock-mass classifications currently in use have reached a high level of development that enables applications to a wide range of engineering problems.

Tunnels and Chambers

The main field of application of rock-mass classifications has traditionally been tunneling. The RSR concept, the geomechanics classification, and the Q-system have all been applied extensively to highway, railroad, and water-conveyance tunnels. In addition, the geomechanics classification and the Q-system were also employed in the design of large rock chambers such as those found in subways and underground hydroelectric schemes. The output of rock-mass classifications for tunneling is the stand-up time of an unsupported roof span. A longer stand-up time can be achieved by selecting suitable rock-reinforcement measures. Today, well-tested support selection guidelines are available for rock tunnels and chambers that feature rock bolts, shotcrete, and steel ribs. However, guidelines for machine-bored tunnels have still to be developed.

The relationships between stand-up time and span length as well as the support guidelines have been developed on the basis of case histories. The geomechanics classification was based on 49 case histories, whereas the Q-system involved 200. Different rock conditions and tunneling practices have clearly affected the selection of stand-up time and length of unsupported spans. As depicted in Figure 2, Scandinavian practice would allow longer unsupported spans than those recommended by the geomechanics classification (29).

Mining

Although mining applications are not within the scope of this paper, they are worth mentioning in passing because mining cases often enable the determination of the limits of rock-mass stability as observed during caving operations. Hence, they are relevant to civil engineering because they offer the opportunity of investigating rock-failure situations. The geomechanics classification has in particular been applied to many mining situations (29) that involved cavability of ore, drift stability, and, more recently in the United States, room-and-pillar coal mining (30).

Rock Slopes

Of the various rock-mass classification systems, only the geomechanics classification has been applied to rock slopes (31,32). For rock slopes, the output from this classification is cohesion and friction data for the five rock-mass classes. In 1976, Steffen classified 35 slopes, of which 20 had failed (32) and, by using the geomechanics classification, he obtained the average values of rock-mass cohesion and friction. With these data, he calculated the factors of safety and plotted the results in the form of a histogram that showed the frequency of occurrence versus the factor of safety. A definite statistical trend was found. However, caution should be exercised when applying this classification to rock slopes since more case histories need to be analyzed. Research in this respect is currently being conducted by K.W. John at Bochum University in the Federal Republic of Germany.

Deformability of Rock Masses

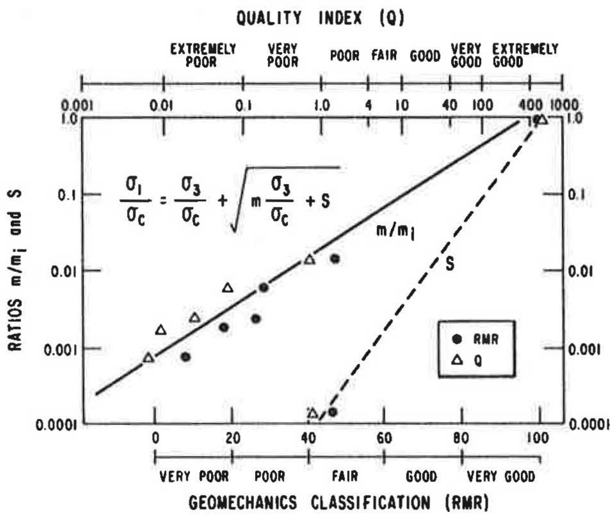
For the design of rock foundations and large rock chambers, knowledge of the modulus of rock-mass deformability is of prime importance. Rock-mass classifications were found useful for estimating in situ deformability of rock masses (33). This is demonstrated in Figure 3, and, as will be seen, the following correlation was obtained:

$$E_M = 2 \times RMR - 100 \quad (1)$$

where E_M is the in situ modulus of deformation in gigapascals (1 GPa = 145 x 10^3 lbf/in²), and RMR is the rock-mass rating from the geomechanics classification.

The above correlation was derived on the basis of 22 case histories that involved a wide range of in situ tests conducted in various parts of the world. The accuracy of prediction of Equation 1 is about 20 percent, which is quite acceptable for rock-engineering purposes.

Figure 4. Hoek's criterion for in situ strength of rock.



Strength of Rock Masses

Hoek and Brown (21) have recently proposed a method for the prediction of rock-mass strength based on rock-mass classifications. In view of the scarcity of reliable information on the strength of rock masses and the very high cost of obtaining such information, Hoek considers it unlikely that comprehensive quantitative analysis of rock-mass strength will ever be possible. Hence, some general guidance based on rock-mass classifications is justifiable.

Hoek proposed a criterion of failure for rock of the following form:

$$\sigma_1/\sigma_c = (\sigma_3/\sigma_c) + \sqrt{m(\sigma_3/\sigma_c) + s} \quad (2)$$

where

σ_1 = major principal stress at failure,
σ_3 = minor principal stress applied to the specimen,
σ_c = uniaxial compressive strength of rock, and
m and s = constants that depend on the properties of the rock and the extent to which it has been broken by being subjected to σ_1 and σ_3.

In Figure 4 [modified from Hoek and Brown (21)], a plot is given of the ratio m/m_i and of the value of s against the geomechanics-classification and the Q-system ratings for Panguna andesite. These relationships may be used as a very rough guide for estimating rock-mass strength. In this procedure, m_i for intact rock is determined from a fit of Equation 2 to triaxial test data. Note that s = 1 for intact rock.

Rippability of Rock

Weaver (34) proposed a rock-mass classification system that enables the assessment of excavation characteristics of earth and rock materials and provided a guide for the assessment of rock rippability. This classification has not found much acceptance. A simpler method was proposed by Franklin, Broch, and Walton (35).

Special Rock Conditions

In situations that involve extremely poor rock conditions, such as swelling and squeezing rock, the Q-system is more effective than the geomechanics classification. The latter is difficult to apply since it was originally developed for shallow tunnels in hard, jointed rock. Although Oliver (36) proposed a rock-durability system for use in conjunction with the geomechanics classification, experience shows (21) that, when work is being done in extremely weak ground, the use of the Q-system is preferable.

RECENT DEVELOPMENTS

Notable developments in the last few years that concern rock classifications fall under the following headings.

Correlations

A number of comparative studies have revealed that there are correlations among the various rock-mass classification systems (17,24,25). In a study of 111 case histories that involved tunnels and chambers in North America, Europe, Africa, and Australia, the following relationship was derived (25):

$$RMR = 9 \log_e Q + 44 \quad (3)$$

where Q is the rock-mass quality (9).

Recently, Rutledge (17) correlated three classification systems on the basis of his tunneling experience in New Zealand. He derived the following relationships:

$$RMR = 13.5 \log Q + 43 \quad (4)$$

$$RSR = 0.77RMR + 12.4 \quad (5)$$

$$RSR = 13.3 \log Q + 46.5 \quad (6)$$

where RSR is the rock-structure rating mentioned earlier (3).

Use of Borehole Data

A trend has emerged for selection of engineering geological parameters on the basis of borehole data, which alone would be sufficient for rock-mass classification purposes without the need for tests in adits or pilot tunnels. As a result of the availability of more-advanced coring techniques such as directional drilling and oriented-core sampling as well as both borehole and core-logging procedures, rock-mass classifications can be conducted on the basis of input data from boreholes alone.

Monitoring During Construction

Although some classification systems tend to rely exclusively on the accumulated case-study experience, it is more appropriate to back predictions of support based on rock-mass classifications by using a monitoring program during construction. The new Austrian tunneling method is a success story of the benefits that can be derived by combining rock classification and monitoring.

Elimination of Two-Tier Support for Tunnels

The traditional concepts of primary (temporary) and secondary (permanent) support for rock tunnels are losing their meaning since the modern tendency is toward a single support system, that is, rock reinforcement necessary to maintain tunnel stability for the life of the project.

Contracting Practices

Although the tunneling-project contracts in Europe have featured rock-mass classifications as a basis of payment for many years, this matter is now receiving attention in some countries outside Europe.

Analytical Procedures

Analytical techniques in the field of rock mechanics have experienced a tremendous growth and, although analytical design cannot as yet replace empirical and observational designs (mainly due to the difficulty of providing reliable input data for the mathematical models), progress can only be maintained if empirical approaches are backed by analytical studies.

LESSONS LEARNED

After so many years of systematic application, the rock-mass classification situation may be compared with that of rock-stress measurements. By the time the first international conference on rock-stress measurements was held (1969 in Lisbon, Portugal), no fewer than 50 stress-measurement techniques could be counted and the problem was how to stop new techniques from being developed for the sake of development and to direct efforts toward perfecting the most promising techniques. No organized approach was made in this respect, and the second international conference on rock-stress measurements (1976 in Sydney, Australia) was nearly canceled for lack of interest. The reason was discouragement because, after so many years, no single reliable and acceptable technique existed.

The same danger looms in the case of rock classifications. There are too many systems available and not enough attention is being devoted to consolidation of efforts on the more-promising techniques.

Positive Aspects

Since the RSR concept was proposed in 1972 by Wickham, Tiedeman, and Skinner (3), three positive aspects have become evident:

1. No matter what classification system is used, the very process of rock-classification procedures enables the designer to gain a better understanding of the influence of the various geological parameters in the overall rock-mass behavior and hence to gain better appreciation of all the factors involved in the engineering problem. This leads to better engineering judgment. Consequently, the lack of general agreement on a single rock-classification system does not really matter; it is better to try two or more systems and, through a parametric study, obtain a better feel of the rock masses.
2. Once a few rock-classification systems have been applied to a given project, it may be found that a simplified classification system particularly suited for this project will evolve. Examples of this approach are the Drakensberg scheme in South Africa, the Dinorwic scheme in Britain, and the Washington Metro project in the United States.
3. Quite apart from the engineering benefits such as design data, rock classifications have been spectacularly successful in ensuring better communication on the project. This leads to high morale as well as economic and technical benefits.

Negative Aspects

In spite of the overall optimism about rock-mass classifications, there are a few negative aspects.

1. There are instances in which rock-mass classifications simply do not work at all. In one case that involved a large cavern, such excellent rock conditions were found that only spot bolting was used in spite of earlier indications, by using rock-mass classifications, that systematic support would be required. This highlights the problem that very few case histories that involved exceptionally good rock were included in the development of the original rock-mass classifications.
2. Even when effective, rock classifications should not be taken too far as a substitute for engineering design. In the case of very complex rock structures such as large multiple caverns, the classification approaches are not sufficient. In such cases, other approaches such as field monitoring or in situ tests may be preferable (used in conjunction with classifications).
3. There may be a tendency to use rock classifications without full understanding of the input and output implications because of the lack of time or the lack of a correct step-by-step application procedure.

NEED FOR STANDARDIZATION

From time to time, it has been questioned whether standardization is progress or is a retrospective and restrictive step. The argument was advanced that standards tend to be followed blindly and that they inhibit the development of new and improved techniques.

On the other hand, a questionnaire circulated by the ISRM Commission on Standardization has shown (37) that the response of industry is overwhelmingly favorable to some form of standardization. It appears that those who have less-ready access to library facilities and less time to choose among the apparently confusing alternative rock-classification procedures published in the literature would appreciate some guidance in the selection and use of classification systems.

PROSPECTS FOR ATTAINING STANDARDIZATION

I believe that, in view of the many rock-classification systems in existence (which has advantages as well as disadvantages), there is a need for some form of standardization. However, the following precautions should be taken:

1. Rather than being prepared as rigid standards, the documents should be termed "suggested methods," which implies that the user may choose to follow one or the other of several alternative methods or to use methods that seem appropriate for that particular project. Thus suggested methods should be written for each major rock-classification system, in particular for these four: Terzaghi, RQD, geomechanics, and Q-system.
2. Each suggested method should have the following warning included in the introduction: "It should be emphasized that the purpose of these suggested methods is to specify rock-classification procedures and to achieve some degree of standardization without inhibiting the development or improvement of techniques."

If the above two precautions are accepted, then the prospects of attaining a degree of standardization in rock classifications are excellent. The TRB Committee on Exploration and Classification of Earth Materials would have little difficulty in applying, evaluating, and correlating the major existing rock classifications, whereas ASTM Committee D-18 could easily prepare suggested methods for intact-rock classifications as well as for rock-mass classifications. In this respect, it should be noted that the existing rock-mass classifications already include both intact-rock and rock-mass properties.

However, the prospect for developing one standard classification that has a universal application does seem low because there is no need for such a system; various engineering applications may have different classification requirements. In fact, it is an advantage to engineers to try a few classification systems in order to compare the results and to get a feel of the important variables in a given project. Thus the lack of agreement among the various classification systems is not a problem; indeed, it may be an advantage. Furthermore, there is no problem with a classification scheme that includes both intact-rock and rock-mass properties. Such systems are already in existence. Concerning intact-rock classifications, these are considered of limited use since they are unable to provide quantitative engineering design data and their main function is one of improving communication.

ASPECTS THAT REQUIRE DISCUSSION

Before a limited standardization of rock classifications is embarked on, the following aspects merit discussion:

1. Classification requirements of various engineering applications,
2. Classification parameters and their determination in the field,
3. Use of classifications in rock slopes (natural and man-made),
4. Collection of case histories for systematic correlation and evaluation of rock classifications, and
5. Determination of whether classification systems are themselves design methods.

CONCLUSIONS

This state-of-the-art review of rock classifications has led to the following conclusions:

1. Rock classifications have reached a high level of development and have been successfully applied in underground construction projects, excavated rock slopes, rock foundations, rock rippability, and mining.
2. Some current rock classifications provide valuable quantitative design data and are thus important aids for the engineer.
3. Intact-rock classifications are of limited practical value; their main function is one of improving communication.
4. Rock-mass classifications include both intact-rock and rock-mass properties, and four such systems are currently in use in the United States. Correlations are available among the most recent rock-classification systems.
5. The lack of agreement among the various rock-classification systems is not a problem but is rather an advantage in that it enables the engineer to compare the data from the various classification systems, which leads to better understanding of the influence of the design variables.
6. There is a distinct need for limited standardization specifications, but these should be in the form of suggested methods (one for each classification system), which would achieve some degree of standardization without inhibiting the development or improvement of techniques.
7. There does not seem to be a need for one standard classification that has a universal application because the various engineering applications have different classification requirements.

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Uniform Rock Classification for Geotechnical Engineering Purposes

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The Unified Rock Classification System (URCS) is used by a large organization, such as the Forest Service of the U.S. Department of Agriculture, to handle projects of all sizes that involve rock. Existing geologic classifications have not provided the necessary information. The usefulness of URCS is that the pertinent natural conditions related to design and strength are emphasized and can be read at a glance, which allows an immediate assessment. A decision is then made as to the appropriate level of detail and the extent of investigation needed to complete or evaluate the project. Efforts can be concentrated toward the rock conditions that are most critical to the project. The data base that covers rock conditions is, in many instances, too detailed for collective analysis. URCS is a type of engineering shorthand to convey maximum design and construction information and omit specific details unrelated to a general evaluation.

The Unified Rock Classification System (URCS) was originally conceived in 1959 and used in simplified form to perform investigations and explorations for the design and construction of major flood-control dams by the Portland District of the U.S. Army Corps of Engineers. The use of URCS materially increased efficiency and produced reliable rock information that resulted in successful design and construction as well as postconstruction evaluation. URCS in its present form dates from 1975 and is used by the Forest Service, U.S. Department of Agriculture (USDA), in Region 6 and parts of Regions 1 and 5. It has been found to be a reliable method of communicating rock conditions (including those in quarries, retaining walls, and extensive rock excavations) for the design of forest access roads. Information on URCS has been published by USDA (1).

PURPOSE AND NEED

The purpose of URCS is to establish a means of making rapid initial assessments of rock conditions related to design and construction by simple field tests that establish natural strength parameters. The purpose is threefold: (a) to present a rock classification for use in engineering geology and geotechnical investigations, (b) to outline field-identification procedures that require simple field apparatus, and (c) to establish the classification relationship to design and performance. Experienced professionals who deal with rock can, and often do, apply the principles of rock mechanics without any formally accepted rock classification. This is not the usual case, especially in an organization that has employees of many experience levels. URCS is not intended to supplant the existing geologic classifications but to implement and to eliminate the inherent confusion of subjective terminology when applied to civil engineering.

Classification is not the chief aim of geotechnical investigations, but a uniform working classification is necessary to effectively supply the needs of a large organization of differing professional disciplines. The assertion that there is no need for a new classification is easily discounted by the statement that a classification is always needed until one is found that meets general approval and acceptance. The statements that there are as many classifications as there are geologists and that no two geotechnical investigators will give the same name to the same rock are unfortunately still true. Because of the number of geotechnical personnel working in the field, it is vital that some uniformity of data exist. Even now, when one reads geotechnical reports, drilling logs, or contract documents, it is not possible to be sure that two different geotechnical specialists who are discussing the same rock are describing sufficiently identical design characteristics. A working classification requires uniform symbols, abbreviations, notations, and definitions that are established to be acceptable procedures (2).

BASIS OF URCS

URCS, as originally conceived, has the following basis:

1. The rock can be defined by simple field tests.
2. The information presented is in simple, understandable, nontechnical terms.
3. The conditions defined are related to design and construction.
4. The design notation is flexible to scale in that it applies to both a very small sample or section of rock or to a large rock excavation and is appropriate to evaluation.
5. The data collected are verifiable, reproducible, and independent of experience but not training.
6. The system is useful to all levels of experience.
7. The system allows immediate assessment, both directly and on notes or documents.

BASIC ELEMENTS

URCS uses the four basic elements, or major physical properties, related to design and construction evaluation: (a) degree of weathering, (b) strength, (c) discontinuity or directional weakness, and (d) gravity or unit weight. By establishing limiting values of these four basic elements by using uniform field tests and observations, terminology, notation, and abbreviations, URCS records and communicates a reliable indication of rock properties and performance. URCS permits a useful estimate of compressive strength, permeability, and shear strength--the three primary properties of a rock. When combined with other geotechnical information (stress history and water-table location), URCS permits a rough estimate of rock performance such as foundation suitability, excavation means, slope stability, material use, blasting character, and water transmittal.

The equipment used for the field tests and observations is simple and available: one's fingers, a 10-power hand lens, a 1-lb (2.2-kg) ball peen hammer, and a spring-loaded scale of the 10-lb (5-kg) range. The fingers determine the degree of weathering and the manual-strength estimate. The hand lens assists in defining the degree of weathering. The ball peen hammer is used to estimate the range of unconfined compressive strength from impact reaction. The spring-loaded scale determines the field-unit weight or apparent specific gravity.

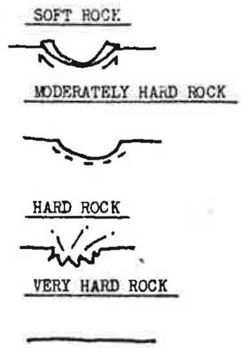
URCS design notation consists of underlined groups of combinations of the letters A through E, which stand for the five categories or design-limiting conditions that define each of the four basic elements, or major physical properties of rock (weathering, strength, discontinuity, and weight). These five limiting conditions will be discussed for each basic element in the sections that follow.

Degree of Weathering

In URCS the degree of weathering is restricted to chemical weathering. There are five design-limiting states or conditions that define the basic element degree of weathering: A, micro fresh state (MFS); B, visually fresh state (VFS); C, stained state (STS); D, partly decomposed state (PDS); and E, completely decomposed state (CDS). MFS is defined by examination by means of a hand lens; VFS is defined by examination by means of the naked eye; STS is weathered but not to the degree that it is remoldable by finger pressure; PDS remolds with the fingers to combinations of rock and soil due to weathering; and CDS remolds to soil.

1. MFS (A) is determined in the field by means of a 10-power hand lens. This condition exists when there is no oxidation alteration of any of the mineral components. It is desirable but not necessary to make this determination for ordinary rock-design evaluation, except for investigations of crushed rock and concrete aggregate.
2. VFS (B), the condition that is representative of the standard of quality for the site and the rock quality with respect to weathering, is not expected to change within the economic limits of the excavation. The mineral components are evaluated with the naked eye, and such an evaluation is usually accept-

Figure 1. Rock hardness related to impact.



able for all foundation and excavation designs. The rock material has a uniform color, usually shades of gray, green, blue, or black. The sample tested and classified is representative of maximum unit weight, maximum specimen strength, and least relative absorption for the site and from which comparisons to STS are made.

3. STS (C) denotes that the rock material is partly or completely discolored due to oxidation but cannot be remolded by means of finger pressure. The mineral components are usually shades of yellow or brown and have a reduced unit weight and a higher absorption of water than VFS. The specimen strength may or may not vary from that of VFS, and a comparison is made at a given site. Weight reduction is expressed as a percentage of the VFS unit weight.

4. PDS (D) is a condition that is defined by moldability and the size of the resulting aggregate. The rock material is remoldable by means of finger pressure to gravel-sized and large rock fragments with or without sand, silt, or clay mixtures. In other words, the material is solid when in place but becomes rock and soil mixtures when excavated. The relative percentage of rock fragments is estimated and the quality of individual fragments is assessed (by URCS), and fines are determined to be plastic or nonplastic. The in-place strength is estimated by manual consistency values or by size, shape, and gradation of the remolded aggregate. The remolded soil aggregate is tested for dilatency, dry strength, and toughness and classified according to field procedures of the Unified Soil Classification System (USCS) (3). Both URCS and USCS symbols are recorded (3,4).

5. CDS (E) is a condition of all remoldable mineral components to sand, silt, or clay, or mixtures of two or more sizes. In other words, the material is rock when in place and becomes soil when excavated. The remolded material is determined to be plastic or nonplastic, and dry-strength, dilatency, and toughness tests are performed. The in-place strength is estimated by manual consistency values. Both URCS and USCS symbols are recorded. Note that in URCS the boundary condition that defines rock and soil on a basis of size is the sieve size that divides gravel and sand (No. 4). It is generally accepted by most investigators, which includes laypersons, that gravel is composed of rock fragments but that sand is composed of minerals.

The degrees of weathering and their URCS symbols are summarized below:

Symbol	Condition	Definition
A	MFS	Fresh by using hand lens
B	VFS	Fresh by using naked eye
C	STS	Weathered but not moldable
D	PDS	Remolds to rock and soil
E	CDS	Remolds to soil

Strength

A reasonable estimate of specimen strength can be made by striking the sample, rock core, or outcrop with the round end of a ball peen hammer (or with the rounded end of a 20-penny nail if the specimen is to be preserved). The resulting characteristic impact reaction indicates a range of unconfined compressive strength (5). The rock specimen or outcrop is struck several times to determine uniformity of response, and a quality is assigned based on the distinct reaction at the point of impact (Figure 1). There are five design-limiting conditions (A-E) in the URCS basic-element category of specimen strength: A, rebound quality (RQ); B, pit quality (PQ); C, dent quality (DQ); D, crater quality (CQ); and E, moldable quality (MBL).

1. RQ (A) rock material shows no reaction under point of impact and is a true brittle-elastic substance in a mechanical sense. This classification quality has an estimated unconfined compressive strength greater than 15 000 lbf/in^2 (103 MPa). The exact unconfined compressive strength is seldom significant with respect to typical civil engineering design and construction once the strength reaches this value. RQ rock material produces free-draining fill that is suitable for road aggregate; however, it is often sharp and angular due to its brittleness and therefore produces a less desirable material. RQ rock material has a very high energy transfer in response to blasting and is difficult to drill and break in the absence of planar separations.

2. PQ (B) rock material produces an explosive departure of mineral grains under the point of impact, which results in a shallow, rough pit. This quality of specimen has an estimated range of unconfined compressive strength of 8000-15 000 lbf/in^2 (55-103 MPa) and is considered hard rock by the construction industry. PQ rock material produces free-draining fill and is suitable for road-surfacing material. It has a high energy transfer in response to blasting, which produces good fragmentation and satisfactory excavation slopes. No special blasting design procedure is necessary.

3. DQ (C) rock material produces a dent or depression under the point of impact. This indicates the presence of pore spaces between the mineral grains. This classification or quality has an estimated range of unconfined compressive strength of 3000-8000 lbf/in^2 (21-55 MPa) and is roughly equivalent to the strength range of concrete. DQ rock material usually does not meet absorption specifications and has a low energy transfer in response to blasting. Special blasting design is necessary to avoid boulders and sand as the end product. DQ material is usually not suitable for road fill or surfacing and is not free draining.

4. CQ (D) rock material has, as the term implies, a reaction under the point of impact that produces a shearing and upthrusting of adjacent mineral grains that is similar in shape to a moon crater. This category has an estimated range of unconfined compressive strength of 1000-3000 lbf/in^2 (7-21 MPa). CQ rock material can usually be recovered during diamond-core drilling operations, has high absorption, and will respond to freeze-thaw stresses by at least cracking and checking. It has a very low energy transfer when blasted and can be excavated by means of machinery, produces poorly drained embankments, and is not suitable for road fill or road-surfacing materials.

5. MBL (E) rock is in a condition in which otherwise visually similar and continuous rock

material can be remolded by means of finger pressure. This category has an unconfined compressive strength of less than 1000 lbf/in^2 (7 MPa). In all cases, the material is examined and tested as a soil and a dual classification is given. The material usually cannot be recovered by diamond-core drilling, can be excavated by machinery, and must be evaluated as a soil for design purposes.

The types of specimen strength and their URCS symbols are summarized below (1 lbf/in^2 = 0.007 MPa):

Symbol	Condition	Range of Unconfined Compressive Strength (lbf/in^2)
A	RQ	15 000
B	PQ	8000-15 000
C	DQ	3000-8000
D	CQ	1000-3000
E	MQ	1000

Discontinuity

Directional weaknesses of a rock mass or rock material are termed planar or linear features. Planar separations are open separations that already exist in the rock mass and are defined by relative capacity to transmit water. Linear features are directional weaknesses due to visual or nonvisual mineral alignment in an otherwise solid rock mass or material that usually requires blasting or mechanical crushing to produce a separation. For purposes of design evaluation, linear features are defined by breakage characteristics. Planar features or open planes of separation are defined by the scale dimension of the rock mass examined and by the geometric determination that defines a plane or a shape. The five design-limiting conditions discussed below are as follows: A, solid random breakage (SRB); B, solid preferred breakage (SPB); C, latent planar separations (LPS); D, two-dimensional open planar separations (2-D); and E, three-dimensional open planar separations (3-D).

1. SRB (A) represents ideal design conditions, in which there is no effect from planar and linear features within the dimension of the rock mass examined. The specimen strength equals the mass strength, so that the strength value of any individual sample tested is directly representative of the entire rock-mass strength. Needless to say, this is seldom the case, except in very limited foundation dimensions.

2. SPB (B) indicates that there is a nonvisual mineral alignment that results in a directional weakness in the rock mass or material. The rock breaks consistently along a constant angle or direction. SPB rock material may produce an undesirable shape or size for rock aggregate or may prevent the achievement of a designed slope in rock excavation. It is adverse in the production of dimension stone.

3. LPS (C) is a category that indicates visual mineral alignment, which may or may not affect the strength or breakage character of the rock mass or rock material during excavation or crushing. The latent planes may be stronger or weaker than the rock mass. The reaction of LPS material to impact defines the strength estimate. Latent planes occur in patterns or at random and are continuous or discontinuous; the plane may be of a measurable thickness. In all cases, the infilling of the material in the latent plane of separation is greater than 1000 lbf/in^2 (7 MPa). LPS material is usually not a foundation-design consideration, because the material is, for practical purposes, a solid. In consideration of rock-slope design or road-aggregate source, blasting energy will, in most cases, be reflected by the latent plane and produce a separation and breakage 90° from the plane alignment.

4. The 2-D (D) category indicates the presence of one or more parallel open planes of separation that pass through the rock mass at the point of examination. The 2-D planar separations may vary in frequency and spacing but do not intersect. The attitude, relief, and continuity of the plane or planes are the fundamental elements of design analysis. Water transmission along the open planes can be determined by observation of the drilling operation or by water testing.

5. The 3-D (E) category indicates the presence of two or more intersecting planar discontinuities or open planes of separation that pass through the rock mass at the point of examination. The planar separation may form patterns or may be random in occurrence. Internal planar separations (IPS) terminate within the rock mass, and mass planar separations (MPS) pass entirely through the rock mass and are infinite in extent in terms of design. By geometric definition, three dimensions form a shape. This shape is often referred to as a joint block, which has an average size and weight that can be estimated. The degree of interlock between joint blocks defines the strength-of-foundation or the stability-of-excavation factor. If MPS occurs, the attitude of the plane or planes with respect to slope or excavation is the chief design factor. Whether or not the planes transmit water is estimated or measured as in category D.

The types of discontinuity and their URCS symbols are summarized below:

Symbol	Condition	Definition
A	SRB	No directional weakness
B	SPB	Nonvisual mineral alignment
C	LPS	Visual mineral alignment
D	2-D	Nonintersecting planes of weakness
E	3-D	Intersecting planes of weakness

Gravity

Density or unit weight has been found to be one of the most useful and reliable means of communicating rock quality to the design engineers or contractors, due to their past experience with rock. The unit weight is determined in the field by using the spring-loaded scale. The apparent specific gravity is determined first; then it is converted to unit weight. Unit weight in pounds per cubic foot is used for a better individual appreciation of weight and changes in weight. Few persons understand the numerical differences of specific gravity without its conversion to unit weight. The URCS basic element related to gravity or weight has five categories or ranges of unit weight: 160 (A), greater than 160 lb/ft^3 (2667 kg/m^3); 150 (B), 150-160 lb/ft^3 (2500-2667 kg/m^3); 140 (C), 140-150 lb/ft^3 (2500-3333 kg/m^3); 130 (D), 130-140 lb/ft^3 (2166-3333 kg/m^3); and 130 (E), less than 130 lb/ft^3. The unit-weight design evaluation establishes the driving force in problems of slope stability, the relative usefulness of the rock material as a surface course or concrete aggregate, or the weight-volume relationship for estimates of haul cost. Unit weight establishes the degree of change due to change of weathering state. As a general rule of thumb, rock material that has a unit weight greater than 160 lb/ft³ is suitable more than 50 percent of the time for use as road aggregate, concrete aggregate, riprap, or jetty stone without laboratory testing. Rock material that has a unit weight of 150-160 lb/ft³ may be acceptable but will require laboratory testing for confirmation. Rock that has a unit weight of 150 lb/ft³ is not usually acceptable for the above purposes, is not free-draining fill, and will degrade. Rock that has a unit weight of less than 130 lb/ft³ can usually be excavated by machinery but will degrade during excavation under the abrasion of excavation equipment.

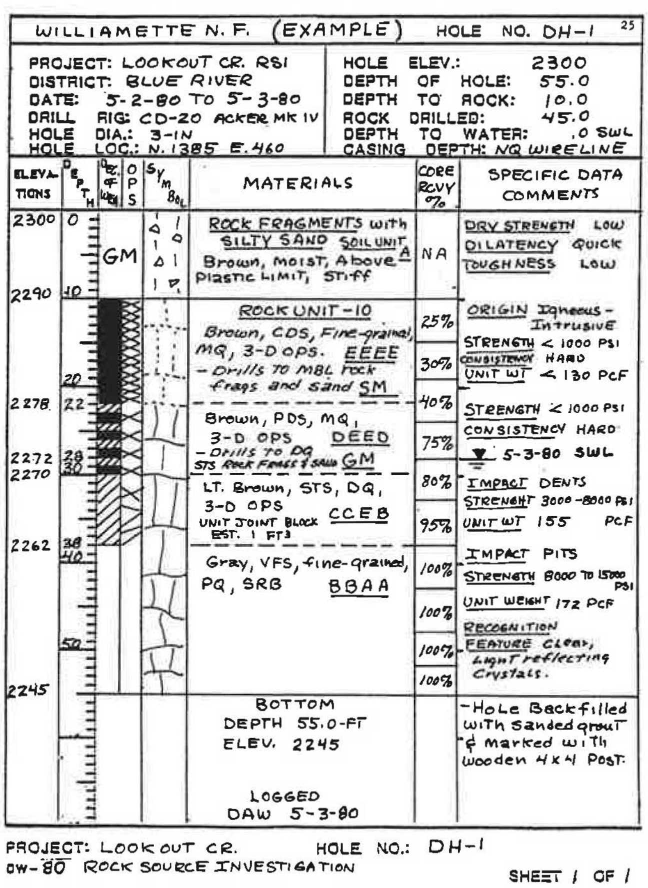
The categories of unit weight, their URCS symbols, and their specific gravity are given below:

Symbol	Unit Weight (lb/ft³)	Specific Gravity
A	160	>2.56
B	150	2.40-2.56
C	140	2.24-2.40
D	130	2.08-2.24
E	130	<2.08

CONTRACT SPECIFICATION OR DESIGN MEMORANDUM

Information that pertains to rock material or rock masses in current contract specifications or design memoranda is sketchy and ambiguous, to say the least, even when supported by laboratory testing. The terminology used in drilling logs and geologic sections usually fails to provide understandable information to the contractor for purposes of bid estimates. Here is an example of a rock description found in a typical contract specification or design memorandum: "Slightly weathered, moderately hard, highly fractured, lightweight, rhyolitic rock." This information is sincerely intended to portray the actual conditions existing at the site and will provide the basis of the design, the cost estimate, or the judgment of the construction method required. Descriptive terms such as these vary widely in meaning, depending on both the individual and the professional experience, and cannot be quantified with any degree of precision or uniformity. Design decisions, cost estimates, or construction methods based on this information vary widely when used by contractors, engineers, planners, or geologists.

Figure 2. Logging example of URCS use.



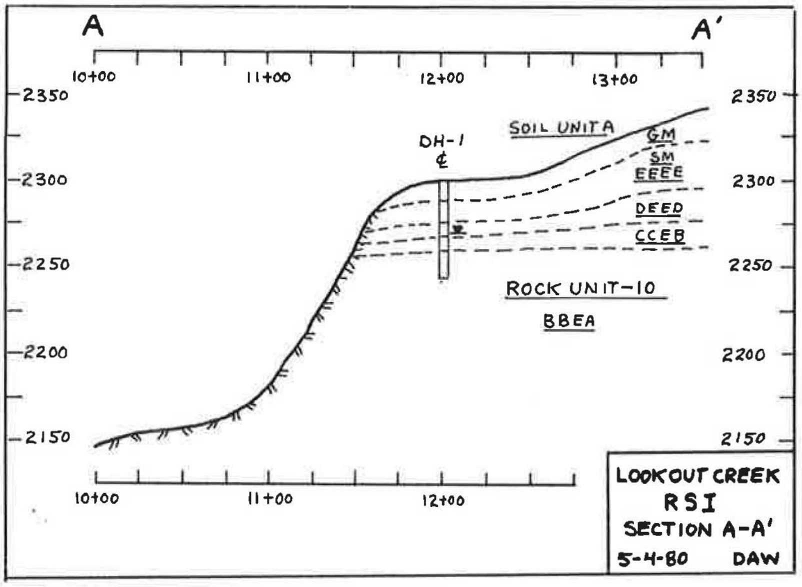
URCS ALTERNATIVE

URCS offers a suitable alternative to this ambiguous descriptive approach. The term "unified" refers to the necessary unification of geology and engineering for geotechnical purposes. The URCS equivalent of the typical rock description for contract specifications and design memoranda is CCED. This simple notation is based on uniform acceptable procedures that define design conditions. This notation indicates that the degree of weathering of the rock is the stained state (STS) or not representative of the standard design condition that exists at the site and that comparative data will have to be determined. The strength of the rock material is dent quality (DQ) and has a range of unconfined compressive strength of 3000-8000 lbf/in², which indicates that it is roughly comparable to concrete in strength when in a weathered state. The rock mass has three-dimensional planar separations (3-D), which will be the primary design and construction consideration with respect to stability, excavation, and material use. The size, shape, volume, and weight of the unit joint block have not been defined and will have to be determined as well as the continuity, attitude, and degree of interlock of the planes. Water transmission will have to be estimated or measured. The unit weight of the rock material is 130-140 lb/ft³, which indicates that there will be full loads for hauling equipment but that the material is probably not free draining nor can it be used in load-bearing fills or for surfacing. This simple but well-defined verifiable design notation is suitable for graphic abstracts, boring logs, plans and sections, and other documentation. Since it is based on basic design elements, the notation provides a reliable means for decision. The notation registers rapidly in the mind during scanning and allows rapid comparison with several rock conditions. Similarities and differences can be established immediately. The simple notation minimizes the drafting effort. The notation prevents subjective connotation and allows recording the significant information on a scale appropriate to the investigation. The information can be checked and verified. See Figures 2 and 3 for examples of how the notation looks in actual use.

CONCLUSIONS

URCS furnishes a means by which a relatively large number of persons from professional and technical disciplines who have different experience levels can work together in a successful team effort. The two government agencies involved were the Portland District Corps of Engineers from 1959 to 1975 and the Forest Service, Region 6, from 1975 to the present (6). URCS, although not universally accepted within these two agencies, did provide reliable information when used, which resulted in

Figure 3. Typical section that shows use of URCS.



effective planning, design, specification, and construction of projects that involved rock. Machiavelli wrote, in The Prince (1513),

> It must be remembered that there is nothing more difficult to plan, more doubtful of success, nor more dangerous to manage than the creation of a new system. For the initiator has the enmity of all who would profit by the preservation of the old institution and merely lukewarm defenders in those who would gain by the new one.

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Part 2
Horizontal Drilling and Drainage

Horizontal Drains and Horizontal Drilling: An Overview

DAVID L. ROYSTER

Subsurface water may act in many ways to reduce the stability of cuts and embankments. Among these are decrease in cohesion, subsurface erosion, lateral pressure in fractures and joints, and excess pore-water pressure. One way of removing subsurface water is to use horizontal drains, which are holes drilled into an embankment or cut slope and cased with a perforated-metal or slotted-plastic liner. The equipment, materials, and procedures used in the drilling and installation of horizontal drains have been improved and refined considerably since the California Division of Highways first introduced the Hydrauger in 1939. The development of polyvinyl chloride pipe, improvements in drill bits and drill stem, and the development of drilling machines capable of producing high thrust and torque have made subsurface drainage a significant and economical alternative in the repair and prevention of some types of landslides.

Water in all its forms (rain, snow, fog, ice, etc.) and in all its occurrences (streams, lakes, oceans, the subsurface, etc.) is the single most troublesome and perplexing substance that must be dealt with by transportation engineers. Of all these occurrences, subsurface water is probably the most perplexing because it is the least predictable, especially as it relates to the stability of cuts and embankments in geologically complex areas.

Subsurface water may act in many ways to reduce the stability of cuts and embankments. These include subsurface erosion, lateral pressure in fractures and joints, decrease in cohesion, reduction in moisture tension, viscous drag due to seepage flow, and excess pore-water pressure (1). By far the most common and significant of these is excess pore-water pressure, which is also referred to as neutral stress and is defined as "the stress transmitted through the fluid that fills the voids between particles of a soil or rock mass" (2). Pore-water pressure increases in a cut or embankment when what may be termed the normal balance among infiltration, migration, and discharge is upset. This can happen rather suddenly during periods of heavy rain when there is high infiltration, or it may develop over longer periods of time due to blockage that results, for example, from consolidation along the contact or along a zone between an embankment and its underlying foundation. A reduction in stability, often to the point of failure, frequently accompanies excess increases in pore-water pressure.

One way of reducing excess pore-water pressure and high seepage forces created by perched water tables or of lowering the normal water table is through the use of horizontal drains. Horizontal drains are holes drilled into an embankment or cut slope and cased with a perforated-metal or slotted-plastic liner.

Although horizontal drains have been used in the stabilization of landslides since about 1939, when the California Division of Highways introduced the Hydrauger (3), the method did not begin to gain wide acceptance for use on a large scale by highway engineers, at least in the eastern states, until many years later. The Tennessee Department of Transportation (TDOT) first used horizontal drains, for example, as recently as 1972 when a series of embankment failures occurred along Interstate 75 in Campbell County (4-7). Drains on this project, which totaled approximately 18 288 m (60 000 ft) in length, were used in conjunction with rock buttresses. Some drains extended as far as 183 m (600 ft) and initially produced flows as great as 26.4 L/min (7 gal/min). Since 1972, TDOT has installed horizontal drains that total more than 53 000 m (175 000 ft) in length. Of the other eastern states, there has been rather widespread use in North Carolina, Kentucky, New York, Virginia, and Mississippi since the early to middle 1970s.

HISTORICAL DEVELOPMENT

Early Equipment, Materials, and Procedures

According to Smith and Stafford (8), the first horizontal drilling rig used in California (the Hydrauger) was a rotary drill mounted on a racked frame in a way that permitted a revolving drill bit to be advanced into the slope by using a hand-operated ratchet lever while water was pumped through the drill rod to cool the bit and wash the cuttings from the hole. Sections of drill rod 1.5 m (5 ft) long were added as the drilling proceeded. The first holes were drilled with a 51-mm (2-in) bit and then reamed to 152 mm (6 in) prior to being cased with a 102-mm (4-in) perforated-metal pipe. It was soon determined that it was more practical to perform the drilling in one operation. This resulted in the development of a 102-mm modified fishtail bit, which was improved over the years by being hardened with various types of alloys. In 1949 the tri-cone roller bit became available and was used in drilling harder materials.

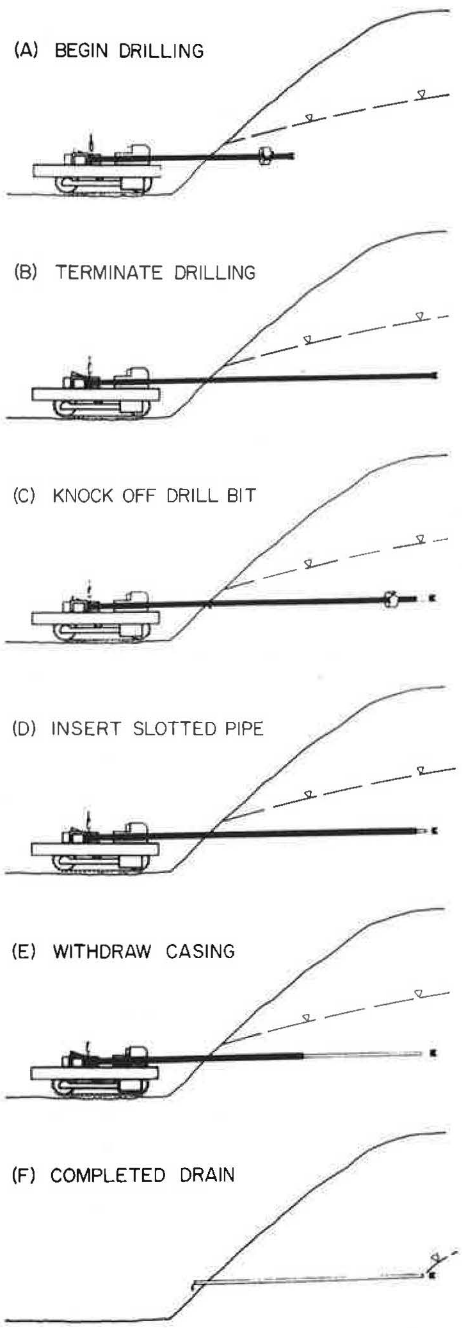
With the improvement of the fishtail bit and the development of the roller bit came further advances. For example, the 102-mm casing was replaced by 51-mm casing. It consisted of standard black pipe perforated with holes 10 mm (0.4 in) in diameter on 76-mm (3-in) spacings drilled in three rows at the quarter points. The pipe was furnished in lengths of 4.8-7.3 m (16-24 ft) without threads or couplings. The casing was then butt-welded as it was jacked into the hole. Smith and Stafford (8) state that the reason for welding the joints rather than using couplings was to hold the perforation rows in alignment and in an upright position.

Since the Hydrauger could be used in only the more-cohesive soils and soft rock, the California Division of Highways developed a more-powerful and versatile machine. This drill, first used in 1951, was a self-propelled unit powered by a 60-hp (45-kW) gasoline engine and equipped with a hydraulic feed to advance the continuous flight augers, which did not require a drilling fluid. In 1953 this machine was further modified so that regular rotary drilling could be accomplished by using N-rod and roller bits. Further improvements were made over the following several years. These involved the design and construction of a machine that had such features as a transmission that permitted control of speed of rotation over a wide range; a hydraulic feed with a minimum 1.8-m (6-ft) stroke capable of exerting a thrust of 17 792 N (4000 lbf); a chuck that could be easily interchanged to accommodate A-rod, N-rod, or casing; and spuds for maintaining alignment.

Present Equipment, Materials, and Procedures

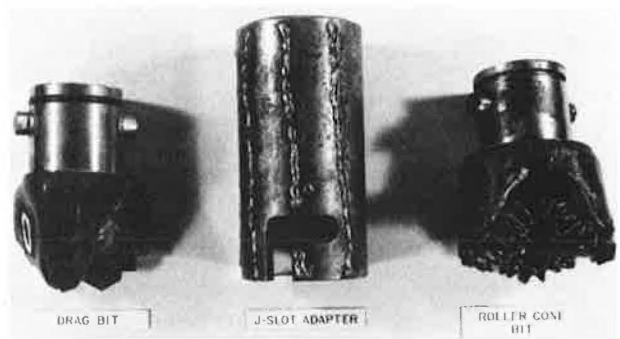
The equipment, materials, and procedures used in the

Figure 1. Standard horizontal drilling procedure.



drilling and installation of horizontal drains have changed considerably since the pioneering efforts of the California Division of Highways in the late 1930s and early 1950s (2,8). The most significant changes have involved the development of plastic polyvinyl chloride (PVC) pipe for use as a liner for the drilled hole; more-frequent uses of air and down-hole hammers; a heavy-walled, flush-coupled drill rod that has tapered threads; expendable drag and roller bits; and a drilling machine capable of developing extremely high thrust and torque. The most frequently used drilling procedure is depicted schematically in Figure 1. It involves the use of an expendable roller or drag bit (Figure 2), which is attached to the drill stem with a slotted adapter. Drill stem is added in 3-m (10-ft) sections as the hole is advanced. Water, pumped at the rate of 19-150 L/min (5-40 gal/min), is used to

Figure 2. Soil and rock bits and slotted adapter for attachment to drill stem.



cool the bit and flush the cuttings from the hole. Once the required depth is reached, the bit is knocked off by reversing the rotating direction of the drill stem. Slotted PVC pipe is then inserted through the drill stem as the drill stem is extracted from the hole.

The plastic pipe used as a liner is Schedule 80, type II PVC that has an inside diameter of 38.1 mm (1.5 in), which conforms to ASTM D1785. It is supplied in lengths of 3 and 6 m (10 and 20 ft). Slot widths usually vary from 0.25 to 1.27 mm (0.010-0.050 in), depending on the type of material to be drained (Figure 3). The smaller slot sizes are used in fine-grained materials, whereas the larger ones are used in the coarser materials. A two-slot circumferential configuration was used on the various Tennessee projects. The specification called for 72 slots per row per meter (22 per foot) that had the 1.27-mm (0.050-in) slot size and 138 slots per row per meter (42 per foot) that had the 0.25-mm (0.020-in) slot size. The two rows of slots are cut with a 120° center-to-center separation. The outer 1.5-3 m (5-10 ft) of the liner are solid (unslotted). The sections are joined together with a fast-setting plastic cement as the pipe is inserted into the hole. Where the sections do not fit firmly, rivets are also used. The space between the liner and the boring (the annulus) is packed with bentonite or some other impervious material to a depth of 0.6-0.9 m (2-3 ft) to direct the flow into the liner and to prevent erosion around the liner.

The drilling machine is track mounted and hydraulically powered (Figure 4). Most are capable of producing 2992 N•m (2200 lbf•ft) or more of torque at 150 rpm. The drill carriage has a 3.35-m (11-ft) stroke and capacity of applying more than 40 940 N (9200 lbf) of thrust to the drill bit. Resistance to torque increases with depth because of friction. This may become especially significant when there is drift or hole deflection or when circulation is restricted. Resistance to torque can be overcome or reduced somewhat by the addition of soap (liquid industrial detergent) to the drilling medium (water, air, or both).

Water has been the principal flushing and cooling agent (drilling medium) used in horizontal drilling; however, air is being used more and more, either by itself or with water. In down-hole percussive drilling, for example, the material being drilled is fragmented by a slowly rotating air-driven piston. This method has proven to be successful in shallow drilling [up to 91 m (300 ft)] in rock formations that have relatively consistent hardness and are not badly fractured or jointed. One contractor reported considerable success in drilling medium to hard granite and gnessic materials in North Carolina (according to John Jensen of Jensen Drilling Company, June 1976). Air was used on the project

Figure 3. Slot sizes and spacings for PVC pipe that has inside diameter of 38 mm and two-row configuration.

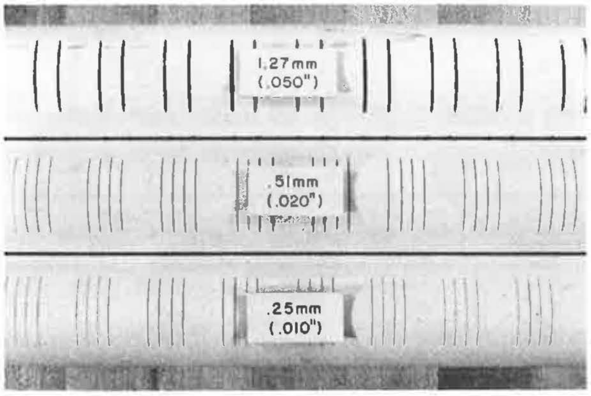


Figure 4. Horizontal drilling rig.

because the drill holes in the softer, more-friable materials eroded severely under the action of the return drill water, which produced deep channels beneath the drill stem. As pressure was applied to the drill bit, the string of drill stem would flex into the eroded channels and break. Jensen states that a number of holes and several sections of drill stem were lost as a result of this phenomenon. Changing to air solved this particular problem.

As is the case with most vertical drilling, it is vitally important in horizontal drilling that circulation be maintained. Cuttings that get trapped between the drill stem and the wall of the boring may create severe torque problems. In a recent project in Tennessee, the driller was having problems penetrating alternating zones of weathered, broken, and relatively solid metasediments. To reduce the problem, rows of hardened buttons or beads were welded on the section of drill stem. These buttons provided the agitation and churning action needed to keep the cuttings moving. Essentially all torque problems were eliminated once this procedure was initiated. As indicated by this example, the success of a drilling project depends a great deal on the skill, technique, and creativity of the driller. The driller must be able to adjust and adapt to each situation as it develops. For example, it is essential to know when to use air, water, and soap, as well as combinations of each; when to vary the input of each; the type of bit to use; and when to adjust the spindle speed and thrust. It would seem that such knowledge would be academic to most drilling firms; yet it may not be, which is why the user should look very closely at the experience of the potential drilling firm before a contract is consummated.

Many firms and agencies that use horizontal drains for the first time are usually quite concerned about such things as borehole guidance and trajectory (even to the point of knowing where the drill bit is at all times); preciseness in hole lengths, spacings, and slope angles; and whether or not every drain produces a sizable flow. The fact is, however, that horizontal drilling for drainage purposes has not reached a point of development at which all these things can be precisely known and absolutely controlled, nor do they need to be. The survey of horizontal holes for determining trajectory involves a very specialized process that would not be cost-effective in the majority of cases. Even if these things were known, the cost of using wedging, or whip stocking, to deflect the drilling assembly would simply be prohibitive in all but a very few situations. The first-time user, therefore, should be cautioned against overrefinements and highly rigid requirements in developing designs and establishing specifications.

The choice of spacings and lengths of drains has, in practice, been done largely on the basis of trial and adjustment. Huculak and Brawner (1) have suggested that the lateral spacing of horizontal drains is dependent on several factors:

1. The quantity of water tapped in the first few installations,
2. The suspected internal drainage pattern,
3. The height and volume of the potentially unstable area, and
4. The permeability of the soil.

They also state that the length of installation for any series of drains is dependent on a number of factors:

1. The height of the cut or distance from crown to toe of the slide,
2. The location of the probable slip plane or firm material, and
3. The distance from the face of the slope to the location of the water source or reservoir.

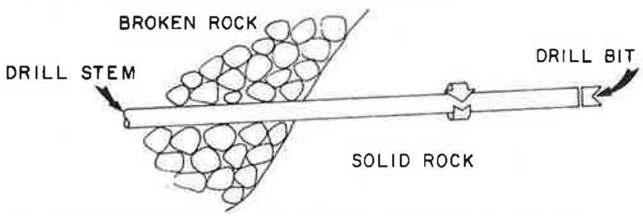
Smith and Stafford (8) have stated, "The spacings of the drains should be dependent upon the drainage characteristics of the soil, the quantity of water intercepted, and the character and magnitude of the slide involved."

Although there are many factors that must be considered in determining lengths, many engineering geologists and geotechnical engineers agree that drains should never extend more than 3-5 m (10-15 ft) beyond the shear zone. Lengths greater than this may only serve to bring additional water into the zone of failure.

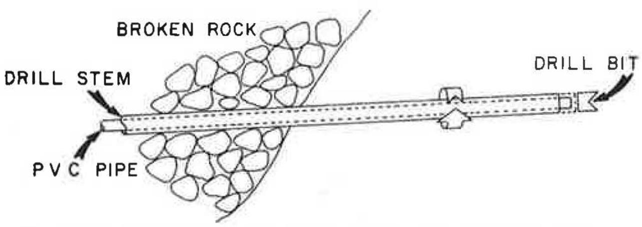
Horizontal borings may be drilled parallel or fanned out from one or more centers. The method chosen is based on topographic expression, subsurface materials and conditions, configuration of the slope or slide, location of the water source or reservoir, etc.

Recent additions to the literature (9,10) have presented discussions of methods for designing drainage systems quantitatively. These methods are based for the most part on ideal conditions; that is, they assume homogeneous and isotropic materials as well as steady-state drainage. Since these methods have not been applied to actual field problems, it is not possible at this time to judge

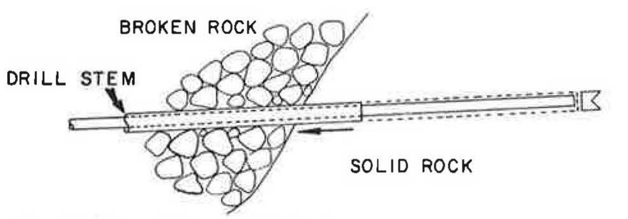
Figure 5. Installation of drains in broken and solid rock.



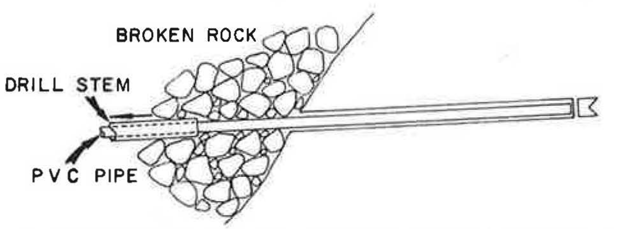
A. Drilling horizontal hole through broken and weathered rock into solid rock.



B. Knock off drill bit and insert PVC pipe through drill stem.



C. Withdraw drill stem. PVC pipe tends to pull out when drill stem is withdrawn from solid rock due to friction and binding between the PVC pipe and the drill stem.



D. Hole collapses in broken rock zone, tending to hold PVC pipe in place while remainder of drill stem is withdrawn.

their validity and applicability.

As indicated previously, it is important that the specifications be rigid enough to ensure that the project objectives are met and yet not so rigid that they severely penalize the contractor for not completing all holes. The many variables in the subsurface make horizontal drilling extremely speculative and very risky, and there will be times when holes simply cannot be completed to the specified or desired depths.

Most lengths should be established, for example, with a plus-or-minus tolerance. That tolerance usually cannot be established until several holes have been drilled, that is, unless a very detailed predesign investigation that used closely spaced borings has been carried out. Even then, a tolerance factor should be applied. As stated before, in many cases drains in landslides should extend no more than 3-5 m beyond the shear zone. If this zone has been well defined in advance, the lengths and tolerances can usually be determined after five or six holes have been drilled. If there is some question as to the location of this zone, especially if solid rock is believed to interface at or just beyond the shear zone, then the specifications could be written for a plus-or-minus length or a certain distance into in-place solid rock, whichever occurs first.

This, of course, causes the questions, What is in-place solid rock and how can one be certain when this point has been reached? One way to tell is by what might be referred to as steady-gauge drilling. This occurs in situations in which the rate of penetration becomes very smooth and uniform and slows to, say, 0.3-1.0 m/h (1-3 ft/h) and in which all the gauges (torque, thrust, and water pressure) level out to a steady and constant reading. Depending on other factors--i.e., experience in adjacent holes, makeup and consistency of cuttings, color of drill water--it may then be determined that the boring should be terminated.

Another way to detemine whether solid rock has been penetrated is by noting the degree of difficulty in holding the liner in place while the drill stem is extracted. When the hole is into rock, the liner tends to pull out when the drill stem is withdrawn due to friction and binding between the liner and drill stem. Once the drill stem is extracted beyond the solid rock zone, the soil and broken rock in the weathered zone collapse around the liner and hold it in place while the remaining sections of drill stem are removed (Figure 5).

CONCLUSIONS

Horizontal drains are not a panacea for correcting or preventing landslides; however, they provide a very workable alternative that most assuredly should be given due consideration in most cases, either as a single remedial or preventive measure or for use in conjunction with other measures. The reason is simple: Water is the principal cause or a principal factor in most landslides, which means that some form of dewatering must nearly always be considered. Furthermore, aside from minimum periodic maintenance on slides that must be lived with, correction by dewatering with horizontal drains is usually the least expensive of all available corrective measures (7).

As with other corrective or preventive measures, an analysis must be made as to effectiveness. It is here, however, that the difficulty lies when drainage (and only drainage) is being considered, because it is not always possible to analyze benefits and effectiveness in a strict quantitative sense. There are just too many variables and unknowns, particularly in geologically complex areas. The decision to use drainage in these areas, therefore, must be based on experience and a sound knowledge and understanding of the geology and geologic structure in and around the slide area.

As to the general state of the art, much more needs to be learned about horizontal drains and horizontal drilling, particularly in terms of equipment capabilities, drilling methods and techniques, hole stabilization, and borehole guidance procedures and capabilities. More information is also needed on existing horizontal drain installations in various soil and geologic materials and environments. There are very few published case histories and current practice papers concerning horizontal drilling and horizontal drains. There are also apparently no textbooks that cover the subject to any significant degree. But in spite of this sparsity of information, it would appear that the use of horizontal drains is increasing. This is especially true in the eastern half of the United States, in which use in the past has been far below that in the western states. As confidence is gained and as information is disseminated about successful installations, the use of horizontal drainage will no doubt increase as a principal alternative for consideration in the repair and prevention of many types of landslides.

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Use of Horizontal Drains: Case Histories from the Colorado Division of Highways

ROBERT K. BARRETT

Horizontal drains have been used on western Colorado highways for several years, in both preconstruction and postconstruction applications. In specific cases, horizontal drains have proved to be cost-effective in preventing and correcting failures in cut-and-fill slopes. The use of horizontal drains has been limited to specific locations at which subsurface mapping and sampling indicates that groundwater is a major detrimental factor and that it can be effectively collected for a required period of time. This report describes four case histories, all located on Interstate 70 in the Vail area of west-central Colorado. At site 1, horizontal drains were used as a postconstruction alternative to a deep underdrain trench that would have cut across an Interstate roadway. At site 2, horizontal drains were used to improve drainage behind a cut slope. At site 3, the drains were used to improve slope stability prior to construction of a major reinforced-earth wall. At site 4, horizontal drains were used for correction of a major cut-slope failure that occurred during construction.

Horizontal drains have been used by the Colorado Division of Highways (CDOH) for several years, in both preconstruction and postconstruction applications. Experience has shown that, in specific instances of slope instability or potential slope instability caused by free water, drainage by drilled holes that have drains installed at an angle that will permit gravity flow can improve slope stability.

In common use, "horizontal drain" refers to flat or low-angle drilled holes that may or may not be permanently cased. Horizontal drains incorporated by CDOH have been limited to small-diameter boreholes that have a 3.7-cm (1.5-in) diameter slotted polyvinyl chloride (PVC) casing installed for permanent drains.

The case histories selected for this paper are located on Interstate 70 in the Vail area in west-central Colorado (Figure 1). I-70 through this area used the US-6 corridor, and in many areas I-70 closely parallels or replaces US-6.

SELECTED CASE HISTORIES

Site 1: Interstate 70 West of Edwards

An unusual groundwater problem developed in the cut slope north of the westbound lanes of I-70 around station 470, about 3.2 km (2 miles) west of Edwards in west-central Colorado. The westbound cut and the eastbound fill were constructed during the summer of 1969. Nothing was observed during preliminary soils and geologic studies to indicate a high water table, and the cut remained dry during construction (see Figure 2).

Each autumn following construction, the cut slope became progressively wetter yet remained dry during the spring and summer. During late summer and fall of 1976, the water problem expanded further to include the median section. It was feared that, should the trend continue, the eastbound embankment could become saturated and fail. An investigation was begun to determine causes for the seasonal groundwater occurrence.

Geologic conditions on the project were fairly uncomplicated. I-70 traverses the lower portion of a major alluvial fan throughout the problem area. The

Figure 1. Location map of I-70 across Colorado.

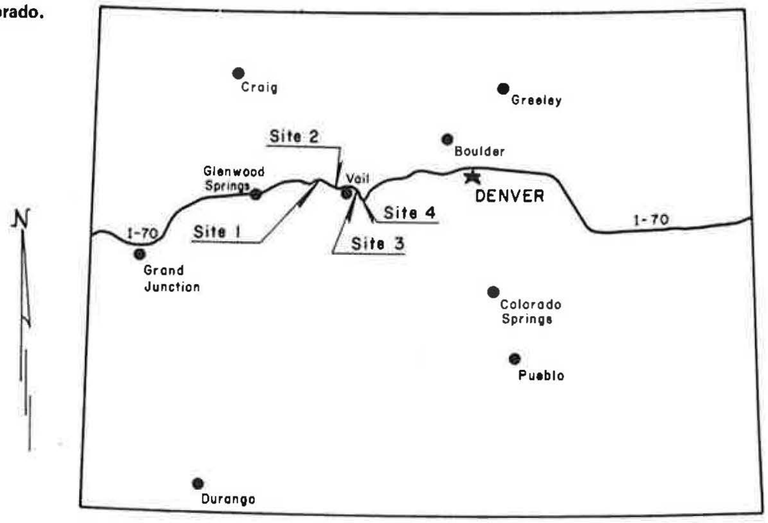
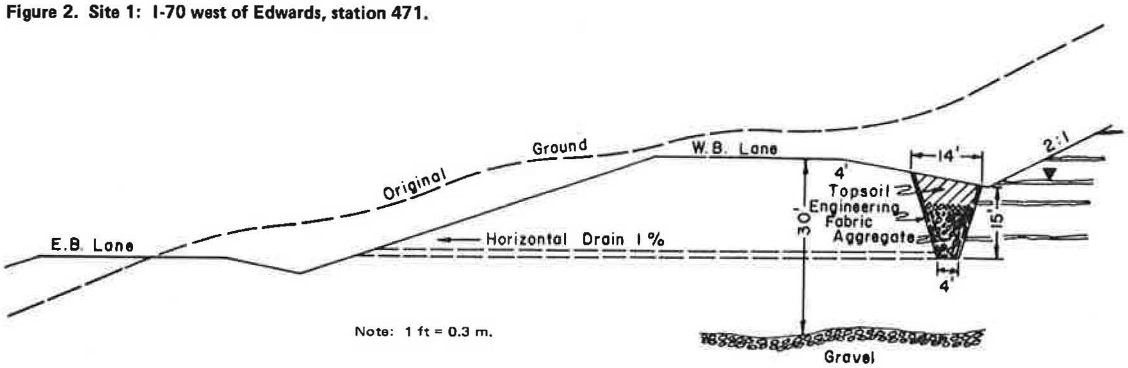


Figure 2. Site 1: I-70 west of Edwards, station 471.



12-m (40-ft) westbound cut through the fan exposes typical alluvial outwash consisting of a locally derived mixture of Eagle Valley Evaporite Formation and Maroon Formation clay, silt, sand, flat pebbles, and cobbles. The eastbound lanes are a fill section that covers most of the lower toe area of the fan.

Vertical drill holes indicated that the alluvial-fan deposit continued in depth about 9 m (30 ft) below the westbound ditch line and lay on an Eagle River gravel deposit. The gravel rests on the Eagle Valley Evaporite Formation. Saturated soils extended only 4.7 m (15 ft) below the ditch. The water that seeped in the lower portion of the westbound backslope exhibited horizontally linear patterns. These patterns appeared to follow lenses of coarser-graded materials, which are probably old stream channels that were buried during depositional (mudflow) events. After such an event, a new channel would temporarily develop on the fan.

It was concluded from the investigation that the seasonal groundwater flow was related to irrigation and snow melt. Water surfaced in the westbound cut from buried stream channels. The eastbound fill acted as a dam as well as compressing underlying soils. These two actions reduced permeability and caused more water to be available to the westbound ditch. Each annual cycle further developed water courses to the westbound ditch area, which caused a greater flow each year at the expense of other, slower paths.

It was concluded that the progressive development of these water courses could lead to sufficient saturation of the eastbound embankment and underlying soils to cause failure and that corrective action was warranted.

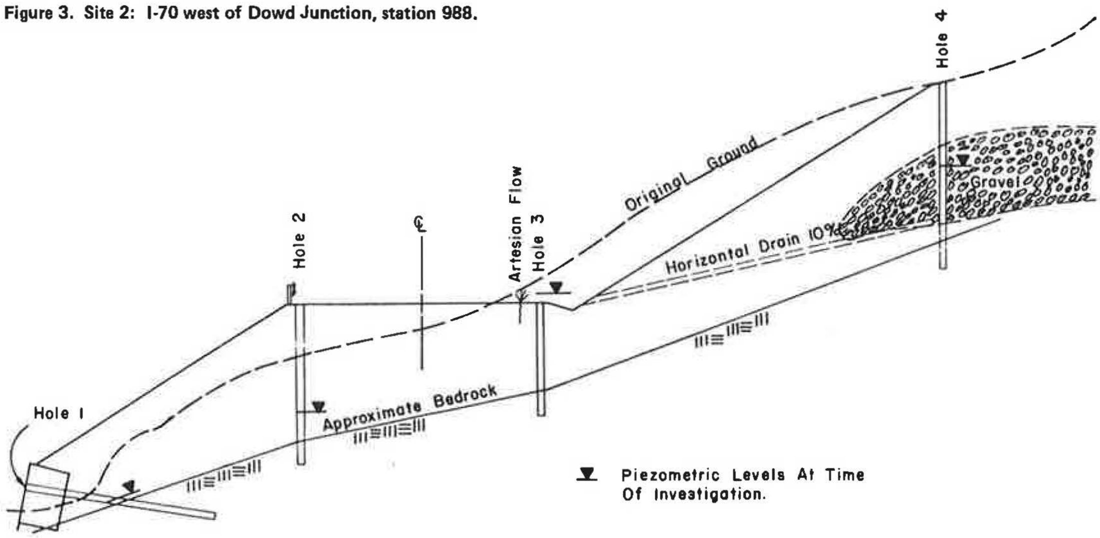
An attempt was made to drain the water into the gravel layer below the westbound ditch line. In semiarid climates, it is not uncommon for fan deposits in steep terrain to be more permeable horizontally than vertically, and improving vertical permeability can alleviate drainage problems in some cases. The gravel deposit proved to be quite impermeable, and none of three vertical drill holes into the gravel would accept a flow of water.

Next, a cost estimate was developed for a typical underdrain correction that included placing 183 m (600 ft) of underdrain under the westbound ditch line. The system would drain into the median inlet of the culvert at station 471+00 and would require cutting a trench about 6 m (20 ft) deep across the westbound lanes. Estimated cost for this work was $53 580.

Construction personnel were reluctant to cut a trench across the westbound lanes. Traffic problems would be considerable, and the trench would be difficult to recompact, which would result in a permanent bump in the road. Several other alternatives were considered, and it was finally decided to try a somewhat unorthodox method that used five small-diameter horizontal drains drilled from the median ditch line and a collection trench lined with engineering fabric in the westbound ditch.

Collection of the water was facilitated by means of the 183-m trench, which was dug in the westbound ditch to a depth of about 4.7 m (15 ft). The trench was lined with a synthetic nonwoven engineering filter fabric and filled to within 1.5 m (5 ft) of the surface with free-draining screened aggregate. After the engineering fabric had been folded over

Figure 3. Site 2: I-70 west of Dowd Junction, station 988.



the aggregate, the ditch was filled with common material and regraded.

The horizontal drain outlets in the median were collected into an open-graded gravel French drain and piped via a short underdrain into the median culvert inlet.

This correction avoided the traffic-control problems. It was constructed at a total cost of $44 344.66, a substantial saving over the conventional underdrain method (1).

Site 2: Interstate 70 West of Dowd Junction

I-70 traverses a geologically complex area as it leaves the Eagle River and turns up Gore Creek, the west drainage of Vail Pass. The Dowd Junction interchange is located on a landslide of several million cubic meters that has moved more than 0.6 m (2 ft) in the past 12 years and has resulted in continuing maintenance operations to relevel the roadway and readjust bridge girders. Immediately west of that slide, a major cut-slope failure occurred during construction and was corrected by using a rock buttress and slope flattening. Immediately west of these two problem areas, artesian flow began from the eastbound lanes in 1970, two years after construction had been completed.

The roadway template in the subject area consists of a narrow-median, sidehill cut-fill section that has a 1.5:1 cut 17 m (50 ft) high for eastbound lanes and a 1.5:1 fill 17 m high retained by a metal bin wall 7 m (22 ft) high located on the shoulder of old US-6 (see Figure 3). When the water was observed, a field inspection was made of the entire area. The inspection revealed that the bin wall was exhibiting distress. Three vertical members were overriding and crushing at the mid-height connections. Total downward movement at mid-height had amounted to about 5 cm (2 in).

It was feared that the westbound fill had inhibited groundwater flow in the underlying soil and that a pore-pressure buildup had resulted that could overturn the wall. A drilling program was launched immediately to determine piezometric levels in the area.

Several holes were drilled from the roadway template--vertically to develop a cross-sectional view and horizontally through the wall to determine soil and groundwater conditions immediately behind the wall. Groundwater tables, subsurface saturation, and piezometric pressures did not approach critical levels. The soil behind the wall was not saturated and appeared to still conform to original design specifications.

Drilling was then extended upslope above the 17-m 1.5:1 cut. A totally unsuspected major gravel deposit was discovered--a buried channel of the Eagle River. This feature was not detectable on aerial photographs and had not been encountered during subsurface investigations during the preliminary engineering phase. Slope-design borings had not extended upslope far enough to encounter the channel.

The springtime artesian phenomenon in the pavement was determined to be related to snow-melt storage in the gravel. Construction of the fill and removal of a significant loading by cutting probably influenced preexisting, preferred drainage paths for the water.

The problem was resolved by installing a series of horizontal drains from the cut-slope ditch line upward into the base of the gravel formation. By the time the drains were actually installed, the artesian flow had ceased. Small amounts of water were initially encountered in the gravel, but this flow also ceased in the autumn. During the following spring, a considerable flow was observed from the drains, and no water appeared in the pavement.

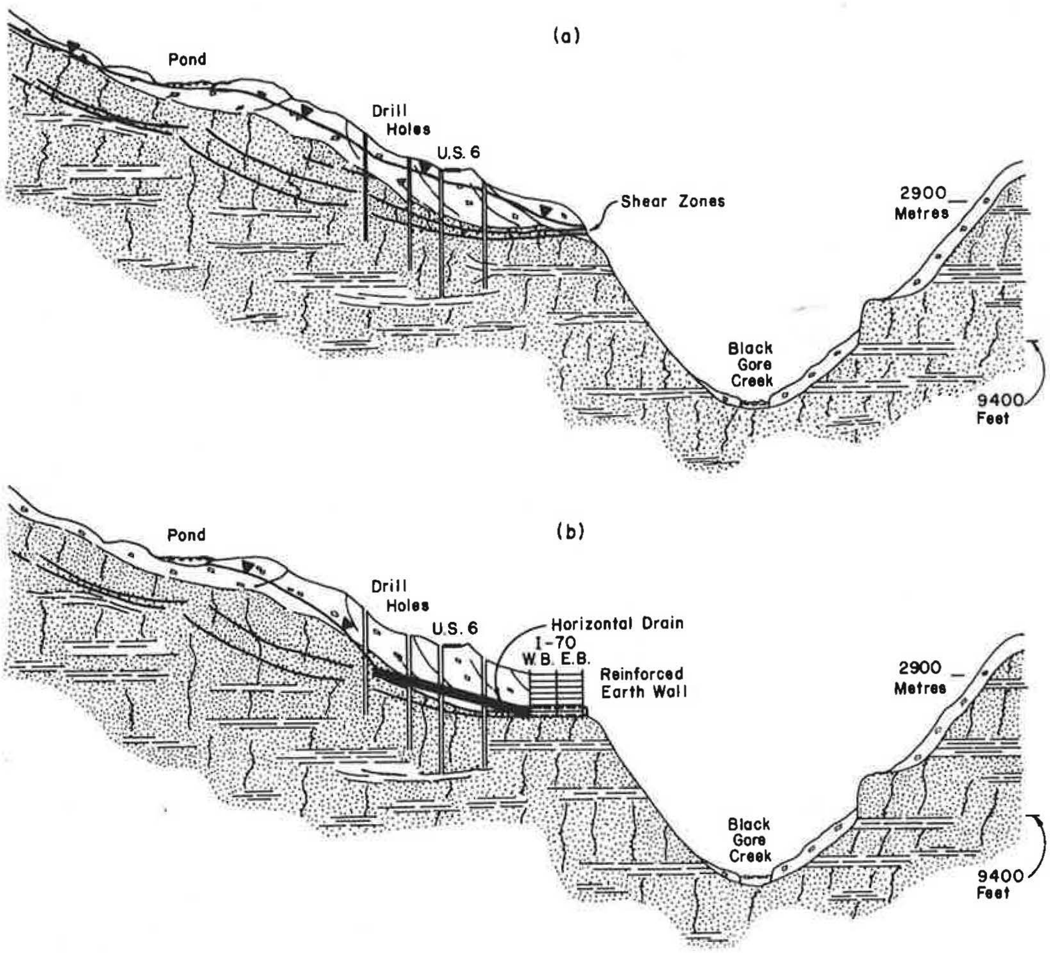
It was determined that movement or readjustment in the bin wall had ceased. The wall was instrumented and monitored for two years; there was no indication of significant vertical or horizontal movement. A likely reason for the distress was some readjustment and settling of backfill material within the bins, which could pull the metal downward and produce overriding at joints in vertical members.

Site 3: Interstate 70, Miller Creek Area, Vail Pass

I-70 over Vail Pass traversed both active and dormant landslides and bedrock failures for 11 of the 23 km (7 of the 14 miles). Geotechnical studies on that section of Interstate 70 encompassed 10 years and involved about 20 person years of geotechnical expertise (2).

The most difficult and challenging problem for geotechnicians on Vail Pass was in the area known as the Miller Creek slide. Maximum dimensions of the slide area are more than 1 km (0.75 mile) long, almost 1 km (0.50 mile) wide, and about 46 m (150 ft) deep. The slide consists of up to 21 m (70 ft)

Figure 4. Site 3: I-70, Miller Creek area, Vail Pass, station 550.



of surficial silty soils that overlie about 24 m (80 ft) of failed bedrock. Water levels varied; in some areas there was water standing in escarpment ponds while other areas were dry (see Figure 4a and 4b).

During the earliest drilling, the lowermost failure plane was thought to be at the interface between soil and rock. Two core drill holes that extended 6 m (20 ft) below this contact indicated intact sandstone and siltstone that dipped about 4°. This matched the geology in the surrounding area. It was found, however, that this did not match a deep core hole drilled previously by the Denver Water Board in conjunction with a proposed water-diversion tunnel (3). A third hole was drilled to 46 m (150 ft) and several obvious shear zones were found. Additional drilling revealed areas of completely disrupted bedrock. Competent, intact bedrock was below this zone. The bedrock failure planes were dipping only 4° in the lower reaches of the slide.

During the course of the drilling, a buried soil profile was discovered in the toe area of the slide. Carbon-14 dating indicated that a sample of organic material from the soil layer was 1000 ± 70 years old, which would indicate that relatively recent movement had occurred.

The topography in the Miller Creek slide area includes a deep canyon below the slide toe into which the slide spills during active periods. Severe grade restrictions, both up- and down-station from the slide area, dictated that the alignment must cross the slide toe. In other critical areas, it was usually possible to vary vertical or horizontal alignment, or both, to optimize alignment and geological conditions. Here, options were few.

The initial proposed alignment included major fills that ramped onto and off the toe area and the roadway in-cut and fill across the toe area. Cuts as long as 15 m (50 ft) were required and, due to the relatively recent movement that had occurred, it was decided that the risk was too great. The safety factor would have been reduced about 25 percent in the immediate area.

A second alignment investigated included twin bridges that had caisson foundations into intact bedrock. In conjunction with this alternative, a consultant presented a European concept that included oversized, elliptical, concrete-lined caisson holes that had free-standing caissons inside. This technique would keep horizontal pressures from acting on the caissons and would allow monitoring of any creep that might develop. Evaluations of existing safety factors, long-term monitoring, and maintenance costs and of the consequences of failure ruled out this alternative.

The alignment ultimately chosen required a high vertical reinforced-earth retaining wall, the toe of which would be placed on the edge of the steep canyon wall and the base of which would necessarily have to be placed on intact bedrock. In place, this design raised existing safety factors by an insignificantly small number; however, excavation of the toe of the slide to permit construction showed a mathematically significant reduction in the safety factor on the critical circle.

A detailed geotechnical investigation followed a tentative decision to try the wall concept. It was decided that, if the water table could be lowered about 11-17 m (35-55 ft) below the groundline for 91 m (300 ft) horizontally behind the excavation, it would be possible to construct the wall. Based on water-level monitoring, it was also decided that the wall would have to be constructed during the winter months--the period of lowest groundwater levels and least groundwater recharge.

In March 1974, a series of horizontal drainage holes was drilled in an effort to lower the groundwater table. A drill access road was attempted along the lower margin of the slide toe along the rim of the canyon. The area was saturated and not frozen. A 2.1-m (7-ft) snow cover had not allowed frost penetration and the bulldozer became helplessly mired. A second road was successfully built about 6 m (20 ft) higher vertically that allowed access of drill equipment. Minimum expectation from drainage holes at this level was to help dry the lower area for a future drainage project.

The horizontal drilling proved successful and interesting. Some intact blocks of rock were encountered that were as large as 24 m (80 ft) across, and almost invariably great bursts of water would flood from the drill holes when the bit broke through and into a shear zone. One such flow was measured at about 575 L/min (200 gal/min). Most flows dropped off substantially in 30 min to 2 h after the aquifer had been tapped.

Water levels were reduced almost 11 m. Due to the low cost and great success of the drainage program, it was decided that it would be worthwhile to drill another series of holes at a lower elevation in conjunction with the construction project. When the second series of drain holes was completed more than two years later, the toe area had, in the interim, dried substantially. The second series lowered water levels to about 17 m below the ground surface within the roadway template.

Prior to construction, three inclinometer holes were installed upslope to monitor any movement that might result from the excavation. The construction plan included very stringent procedures to minimize the amount of slide-toe area excavation to be left unsupported at any given time. These restrictions, coupled with a winter work requirement, caused considerable consternation both to CDOH construction personnel and to the bidding contractors. Some painted a bleak picture for success of the plan.

An unusually dry autumn and snow-free beginning of winter in 1976 permitted construction to proceed as planned. The backfill for the retaining wall was all coarse [2.5-20 cm (1-8 in)] gravel and cobblestones obtained from tailings from early gold-dredging operations in the Blue River. This material was workable at all temperatures, and achieving density even on subzero days was not a problem. The high [21-m (70-ft)] steep (1:1) temporary slopes in the slide toe remained stable and no movement was detected by means of the inclinometers.

This project was completed the following summer and opened to traffic that fall. The inclinometers were read on a weekly basis through the first spring and monthly for the following three years. The drain outlets were observed and the wall was inspected at least bimonthly for two years after completion of the project. No movement has been detected by the inclinometers, the drains are continuing to function, and no distortion can be observed in the wall.

Site 4: Interstate 70 West of Vail Pass Summit

The first major Vail Pass construction contract was awarded in 1979. During the fall of 1974, construction had proceeded to the 33-m (100-ft) level of a through-cut at station 712 planned to be 46 m (150 ft) deep when water began to flow from the floor of the cut. By the following day, tension cracks had appeared 91 m (300 ft) upslope. The contractor was ordered to cease work in the immediate area. An investigation was initiated immediately, which included drilling and instrumentation (see Figure 5a and 5b).

Based on geologic information gathered during the slope-design phase of project development, this cut was being constructed in a failed bedrock slope (4). In fact, these failures extended up and down the valley continuously for more than 3.2 km (2 miles). Drill information indicated that the bedrock (composed of red micaceous sandstone and siltstone and some shale) had been reduced to rubble in many areas. No groundwater table had been observed in the immediate area of the incipient failure.

Although it was recognized that cutting into this slope carried an element of risk of failure, several factors were considered that suggested this risk was acceptable. The bedrock failures were all quite old and apparently had occurred at approximately the same time. The failures had reduced the bedrock to rubble in many areas, which had destroyed the thin shale slip planes. Study indicated that the area was significantly more stable now than at the time of the initial failure.

Alternatives to the cut were not attractive. Avoidance of the cut would have required a twin viaduct in the narrow Black Gore Canyon. This alternative was explored, and the cost was $2 million more than the cut. In addition, the viaducts would have significantly affected Black Gore Creek proper. Placing Black Gore Creek in a culvert and filling the canyon was not even suggested due to environmental concern.

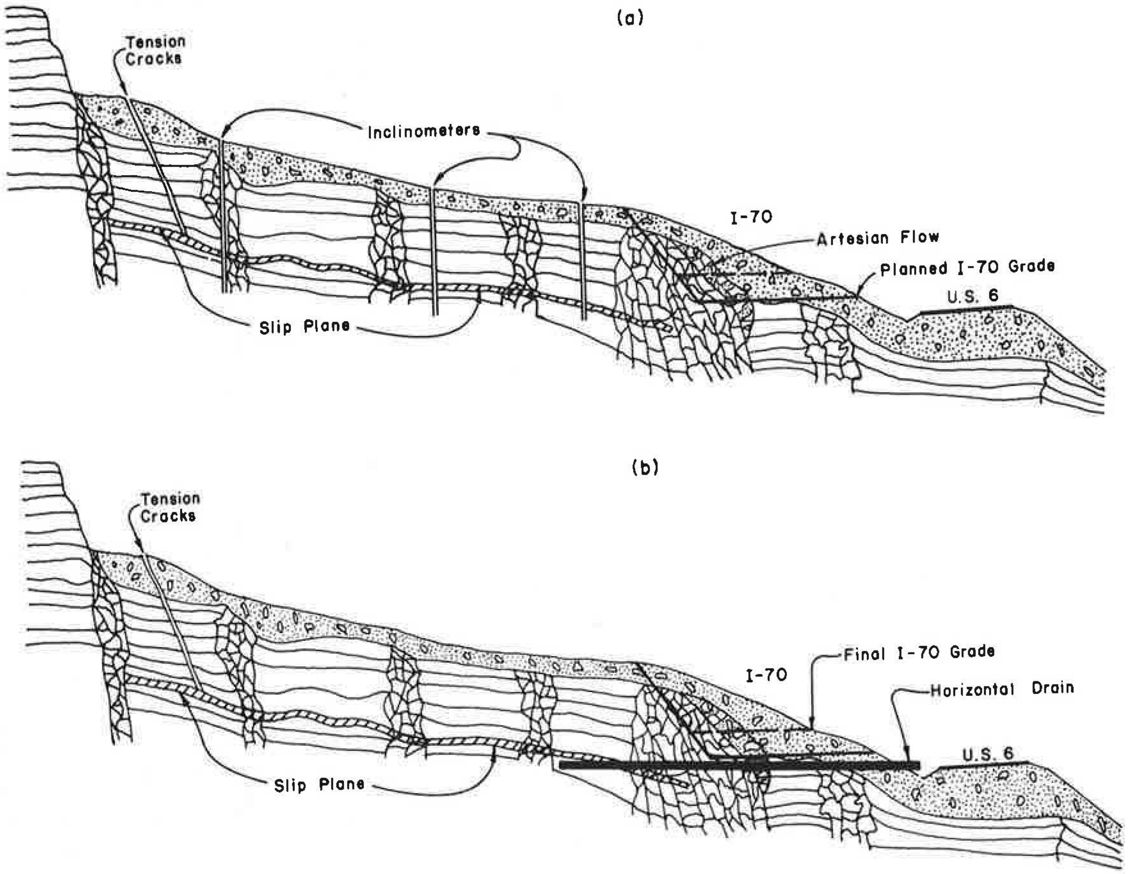
The investigation revealed that the failure was a reactivation of an original failure along a very thin, weak clay shale seam that underlay a sandstone-siltstone sequence. This dipping slip zone was impermeable to the extent that groundwater trapped underneath was under artesian pressure. Exit paths that appeared in the cut floor were quite selective, even though the cut proper was in a highly rubblized zone. It was apparent that a vertical drill hole could have missed the groundwater. The single upslope slope-design drill hole was also drilled in a shear zone. Hence, it was felt at that time that a 1.5:1 slope would have a reasonable probability of stability. Drilling and instrumentation subsequent to the failure located failure planes and piezometric levels in the slide. It was apparent that high piezometric levels were the cause of the failure.

Installation of horizontal drains began some three weeks after the problem was first observed. By that time, total movement exceeded 2 m (6 ft) in some areas, and the failure had spread both upslope and laterally until about 152 000 m^3 (200 000 yd^3) were involved. Drain holes were drilled below the slip plane from US-6, which was about 12 m (40 ft) vertically below final grade and about 18 m (60 ft) below the slip plane.

The initial holes produced spectacular results. Flows that exceeded 378 L/min (100 gal/min) were encountered in the vicinity of the contact between the slip plane and the shear zone. Water in the grade quickly dried and piezometric levels dropped substantially over the lower portion of the slide. Movement slowed and ceased shortly thereafter.

The final correction included a grade raise of 6 m (20 ft) along with provisions for positive, long-term flow capability from the various drains. Total cost of the slide correction was $785 000, due

Figure 5. Site 4: I-70 west of Vail Pass Summit, station 712.



in part to the earthwork imbalance caused by the grade raise. (No cost was assigned to the adverse grade that was introduced.) Thus, even with the failure, the cut alternative was less expensive than the viaduct.

CONCLUSIONS

Water is a major contributor to practically all stability problems on Colorado highways. Drainage is one of the obvious choices for correction of cut-and-fill failures; however, it is a method that seldom can be used alone with confidence. The cases described herein were all successful, but none of the CDOH failure corrections to date rely solely on drainage. Drainage has been used without additional measures for prevention, however, as at sites 1 and 2 discussed in this paper.

There is a tendency to be overly conservative when choosing drainage as one feature of a failure correction. Buttresses and other features are frequently designed for maximum hydrostatic loads on the assumption that drainage installations may cease to function. This redundancy is sometimes unwarranted and expensive. However, there are enough uncertainties associated with long-term reliance on drainage alone so that the redundancy can sometimes be justified.

Some type of drainage is often required to complete the construction phase of an instability correction. Whether or not this drainage is considered permanent and incorporated into the overall design is a question that must be considered carefully. Consequence of failure of the correction is often a guide to the degree of reliance on a drainage system.

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Rockwood Embankment Slide Between Stations 2001+00 and 2018+00: A Horizontal Drain Case History

WILLIAM D. TROLINGER

The most troublesome interval in regard to landslides on Interstate 40 near Rockwood, Tennessee, was along the eastbound lanes between stations 2001+00 and 2018+00. During an extensive investigation of the slides, subsurface water was determined to be one of the major factors in the failures. At the time that alternatives were being prepared as remedial measures, economical methods for the installation of deep horizontal drains were not available. Initially, minimum stabilization of the slide was accomplished by an alignment shift and grade change. Stabilization of the slide to tolerable limits was finally accomplished by the installation of 3120 m (10 325 ft) of horizontal drains in 1976. Flow records from the drains give insight into the reasons for the success of the drainage.

"Without question, the most complicated and frustrating landslide problems in the history of highway construction in the state of Tennessee have occurred along a 6.4-km (4-mile) section of Interstate 40 near Rockwood" (1). Construction on this segment, which began in late 1967, was soon delayed by the failure of a massive fill in January 1968 along the eastbound lanes between stations 2001+00 and 2018+00. Ironically, this failure, although it was one of the more than 30 slides that had to be corrected before all four lanes could be opened to traffic in the late summer of 1974, was the last area to be stabilized to tolerable limits. Remedial measures for these slides included partial to total removal, minor grade and alignment changes, various restraint devices, and various drainage dewatering systems such as French drains, vertical wells, and horizontal drains.

By far the most troublesome interval on the Rockwood project was along the eastbound lane approximately between stations 2001+00 and 2018+00. This paper presents a history of this slide from the time of the initial grading in August 1967 through the initial attempts at stabilization and the later installation of the horizontal drainage system. Finally, it is looked at in its present condition.

GEOLOGIC SETTING

The alignment of Interstate 40 at Rockwood traverses the steep southeastward-facing slope of the Cumberland escarpment through approximately 244 m (800 ft) of elevation in the 6.4-km (4-mile) problem area (Figure 1). The escarpment lies at the juncture of two physiographic provinces, the Valley and Ridge provinces to the east and the Cumberland Plateau to the west (2).

The face of the escarpment, in which the strata dip northwest 30-60°, is composed of a rather distinct and identifiable yet highly complex assemblage of soil and geologic materials and conditions (1). The rock formations include sandstone and conglomerate cap rock of the Crab Orchard Mountains and Gizzard groups, highly variegated clay shale that has interbedded siltstones and sandstones of the Pennington Formation, and Newman limestone near the base of the escarpment (Figure 2). Along the escarpment the strata are severely faulted, folded, jointed, and weathered. These lithologies are overlaid by accumulations of up to 15 m (50 ft) of colluvium. The colluvium, or slope debris, consists of a silty heterogeneous mixture of sand and clays that has conglomeratic sandstone fragments that range from the size of a pebble to close to that of a boxcar.

FAILURE MECHANISM

The landslide between stations 2001+00 and 2018+00 involved both a failure zone along the interface between the colluvium and the weathered shale and a deeper failure zone that passed through sandstone, siltstone, shale, and limestone formations that have been severely folded, jointed, and fractured. Basically, the existence and interaction of clay shales, colluvium, and water underlie the principal reason for failure along the interface between the colluvium and the weathered shale. Accumulations of the colluvial veneer vary considerably in thickness over a relatively small area. This variance is due primarily to the troughs and lows filled with colluvium in the highly undulating and irregular subtopography of the area rather than to buildups of colluvium over a uniform surface. The colluvium, which is very permeable, permits ready access to water from both surface and subsurface sources. These sources are highly variable and unpredictable where springs and seep outlets are covered by colluvium. The underlying clay shale and residual clays are virtually impermeable. The difference in permeability allows the groundwater to collect and sometimes become trapped along the contact between the two materials. Trapped water along the interface reduces the shear strength, and failure occurs.

In an effort to locate the zones or strata in which water pressure was the highest and to define the groundwater conditions, piezometers were installed in 10 different borings. The piezometers were set at varying levels based on the relative porosity and permeability as shown by lithology, structure, and degree of weathering. Although the results obtained were quite inconclusive for defining zones of excessive pore pressure or direct sources of infiltration, quite erratic conditions in the groundwater were found. These were attributed to two factors--the varying permeabilities of the severely jointed and fractured lithologies and the weight of the fill, which closed or blocked drainage in the lower formations and tended to dam up the groundwater and produce higher groundwater levels.

The highly irregular groundwater levels made it difficult to establish the effect of groundwater quantitatively. However, the circumstantial evidence of the effect of water was confirmed by stability analyses that showed that the higher the groundwater level, the less stable the mass (1 m = 3 ft):

Water-Table Location	Safety Factor
6 m below fill surface	0.84
7.5 m below original ground surface	1.00
Below failure surface	1.39
At surface, fill removed	1.24

Considering the observed groundwater levels at their deepest, about 7.5 m (25 ft) below the original ground surface, the analysis showed a safety factor of approximately 1. Considering that the groundwater levels had been altered by the fill to be above the original ground surface, the safety factors were

Figure 1. General alignment of Interstate 40 across the Cumberland Plateau (shaded area).

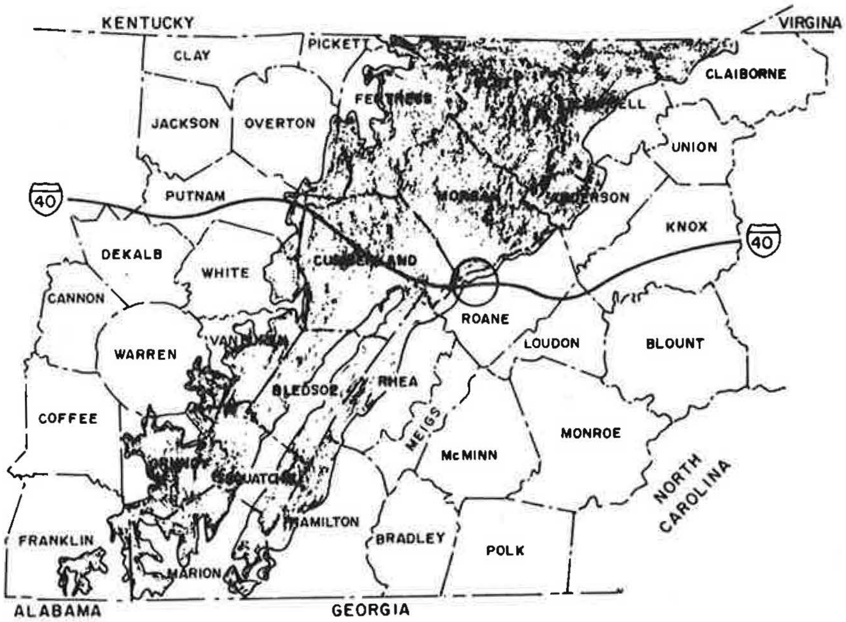
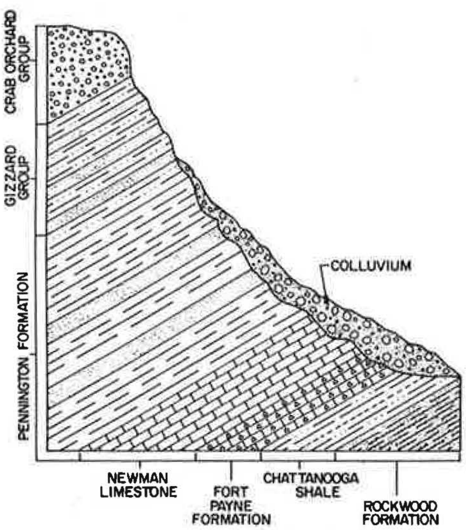


Figure 2. General sequence of geologic formations and material encountered along the plateau escarpment near Rockwood.



well below 1. The actual effective groundwater table was probably between these two limits, equivalent to a calculated safety factor of approximately 0.9 (Figure 3).

It was concluded that water that affected the stability was migrating along the strike of the steeply dipping beds and collecting in a colluvium-filled trough centered beneath the embankment. Movements along the relatively shallow interface--3-12 m (10-40 ft)--were correlated with average infiltration of water or conditions of a perched water table above the interface between the weathered shale and the colluvium. Movements in the deeper failure zone [18-30 m (60-100 ft)] apparently occurred during periods of heavy and prolonged infiltration. During these periods the high water table reduced effective stresses in the lower strata until failure occurred.

CHRONOLOGY OF EVENTS

Construction on the eastbound lane of Interstate 40 began in May 1967 by using clearing and grubbing. Grading of the 42-m (140-ft) high embankment began in August 1967. The initial failure occurred in January 1968 when the embankment was about 27 m (90 ft) below the proposed grade. Grading of the landslide area was halted in March 1968.

The installation of a drainage facility for the slide area was begun in April 1968. The facility consisted of shallow interceptor drainage trenches dug to the left (uphill) side of the embankment. Grading was resumed in mid-April 1968 and, when continuing movement was observed, an effort was made to alleviate excess pore pressures beneath the embankment. A line of vertical shafts 0.9 m (3 ft) in diameter and 10 m (36 ft) deep was drilled outside of the right (downhill) embankment toe and filled with crushed stone. Movement continued, however, and grading was again halted in June 1968.

After preliminary investigations by state highway personnel were performed in the spring of 1968, a consultant was retained to investigate the failure and develop remedial measures. After an extensive investigation in June 1969, several alternatives were developed and cost analyses and degrees of effectiveness were given (3). The alternatives proposed included total removal and replacement, deep drainage that used galleries and wells, drilled shaft restraint, relocation, bridging, and a minimal stabilization plan that included drainage, as shown below:

Alternative	Cost ($000 000s)	Effectiveness (safety factor)
Removal and replacement	4.0	1.4+
Deep drainage	1.75	1.4
Drilled shaft restraint	10.45	1.4
Relocation	1.5	-
Bridge	1.4	1.5
Minimal stabilization	0.6	1.0

Because of costs, the minimal stabilization plan was chosen even though the factor of safety was calculated to be only of the order of that of the original slope prior to construction. The design consisted of removing much of the weight of the existing embankment by a minor alignment change and by lowering the proposed grade about 21 m (70 ft). A drainage system that consisted of interceptor drains at both the uphill and downhill toes of the embankment was included to increase the safety factor to a low, but tolerable, level. This program would involve continued movement, especially during wet periods, and continued maintenance by means of periodic paving on each side of the slide area (Figure 4).

From 1969 to 1972 (when the redesigned embankment was completed) movements continued to occur that totaled 152-203 mm (6-8 in) vertically per year.

HORIZONTAL DRAINAGE SYSTEM

The first horizontal drains in the state were installed in 1972 as part of the remedial measures for several embankment failures on Interstate 75. Since the initial results from these drains appeared to be good, it was felt that additional stability could be gained by installing drains along the toe of the I-40 embankment. These would reduce the driving force of the slide by removing water from the perched water table in the embankment. During

Figure 3. Typical section of embankment at time of failure.

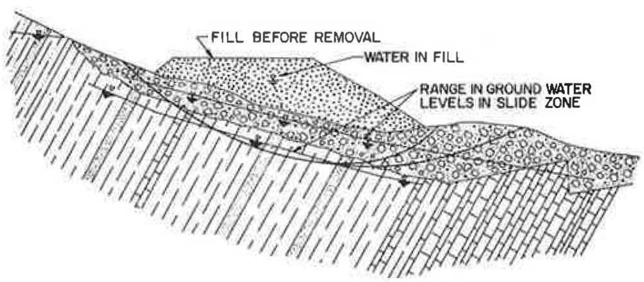
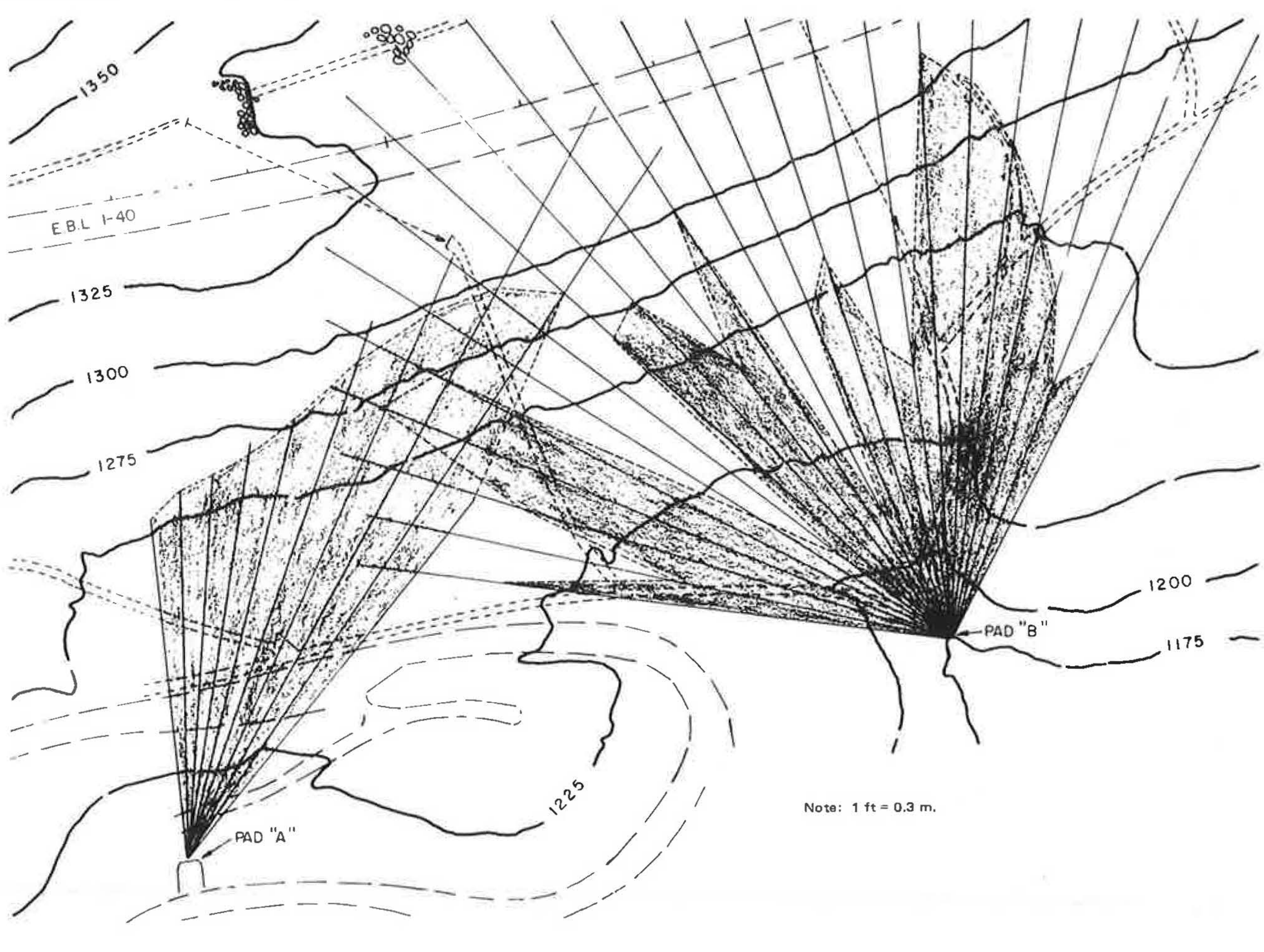


Figure 4. The Rockwood fill slide between stations 2001+00 and 2018+00 (paved areas delineate lateral limits of slide).

Figure 5. Horizontal drain layout at pad A and pad B, 2001+00–2018+00 slide.



the winter of 1972, 44 drains on 5-10 percent grades were installed to lengths of 23-46 m (75-150 ft). Although only approximately one-half of the drains were producers initially, most of them became active during wet periods (3).

Between 1972 and 1975 vertical movement was of the order of 76-127 mm (3-5 in) per year. Although this installation had reduced the amount of movement somewhat, it was felt that additional measures were necessary to further increase the stability of the embankment.

Because of the success of horizontal drains on the I-75 projects it was felt that a modified deep drainage system as proposed in June 1969 could be installed at a much lower cost by using horizontal drains. As a result, a project was begun in August 1975 to install horizontal drains in the two ravines below the fill. Along the slide between stations 2001+00 and 2018+00, 33 drains were to fan out from two centers, one in each ravine (shaded area, Figure 5). At these two centers (pad A and pad B), drilling platforms were excavated at an elevation of 367 m (1175 ft). The excavations into natural slopes of approximately 4H:1V provided a vertical face into which drilling progressed. Grades of 5 percent and lengths that ranged from 27 to 146 m (90-480 ft) were installed in the 10 holes at pad A and 23 holes at pad B. Lengths of up to 183 m (600 ft) were planned for several holes in order to provide drainage from beneath the embankment; however, because of the geologic conditions and the equipment and procedures available at the time, these lengths could not be achieved. Most of the holes were terminated after they had penetrated the heterogeneous colluvium and had been refused in relatively thick [2-5 m (6.5-16 ft)] zones of sandstone (4). Penetration rates were very erratic due to the variation of materials--clay, shale, sandstone--encountered by both drag and roller bits. Several holes were terminated due to lost bits or broken steel. Water used as the principal drilling medium was collected and recirculated during drilling.

Figure 6. Completed horizontal drain installation at pad B.

Schedule 80, type II polyvinyl chloride plastic pipe that had an inside diameter of 38.1 mm (1.5 in) was inserted in each hole as a liner. The pipe, supplied in 6.1-m (20-ft) sections, was inserted through the drill stem as it was extracted from the hole. Because of the variation in the grain size of the materials in the embankment, slot openings of 0.51 mm (0.020 in) were used. After all drains at a pad had been completed, the outer 3-6 m (10-20 ft) of the plastic pipe was encased in corrugated pipe 152.4 mm (6 in) in diameter and the slope reshaped by using shot rock (Figure 6).

Initial rates of flow ranged from drips at pad A to relatively high rates of 2.54-56.8 L/min (0.67-15 gal/min) at pad B. For the most part, although the rates varied considerably, the drains were very active after installation (Table 1). Total flow from the drains varied according to the influence of two factors. One of these--the season of the year--influenced the average total flow. During the wet season (November to June), corresponding to a high water table, flows were generally high. The dry season (June to November), when there was a lower water table, yielded lower flows. For example, average total flows during the wet season were 46.2 L/min (12.2 gal/min) on March 10, 1976, and 66 L/min (17.4 gal/min) on May 21, 1976. During the dry season flows averaged only 2.22 L/min (0.58 gal/min) on August 4, 1976, and 1.92 L/min (0.5 gal/min) on September 1, 1976. The table below presents the total flow rates for each pad throughout the year (1 L/min = 0.264 gal/min):

	Flow (L/min)		
Date	Pad A	Pad B	Total
3/10/76	1.2	44.6	45.8
3/30/76	14.4	106.7	121.1
5/20/76	2.5	63.2	65.7
9/04/76	0.53	1.67	2.2
10/01/76	0.45	1.44	1.89

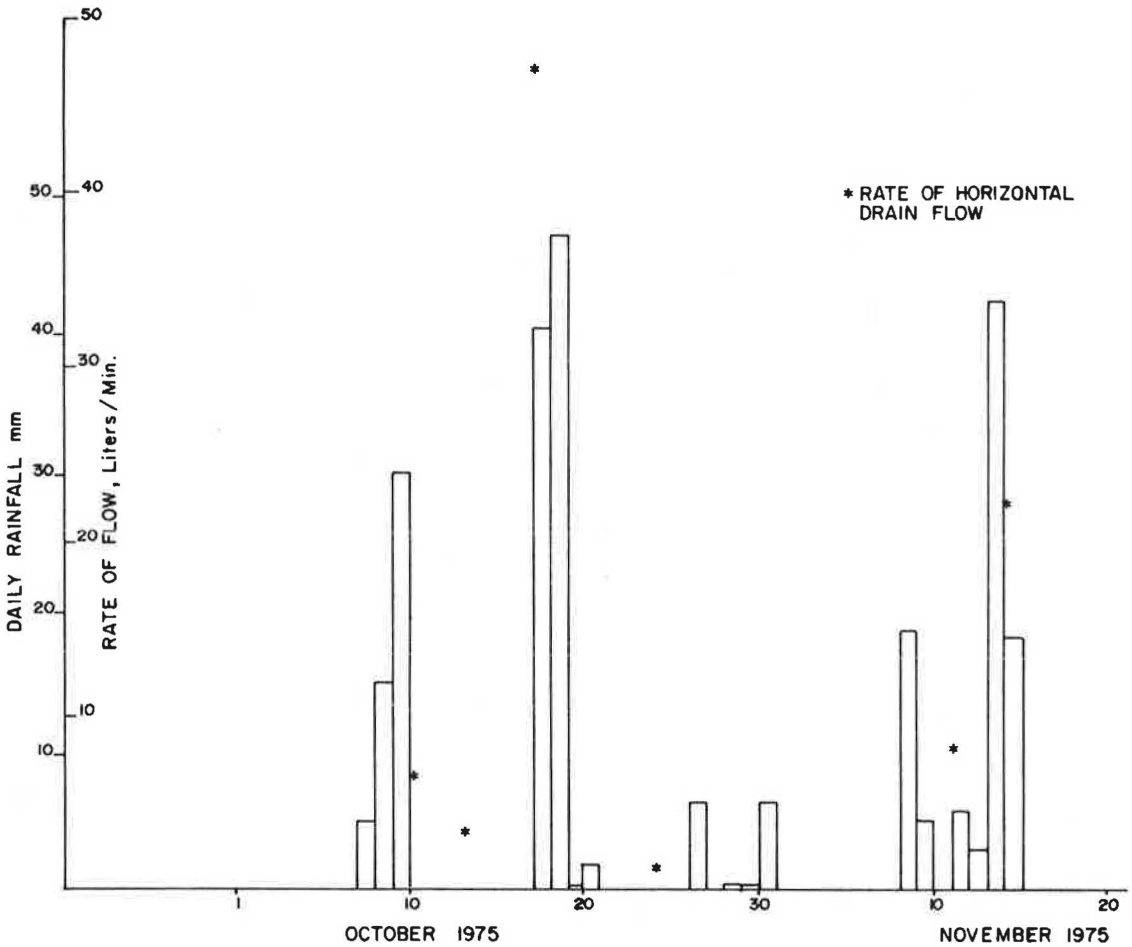
The second factor that influenced the total flow consisted of the almost-immediate response to short-duration heavy rainfall. Figure 7 shows the daily rainfall and the corresponding rate of flow for drain number 23 during October and November 1975. On both October 17 and November 13, the rate

Table 1. Rate of flow of horizontal drains at 2001+00–2018+00 slide, 1975.

Pad	Drain No.	Date	Flow (L/min)	Date	Flow (L/min)	Date	Flow (L/min)	Date	Flow (L/min)	Date	Flow (L/min)
A	5	12/5	0.19	12/8	0.19	12/12	0.38	12/22	0.3		
	6	12/4	1.25	12/5	1.32	12/12	0.08	12/22	0.08		
	10	12/5	0.08	12/8	0	12/12	0.08	12/22	0.08		
B	1	10/6	3.79	10/10	3.03	10/13	2.54	10/22	3.03	11/4	1.25
	3	10/6	7.57	10/10	3.41	10/13	2.65	10/22	3.79	11/4	2.27
	5	10/8	3.22	10/10	3.22	10/15	3.8	10/22	2.54	11/5	1.97
	6	10/8	2.54	10/10	2.54	10/15	2.08	10/22	2.08	11/5	1.25
	18	10/17	18.9	10/20	9.84	10/27	6.06	11/3	3.79	11/24	3.79
	20	10/15	49.2	10/16	49.2	10/23	3.79	10/30	2.84	11/24	0.95
	21	10/14	56.8	10/15	34.1	10/21	9.08	10/28	0.49	11/24	0.08
	22	10/10	3.79	10/13	3.79	10/17	27.3	10/24	1.21	11/10	1.7
	23	10/10	6.43	10/13	3.48	10/17	49.2	10/24	1.4	11/10	0.76

Note: 1 L/min = 0.264 gal/min.

Figure 7. Daily rainfall versus rate of flow at drain number 23.



of flow increased either during or immediately after heavy rainfall.

CONCLUSIONS

Of the 4842 m (15 885 ft) of proposed horizontal drains, 3120 m (10 325 ft) were installed at a total cost of $113 692. This amounted to more than $1.5 million less than the original deep-drainage alternative. Granted that the horizontal drainage system was not nearly so extensive as the original alternative, it has reduced movements at the slide to well within tolerable limits that can be maintained at a reasonable cost.

In the more than three years since the drainage installation was completed (March 1976), vertical movement measured at the scarp where it crosses the roadway (at approximately station 2004+00) has averaged approximately 7.5 mm (0.3 in) per year. When compared with the vertical movements of 152-203 mm (6-8 in) per year before the horizontal drains were installed, the results must be considered excellent.

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Horizontal Drilling by Using an Oriented Core at Wheeler Junction, Colorado

CHARLES S. ROBINSON

A horizontal diamond-bit core hole (7.30 cm in diameter) was drilled near Wheeler Junction in 1969 in conjunction with the studies for a possible tunnel on Interstate 70 about 148 km west of Denver. The tunnel, about 450 m long, was to be driven through a prominent ridge to reduce the length and the environmental impact of construction of I-70. The ridge is composed of highly sheared and partially altered Precambrian metasedimentary and granitic rocks within the Mosquito fault zone. The core hole was drilled along the east side of the proposed twin tunnels at about the invert of the westbound lane. The bearing and plunge of the hole were controlled by surveying and the use of wedges, and the core was oriented. Geotechnical and geologic data in their true relation to the alignment of the proposed tunnel were obtained from the oriented core. Horizontal drill holes from which oriented core is obtained can be used to obtain geotechnical and geological data for the design and construction of a tunnel at a much lower cost than would have been required for the driving of a pilot tunnel. The only limit to the use of the technique is the present equipment's ability to drill holes of no more than 1750 m.

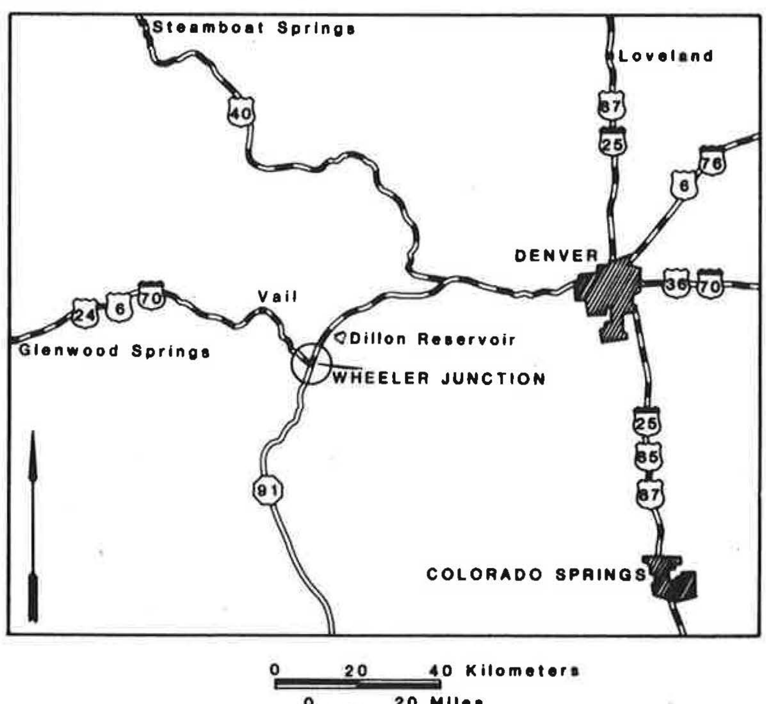
In the studies for the alignment of Interstate 70 west of Denver, Colorado, the feasibility of constructing a tunnel through a prominent rock ridge near Frisco, Colorado, was considered. To evaluate the geologic conditions, a diamond-bit core hole 7.30 cm in diameter was drilled. By controlling the core hole and by orienting the core, it was possible to obtain knowledge of the geotechnical and geologic conditions at depth along the tunnel at a considerably lower cost than would have been required to drive a pilot tunnel.

The rock ridge was along Ten Mile Creek about 9 km southwest of Frisco and about 1 km northeast of Wheeler Junction--the junction between I-70 and CO-91 (Figure 1). The area is typical of the high glaciated mountain areas of Colorado; it has deep, U-shaped valleys bounded by steep cliffs. The area was at the south end of the Gore Range in the central part of the southern Rocky Mountains. The rocks in this area are Precambrian metasedimentary rock (chiefly varieties of gneiss) that were intruded by Precambrian granitic rocks. At the proposed tunnel site the bedrock is cut by the Mosquito fault. The Mosquito fault is a major structure in Colorado. About 22 km southwest of Wheeler Junction, at Climax, Wallace (1) estimates that there has been about 2700 m of displacement along the fault. At the proposed tunnel site the rock is highly sheared, silicified, and cut by quartz veins that contain minor amounts of sulfide minerals.

The crest of the prominent ridge is more than 122 m above Ten Mile Creek. The purpose of a tunnel through the ridge was to shorten the highway between Frisco and Wheeler Junction and to reduce the environmental impact of the highway. The alternative to the tunnel was a high cut along Ten Mile Creek.

The investigations for the proposed tunnel included detailed geologic mapping of the surface in the vicinity of the tunnel and the drilling of a core hole at about the invert on the east side of the westbound lane. The geologic mapping was initiated by Charles S. Robinson and Associates under a contract with the Colorado Division of Highways in July 1969. The core drilling was done by Boyles Brothers Drilling Company. The core hole was surveyed and directionally controlled, and the core was oriented and logged by Charles S. Robinson and Associates. Drilling started August 1, 1969, at the

Figure 1. Index map that shows location of Wheeler Junction, Colorado.



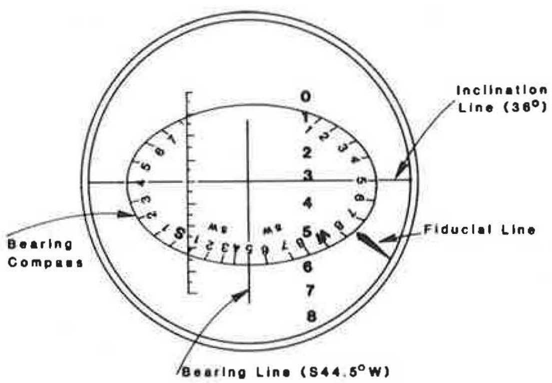
west end of the proposed tunnel and was completed to a depth of 359 m by November 15. A hole from near the east portal was started December 15, 1969, and drilled to 157 m by March 15, 1970.

HORIZONTAL DRILLING

Horizontal drilling has been conducted for drainage of slopes, for removal of methane from coal mines, and for exploration in the mining and the construction industries. The prinicipal reason for the use of horizontal drill holes in underground construction is to determine geologic conditions before construction. The alternative, in tunnel construction, is a pilot tunnel, which gives more-complete information but, depending on conditions, will cost about 10 times as much as will a horizontal drill hole. Horizontal drilling for tunnel construction apparently was first used about 1964 (2). Drilling methods have included diamond-core drilling, rotary drilling, and in-hole percussion-type drilling. The longest hole (1615 m) was drilled as part of the investigations for the Seikan tunnel project in Japan and was drilled by using a rotary bit. Most holes more than 610 m long are drilled by using diamond-coring techniques (2). The longest horizontal diamond-core hole (1220 m) is reported (2) to have been drilled in South Africa.

To drill the hole at Wheeler Junction, a Longyear 44 was mounted on a 3-m^3 block of concrete. The contract between the Colorado Department of Highways and Charles S. Robinson and Associates required a continuous core hole of about 457 m that had the core oriented and no more than 1 percent deviation. The drill was aligned by transit along the line of the hole. The drilling was NX size by using wire line and a double-tube core barrel. Drilling from the west end had advanced to a depth of 360 m when the hole had to be abandoned because two wedges used to control the direction of the hole became stuck. A second core hole was started near the east portal of the proposed tunnel and drilled to a depth of 157 m.

Figure 2. Survey disk for Eastman R Single Shot.



SURVEYING AND CORE ORIENTATION

The surveying of the drill hole and the orientation of the core were done with the use of an Eastman Oil Well Survey Company R Single Shot instrument. The R Single Shot records on a photographic disk the magnetic direction and the inclination of a point in an uncased drill hole. The recording is made by inserting the instrument to a predetermined depth. The position of a very sensitive plumb bob is photographed in reference to a calibrated compass card and a concentric-ring glass. Figure 2 is an example of the survey disk for an Eastman R Single Shot. The hole was surveyed at intervals of 8-30 m or whenever an orientation of core was required.

Core orientation was done by using a special core-scribing tool that was aligned with the Eastman R Single Shot. Figure 3 is a schematic drawing of the scribing tool.

To orient the core, the retaining ring in the core barrel and the liner of the core barrel were removed and approximately 25 cm of core was drilled. The rods, core barrel, and bit were removed from the hole. An aligned assembly (which consisted of the scribing tool, the surveying instrument in the instrument case, and four aluminum rods) was attached to the drill rods and inserted or pumped into the hole. The fiducial line in the surveying instrument is aligned with the fiducial knife edge in the scribing tool. The scribing tool was forced over the stub of rock left in the hole in order to scribe lines down the side of the core. The assembly was left stationary in the hole long enough for the survey instrument to record the bearing and inclination of the hole and the fiducial line. The drill rods were rotated about a quarter turn to break off the rock stub, and the rods, surveying tool, and scribing tool were extracted from the drill hole together with the rock stub. The scribed rock stub was removed from the scribing tool and placed in the core box so that it lay with the fiducial line up. Core in the core box and additional core drilled were oriented by matching pieces with the scribed stub. The fiducial line was marked on the matching core by using a chalk line or by using a straightedge and a felt-tipped pen.

Figure 3. Core-scribing tool.

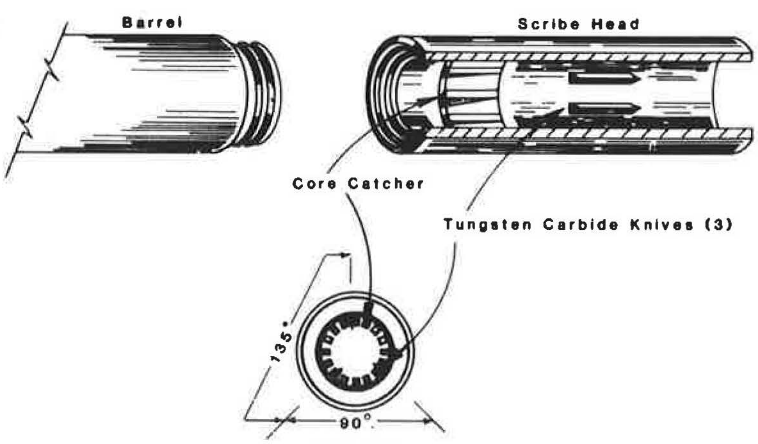
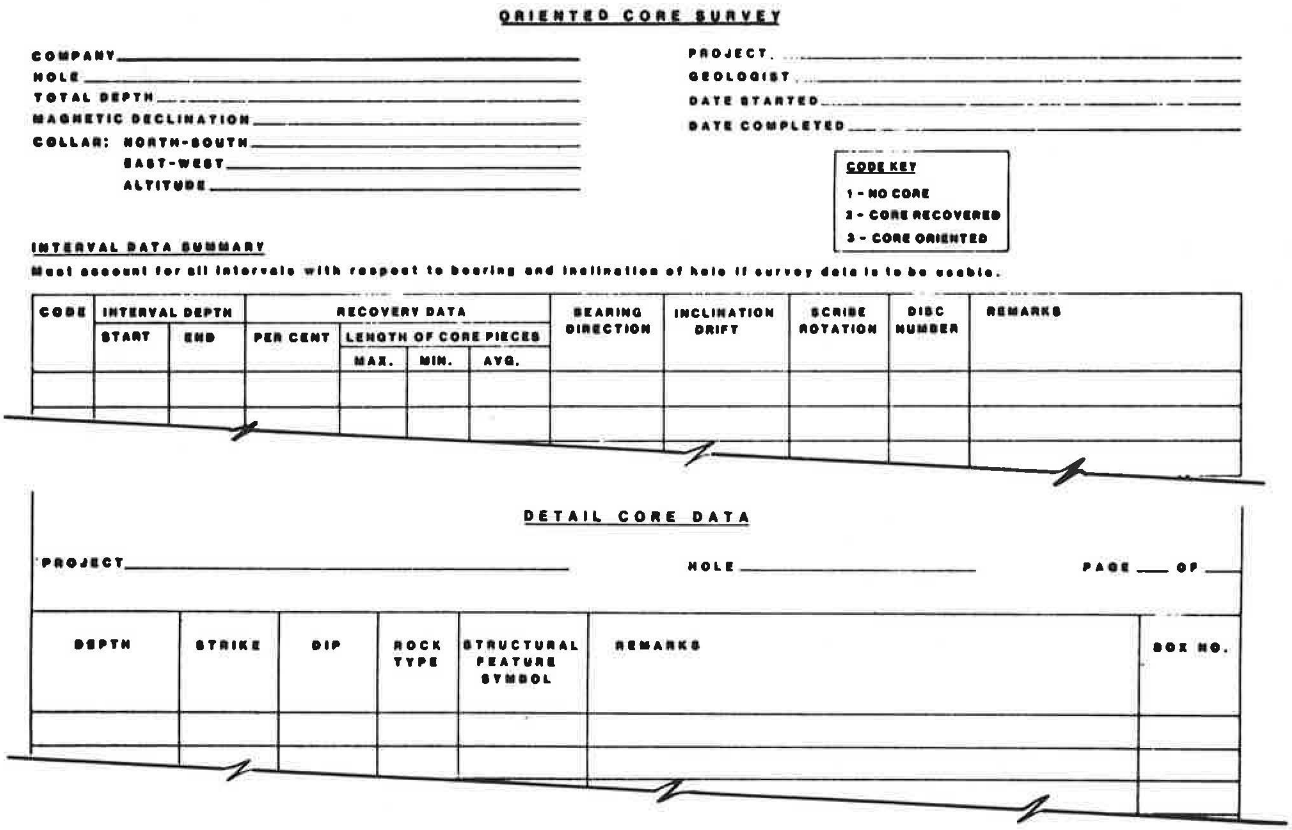


Figure 4. Data recording sheets for oriented core.



ORIENTED CORE SURVEY

COMPANY

HOLE

TOTAL DEPTH

MAGNETIC DECLINATION

COLLAR: NORTH-SOUTH

EAST-WEST

ALTITUDE

PROJECT

GEOLOGIST

DATE STARTED

DATE COMPLETED

CODE KEY

1 - NO CORE

2 - CORE RECOVERED

3 - CORE ORIENTED

INTERVAL DATA SUMMARY

Must account for all intervals with respect to bearing and inclination of hole if survey data is to be usable.

CODE	INTERVAL DEPTH		RECOVERY DATA				BEARING DIRECTION	INCLINATION DRIFT	SCRIBE ROTATION	DISC NUMBER	REMARKS
	START	END	PER CENT	LENGTH OF CORE PIECES							
				MAX.	MIN.	AVG.					

DETAIL CORE DATA

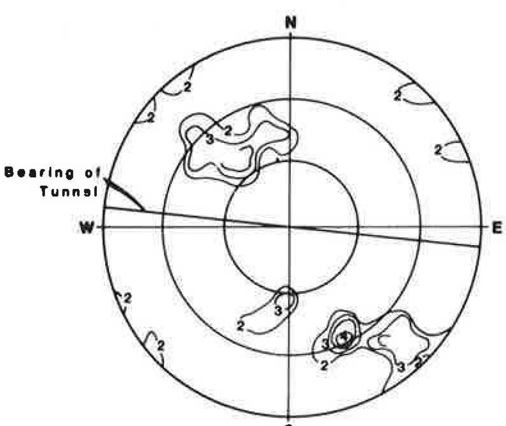
PROJECT

HOLE

PAGE ___ OF ___

DEPTH	STRIKE	DIP	ROCK TYPE	STRUCTURAL FEATURE SYMBOL	REMARKS	BOX NO.

Figure 5. Equal-area diagram of the attitude of joints in core from the Wheeler Junction hole.



ORIENTED CORE
(Contoured in per cent per one per cent of area, 389 poles)

Orientation was lost when the pieces in the box could not be matched--for instance, when drilling through a fault zone. The procedure for orienting the core was then repeated. Orientation runs were made whenever orientation was lost or at least every 20 m.

RECORDING GEOLOGIC DATA

The core was logged geologically, and structural data were recorded based on the assumption that the fiducial scribe line and the axis of the core defined a plane that passed through the center of the core. The plane was assumed to strike north and dip vertically. Figure 4 is an example of the data recording sheets.

The standard information (company, hole number, total depth, geologist, date started, date completed, coordinates of collar, and magnetic declination) are recorded in the heading of the first sheet. In addition, a code is used to specify whether core was recovered and whether the core was oriented. The interval drilled, data on core recovery, and in-hole survey data are tabulated.

The geologic data are recorded on a separate sheet (lower part of Figure 4). Rock type and the attitude of structural features such as the foliation, joint, or veins are recorded in reference to the orientation plane. A code system for different types of structural elements has been developed to facilitate computer input of data. The code used is designed for each geologic environment.

DATA REDUCTION

At the time when the horizontal hole at Wheeler Junction was drilled, the only technique used to determine the true attitude of geologic features was the use of a core goniometer. The goniometer consisted of three graduated circles of right angles and a core holder. An oriented piece of core is placed in the core holder and the three graduated circles are rotated until the core is in its surveyed position. The planar features can then be measured in relation to true bearing and horizontal plane by a compass or protractor.

The data for this project were reduced by using a Schmidt equal-area net. A data point, such as the attitude of a joint, was plotted as a pole on a Schmidt equal-area net and the net rotated to the bearing of the drill hole. The true attitude of the joint could then be read and correctly recorded. The true attitudes of foliation, joints, faults, and veins were then compiled on equal-area diagrams. Figure 5 is the resultant equal-area diagram of the joints in the core.

In present practice, the data reduction has been greatly simplified. Survey and geologic data are now punched on cards, and a program has been developed to plot a plan and profile of the drill hole, to correct the attitude of planar elements to their true position, and to plot and contour the equal-area nets.

HOLE DEVIATION

Each time that a core orientation is taken, the bearing and plunge of the hole are known. When deviation in the bearing or plunge of the hole

Figure 6. Standard 1.5° wedge, orientation assembly, and drilling assembly.

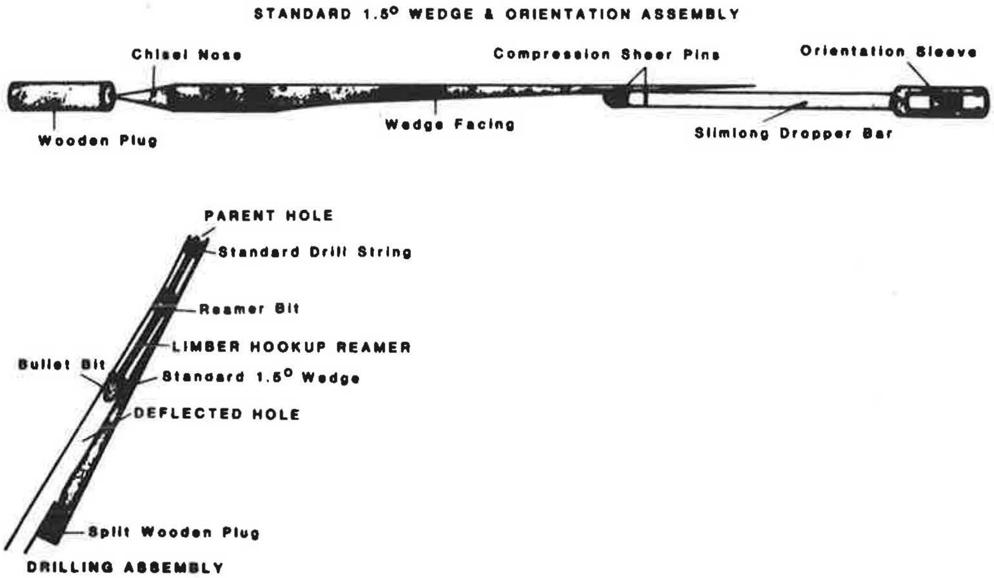
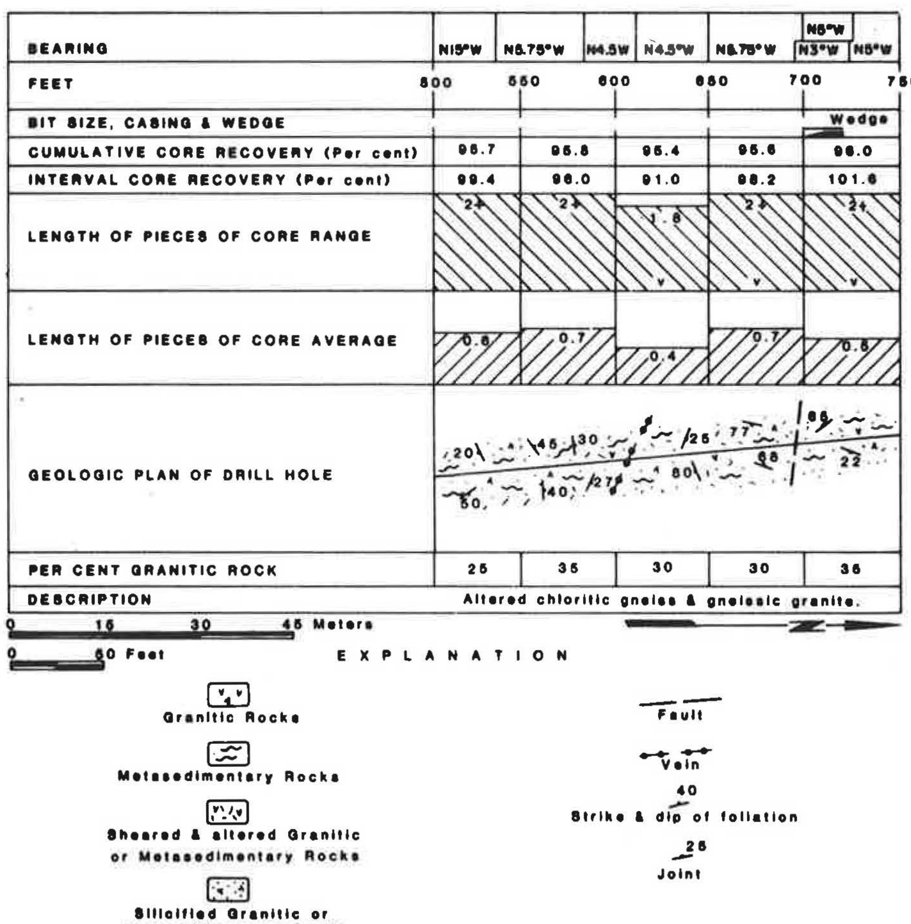


Figure 7. Portion of geologic map of drill hole no. 1, Wheeler Junction.



exceeded 1° from the planned alignment, the hole was whipstocked to the correct bearing and plunge by the use of a wedge. The wedge (Figure 6) is a carefully machined bar of steel. An NX wedge is 7.30 cm in diameter and 2.44 m long. The face of the wedge is machined to form a curved face with an angle of 1.5°. The nose of the wedge is a chisel point. The position of the wedge is fixed in the hole by a fast-swelling wooden plug that is split by the chisel point of the wedge.

The wedge is positioned in the hole by using the Eastman R Single Shot surveying instrument so that the face is in the required direction to deflect the hole. As in taking oriented core, the fiducial line of the instrument is aligned by using an orientation sleeve and a slimlong dropper bar. The slimlong dropper bar is aligned with the face of the wedge by shear pins. The wooden nose plug and the wedge and instrument assembly are inserted in the hole at the end of the drilling rods and the orientation of the face is determined by taking a survey. The face of the wedge is then rotated to the desired position and a check survey is made. When the correct position of the face of the wedge is obtained, hydraulic pressure is applied to the drill rods, the chisel point on the wedge splits the wooden plug, and the pins that hold the wedge to the slimlong dropper bar are sheared. The entire orientation assembly is then withdrawn from the hole.

The new hole direction is obtained from the face of the wedge by drilling with limber hookup. A bullet-nosed bit smaller than the desired diameter of the hole is put on the front of a small-diameter steel rod. A reamer bit the size of the desired hole and a short length of solid rod are attached behind the bullet-nosed bit and rod (Figure 6). The entire limber hookup is attached to the standard drill string.

RESULTS

Figure 7 is a portion of the geologic map (plan view) of drill hole 1 at Wheeler Junction prepared from the oriented core. Geotechnical data recorded included the bearing of the hole; footage; bit size, casing, and location of wedges; percentage of core recovery; and the range and average size of pieces of core recovered. The geologic data recorded included rock type and the attitude of the foliation, joints, faults, and veins.

CONCLUSIONS

Oriented-core horizontal diamond-drilled holes can be used to determine the geological and geotechnical data for the design and construction of tunnels in rock. Technology is available to drill and control the bearing of core holes and to obtain oriented core from which geologic data can be recorded. The cost of drilling oriented diamond-core holes is considerably less and requires less time than the construction of a pilot tunnel to obtain the same data. The apparent limit on the use of oriented diamond-core holes is length. Present drilling equipment is capable of drilling holes to about 1750 m.

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Mississippi's Experience with Horizontally Drilled Drains and Conduits in Soil

WENDEL T. RUFF

This paper relates the experience of the Mississippi Highway Department (MHD) with horizontally drilled drains and conduits in soil. MHD has used horizontally drilled drains that had an inside diameter of 3.8 cm (1.5 in) and conduits that had an inside diameter of 12.7 cm (5 in) with interconnecting vertically drilled drains to effectively achieve subsurface drainage. The advantages of drilled drains and conduits over trench excavations (to eliminate the risk associated with the latter) are discussed. Two case histories are described. The first describes the use of large-diameter interconnecting vertically drilled shafts to create a drainable reservoir in a deep irregular formation. The second describes the use of long horizontally drilled perforated drains to reduce the water table in an active landslide.

The primary cause of landslides in Mississippi can usually be traced to inadequate subsurface drainage. Likewise, the correction of all stability problems generally includes some method of relieving subsurface hydrostatic pressures or controlling seepage. This paper describes the experience of the Mississippi Highway Department (MHD) in accomplishing subsurface drainage by using horizontally drilled drains and conduits.

Simply defined, horizontally drilled drains are small-diameter wells drilled nearly horizontally into a hill or embankment for the purpose of removing groundwater and controlling seepage (1). Their purpose and benefit is to lower the water table rather than to serve as a seepage cutoff wall, as would be the case in trench-excavated interceptor drains. The most common horizontally drilled drainage pipe is Schedule 80, type II polyvinyl chloride that has an inside diameter of 3.8 cm (1.5 in), which conforms to ASTM D-1785. The pipe has two rows of slots cut around the circumference of the pipe on two of the one-third points (120° apart).

The width of the slots can be specified depending on the gradation of the material in the aquifer. The lengths of stock pipe vary from 1.53 to 6.1 m (5-20 ft) and are simply glued together in the field. The drainpipe is inserted into the ground through the center of the drill rod as the rod is retracted.

Horizontally drilled conduits provide an alternative method to the installation of jointed pipe in an open-trench excavation. The same type of horizontal drilling equipment is used for installing both drains and conduits.

Figure 1. Approximate angle of repose for sloping sides of excavations.

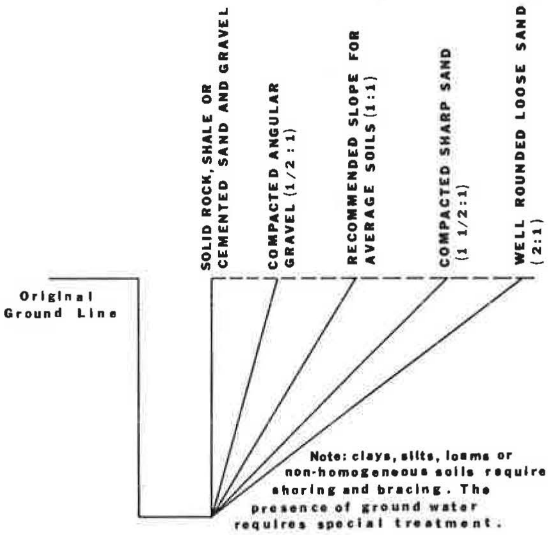


Figure 2. Connecting vertical drains.

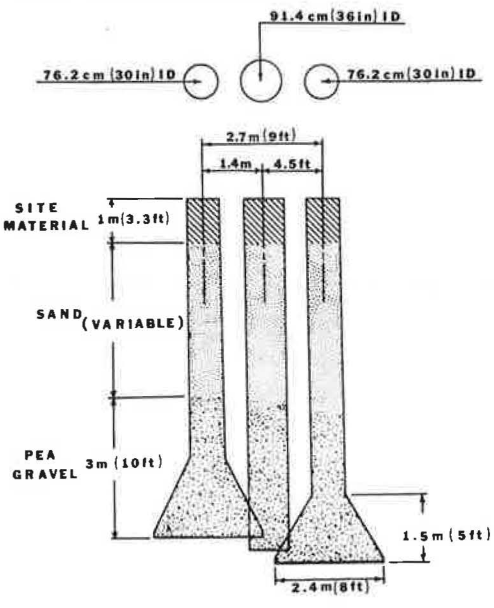
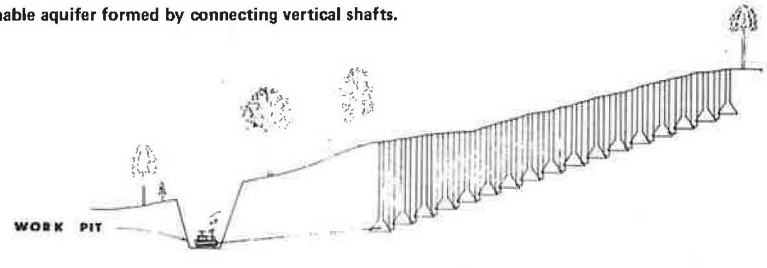


Figure 3. Drainable aquifer formed by connecting vertical shafts.



OPEN EXCAVATION VERSUS DRILLED DRAINS

The practicing engineer has to evaluate the risk and cost-benefit ratio of each type of drainage system before the final selection is made. The type of system chosen to effect subsurface drainage depends on the subsurface and site conditions. The depth of the aquifer below the surface of the ground is usually the primary determining factor. If the depth of the aquifer is as great as 6 m (20 ft), the method of excavation to install a drainage system is, in reality, usually something other than that commonly referred to as a trench. Excavation depths greater than 6 m for no other purpose than to install a subsurface drain should be carefully evaluated.

Since passage of the Occupational Safety and Health Administration (OSHA) Act of 1970, which specified trenching requirements, the cost of trenching has increased significantly. OSHA regulations require that banks higher than 1.5 m (5 ft) be shored or laid back to a stable slope or that some other equivalent means of protection be provided when employees may be exposed to moving ground or cave-ins (2). The recommended guide for sloping banks is presented in Figure 1.

When it is necessary to install a subsurface drainage system across or within an active landslide or inactive failed mass, the excavation is very hazardous. The existence of cracks within the mass makes it mandatory that stringent requirements be placed on the method of excavation. Alterations of the mass are usually accompanied by some additional movement, since the factor of safety is usually close to 1.0 against a sliding failure. Sliding forces must be taken into consideration in the design of a bracing system for such excavations. Excavations must be conducted in small sections that have a sequence of operations that call for the backfilling to proceed along with the excavation. Contractors usually dislike such restrictive work plans and tend to reflect this in their bid price. Another factor that affects the design and selection of a drainage system is the existence of buildings and other structures adjacent to an area in which an excavation is being considered. It is often desirable to avoid deep excavations in which the risk to an adjacent structure is high.

HORIZONTALLY DRILLED DRAINS AND CONDUITS REDUCE RISKS

The installation of horizontally drilled drains and conduits eliminates the risk associated with trench excavations. A drilling plan is much more flexible than an excavation plan. Regardless of the time and expense spent on a subsurface investigation of a project, changed conditions still occur. When these conditions occur on a trenching job, the plans are usually altered to include deeper excavations and greater project costs. Alterations in a drilling plan can be made without affecting the completed portions of the drain.

The following case histories describe the use of horizontally drilled drains and conduits to achieve subsurface drainage. In both cases, the installation of drains and conduits by horizontal drilling was considered to be more economical and safer than the installation of drainpipe in an open-trench excavation.

Case History 1

In 1975, a landslide occurred in a side-hill fill of US-61 in Warren County, Mississippi. The maximum fill height of the four-lane corridor was approximately 18 m (60 ft) from the toe of the slope to the roadway crown. The subsurface investigation revealed approximately 9 m (30 ft) beneath the surface a stratum 1-2 m (3-6 ft) thick of silty sand that had clay layers and that channeled water into the slide area. The sandy stratum, approximately 122 m (400 ft) wide, varied considerably in depth and thickness. Due to the depth of the deposit, a trench-excavated interceptor drain was not considered practical. Likewise, due to its irregular cross section, it was not felt that sufficient intersection could be achieved with horizontally drilled drains. Therefore, it was decided to drill a series of vertical connecting shafts, which could be backfilled with pea gravel to create a drainable artificial aquifer or reservoir. The bottoms of the shafts were stairstepped in such a manner as to provide a sump or low point in the series. A typical section of the interconnecting shafts is presented in Figures 2 and 3.

The system, which contained 37 belled shafts and 36 straight shafts, was completed in approximately one month. Construction was accomplished by drilling the belled shafts first and then making the connection by means of straight shafts. At the low point in the system, a well casing was inserted so that dewatering could temporarily be accomplished by using a submersible pump. A similar casing was also installed at the high point in the system to permit the injection of dye into the aquifer to determine whether continuity had been achieved between the connecting shafts. The tests indicated that continuity had been achieved. At a later time in the construction sequence, a plastic discharge conduit that had an inside diameter of 12.7 cm (5 in) was installed from a point at which gravity flow could be obtained to the low point in the artificial aquifer. Installation was achieved through an oversized shaft by placing the drill stem inside the conduit and pushing against a plug affixed to the end of the conduit. Since plans for a conventional trench-excavated interceptor drain alternative had not been fully developed, an exact cost comparison could not be made. However, it was estimated that the vertical shafts and the horizontally drilled conduit were installed for approximately 40 percent of what a braced excavation would have cost.

Another part of the restoration plan called for the construction of a sand shear key along and adjacent to the toe of the embankment. The shear key was installed by excavating a trench to specific widths and depths and then backfilling it with sand. A perforated interceptor drain was installed along the base of the key to keep seepage water from building up in the shear key. At the low point in the shear key drain, a discharge conduit was required. A conventionally constructed open-trench discharge conduit would have required an excavation 4.5-15 m (15-50 ft) deep for a distance of 189 m (620 ft). In lieu of an open-trench excavation, a plastic conduit pipe that had an inside diameter of 12.7 cm (5 in) was installed through a horizontally drilled shaft (Figure 4).

Installation was achieved by removing the slightly oversized bit from the stem at the point at which it emerged and replacing it with an adaptor that permitted the attachment of the conduit to the drill stem. The conduit was then pulled toward the drilling rig as the stem was retracted. The grade of the 12.7-cm (5-in) conduit was checked by connecting a vertical section of pipe to the upper end of the conduit and discharging water into it until the outflow at the lower end was equal to the inflow. The inflow was then stopped and the head in the vertical section was allowed to equalize. The test indicated that at some point in the line the conduit actually rose 0.46 m (18 in) above the inlet flowline elevation. Since the discharge end was about 1.2 m (4 ft) below the inlet flowline, the grade of the horizontally drilled conduit fluctuated considerably over the 198 m (620 ft). The results were considered acceptable in view of other circumstances and the fact that the shear key was designed to function under a larger head. It was

Figure 4. Horizontally drilled conduit.

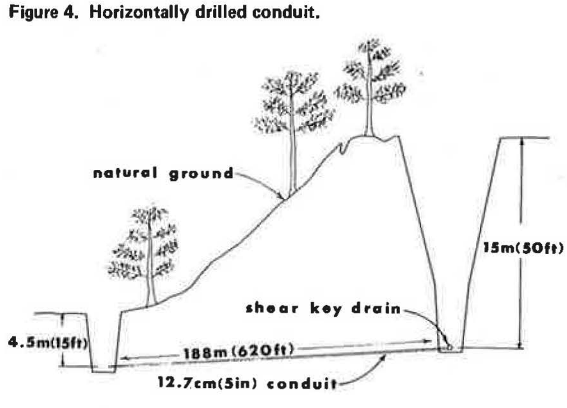


Figure 5. Generalized soil profile.

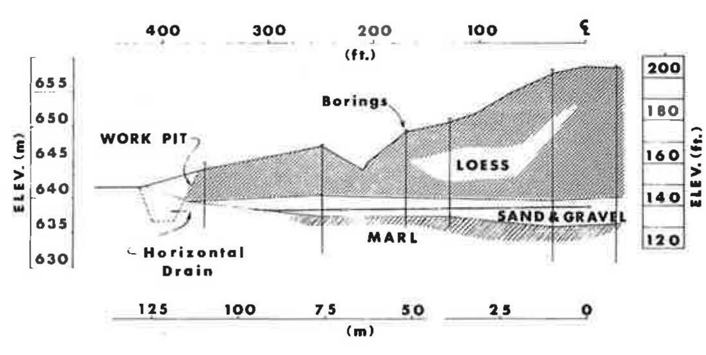
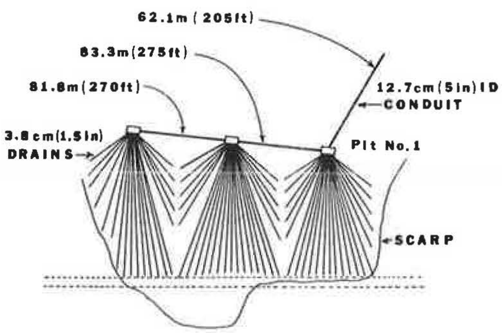


Figure 6. Horizontally drilled drains and conduits.



estimated that the discharge conduit line was installed in approximately 25 percent of the time that would have been required for an open-trench installation and at a substantial cost savings.

Case History 2

The telltale signs of a major stability problem were noted in 1974 along a section of MI-3 approximately five miles north of Redwood, Mississippi, in Warren County. In this vicinity, MI-3 meanders in and out of the loess hills that separate the Mississippi delta from the upland region. The field investigation revealed that the failure was occurring in a wet stratum of medium-dense sand that had gravel and clay seams between the upper loess formation and the lower impervious hard marl and rock (Figure 5).

Studies were conducted to determine the groundwater levels, and slope inclinometer tubes were installed to determine the location of the shear plane. The permeability characteristics of the sand stratum were determined by rate-of-flow tests and draw-down data obtained from two test wells. The laboratory investigation was confined to determining the permeability and grain-size distribution of the sand stratum. By using the data obtained by both the field and laboratory tests, the permeability was found to range between 1×10^{-3} and 1×10^{-4} cm/s. The tests also indicated that the permeability of the sand stratum decreased toward the toe of the bluff.

The major contributing factor to the failure was the buildup of hydrostatic pressure in the sand stratum during periods of high rainfall. The permeability of the loess allowed water to be absorbed during rainfall periods. Since the aquifer did not have a natural outcrop in the vicinity, large hydrostatic forces developed. Stability analyses indicated that a factor of safety of 1.3 against a sliding-block failure could be achieved by reducing the hydrostatic head 3.1 m (10 ft) in the vicinity of the centerline.

Drainage was achieved by installing horizontally drilled perforated drains that had an inside diameter of 3.8 cm (1.5 in). Plans were developed that called for the drains to be fanned out from three work pits. The work pits were necessary in order to obtain gravity flow. The drains were installed from the lowest pit first, which then served as a collector for the other two pits. After the drains in the first pit had been completed, a discharge conduit that had an inside diameter of 12.7 cm (5 in) was horizontally drilled to a point at which gravity flow could be achieved. The second pit was then opened and a connector pipe that had an inside diameter of 12.7 cm was drilled between the first and second pits. Drilling then proceeded in the second pit as the drains in the first pit were connected to a cast-in-place junction box. A plan view of the design is presented in Figure 6. The junction boxes were designed to provide surface access so that the rate of flow from each drain could be monitored in the future. Care was taken to place a sand blanket around each drain hole so that the seepage that occurred along the outside of the drainpipe could eventually find its way into the junction box through weep holes.

The average length of the 68 drains installed was 94.5 m (310 ft). The maximum length was 128 m (420 ft). The percentage of grade varied between +0.7 and +3.3 percent. Installation of the 6718 m (22 026 ft) of drilled drains was completed in June 1977 at the contract price of \$21.33/m (\$6.50/ft). The drilling was achieved at an average rate of penetration of 0.46 m/min (1.5 ft/min). The combined flow rate one month after completion was 232 m^3/day (61 272 gal/day). After three months, the flow rate had dropped to 191 m^3/day (50 472 gal/day). An analysis of the piezometric data indicated that the water table had declined 3.3 m (10.8 ft) on completion of the installation. Approval was then given to proceed with the restoration of the roadway, which had been closed to traffic for approximately one year. The restoration work consisted of remolding the surface area to close all cracks and reconstructing the embankment.

CONCLUSION

It has been the experience of MHD that horizontally drilled drains can be effective. There are, however, areas in which this technique needs to be improved. One of the major deficiencies is the horizontal and vertical control. The desired slope of the horizontal drain is set by adjusting the 4.6-m (15-ft) long drill carriage. Maintenance of the desired slope is difficult and depends on the type of soil encountered and the drain length. Measurements of the slope after the drill stem enters the ground are not very accurate. The elevation of the bit is determined by filling the stem with water and measuring the height to which the water rises in a manometer attached to the daylight end. The results are affected by the pump-induced pressure and a vacuum caused by the foot valve located in the bit. Deflections obviously occur in every drilled hole. A better method of determining horizontal and vertical location is needed.

Care should be exercised to keep the drain slots clean when the drainpipe is stored on the site and when it is being installed. The use of some additives such as drilling mud may reduce the efficiency of the drain. The torque capacity of the drilling rig can be exceeded by drilling long drains through collapsible soils. It can therefore be necessary to use something to reduce the shaft friction. The use of drilling soap is recommended rather than commercial drilling mud. Preventing the slots from becoming clogged is of primary importance due to the difficulty of cleaning the drains after they have been installed.

One of the major benefits of horizontally drilled drains is the ability to effectively reduce hydrostatic pressures with little or no alteration to the site. This benefit saves money and reduces construction risk.

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Long-Term Performance of Horizontal Drains

DUANE D. SMITH

A history is given of the 40 years of development of horizontal drain installation techniques and equipment in California. The long-term performance of three drain installations—Cloverdale, 1941; Nojoqui Grade, 1940; and Pacific House, 1969—is evaluated. Major factors that influence the long-term performance of horizontal drains are discussed, such as location and installation methods, the frequency and quality of inspections, cleaning and maintenance programs, the types of casing used, the lithologic characteristics of the site, the pH and mineral contents of the groundwater, and the protective measures necessary to preserve external features such as outlets and collector systems.

Construction and maintenance of highways in hilly and mountainous terrain are often complicated by the reactivation of old landslides and by the development of new ground-mass movements in unstable material during and following construction.

The presence of groundwater is the most important factor that influences the development of slides and embankment slipouts. Subsurface water reduces the stability of cut slopes and embankment foundation soils through reduction of the shearing resistance of the soil, increase in the weight of the ground mass, and seepage forces that add to the driving force.

For the past 40 years, horizontal drains have been used effectively in California as an economical method of draining unstable areas. This paper, in addition to providing a brief history of the development of horizontal drains in California, discusses their long-term performance and the more-important factors that affect their performance.

HISTORY OF HORIZONTAL DRILLING IN CALIFORNIA

Hydrauger

As the need for more-economical methods of draining and stabilizing landslides was recognized, the California Division of Highways (CDH) in 1939 developed a method of installing perforated metal pipe drains in horizontal or slightly inclined holes drilled by a machine called the Hydrauger. The original equipment, purchased from the Hydra Auger Corporation of San Francisco, was designed for placing pipes under sidewalks and streets without disturbing the surface.

It consisted of an air-driven rotary drill mounted on a racked frame in such a manner that a revolving auger-type bit could be advanced into the earth by means of a hand-operated ratchet lever while water was pumped through the drill rod to cool the bit and wash cuttings from the borings (Figure 1).

The first horizontal drains were drilled by a 6.35-cm (2.5-in) pilot bit followed by a 15.0-cm (6-in) reamer. In 1947, rock-type auger bits with seven carbaloy inserts set in the lead and cutting surfaces of the bit were used for drilling in shale, sandstone, and partially decomposed granite. At Camp Tejon, in Kern County, three holes were drilled with this new bit in partially decomposed granite to an average depth of 37 m (122 ft). Several years earlier, when the same slide had been drilled by using the older auger bit, the greatest depth attained had been 24.5 m (80 ft).

A better bit was found in 1949 when the tri-cone roller rock bits commonly used in the oil fields became available in small sizes (Figure 2). These bits were tried and proved superior in all formations except possibly stiff plastic clays. They have been used almost exclusively from that time to the present.

With experience and progress in horizontal drilling techniques, it was recognized that more-powerful drilling equipment was needed to supplement or replace the lighter equipment then in use.

McCarthy Rock-Boring Machine

In June 1951, a more-powerful machine (initially designed for drilling blast holes in eastern coal mines) was purchased. This drill was a self-propelled unit capable of moving about within limits of a job on its three-wheeled undercarriage. Continuous 15.2-cm and 20.3-cm (8-in) flight augers, which required no water for drilling, were used (Figure 3). The drill engine transmission and drill head were mounted on a track that could be moved back and forth during the drilling operation for adding flights or advancing the boring (1).

This machine proved to be an effective, rugged piece of equipment, although the continuous flight augers were limited to drilling in soil or soft rock formations. The practical drilling depth due to lack of directional control caused by the necessary flight-coupling arrangement, inability to drill hard rock, and excessive torque on the flight augers proved to be only about 46 m (150 ft) in most formations.

In 1953, accessory equipment was fabricated so that regular rotary drilling with drill fluid could be accomplished by using diamond N-rod and 11.4-cm (4.5-in) tri-cone roller rock bits. The degree of success on converting the machine to a rotary drill led almost immediately to the present phase of equipment development.

California Horizontal Drill Rig

The CDH Materials and Research Department had for several years realized the need for an improved horizontal drill. Since no completely suitable machine was commercially available, the decision was made in early 1954 to design and build a drill unit specifically for horizontal drilling. The drill rig that evolved was field tested in June 1954 (Figure 4). It was, for the most part, made up of standard or proven parts of subassemblies similar to those used in manufactured drills.

The California horizontal drill rig incorporated the desirable features of various machines into a lightweight, compact drill rig especially suited for the type of drilling required for installation of horizontal drains. These light portable units had the advantage of greater mobility when access and setup room were problems. In addition, they greatly reduced time required for casing a boring because of the longer lengths of casing that could be used. The California drill had an optimum range of up to 107 m (350 ft) and a maximum of approximately 152 m (500 ft). Beyond this depth, drilling was usually very slow because of lack of power and water pressure. Although these rigs are no longer operated by the state, they remain in use by private drilling contractors.

Modern Horizontal Drilling Equipment

As attention to the effect of highway construction

Figure 1. Hydrauger, first used in 1939 to install horizontal drains by CDH.

Figure 2. Progressive development of drill bits (left to right): folding bit, fishtail bit, and tri-cone roller rock bit.

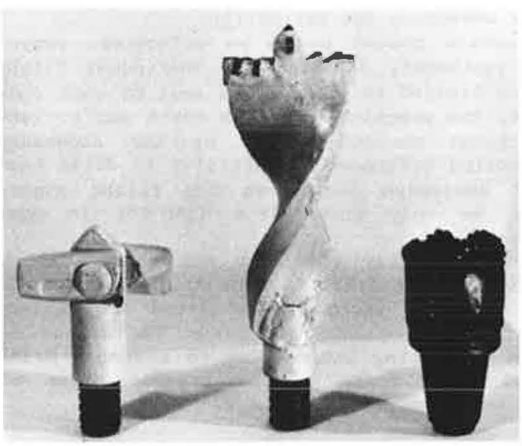
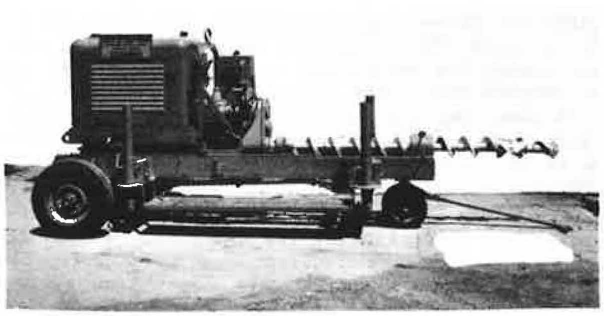


Figure 3. McCarthy rock-boring machine, June 1951.



on the environment increased in the mid-1960s, there was a great deal of concern over the disturbance of the natural landscape by newly pioneered drill roads and drill pads. Location of drill sites in areas that would greatly reduce or eliminate drill access roads often resulted in longer drains in order to intercept the target areas of saturated soil and rock. Longer holes therefore produced the need for more-powerful equipment and heavier drill rod that would not twist off under large torques. The lack of access roads called for a drill rig that could

Figure 4. California horizontal drill rig, June 1954.

traverse fairly rough terrain and carry sufficient drill rod and accessory equipment to do the job. These requirements gave birth to our present drill equipment, which is track-mounted and powered by a diesel engine.

In place of using the conventional drill chuck, the Jensen drill was purchased in 1970; it has a power swivel or drill head that travels a distance of approximately 3.4 m (11 ft) on a side-mounted carriage (Figure 5). Drill rods 3 m (10 ft) long are used. The rig can be adapted to drill with NW, NX, or NX-wire-line drill rod or NX and BX casing, if desired. The power swivel has a 7.6-cm (3-in) opening, which makes it possible to run 5.1-cm (2-in) standard steel pipe through it.

A second problem that had been present since the first horizontal installation is casing a boring that has caved in or is in broken rock. In October 1969 (according to A. D. Hirsch, formerly of the California Division of Highways), an installation at Pacific House in the Sierra Nevada Mountains was completed by using NX-wire-line drill rod. A modified bit adaptor was designed that had a J-shaped slot that received a modified 11.4-cm (4.5-in) roller rock bit that had a 0.95-cm (0.4-in) pin that extruded from the sides of the bit shank (Figure 6). After the required depth of drain had been reached, the drill rod was rotated one-quarter turn counterclockwise, so that the bit was dropped at the end of the hole. Schedule 80 slotted polyvinyl chloride (PVC) casing that had an inside diameter of 3.8 cm (1.5 in) was then inserted inside the drill rod, and the rod was retracted from around the casing without the bit, which left the boring cased to its full length. By using this scheme, drilling can progress as rapidly as possible without the need to maintain an open hole. This process, with various modifications, has been widely adapted by the drilling industry.

Horizontal drains more than 305 m (1000 ft) long have been drilled by using this equipment. Grades of 0-20 percent can be obtained by a simple adjustment of the drill carriage. Horizontal drains 107 m (350 ft) long are commonly drilled and cased during an 8-h shift, and 137-m (450-ft) drains can be installed within the same time frame under ideal conditions.

CASE HISTORIES

Horizontal drains have been used successfully in California since 1939. The following case histories are typical of the earlier installations and should serve to illustrate the long-term performance of horizontal drains.

Figure 5. Jensen drill, January 1980.

Figure 6. J-slot bit adaptor (top) and dropoff bit (bottom).

Cloverdale, 1941

During extremely heavy rainstorms in February 1941, a large landslide occurred on CA-101 near Cloverdale, about 145 km (90 miles) north of San Francisco. Portions of a high cut slope and side-hill embankment in poorly bedded, sheared shale with minor lenses and beds of sandstone slid into the Russian River and severed approximately 335 m (1100 ft) of the roadway (Figure 7). Large quantities of water in the form of springs and saturated slide debris were associated with the failure.

Corrective measures included a benched 1.5:1 cut slope and a reconstructed 2:1 embankment slope. Between March and June 1941, 97 horizontal drains were installed by using the Hydrauger equipment and 5.1-cm (2-in) perforated steel casing placed in 10.2-cm (4-in) drilled holes. The loose broken shale caused a great deal of difficulty during the installation. The average drain length was only 16.7 m (55 ft). Several holes were abandoned because of caving and the lack of a workable method to case the holes under these conditions. A complete record of initial flows is missing from the original report, although some of the individual drains were known to have produced between 568 m^3/day (150 000 gal/day) and 757 m^3/day (200 000 gal/day).

Drain outlets and downpipes were connected to corrugated metal pipe that was buried below the shoulder, which thus prevented periodic inspection of the drains. Heavy rains during the winter of 1955-1956 produced appreciable quantities of water, which appeared in various places along the toe of the cut slope. Investigation of the 15-year-old installation showed that this water was coming from around the horizontal drains, which had ceased to function properly because of heavy deposits of rust, gypsum, and root growth (Figure 8).

As a result of the inspection, a drain-cleaning and restoration program was recommended and completed in the summer of 1956. A total of 49 drains was located. Prior to cleaning, the drains produced a combined total flow of 697 m^3/day (184 250 gal/day); one drain produced 541 m^3/day (143 000 gal/day) of this total. Immediately after being cleaned, they produced a cumulative total flow of 1077 m^3/day (284 440 gal/day). This flow increase clearly illustrates the value of the work performed (Table 1).

In addition to the cleaning and reconditioning of the collector system that had easily accessible cleanouts, three new drains were installed in the most critical areas. A fourth drain was attempted but was abandoned without casing. Although considerable difficulty was still encountered during the installation of the three new drains, by using the McCarthy rock-boring machine and N-rod and rock roller bits, an average length of 35 m (114 ft) was obtained. These three drains produced initial flows that totaled approximately 106 m^3/day (28 000 gal/day).

During October 1956, 11 additional horizontal drains that had an average length of 38 m (125 ft) were installed in a slipout immediately south of the original failure. Initial flows totaled 992 m^3/day (261 989 gal/day). More drains were recommended at that time to stabilize the large area. In 1959 these drains were installed. Unfortunately, there is no record of the number of drains in this installation or of their performance.

In 1974 the installation was again evaluated. Of the 147 drains examined, approximately 40 percent had flows. Many of the original drains installed 38 years before were still functioning; one of these had a flow of 136 m^3/day (36 000 gal/day). Most of the steel casings were severely rusted. Root growth from willows and other native plants had clogged many of the drains and the 20.3-cm (8-in) collector system. Sloughing of the weathered slopes had buried many of the drain outlets both on the bench and at grade.

Nojoqui Grade, 1940

In December 1940 approximately 61 m (200 ft) of the northbound lanes of CA-101 was lost or endangered by the slipout of a portion of a side-hill embankment. Vertical borings in the foundation area indicated the presence of a high water table in soft, poorly bedded claystone and siltstone. A smaller slipout of a similar nature occurred about 244 m (800 ft) north of this site at approximately the same time.

During the summer of 1941, 42 horizontal drains were installed after the embankments had been reconstructed. They were placed into the hillside from locations immediately below the toe of the slope in the saturated foundation area. These drains

Figure 7. Cloverdale landslide and slipout, December 1941.

Figure 8. Rusted and rootbound 5.1-cm (2-in) casing from Cloverdale.

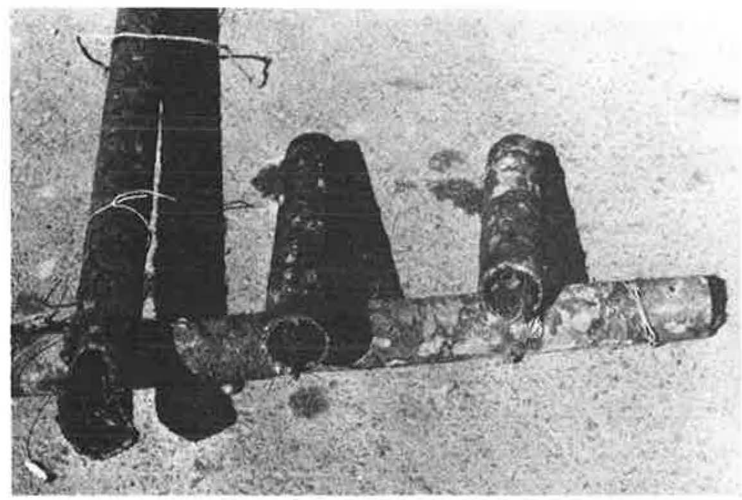


Table 1. Effect of drain cleaning on flow at Cloverdale, 1956.

Drain No.	Hole Length (m)	Length Cleaned (m)	Flow Before Cleaning (m^3/day)	Flow After Cleaning (m^3/day)	Remarks
3	19.8	19.8	3.1	4.1	Heavy root growth and rust
6	18.3	18.3	541	541	Gypsum and heavy rust
9	15.2	15.2	9.3	16.4	Gypsum and heavy rust
11	22.9	22.9	1.1	162.8	Very heavy root growth
13	24.4	24.4	5.5	27.3	Heavy rust
18	13.7	13.7	0.38	40.8	Heavy root growth and silt
19	9.1	3.4	10.9	27.3	Heavy root growth and rust
21	7.9	7.9	0.38	16.4	Heavy root growth
22	10.7	1.8	1.4	4.1	Heavy root growth
23	10.7	2.4	25.1	40.8	Heavy root growth and rust
24	11.3	11.3	0.76	46.7	Heavy root growth and rust
27	17.4	17.4	2.7	5.8	Heavy rust
29	13.7	13.7	1.1	65.4	Root growth first 6 m; sand and rust

Note: 1 m = 3.2 ft; 1 m^3/day = 264 gal/day.

were drilled by the Hydrauger; they ranged in length from 22 to 58 m (72-191 ft)--a record at that time. Perforated 5.1-cm (2-in) steel casing was used in 10.2-cm (4-in) holes that used auger bits and diamond A-rod. Grades ranged from 2 to 10 percent. Records that show initial flows are not available and may not have been made.

Although the drains may have been cleaned from time to time, the first record of cleaning was in December 1962, about 21 years after they had been installed. Only 27 of the original 42 drains were found. The others had been buried by end-dumping of slide material from a cut slope immediately south of the embankment. A heavy accumulation of roots, rust, and silt was evident in most of the drains cleaned. No appreciable increase in flow was noted after cleaning (Table 2), although cleaning was done at the end of a hot, dry summer. Thus flows undoubtedly increased considerably during the wet season.

In late 1974 a contract was entered into to widen the embankment and construct two additional lanes for traffic. Shortly after commencement of the project, it became evident to the resident engineer that additional subdrainage facilities would be necessary to assure construction and maintenance of a stable roadway. Boggy conditions, especially along the toe of the existing embankments in the area of buried drains, caused very slow and difficult operation of the contractor's excavation equipment.

Table 2. Effect of drain cleaning on flow at Nojoqui Grade, 1962.

Drain No.	Length Cleaned (m)	Flow Before Cleaning	Flow After Cleaning	Remarks
2	35.1	Dry	Dry	Rust and silt
3	16.8	Dry	Dry	Rust and silt; pipe broken
8	36.6	Damp	Drip	Bit went through pipe at 34 m
9	6.7	Damp	Damp	Rust, roots, silt; pipe broken at 7 m
11	1.8	Slow drip	Drip	Pipe bent at 2 m
12	9.8	Damp	Damp	Roots, rust, silt; pipe bent or broken at 10 m
13	54.9	Damp	Damp	Roots, rust, and silt
14	47.2	Dry	Dry	Heavy rust, roots, and silt
16	57.9	Dry	Dry	Roots, rust, and silt
17	53.3	Drip	Fast drip	Roots, rust, and silt
19	57.9	Drip	Drip	Roots, rust, and silt
21	53.3	Dry	Dry	Roots, rust, and silt

Note: 1 m = 3.2 ft.

Figure 9. Iron oxide and algae build-up, Nojoqui Grade, 1978: distant view of two drains (top) and close-up of drain at right (bottom).

Several of the buried drains were uncovered by the contractor's equipment and were still functioning. The casings were mostly rusted through. Those drains that had exposed outlets had large accumulations of rust and algae on the ground below the casing (Figure 9).

A total of 32 new drains was installed in the spring of 1975, all of which showed some flow at the time of installation. The combined maximum initial flow was approximately 596 m^3/day (157 480 gal/day). The combined flow at the completion of the job was approximately 43 m^3/day (11 350 gal/day). The drains ranged in length from 46 to 137 m (150-450 ft); 25 are 91 m (300 ft) long.

A very successful horizontal drain installation in terms of drilling progress and quantities of water produced was completed at Nojoqui Grade. An average of 84 m (275 ft) of drilled and cased drain was placed per 8-h shift. There were several days when the length was of the order of 107 m (350 ft) per 8-h shift by using modern equipment.

In March 1978, the installation was again inspected. Of the original 42 drains, all but 8 were destroyed or replaced during the widening project of 1974. Of the eight remaining drains, two were dry, one had a drip, two displayed a trickle, two produced 0.3 m^3/day (90 gal/day), and one had a flow of 2.7 m^3/day (720 gal/day). The steel casings of the 38-year-old drains were severely rusted and would probably be totally destroyed if disturbed by a cleaning operation.

Pacific House, 1969

A field review of an unstable cut slope at Pacific House, located on CA-50, was made in August 1969. Saturated clayey soil and decomposed coarse-grained granitic debris were encroaching on the eastbound lanes from a 0.75:1 slope 90 m (30 ft) high, which caused a traffic hazard. The slide mass was fed by a spring, which may have been partly sustained through irrigation of a large apple orchard upslope from the top of the cut.

Ten horizontal drains were installed in late October and early November 1969. A total of 549 m (1800 ft) of drain was installed, which ranged in length from 47.5 to 64.9 m (156-213 ft). Grades varied from 2 to 4 percent. Schedule 80 PVC casing that had an inside diameter of 3.8 cm (1.5 in) was used in 4 of the 10 drains, and conventional 5.1-cm (2-in) perforated steel casing was used in the remaining 6 drains. The combined initial flow was 99.4 m^3/day (26 285 gal/day). On completion of the project, the flows totaled 58.4 m^3/day (15 438 gal/day).

On November 1, 1978, the site was again inspected. Although the cut slope has remained stable for the past 10 years, wet spots are common around the drains and two of the drains show more water coming from around the casings than through them. Heavy root growth from a dense stand of willows has plugged most of the drains. Approximately 11.4 m^3/day (3000 gal/day) was measured during the review.

This was the first project where PVC was used as casing by the California Department of Transportation. Both steel and PVC were used on the same job so that a meaningful comparison of performance could be made. The 10-year-old PVC casing was in excellent condition and should perform well for many years. The steel casing, although somewhat rusted, was in good condition and should last an additional 20-30 years in this environment.

FACTORS THAT AFFECT LONG-TERM PERFORMANCE

By means of the three case histories, the long-term performance of horizontal drain installations in California has been demonstrated. Some drains that are nearly 40 years old are still producing large quantities of water, although their steel casings have largely deteriorated to thin shells of rust. In most cases, an attempt to clean them at this stage would result in severe damage or total loss of their structural integrity.

It would appear that a 40-year life span is about the maximum that can be expected for drains that have steel casings and perhaps 30 years is a more-practical guide. Although our oldest installation

Figure 10. Paddle marker used to locate drain.

that used PVC casing is only 10 years old, we anticipate that, by means of an effective maintenance and cleaning program, the 40-year life may be considerably extended, certainly well beyond the design life of the highway.

Several factors influence the long-term performance of horizontal drains, such as frequency and quality of a maintenance program, type of casing, pH and mineral content of the water, lithologic characteristics of the installation, and protective measures taken to preserve external features such as outlets and collector systems.

Maintenance

Perhaps the single most important factor in the long-term performance of horizontal drains is a well-developed and well-executed program of inspection, repair, and cleaning. To start, a drain cannot be maintained if it cannot be located. Lack of good records, which includes plans that show the locations of the drains in relationship to survey monuments or permanent landmarks, is common. These sites can be obscured or lost in time because of transfers, retirements, or promotions of personnel who are aware of them only because of personal experience.

The establishment and maintenance of a central file of all drain installations as well as the placement of paddle markers (Figure 10) or other means of marking each drain in the field (such as well-marked steel posts or signs) is good practice.

Drains located near the toes of embankments are sometimes lost through the practice of end-dumping waste material over the side of the fill, which covers the outlets. This practice, in addition to burying the drains, can reactivate movement in unstable areas, particularly when the load is placed at or near the head of the slide mass.

Dense growth of water-seeking plants, such as willows, around the outlets tends not only to conceal the drains but also to curtail their performance by extensive root growth within the first 3-6 m (10-20 ft) of the drain openings. Selective herbicides have been used successfully around the drain outlets to retard or eliminate undesired vegetation. Solid pipe is used for the outer 6 m (20 ft) of each drain to discourage roots from entering the drains.

Horizontal drains are also lost or damaged because they are not protected against rockfall, particularly in the case of exposed PVC casing, or because they are vulnerable to snowplows, rockplows, or vehicles that stray near the edge of the traveled way. A good practice is to protect the PVC casing at the outlet with a galvanized pipe sleeve that slips over the PVC casing and is inserted into the drilled hole approximately 3 m (10 ft). The exposed end of the metal pipe is then connected to a cleanout plug by an elbow that allows the surface pipe to lie adjacent to and parallel with the slope. The pipe is then connected to a buried collector system located at the base of the slope. Cleanouts should also be provided for the collector system.

Drain cleaning and maintenance records should be kept for each installation. These records should include dates of cleaning and repairs; flows recorded for each drain prior to and after cleaning; depth of cleaning, shearing, or damage of drains due to slide movement or external forces; the general condition of each drain; and the person responsible for the operation.

pH and Mineral Content

The long-term performance of horizontal drains is also a function of the pH and mineral content of groundwater. These factors appear to be more important than the type of environment (coastal, valley, or mountain). High acidity (low pH) or the presence of corrosive elements commonly found in fault zones or highly mineralized areas may significantly shorten the life of the drain, particularly when steel casing has been used.

Groundwater highly charged with calcium sulfate or iron, for example, may also reduce the performance of the drain by plugging the slots or perforations. Mineral compounds are usually precipitated near the outlet at which the ionic solutions undergo changes in temperature and pressure.

Lithologic Characteristics

The lithologic characteristics of the formations in which drains are placed is a definite factor in their long-term performance. An installation located in moderately hard broken rock will usually have a long life because of a minimal amount of silt and clay fractions that can gradually build up around the outside of the drain and block the passage of the groundwater. Conversely, drains placed in fine sands or silts may have a shorter life because of an abundance of fines and may require more-frequent cleaning or replacement.

A 5-mm (0.020-in) slotted PVC casing is used in the majority of installations made in California today. When poorly cemented fine-grained sands are encountered, a mechanical analysis is run to determine the proper slot size. Three of these installations have required the use of 2.5-mm (0.010-in) slotted casing to prevent the fine-grained sand from entering the drains. The oldest of the three is now seven years of age and is still performing well but requires cleaning with a high-pressure water system every second year. Installations in rock or well-cemented sediments may not require cleaning for as long as six or even eight years unless root growth is a problem.

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REFERENCE

1. T. W. Smith and G. V. Stafford. Horizontal Drains on California Highways. Presented at ASCE Meeting, New York, Oct. 1955.

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