CHARACTERIZATION OF WEAK AND WEATHERED ROCK MASSES

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edited by Paul M. Santi and Abdul Shakoor

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PREFACE

This Special Publication was produced because of our perceived need for an overview of the state-of-the-art methods for dealing with weak and weathered rock. As such, the authors were asked to prepare a review, summary, and analysis of previously published work, while minimizing presentation of new research. It is our hope that this Special Publication will serve as a useful reference manual for those dealing with weak and weathered rock materials.

The Special Publication is organized in the same manner as the symposium on which it is based, which was held at the 1997 AEG Annual Meeting in Portland, Oregon. An introductory paper is followed by papers grouped under headings intended to address the range of needs to characterize weak rock masses, including "Formation and Weathering," "Measurement and Prediction of Engineering Properties," and "Classification." Three case studies are presented in an "Application" section. The Special Publication concludes with a short summary of the authors' responses to a questionnaire covering weak and weathered rock characterization methods.

Each of the papers in this Special Publication, except the final summary paper, was read by three reviewers. Without exception, these reviewers provided insightful comments and improved the quality of this publication as a whole. Our thanks go to these generous individuals: Steve Brandon, Marcia Bjornerud, Richard Gertsch, Jerry Higgins, Dave Knott, Hardy Smith, Stan Vitton, John Wirth, and John Zellmer. Thank you also to three students, Tara Algreen, Jennifer Grossman and Sam St. John, who provided the final proofreading. In spite of the hard work of these people, errors may yet surface. For these, we apologize and accept responsibility.

Paul M. Santi
Rolla, Missouri

Abdul Shakoor
Kent, Ohio

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Summary

The Locations and Engineering Characteristics of Weak Rock in the U.S.

PAUL M. SANTI
BRIGET C. DOYLE
Department of Geological and Petroleum Engineering, University of Missouri-Rolla, Rolla, MO 65409

ABSTRACT

This discussion is an overview of the definition, location, and engineering problems associated with weak and weathered rock. The terms "weak rock" and "weathered rock" may be defined based on engineering characteristics as materials with one or more of the following properties: compressive strengths between 1 and 20 MPa, high reactivity to water, high clay content, poor induration, a significant amount of matrix between hard blocks, or measurable loss of strength in a human time frame. An overview of engineering definitions of weak rock is given.

Earth materials with these properties generally fall into one of five categories: materials with high clay content (overconsolidated clays, cemented clays, marls, flysch), young materials (quaternary carbonates, tertiary sediments, young volcanics), highly weathered materials (saprolites, weathered igneous and metamorphic rocks), metamorphosed materials (melange, metashale), and hardened soils (hardpan, caliche, tropical duracrusts). Maps for U.S. locations of each of these materials are presented.

INTRODUCTION

There have been many technical articles presenting general overviews of the engineering problems associated with weak and weathered rock. Rather than adding one more to the list, we have decided to focus this article on two aspects of weak and weathered rock science that, to our knowledge, have not been concisely summarized in a single reference. These topics are:

1. Identification of the full range of weak or weathered rock types based on engineering properties.

2. Presentation and discussion of maps showing U.S. locations of most types of weak rock.

Throughout this paper the term "weak rock" will be generally used to refer to intact, unweathered to slightly weathered material which has low compressive strength or is highly fractured. The term "weathered rock" will be generally used to refer to material which shows significant deterioration, particularly near the ground surface or along fractures. Although these types of material show markedly different genetic and post-depositional histories, they both represent a range of properties intermediate to soil and rock. As a result, they will usually be grouped together for discussion.

IDENTIFICATION BASED ON ENGINEERING CHARACTERISTICS

A number of researchers have proposed identifying weak rocks based on their engineering properties. The typical proposed divisions are between weak and strong, durable and non-durable, soil-like and rock-like, and shale and non-shale. Some of these divisions are summarized and referenced on Figures 1, 2, and 3 and are discussed below.

Strength-Based Tests

The identification of weak rock based on uniaxial unconfined compressive strength is by far the most popular method. As shown on Figure 1, there is a wide range of opinions regarding strength divisions between soil, weak rock, and strong rock. In general, most workers place the division between soil and weak rock between 0.6 and 1.8 MPa, and a simple rule of thumb would seem to be 1 MPa. There is less agreement on the division between weak rock and strong rock. This is likely a result of the various purposes addressed by each researcher: tunneling, rippability, geologic genesis, etc. The values reported in Figure 1 range from 6 to 70 MPa, and a popular central value appears to be 20 MPa.

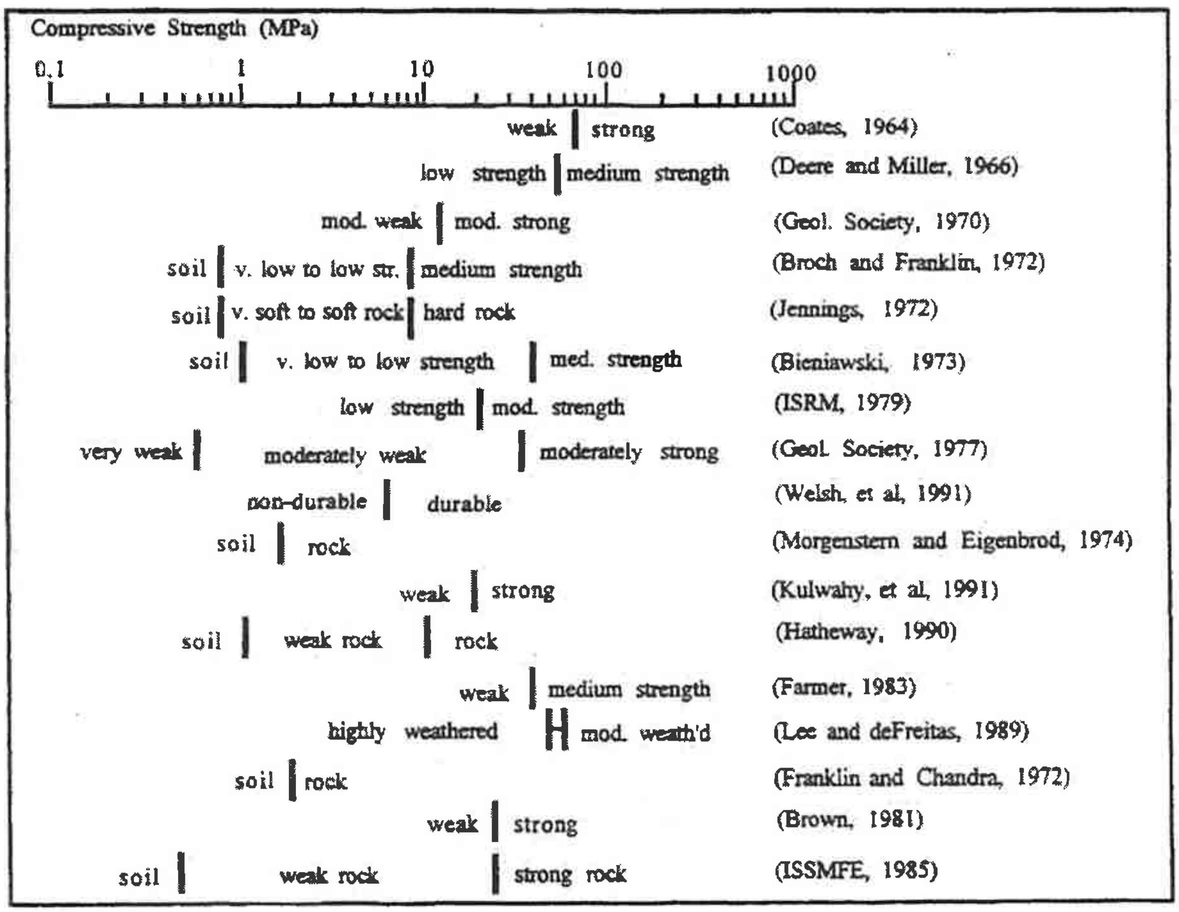

Figure 1. Identification of weak rock on the basis of compressive strength (modified from Afrouz, 1992).

While the compressive strength reflects the rock baseline conditions, the rate of change of strength is also an important factor for weak rocks. Morgenstern and Eigenbrod (1974) noted this in their evaluation system, a part of which is shown on Figure 2. They define soil as a material for which the softened shear strength after immersion, S_s, is less than 40% of the dry shear strength, S_u. They define "stone" or rock as material for which S_s is greater than 60% of S_u.

Although the Standard Penetration Test (SPT) is very popular, few hypotheses have been made regarding weak and weathered rock. This is probably because SPT values are seldom collected over 100 blows/foot. As shown on Figure 2, White and Richardson (1987) suggest that weathered rock may be identified by SPT values exceeding 80 to 100 blows per foot, but less than 100 blows for 4 inches (300 blows per foot). Sowers (1973) distinguishes soil or saprolite from weathered rock based on an SPT value of 50 blows per foot.

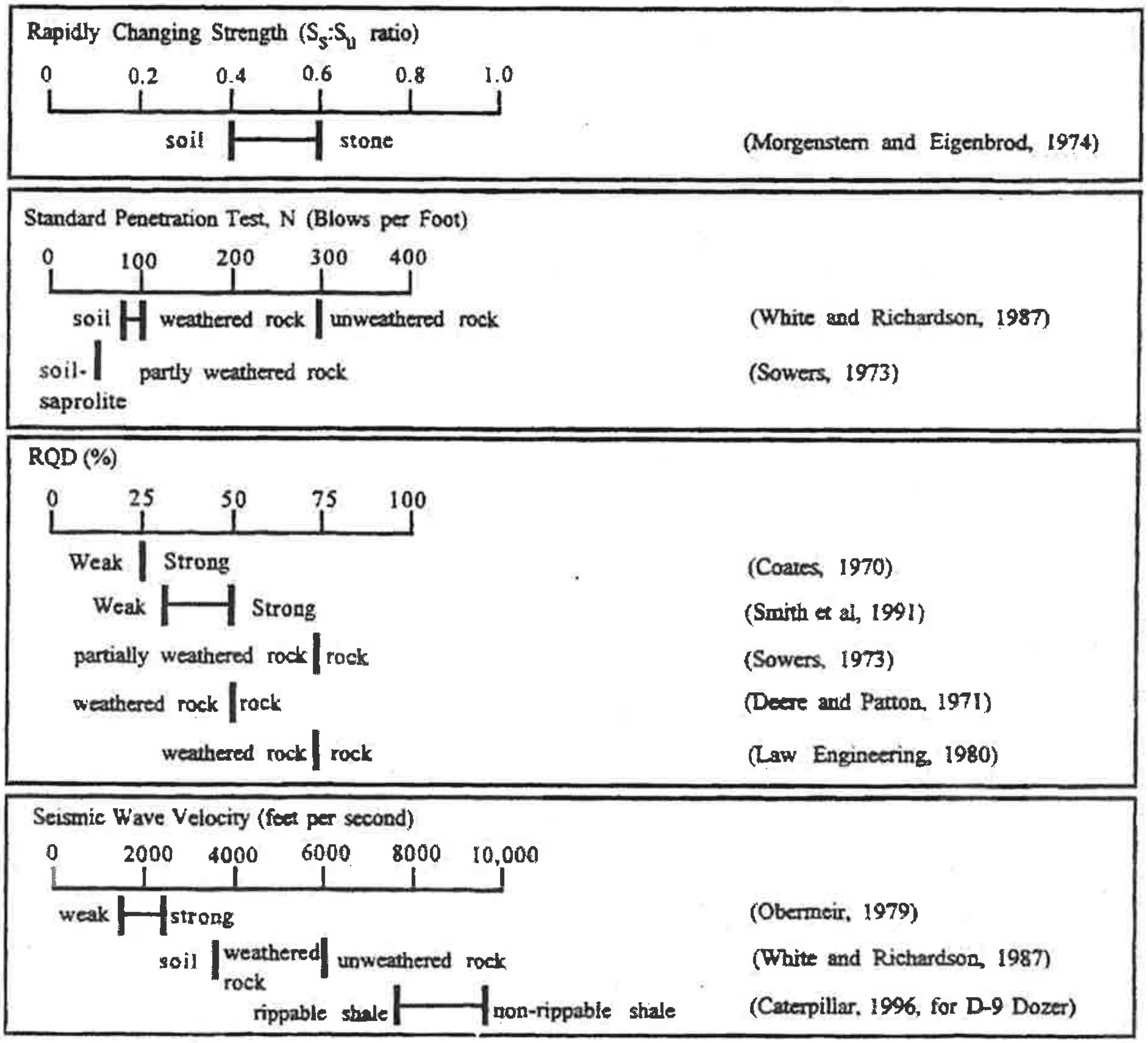

Figure 2. Identification of weak rock on the basis of strength-related tests.

Recognizing the importance of rock mass behavior apart from individual block strength, the Rock Quality Designation (RQD) can also be used to distinguish weak and strong rock. Research summarized in Figure 2 indicates that the RQD for weak rock is less than 25 to 75%.

The distinction between weak and strong rock is even less well defined using seismic compressive wave velocity than using compressive strength. This may be explained by the dependence of seismic wave velocity on both rock strength and discontinuity nature. For example, the range of compressive strength definitions of the weak:strong boundary is roughly 6 to 70 MPa, and the discontinuities are represented by the range of RQD from 25% to 75%. These ranges represent variabilities of 1100% and 200%, respectively. If compounded, as for seismic wave velocity, the variability would be even larger. Based on Figure 2, the variation in values distinguishing seismic wave velocity of weak rock and non-rippable shale is 1700 to 9500 feet per second, a variation of 450%, which is less than the expected variability based on the definition of compressive strength alone. Therefore, although the seismic wave velocity does not appear to be a good tool to distinguish weak and strong rock, its variability is simply a reflection of disagreements on the strength and discontinuity factors which it represents.

Component-Based Tests

Weak rock may also be identified on the basis of matrix amount. Weak and weathered rock often consist of two phases: strong relatively unweathered blocks embedded in a weaker weathered matrix. The studies included in Figure 3 indicate that when the matrix volume exceeds 50-75% of the total volume, the matrix controls the engineering behavior and the blocks have little influence.

While geological definitions of "shale" or "argillaceous rocks" tend to require at least 50% clay minerals or clay-sized particles, an engineering definition may require less, since the engineering properties may be modified by only a small clay component. Mead (1936) noted this early, and proposed that a clay-sized particle content (which may or may not be clay minerals) of roughly 15% was sufficient to characterize the material as a shale, as shown on Figure 3. His classification is shown in detail in a paper by Santi later in this volume.

Water Reaction-Based Tests

The jar slake test, which evaluates the mode of reaction to water, may also be used to distinguish weak or non-durable rock from strong or durable rock (Figure 3). The test categories are as follows (based on 24 hours soaking, as described in Wood and Deo, 1975):

Jar Slake Value	Behavior
1	Degrades to a pile of flakes or mud
2	Breaks rapidly, forms many chips, or both
3	Breaks slowly, forms few chips, or both
4	Breaks rapidly, forms several fractures, or both
5	Breaks slowly, develops few fractures, or both
6	No change

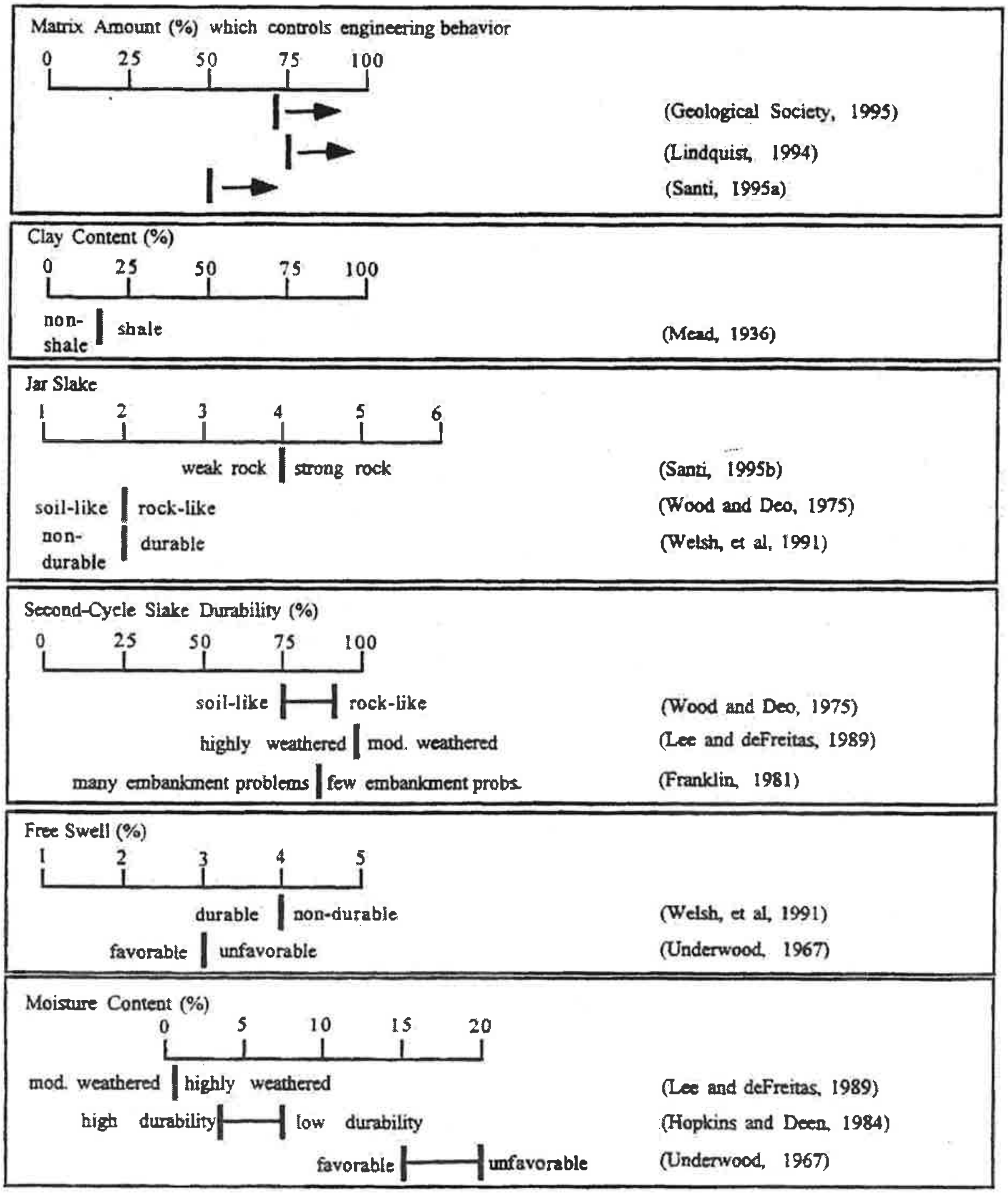

Figure 3. Identification of weak rock from component-based and water reaction-based tests.

Researchers differ in their categorization of material based on jar slake, although they agree that any response to water immersion other than development of a few fractures is an indication of weakness or lack of durability.

Second-cycle slake durability is a more quantitative test than the jar slake test, and the consistency of interpretation reflects this (Figure 3). In general, second-cycle slake durability above 90% indicates durable, rock-like material, and below 75% indicates non-durable, soil-like material. This interpretation implies that, when the issue is durability, the loss of merely 25% of the material causes soil-like behavior. This 25% may be viewed as non-durable matrix within a durable mass. When the focus is strength, such as for the evaluation of matrix amount discussed previously, 75% matrix is required before the behavior is considered soil-like.

Regarding swelling behavior upon exposure to water, rocks are considered non-durable with a free swell of 3 to 4%, as shown on Figure 3. Natural moisture content may also indicate the degree to which a rock has broken down. Figure 3 shows a large variation in interpretation of the significance of natural moisture. Since Lee and deFreitas (1989) dealt with igneous rocks and the other two studies on Figure 3 dealt with clays and shales, one might conclude that igneous materials show the effects of weathering at a much lower moisture content than do shaley materials.

CATEGORIES OF WEAK ROCK

While an assessment of the engineering properties of weak rock is useful, the true definition of a weak rock body should be based on genetic and post-depositional processes. Such a definition of weak rock bodies might include five general categories:

1. *materials with high clay content*, such as overconsolidated clays, cemented clayshales, marls, and flych,

2. *young materials*, such as Quaternary carbonates, Tertiary sediments, and Tertiary volcanics,

3. *highly weathered materials*, such as saprolites and weathered igneous and metamorphic rocks,

4. *metamorphosed materials*, such as melange and metashale, and

5. *hardened soils*, such as hardpan, caliche, and tropical duracrusts.

The locations in the United States of some of the specific weak rock types within each category are discussed below. Maps are grouped together as Figure 4 through 10.

LOCATIONS IN THE U.S.

The great extent of weak rock bodies in the conterminous United States makes it necessary to know the types and locations of weak rock in the planning and reconnaissance engineering projects. Little work has been done, however, to summarize the locations of major weak rock bodies.

The weak rock types which have been mapped for this paper include young volcanics, marls and chalks, Tertiary conglomerates and pediments, shales and siltstones, mixed and

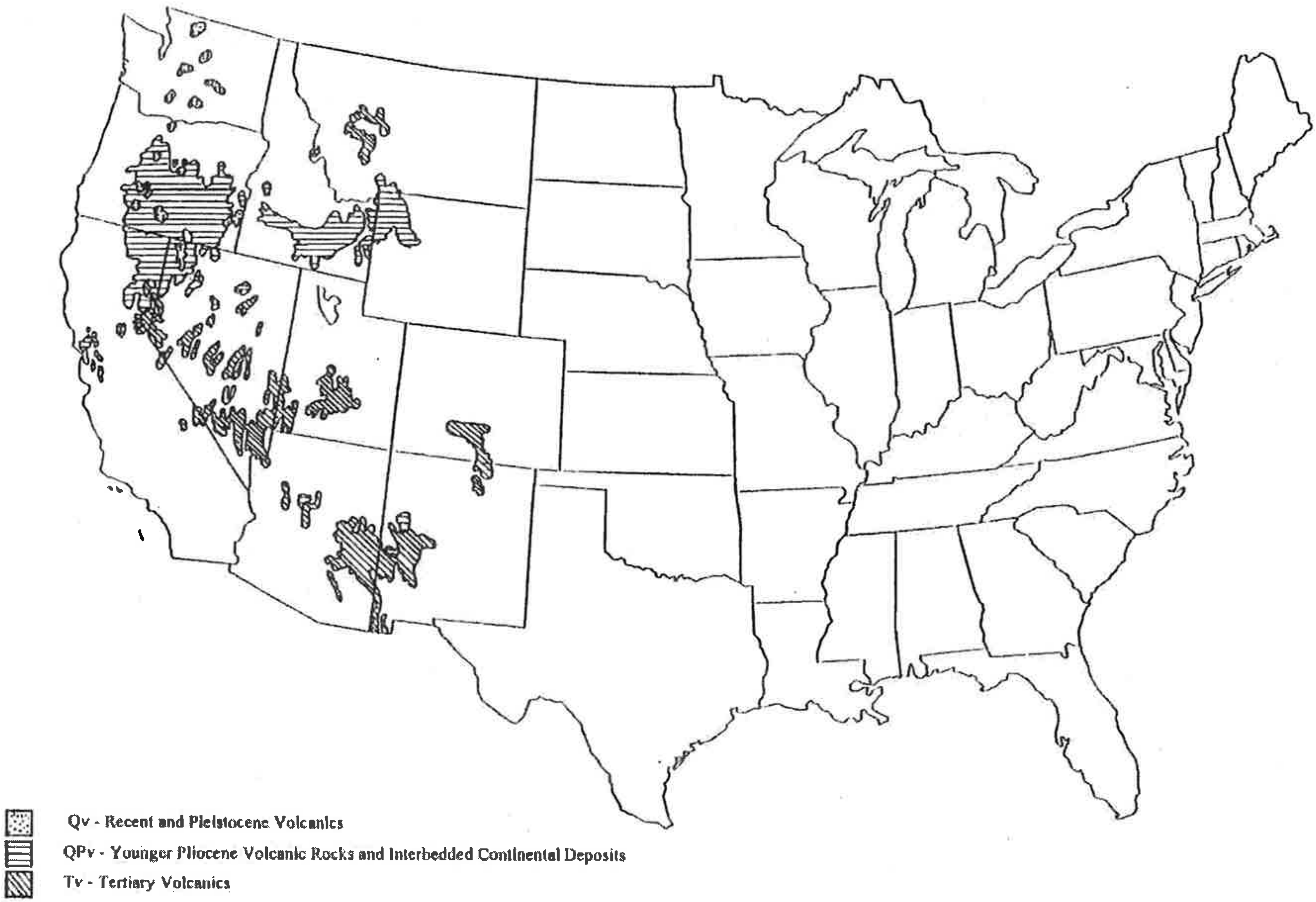

Figure 4. Locations of young volcanics.

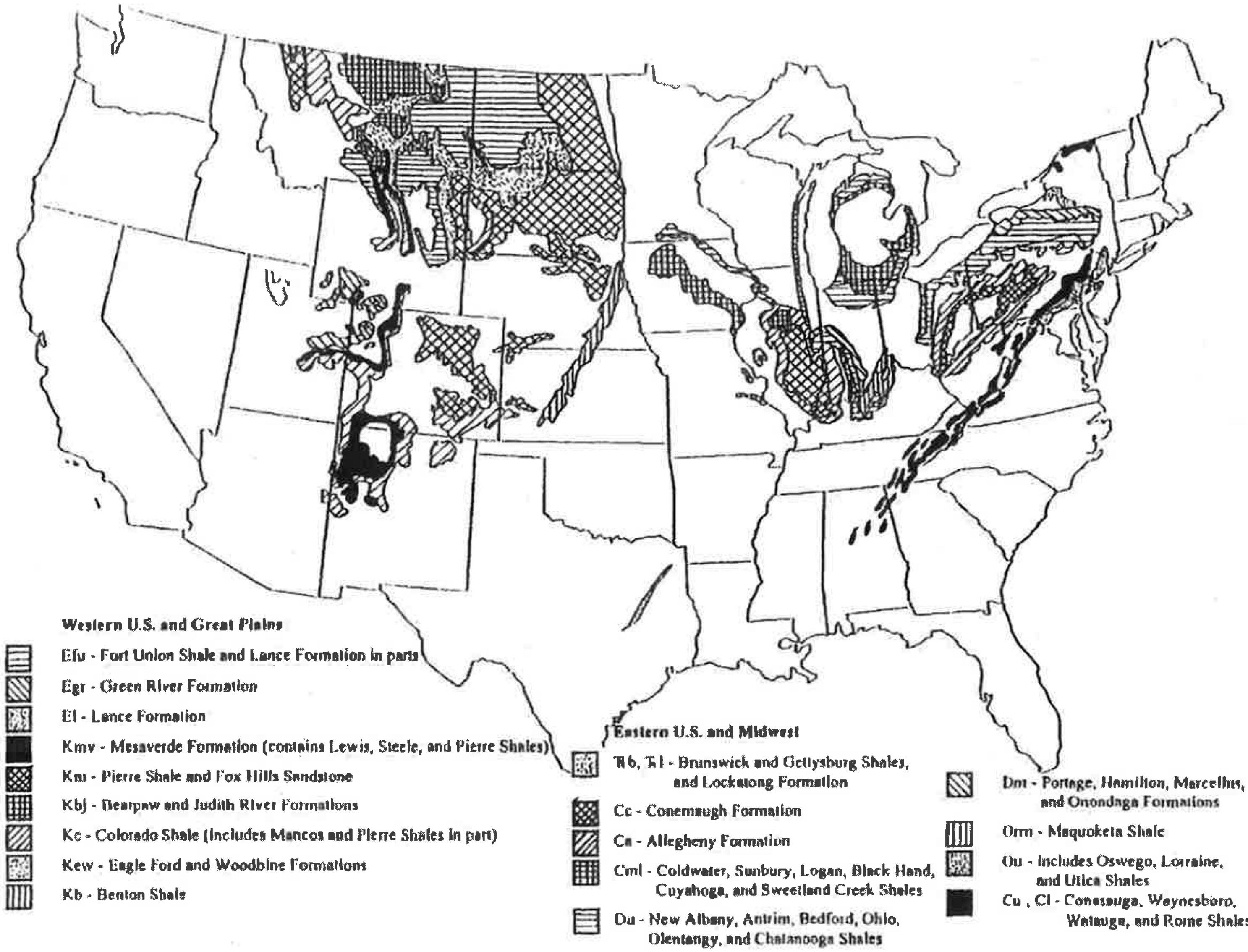

Figure 7. Locations of shales and siltstones.

Figure 8. Locations of mixed and interbedded rocks.

Figure 9. Locations of saprolites.

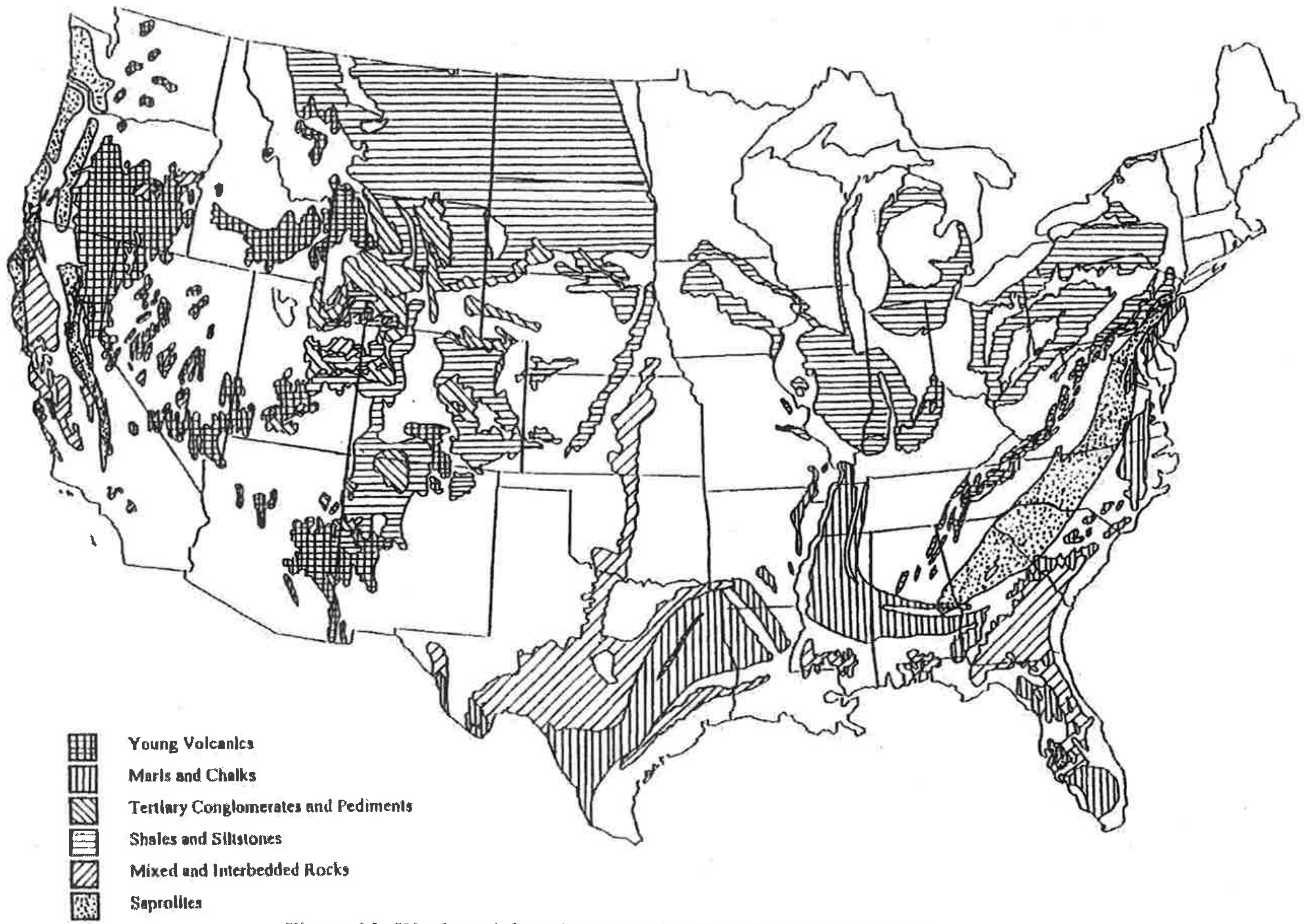

Figure 10. Weak rock locations in the conterminous United States.

interbedded rocks, and saprolites and deeply weathered rocks. The mixed and interbedded rocks include small areas of weak rock which do not fit into the above categories, or rock bodies which consist of material which could be placed into more than one weak rock category.

Weak rock types not illustrated on the maps include Quaternary carbonates, and karst limestone formations. Quaternary carbonates were excluded since most of the weak rock problems associated with Quaternary carbonates occur within 20 degrees latitude of the equator, a region which does not include the conterminous United States (Hatheway, 1990). Karst formations were not included because the degree of karst varies so strongly with microclimatic and other local conditions.

Due to the scale of the maps, many small weak rock bodies could not be included. In addition, the scale of the maps necessitated the approximation of the unit boundaries. However, the maps still provide a good regional-scale reference for various types of weak rock. Once identified on a regional basis, local geologic maps should be used to determine the actual types and boundaries of weak rock in an area.

Lithologic descriptions of the various weak rock bodies included below were taken from Frazier and Schwimmer (1987), except where indicated. Maps were summarized primarily from Stose and Ljungstedt (1960), Oetking and others (1966), Feray and others (1968), Renfro and Feray (1970), Renfro and Feray (1972), Renfro and others (1973), Norris and Webb (1976), Hunt (1986), and Christiansen (1991).

Young Volcanics

Young volcanic rocks are defined as weak rock for several reasons. Volcanic rocks such as tuffs and volcanic breccias may exhibit low strengths. Continental volcanics can often be interbedded with continental detrital deposits, forming planes of weakness and regions of increased hydraulic conductivity between the layers. The relatively young age makes these volcanic rocks quite susceptible to weathering and geochemical alteration, especially in wet environments such as the Pacific Northwest.

The young volcanics shown on Figure 4 include Recent and Pleistocene volcanics, young Pliocene volcanics, and Tertiary volcanics. The Recent and Pleistocene volcanics consist mostly of lava flows and ash beds on the peaks of the Cascade Range. The young Pliocene volcanics contain the Snake River basalts and several tuffs, including the Tuscan Tuff and the Pinole Tuff. In many areas, the Pliocene volcanics are found to be interbedded with continental detrital deposits. Both the Recent and Pleistocene volcanics and the young Pliocene volcanics are concentrated in the northwestern United States. The Tertiary volcanics shown on Figure 4 consist mostly of tuffs and volcanic breccias, and occur mostly in the Basin and Range Province and in the Open Basin section of the Mexican Highlands.

Marls and Chalks

Marls and chalks were considered as one category for mapping because of their compositional and formational similarities, as well as their similar weak rock properties. Both marls and chalks are composed almost entirely of calcium carbonate ($CaCO_3$), although the marls contain substantial clay impurities. The high $CaCO_3$ content of marls and chalks makes them vulnerable to dissolution in acidic environments. Both the $CaCO_3$ and the clay contents of the marls and chalks make them susceptible to softening or expansion in the presence of water. Moreover, these materials are often poorly consolidated, which contributes to their low strength.

The locations of eleven of the major groups and formations of marls or chalks are illustrated on Figure 5. All of the units shown occur partly or entirely in the Atlantic and Gulf Coastal Plain. Part of the Navarro Formation, which consists of marls, chalks, and some sands, occurs in the Great Bend Highlands in southwest Texas, where the sand-rich areas are concentrated. The Taylor Marl and the Austin Chalk, both found in Texas, are two well known weak rock formations. Other important chalks and marls shown on the map are the Calvert Formation, which includes the Trent Marl in North Carolina, and the Jackson group, which includes the Castle Hayne Marl in North Carolina, the Cooper Marl in South Carolina, and various other marls and limestones throughout the Southeast.

Tertiary Conglomerates and Pediments

Tertiary conglomerates and pediments are rock bodies formed from accumulations of continentally derived sediments. The proximity of these rock bodies to their sources results in a large component of easily weathered minerals, such as micas and feldspars, which contribute to their weak rock classification. Because of their young age, these rock bodies are generally poorly consolidated and poorly cemented, resulting in low strength.

The two main bodies of Tertiary conglomerates and pediments occur in the central-western United States, as shown on Figure 6. The White River Group occurs almost entirely in the Great Plains Province, and consists of the Castle Rock Conglomerate, as well as many other formations containing shales and sandstones. The Wasatch Formation is found in the Middle and Southern Rocky Mountain Provinces, in large areas of the Colorado Plateau, and in small areas of the Great Plains Province. The Wasatch Formation contains large areas of sandstones and conglomerates as well as shale beds.

Shales and Siltstones

Shales and siltstones are the most widespread and commonly encountered of the weak rock types. The shales, siltstones, and mudstones included here are derived from volcanic, continental, and marine detrital materials. Shales, for the purpose of this paper, are defined as weak rock material containing a high clay mineral content and exhibiting fissility. Siltstones and mudstones are composed of fine-grained, non-fissile materials.

Shales and siltstones present many different problems from an engineering perspective. Shales and siltstones are often poorly compacted and cemented. These weak rocks experience a wide range of strength values, which depend partly on depth of burial, amount of compaction, and amount and type of cementing material. Shales may expand and slake on contact with water, depending on the clay mineralogy and degree of cementation. Interbedded evaporite and bentonite layers also increase the degree of swelling and shrinking. The high organic content of oil shales creates a strong fissility which serves as a plane of weakness. Rebound and expansion due to stress relief in shales has caused many engineering difficulties, especially in the Prairie Shales of the northern Great Plains.

Figure 7 illustrates the locations of some of the major shale and siltstone formations in the conterminous United States. The formations have been divided for easier reference into two groups; the shales and siltstones of the Western U.S. and the Great Plains, and the shales and siltstones of the Eastern U.S. and the Midwest. The eastern borders of the Dakotas, Nebraska, Kansas, Oklahoma, and Texas separate the two regions.

The shales of the western U.S. are fairly recent deposits, of Cretaceous age or younger. This group includes the Colorado Shale, which includes parts of the Mancos Shale, a well known weak rock formation. Also included are the Prairie Shales, comprised of the Pierre Shale, which occurs partly in the Lance Formation, the Fort Union Shale, and the Bearpaw Shale. The Prairie Shales are frequently encountered in engineering projects, due to their great extent. The most notable of these are the Missouri River Basin Dams, which have experienced problems as a result of stress relief in the Prairie Shales.

The shales of the Midwest and the Eastern U.S. are older sedimentary deposits, all having been formed in the Triassic or earlier. The Cambrian shales of the Appalachians, stretching from northern New York to Alabama, are among the oldest weak rocks encountered in the United States. Many of the shales in the Great Lakes region, especially in southwest Michigan, may be buried under glacial till. Where till and other glacial deposits are thick (greater than 10 feet) the underlying weak rock will probably not impact surface and near-surface geologic properties. Investigation of the depth of surficial materials in glaciated areas is necessary to determine the probable impact of the underlying weak rocks.

Mixed and Interbedded Rocks

Several weak rock areas of the conterminous United States do not easily fit into one of the previously described weak rock categories. Many of these formations contain weak rock materials which could be placed in more than one category, such as marly shales. Other formations consist of mixed or interbedded weak rock material. Flysch and turbidite deposits, and melanges effected by low grade metamorphism can also be found in this category.

The mixed and interbedded rocks shown on Figure 8 range in age from Carboniferous to Pliocene, and are generally found in the western U.S., along the Pacific Coast, and in the Atlantic and Gulf Coastal Plain.

The Jurassic-Triassic aged Franciscan Complex of California, Nevada, and Oregon is among the best examples of a melange. The Franciscan Complex, which includes the Dothan Formation in Oregon, is a complex mixture of greywacke, shale, siltstone, pillow basalts, and radiolarian cherts. The Franciscan Complex also contains gabbro, serpentine, and other ophiolitic lithologies, and blocks of material which have undergone low grade dynamic metamorphism. The various lithologies of the Franciscan are often embedded in a sheared, fine-grained argillaceous matrix. The Knoxville Formation, which is also included in the Franciscan Complex, is composed of thin greywackes and siltstones interbedded with shales, deposited as a result of debris flows and turbidites.

The Bridger Formation of the central-western U.S. is a true assortment of weak rock types. It contains not only marls and limestones, but also tuffaceous mudstones and sandstones interbedded with volcanic ash layers. The mixed and interbedded formations of the Atlantic and Gulf Coastal Plain are dominated by carbonate deposits, such as limestones, dolomites, chalks, and marls, but contain many anhydrite deposits, as well as shales, mudstones, and sandstones.

Saprolites

Saprolites consist of residual soils and weathered rocks which were formed by the alteration of rock materials to clays and other residual minerals (Hunt, 1986). Due to the *in situ* alteration of the parent rock, saprolites often retain the original structure of the parent rock. The highly weathered state of saprolites combined with their rock-like structure contribute to the classification of saprolites as weak rock.

Although a variety of saprolites cover large areas of the United States, many of the saprolites are quite thin. The saprolites shown on Figure 9 are bodies which are greater than 10 feet thick. As with glacial deposits, it is assumed in areas of saprolites less than 10 feet thick the underlying rock material will control surface and near-surface engineering geologic properties.

The three weak rock types shown on Figure 9 have quite different compositions, but are all classified as saprolites. The phosphate clays of Florida consist of poorly sorted clay and phosphate nodules in a sandy matrix. The saprolite is cemented in areas by iron oxides. This deposit has been extensively strip mined as a source of phosphate for fertilizer (Hunt, 1986).

The two other saprolites occur on the Piedmont Plateau and Atlantic Coastal Plain in the east, and on the Pacific Coast in the west. The most extensive of the saprolites on Figure 9, is a micaceous residuum without much quartz sand, formed on slates, schists, and metamorphosed volcanic rocks of the Piedmont Plateau. The similar saprolite on the west coast is formed over volcanic and metamorphosed sedimentary rocks of the Cascade Ranges, and over mixed weathered rocks, often of the Franciscan Complex, north of San Francisco (Hunt, 1986).

A saprolite consisting of clayey residuum with considerable quartz sand is found within or adjacent to the micaceous residuum described above. The clayey residuum is formed over granitic and gneissic rocks. On granite, the saprolite is massive and contains little internal structure other than occasional quartz veins. On gneiss, however, the saprolite has a well developed structure (Hunt, 1986).

CONCLUSIONS

Figure 10 depicts the approximate U.S. locations of all weak rock types discussed in this paper. Although specific weak rock types, such as young volcanics and marls and chalks may be limited to certain regions, weak rocks as a whole cover the majority of the conterminous United States. Additionally, weak rocks can be defined or characterized based on their engineering properties. While there often is disagreement on the exact numerical limits of these property values, some generalizations may prove helpful in defining weak rocks:

1. compressive strength ranging from 1 to 20 MPa,
2. softened (saturated) strength ranging from 40 to 60% of the dry strength,
3. Standard Penetration Test (SPT) ranging from 50 to 300 blows per foot,
4. Rock Quality Designation (RQD) less than 25 to 75,
5. Seismic Wave Velocity less than 7000 feet per second,
6. weathered matrix component greater than 50 to 75% of the entire mass,
7. clay content greater than 15%,
8. jar slake values less than 2 to 4,
9. second-cycle slake durability less than 90%,
10. free swell greater than 3 or 4%, or
11. natural moisture content greater than 1% for igneous and metamorphic rocks or greater than 5-15% for argillaceous rocks.

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Origins and Formation of Weak-Rock Masses: A Guide to Field Work

ALLEN W. HATHEWAY
Department of Geological & Petroleum Engineering, School of Mines & Metallurgy, University of Missouri-Rolla, Rolla, MO 65409-0230

ABSTRACT

Broadly speaking weak, altered, and weathered rock masses together constitute a field of specialty that may never reach a common, generalized geotechnical solution. The causes for their inherently weak character are dependent on the subtleties of geologic history and features at microscopic dimension that only the most astute practitioners are able to predict their performance in engineered works. Additional pressures such as the unfortunate trend to competitive bidding and lower budgets, sites of poorer quality, and short-notice work assignments emphasize the need for weak-rock recognition and characterization. We must come to expect site geologic associations that may indicate the hidden or obscured presence of weak rock.

Heightened awareness constitutes the manner in which we must deal with weak rock in the field. This awareness is made up of knowing the historic relationships that are associated with weak rock, the means of field characterization, and enough case histories to maintain that consciousness.

INTRODUCTION

Weak rock, made up of a broad array of earth material types and effects of historical geological processes, represents the most difficult area of practice for engineering geologists and geological engineers. The difficulties inherent in dealing successfully with weak rock masses are under-appreciated and under-assessed by many engineering geologists and nearly all geotechnical engineers.

Symposium conveners Santi and Shakoor are to be commended for providing this opportunity for us to take stock of what it is that we need to know, how we should apply that knowledge, and, even more importantly, what it is that we yet do not know. In selecting this particular topic, the conveners have taken note of two general shortcomings in our knowledge about weak rock; 1) There has not been enough done in characterizing origins of weak rock, and; 2) There is relatively little awareness that weak rock exists in three-dimensional site mass. Given this attention, we will be successful in providing accurate design-related geotechnical input.

In this paper, the author wishes to make clear that his use of the term "rock" is always meant to include weathered and altered varieties.

WEAK ROCK DEFINED

What is "*weak rock*"? The author's personal definition is:

a *consolidated earth material possessing an unusual degree of bedding or foliation separation, fissility, fracturing, weathering, an/or alteration products, and a significant content of clay minerals, altogether having the appearance of a rock, yet behaving partially as a soil, and often exhibiting a potential to swell or slake, with the addition of water. Some weak rocks are also subject to time-dependent release of stored tectonically-induced stress. When weak for reasons other that weathering or alteration, weak rock is generally Cretaceous or younger in age.*

Some weak rock originated as a highly overconsolidated soil, and as such, has never possessed better quality in terms of any measure of rock strength. Other weak rocks are what the author terms coincidentally-weak rock, such as altered or weathered crystalline varieties. These may have been the hardest of rock at one time, yet have been altered (deteriorated by passage of thermal fluids at some depth below the present ground surface) or weathered (at or near the present topographic surface) to be reduced to a weak-rock condition.

In the minds of most engineers, soil, rock and water are the traditional earth materials. These terms are as friendly a connotation of earth materials as you can find. But, water aside, the terms "soil" and "rock" simply do not cover the essentials of earth materials. We need this term "weak rock" as one that demands respect and which sets the stage for a proper scope of work in site characterization.

WEAK ROCK VIEWED HISTORICALLY

History has a considerable amount to impart to the practice of weak-rock engineering geology. There has been an incremental increase in the state of knowledge about weak rock and only now are practitioners becoming pro-active about the ways in which we have dealt with this earth material. We have advanced to the present state of knowledge through a series of timid steps, from one disaster or failure to another.

PREDICTING WEAK ROCK

Weak-rock properties are basically marginal by nature and general characteristics of weak rock are unfavorable to any sort of engineered construction. Weak rock, due to the dominance of shale as a rock type, represents a significant percentage of exposed rock material on the earth's crust. Weak rock makes up perhaps 80 percent (my estimate) of all Cenozoic rock and it never has possessed worthy engineering properties or behavior. Weak rock is simply never to be trusted and must be subjected to additional levels of effort in site characterization, above the degree normally expended on rock sites.

Table 1 is a list of weak-rock geologic associations which have been personally discovered by the author. With factors such as these integrated into your own personal experience, you will know how to predict the occurrence of weak rock before you reach the site. Age and lithologic associations are found in the stratigraphic nomenclature of State geologic maps, and a good many national geologic maps worldwide. These constitute the basic indicators of weak rock facing the practitioner headed for the field. Secondary consideration needs to include present and probable Pleistocene climatic influences in further degrading rock properties.

FIELD TESTS TO IDENTIFY DEGRADATION POTENTIAL OF WEAK ROCK

Weak rocks, once suspected in terms of geologic associations, should be described, mapped, logged, sampled and tested. There is no standard method of testing to detect or assess the degree of unfavorability of the characteristics and properties of weak rock. The following tests (Table 2) measure properties; some before and some after exposure to various forms of stresses that tend to degrade weak rock.

SEQUENCE OF WEAK-ROCK MASS CHARACTERIZATION

Most of the normal sequence of steps in site characterization will serve to assist toward identification of weak-rock characterizations (Table 3). Many of the indications can be discovered during the pre-field desk study (Steps 1-5).

Table 1. Geologic Associations of Weak-Rock Masses

Age and Lithologic Associations	Typical Degradational Associations
1. Cretaceous or younger age; Insufficient time for lithification	Plate margins, former subduction zones and zones of transform or transcurrent faulting; common provenance of volcaniclastic and/or alteration-degraded ultramafic rock.
2. Permian and younger; *Marl, chalk*, or other dirty (clay-rich) carbonate rock types	Zones of tectonic deformation or shearing; mainly physiographic region boundaries of a structural nature, or plate-margin, terrestrial faults or major intraplate faults.
3. Most rocks classified as *claystone* (preferred term) or *mudstone* (non- preferred; Underwood, 1967).	Volcanic and volcaniclastic rocks found in a presently-topical climate and within the zone of weathering. AKA *mudrocks* (Commonwealth countries); those rocks which have lithified only as a result of *consolidation* (*compaction* of classical geology) and which have little or no cementation (Mead, 1937).
4. Rock classified as *flysch/turbidite*, the interbedded sequence of shale and sandstone of a shallow marine origin.	Extreme lateral variation due to deposition by turbidite-forming conditions; anisotropic rock mass hydraulic flow conditions are to be expected.
5. Sedimentary rock with an organic depositional origin resulting in the presence of sulfate minerals, calcium, iron, or magnesium.	When excavated or otherwise exposed to the elements, these materials are prone to swell and to experience mineral-bond disintegration.
6. Sedimentary rock lacking a cementing agent, or subject to previous dissolution or cation-exchange-removal of pore and void cement.	Speaks of any basic property that will allow for on-going cation exchange alteration or the entire spectrum of weathering agents and phenomena.
7. Clay-rich sedimentary rock having been in near proximity (say hundreds of meters) of a dike or sill; the result of thermally-induced alteration short of low-grade metamorphism.	Resulting physical damage probably is greater if the host rock was wet and poorly lithified at the time of the intrusion. Such a situation is doubly-difficult to anticipate or to detect in site exploration.
8. Rocks of greenish hue, denoting alteration susceptible mineral content.	Denoting presence of chlorite as an undesirable mineral or precursor to swelling clay minerals.
9. High clay or silt content.	When unmetamorphosed, these are the *shaly* rocks, which are also thinly-bedded and subject to considerable strength-reducing jointing.
10. Volcaniclastic rock in general.	Unfavorable combination of clay minerals, as products or geochemical alteration, along with pyroclastic glass shards, or their alteration products, and ferro-magnesian minerals subject to degradation through oxidation.
11. Tropical marine limestone, generally +/- 20° of the Equator.	Expect a high degree of mineral alteration from humid-process weathering and cation-exchange alteration.

Table 2. Field Tests that Selectively Indicate Degradation Potential of Weak Rock

Test	Indication
Drinking Water Glass	Leave representative specimens in a glass of water overnight at your motel; weak-rock disintegration may show by morning.
Roof-Top Exposure Test	Leave representative specimens on the roof of the field office during field work.
Size Gradation	Indicates presence of loose or friable material. Can be used before or after other forms of physical or chemical test stressing.
Particle Shape	Inspect after swelling or slaking has occurred.
Clay Mineralogy	X-Ray diffraction to detect the presence of swelling minerals.
Plasticity	Indication of presence and release of swelling clay minerals.
Slaking	Rate and magnitude of breakdown in air or water.
Ethylene-Glycol Sorption	Qualitative test for swelling or slaking potential.
Methylene Blue Sorption	Qualitative test for swelling or slaking potential.
Cation Exchange Adsorption of Lime	Suggestive of presence of swelling clay minerals.
Ultrasonic Desegregation	Model a variety of field environments; Wetting and chemical agents; forces or accelerates breakdown of the rock.
Relative Compressive Strength of Run-of-Excavation Fragments	Indication of degree of lithification.

Table 3. Sequence of Steps Toward Weak-Rock Mass Characterization

Step	Purpose
1. Compile a Site-Region Stratigraphic Column	Standard practice, but pay special attention to units that have age or lithologic affinity with weak-rock characteristics.
2. Incorporate the standard geologic references	State and Federal geologic reports, especially those dealing with economic geology and ground water; weak rock may have influenced occurrence of economic mineralization or the movement of ground water.
3. Establish number and sequence of major tectonic events within 320- km radius.	Add a column to your stratigraphic column, dealing with plate tectonic, volcanic and plutonic events; spot their general locations and look for evidence of their presence as alteration damage to specific rock units at the site, if the nearest edge of intrusives are within several kilometers.
4. Enlarge the largest-scale existing geologic map of your Site-Area (8-km radius).	Visually review the rock units and their relation to topography. • Are any of the units suspect for being weak rock? (Same as 1, above). • Look at the drainage patterns, using aerial photos; any evidence of mass wastage, unusual or closely-spaced drainage patterning, or lack of outcrops? All of the above will point to potentially weak rock, in a relative sense.
5. Plan and conduct your site-area reconnaissance.	Collect representative samples of each rock type; inspect them for weak-rock characteristics. Any trends of increasing affinity to weak rock, such as increasing zones of low to medium-grade metamorphism?
6. Undertake drilling and borehole logging	Examine boring logs, rock core, and other evidence, such as borehole geophysical logging and video imaging, use to identify the presence of weak rock.
7. Complete Geologic mapping	Take careful notice of the lateral and vertical extent and variation in any evidence of weak-rock characteristics. Note variation in weak-rock characteristics across the project bounds as well as the off-site extent to which construction and operation of the project may represent negative environmental impacts. Examine points-to-poles equal-area stereographic fracture plots to detect potential structural domains of characteristically different discontinuities, between candidate weak-rock units and their three-dimensional masses.
8. Establish Structural Domains at the site	Return to the field for additional collection of strike and dip and other rock-mass classification data, in accordance with ISRM *Recommended Guidelines for Rock Mass Characterization.*
9. Formally review all available geologic data to date	Declare each identified weak-rock unit; Delimit probable geologic map extent of each weak-rock unit. Make a statement whether weak-rock nature is either homogeneous or heterogeneous; Estimate the depth of weak rock nature and its lateral extent; take care to establish apparent variations and the vector (bearing and plunge) of any variations.
10. Add another column within your project Stratigraphic Chart	Specify the apparent weak-rock characteristics. Generate an opinion as to their cause, in terms of site-area geologic history. Refine your previous assumptions, if necessary.
11. Obtain thin sections of each rock type	Don't forget to get two sections if there is visible anisotropy in the rock. Conduct a formal, one -page petrographic evaluation of each rock type. Pay special attention to secondary mineralization and its location, as zonation, overall alteration, or microfracture filling.
12. Subject the suspect specimens to X-ray diffraction	Inspect for presence and relative abundance of clay minerals and related layer silicates. Relate the mineralization to the petrographic analysis report.
13. Conduct appropriate laboratory tests	Specify the tests to suit not only project design requirement, but especially to confirm or refute particular, assumed unfavorable weak-rock properties. Relate the test results to the rock type and all other evidence to date.
14. Add yet another column to the project Stratigraphic Chart	Declare or speculate about possible ways the weak-rock units will impact construction, operation and maintenance of the project; include excavation, ground support, foundation bearing, presence and inflow of ground water to surface and underground excavations; describe intended or speculative use of spoil or muck and how it will be disposed; alert colleagues to possible negative impacts.
15. Establish Structural Domains at the site	Collect additional strike and dip and other rock-mass classification data, in accordance with ISRM *Recommended Guidelines for Rock Mass Characterization.*
16. Tabulate your findings	Identify weak-rock units; tie them to the Site Geologic Map and Geotechnical Profiles. Prepare tables of observed weak-rock characteristic, your findings, and your guidance as to potential negative impacts and toward possible mitigation of the situations.

CASE HISTORIES ON WEAK ROCK

Successful field work in weak rock requires not only a thorough and flexible characterization method, but also a familiarity with a broad range of materials and characteristics. For this reason, case histories are usually helpful in illustrating the realm of weak rock outside of the laboratory, where pitfalls in weak rock characterization abound.

The following case histories are drawn mainly from the author's own weak-rock learning experiences and are not otherwise documented. The first two cases, however, are cited for their historical precedence.

Panama Canal: Eocene Weak Rock

Donald F. MacDonald, expatriate Canadian practitioner of American engineering geology, took an assignment from the U.S. Geological Survey, through the U.S. Bureau of Mines, to serve as the senior resident geologist for the peak period of construction of the Panama Canal (MacDonald, 1915). MacDonald was so impressed with the universal presence and problems caused by poor-quality sedimentary rock on the project that he developed the term "weak rock" for the situation. His 1915 paper is a must for appreciating weak rock.

Lessons Learned: MacDonald's report typifies most of what can occur and happen on a weak-rock project. For large-scale projects, field work should start with mapping of bodies of apparently similar weak-rock characteristics, followed by analysis of rock-mass water conditions and site geometry, both before and after the intended construction.

Upper Great Plains (North Dakota to Nebraska: Cretaceous-Aged Bearpaw Shale)

Geotechnical practitioners of the upper Great Plains know that the Bearpaw Shale (as well as the Pierre Shale) is the local "junk rock". The 1939 report of the Chief of Engineers, U.S. Army, on the construction slide-failure of Fort Peck Dam, Fort Peck, Montana, remains a classic introduction to Bear Paw rock. Even when excavated and recompacted under optimal conditions, seepage and uplift pressure caused failure of the extreme eastern portion of the Bearpaw Shale embankment of this large compacted embankment dam. The 22 September 1938 incident excited then-President Franklin D. Roosevelt to order that a Chief Geologist (Ed Burwell) be appointed to operate from Washington, D.C. and to visit the field Districts constantly, to ensure that such failures would not occur again (Hatheway, 1997, in press).

Ringheim's 1964 paper on dam building in Bearpaw terrane provides good insights.

Lessons Learned: These shales, in all their lithologic variety, behave according to a general rule of the basic unreliability of Cretaceous or younger rock (Hatheway, 1990). For Bearpaw and it sister rocks, those of Pierre Shale, this rule is exacerbated by their clay-rich nature. In these terranes, one must pay close attention to potentially unfavorable mineralogical, structural and groundwater conditions (Ringheim, 1964).

Arkansas (Northwestern) and Oklahoma (South Central): Atoka Formation, Permian-Aged Turbidites

Turbidites (flysch), characterized for their thinly-bedded, discontinuous-facies combinations of claystone, siltstone and fine-medium sandstone, create a difficult set of contaminant-transport challenges. These rocks are found in the immediate subsurface of two landmark uncontrolled hazardous waste SUPERFUND sites, both early listings on the National Priority List (NPL). At both the Hardage-Criner chemical-waste dump site in McClain County, Oklahoma (USEPA, 1989) and at the former VERTAC Chemical Plant, at Jacksonville, Arkansas 1990 and 1993), the turbidites offered distinct pathways for downward migration of dense, nonaqueous-phase liquids (DNAPLs), many of which were known or suspected human carcinogens.

Three particular technical challenges to lateral containment of the broadly-variable contamination were apparent at the Hardage-Criner site (operated 1972-1980), a Federal-lead cleanup.

First of all, the PRP (Potential Responsible Parties) legal Steering Committee found itself prematurely committed to the in-place closure of the site after an off-handed reference to the possible need for vertical subsurface containment by the slurry-wall technique. This method was envisioned by engineers inexperienced with turbidite sequences, and who considered the turbidite series to represent a backhoe-excavatable medium, then proven only in soils. Site characterization showed a moderately-to-highly jointed and horizontally-bedded, sandstones dominated the upper hydrostratigraphy. Not until an average depth of 33 m did the most consistent claystone and siltstone beds offer hope of effectively embedding the slurry wall. Predicted costs of the slurry wall went from less than \$45 per m^2 (one-face surface area) to more than \$270. This potential cost escalation came from two geologically-related factors; 1) Resistance to digging from the sandstone horizons would require use of a crane-mounted rock-milling machine to the required 33 m depth, and; 2) A need for alternately-excavated slots (about 1 m wide by 5 m in length), filled with backfill, followed by excavation of the intervening rock mass after initial plastic-cement curing.

Second, DNAPL penetration of this weak rock was encountered in a logged borehole at some 20 m of depth and about 60 m laterally from the base of the dump mass. This fortuitous incident indicated the natural ability of these contaminants to move a considerable distance through what was thought to be rock of very low hydraulic conductivity. Part of this surprise was the fact that DNAPLs do not behave as water, and the appropriate term would have been "permeability" (to DNAPLs as the pore or cleft fluid, not water).

Third, the wide spectrum of chemical contaminants were known to have been dumped (uncontrolled disposal) within the 345,000 m^3 of the 4.5 ha mass of on-site waste, up to 13 m thick, constituted a serious potential for chemical attack of slurry-wall backfill. Plastic cement, containing industrial clay minerals was necessarily specified in order to prevent formation of contaminant-permeable backfill fractures.

Lessons Learned: Weak rock often is thought of as "impermeable" (more properly, of very low "hydraulic conductivity"), simply because of the obsolete reputation of clay and silt-sized sedimentary material as representing "impermeability." The truth is that these rock masses, by virtue of their inherent weakness, are subject to high frequency of jointing and also of potentially open bedding planes. Over the last 15 years we have learned that DNAPLs possess an inordinate capacity to move through such natural fractures, especially when there is a large supply of free liquid present within some pre-regulation, uncontrolled hazardous waste dump sites.

California: Pliocene-Aged Weak Rock at Los Angeles

The Los Angeles Central Business District had sprung into a new era of high-rise construction by about 1965, and massive foundations were being constructed down into the Pliocene-aged Puente and Fernando Formations, sequences of interbedded siltstones and claystones. These overconsolidated "shales" had as many as 50 or more distinct bedding planes per lineal meter of thickness, relatively high moisture content, and dips averaging about 25 degrees. Building code requirements called for a significant amount of below-ground parking space for building occupants, and basement space went straight down, approaching 30 m in depth. Premium space value called for vertical excavations right up to the inner sidewalk line.

Rock strength began to deteriorate immediately upon excavation. The foundation pit was dug vertically by lifts (there were no benches) went down at 3.5 m each, with rapid bitumen-spraying on the open-cut overlying soils and highly weathered strata to hold moisture. Prior to actual excavation, wide-flange H-beam soldier piles had been sunk to embedment below the eventual floor level and were joined laterally by steel waler beams and held in place with 32 mm reinforcing-iron tie-back rods. The tie-back anchors typically were belled at the embedded end, within a double conical anchor space infilled with concrete up to the active pressure line. Following curing of the anchor-bell concrete, the anchors were tensioned and locked off to retain the vertical basement wall.

Lessons Learned: There is no reason to expect fine-grained shales, especially of Tertiary age to be anything but weak, in all senses of engineering properties and characteristics. In the southern California tectonic regime, these rocks were highly jointed and so thinly bedded that design was based not directly on attitude of bedding, but on a composite active-wedge failure plane presumed to act perpendicular to the azimuth of the supported wall.

California (Southern): Miocene-Aged Monterey Formation

Of the Monterey Formation it is said that you will not pass the California registration exams for Geologist or Engineering Geologist if you possess a fundamental ignorance of this rock unit. Monterey strata hold the lead in slope stability damages, not only for the State, but probably nationally. Thin (a few mm) bedding, lack of induration, secondary growths of soluble gypsum, broad dip planes are some of its characteristics and pesky bedding-plane faults often worsen the stability conditions. Monterey rocks have just enough clay mineral content to indulge in sorption of long-term, slow, spring rainfall and then fail along bedding planes in a flash. Worst of all, Monterey rock is so very plentiful in southern California!

Lessons Learned: Geologic mapping is so far advanced that early warning of Monterey strata generally is available before field verification. Probably the greatest challenge is to recognize slivered bodies of fault-displaced Monterey strata in the detail not shown by larger-scale site-area or quadrangle-sized published geologic maps.

California (Mid-Coastal): Cretaceous Weak Rock of the Hosgri-San Simeon Fault System

Even as an undergraduate, mapping in my third of four UCLA field courses (1959; Tar Springs-Huasna area of San Luis Obispo County), the prospect of suddenly entering a flat, featureless band of ground 1.6 km in width and strikingly dissimilar to the Miocene terrane of out of which I had stepped, seemed incongruous. It was indeed. Only the occasional knocker of brightly-hued chert breccia broke the surface. I was standing on a fault zone of major proportions, both in width and in tectonic implications. The first observation came early; the latter, only after some years.

Within the fault zone, only, large (a few m^3), isolated blocks of silica breccia (float) were observable. The remainder of the zone was probably filled with sheared and deteriorated weak rock affected by cation exchange of migrating ground water. I reflected on how difficult it would be to drive a tunnel through this rock, never mind the thought of earthquakes!

Lessons Learned: This terrane is instructive in any case of long-term fault activity, in which host rock quality has been degraded from repeated tectonic displacements and in which a good deal of the harder knobs or pockets of rock have been reduced by shearing and mineralization-demineralization, to a general lack of exposure.

California (Northern): Jurassic-Aged Franciscan Assemblage

Franciscan Group rocks, simply, are the generally-ferromagnesian, sperpentinized wreckage of pre-Cretaceous plate tectonic subduction. Found along the mid-coast of California north into Oregon, they seldom leave decent outcrops other than patches of silicified knobs ("knockers") on oak-and grass hillsides. North of Cape Mendocino, in the rugged forested terrain of the northwest quarter of the State, Franciscan rocks support entire hill masses. These offer the greatest of engineering geologic challenges, considering the climate, which drops more than 350 cm of annual precipitation.

Three geotechnical rules apply to Franciscan terrane; 1) Always count on it being weak and discontinuous; 2) Never count on it to be of assistance in holding stability in any form of cut; and 3) Always consider it to be loaded with cleft water and in a state of marginal stability. I recall inheriting geotechnical responsibility for the national forests of northwest California, on joining the Geotechnical & Materials Engineering Group of Region V of the U.S. Forest Service, in 1971. An innocent-enough appearing, yet highly challenging, logging road cut on the Gasquet Ranger District of the Six Rivers National Forest, occupied about 100 m of road above a 100-m steep hillside to the sediment-sensitive creek below. I was scratching my head at the end of a day's climbing on and around the unraveling cut, a mass of water-seeping, slippery, movement-polished rhombs of peridotite when the color aerial photographs of the Forest reached me.

This tiny, bulging mass of slippery rhombs was only part of a slightly larger bulging mass, which was embedded in a slightly larger and geomorphologically unstable-looking mass. So the amalgamation went, straight up the ridge four km to the east and a thousand m higher. Overwhelmed, I got truly serious about installing French drains and keeping the road width to one lane. Nothing else could be done, except to warn the Ranger District road-maintenance crew to have respect.

Lessons Learned: There are certain geologic "terranes" of which engineering geologists must become informed and respectful. Franciscan hillsides, more than any other California rock sequence, are highly heterogeneous over the expanse of even a single project and have been formed and sculpted by hill-forming processes, to marginal stability. You are in harm's way in even scratching the site. Inform your client and move with studied respect for whatever your purpose may be.

Caribbean Islands: Tropical Marine Limestone of Puerto Rico

Caribbean islands generally have formed by lateral accretion of Pliocene-aged limestone shelves on to early to pre-Tertiary volcanic or ultramafic cores. These islands also are mantled by Pleistocene-Holocene backreef-hash limestones made up to a high degree by uncemented broken coral and unindurated carbonate infilling. Rock masses here are largely random void spaces.

Lessons Learned: Even the finest and seemingly most intact horizons of these rocks are of variable excavating character, sometimes requiring only a large dozer with a ripping device, and other times requiring drilling and blasting. Depending on shoreline micromorphology, relatively large pockets of weak rock are encountered, as containing troublesome quantities of fine-grained infilling deposited by storms. Site characterization for construction calls for a higher-than normal allocation of site investigation effort, if for no other reason to avoid placement of high-capacity foundation elements on potentially compressible ground.

Caribbean Islands: Tertiary-Aged Marine Volcaniclastic Rocks

The experienced mind should suspect weak-rock conditions here, based on age alone. This Miocene volcaniclastic sedimentary sequence, however, is further complicated by the presence of pyroclastic shards in various tuffs. At Cerrillos Dam, just northeast of Ponce, inward from the south coast, the Jacksonville District, Army Corps of Engineers, engaged in a long and arduous job of drilling and mapping in the thick underbrush of the steep-sided hills. So slow was the degradation of a key tuff bed, to be exposed in the massive right-embankment emergency spillway cut, that its tendency to break up was not detected during the Design Memorandum and bidding phase.

Only during excavation did the slaking nature of the tuff and other sedimentary beds become apparent. The contractor's choice of rock processing equipment had been approved and then was it observed that there was a significant under-run of one of the smaller embankment-zones of processed (as broken and sized) earth-rockfill dam material. Incidental to the undesirable breakup of the volcaniclastic rock was the presence of a uniformly-thick (2.5 m) slightly post-depositional andesite dike. No evidence of contact fusing was apparent but this dike is believed to have thermally and ionically damaged much of the embankment rock, likely by heating the young pile of sediments to an outward range of a few hundred meters.

Corps oversight personnel advised that the contractor be allowed to drill and blast in the spillway cut. It is now believed that this additional, systematic shocking of the rock spoil gave further damage and contributed in the excess breakup of the rock and the ensuing under-run of the sized embankment construction material.

Lessons Learned: The potentially deleterious presence of volcanic ash (largely glass) in an already suspicious post-Cretaceous sedimentary sequence, led to this situation. The surprise was, of course, the degree of strength damage, through alteration and breakdown of mineral bonds and subsequent devitrification of the volcanic glass. The author believes that the contractor, in selection of rock processing methodology, made the situation worse by giving an unnecessarily severe drubbing to the rock during handling and processing, thereby creating a shortage of specified rock and a corresponding excess of waste.

Ecuador (Coastal): Island-Arc Wreckage

Most weak-rock coastlines have been affected by plate-tectonism. This was the case at San Luis Obispo County, California (above), though its fault setting is that of a major transcurrent, strike-slip fault rather than the usual subduction scenario. Coastal Ecuador, on the other hand, is simply a common example of volcanic arc sedimentary rock buckled up into high-angle to recumbent folds as the terrane rode out the underlying subduction activity creating the Andes. The intense disruption of west-to-east compressive forces occurred relatively shortly after deposition of the strata themselves and early in the process of induration, bringing about considerable deformational shearing along bedding planes as well as creating highly-fractured axial-plane cleavage surfaces. The climate is wet enough along the coast to produce a continual degradation since exposure of the sequence to the elements.

Lessons Learned: This is "try to find a good-site country." The rule is to recognize the extreme degree of structural damage and humid-climate overprint of weathering, then carefully map and explore the site. In my mind, the key indicator of the degree of magnitude of these possible effects is the degree of ruggedness of site and site-area topography. The greater the relief, the greater the associated geotechnical problems.

Great Lakes Shoreline (Ohio & Pennsylvania): Ordovician-Aged Chagrin Shale

The geomorphically inconvenient situation of glacially planed Ordovician strata topped with lodgment tills is well known to Great Lakes shoreline practitioners. My own experience dwells mainly on the shorelines of western Pennsylvanian and eastern Ohio, where storm waves gnaw away at the Chagrin Shale and the joint-bounded blocks of till topple into the surf, to be eroded away. Just about everyone finds that the ensuing shoreline retreat is about 30-100 cm per year.

Lessons Learned: The universal lesson is that juxtaposition of two earth materials of strikingly different material/strength types, when found in a site of active geologic processes, make for a challenging engineering problem.

Mid-Continent Minnesota to Missouri: Ordovician St. Peter Sandstone

St. Peter Sandstone is weak only by virtue of its selective cementation. Typically, the rock is a poorly-cemented assemblage of well-graded silica sand which can be mined with little effort. St. Peter ground represents weak rock of the most fundamental type - - characteristically weak by the same depositional characteristics and lack of diagenetic modification for which it is known.

Lessons Learned: It is hard to be fooled by the St. Peter Sandstone and extremely embarrassing when such does happen. Each St. Peter site must be carefully explored for its facies differences in degree of cementation.

Missouri: Pennsylvanian-Aged Hushpuckeny Shale Member, Greater Kansas City

For all the strangeness of its name, this 3 m Member of the Pennsylvanian Swope Formation is a famous geotechnical personality in these parts. Its stratigraphic position is directly below the Bethany Falls Limestone Member (Swope), which has historically been the favorite high-grade crystalline limestone for dimension stone, and more recently, for aggregate. Topographically, the Bethany Falls stratum (it is massive) forms the base unit of most of the incised bluffs in the area.

For perhaps 20 years Hushpuckeny rocks have been recognized to sometimes contain, in places, enough iron pyrite to cause severe buckling of the cleared floors of the limestone mines. Room-and-pillaring has traditionally been used to recover rock and, since the early 1950s, provide for commercially valuable, secure storage space. Most astute mine/space owners now know enough to leave about 60 cm of Bethany Falls rock in place as a means of protecting the shale from infiltration of incidental water. Therefore, nearly all of the mines are dry and do not experience swelling pressure, except where nature's geologic rules have been ignored.

In the recent environmental era, Kansas City sanitary landfills ran into site characterization problems in trying to install and develop groundwater monitoring wells in the "impermeable" underlying Hushpuckeny rocks. By 1986 permit applications to the Missouri Department of Natural Resources (MDNR) were being rejected on this basis. The author was caught up in this fact early and made enough inspections of exposed Hushpuckeny rock, in quarry floors, to develop an understanding of the problem. The answer was solved by recognition that Hushpuckeny rock is broadly jointed, with two sets, spaced at 45 cm to a meter. By using inclined borings when possible, drilled perpendicular to the usual joint sets, a developable monitoring well generally can be established in seven or eight attempts.

Lessons Learned: Hushpuckeny rocks are another example that "shale" is not necessarily "impermeable". Rather, as we have learned elsewhere, rock-mass characteristics (i.e. jointing) can make fine-grained rocks monitorable for groundwater flow and groundwater chemistry.

Missouri: Mississippian/Pennsylvanian Ground of the Tri-State Lead-Zinc District

Joplin, Missouri, lies at the heart of one of the three separate mineral zones of the Mississippi-Valley-type deposits of Kansas, Missouri and Oklahoma. Here, significant construction projects, and environmental restoration programs, have been seriously impacted by weak-rock conditions. Joplin District represents a constructive introduction to an entire genre of troubles related to previously-mined ground. Mineralization would not have been present had the host ground not been somewhat structurally weak before mineralization. Ore deposition most importantly requires host ground that is just weak enough to admit the ore-bearing fluids. Once in the weak-rock or marginally-weak mass, the ore-bearing fluids lose hydrostatic pressure and temperature and are forced to precipitate into the host ground. The ground is again weakened in the process of ore extraction.

More often than not the ore found its way into the host ground through and along existing discontinuities. At Joplin, en-echelon faults host the ore deposit. Ore deposition occurred in several ways within two types of weakened rock masses:

- Stratabound ore, governed largely by the presence of open bedding planes and masses of secondary chert;
- Rings and circles, somehow related to the development of circular to semi-circular karstic pits known as "sinkholes" and "chimneys".

Where ore is not found, the ground is likely to be sound and unbroken cores of three (3) meters in length may be recovered.

Lessons Learned: Ore was deposited by fluids, likely under only modest pressure, finding their way through a fracture network of an otherwise sound rock. Therefore, they also were the areas most intensively mined. Today's geotechnical investigtions must start with an informed interpretation of the mining evidence. Geotechnically-worst ground represented the best ore values and now carries the complications related to mining, without any form of governmental control and no requirement to restore the underground workings. Typical geologic constraints are subsidence and catastrophic roof collapse, acid mine drainage, and ready-made contaminant transport pathways.

New England: There is Weak Rock in the Lower Paleozoic Ground

One might normally say that weak rock is absent in New England. My response is, "not quite." At lease two exceptions come to mind, as two examples come to mind. First is the area around Lake Winnepesaukee, New Hampshire, on the highly irregular contact fringes of the lower-Paleozoic Winnepesaukee Quartz Diorite. Here you will frequently encounter, at the glacier-beveled, grooved and smoothed top-of-rock surface, large (several m^2 of surface area) ferromagnesian/biotite-rich xenoliths which have been altered in-place to soft, micaceous soil that you can dig with your field knife. Some of these pockets are large enough size to represent differential settlement problems for highly loaded spread footings.

The second situation is foliation-trend stringers and thin zones of lightly-metamorphosed metavolcanic ash beds of the Silurian-aged Ammonusac strata of New Hampshire. When far enough away from the plutons to have escaped high-grade metamorphism, these units can represent site and facility-specific structural problems. This lesson was discovered during the upgrade re-routing of Interstate 93 near the Vermont border.

Lessons Learned: Fine-grained metasedimentary or metavolcanic (formerly tuffs and ashfalls, sometimes ashflows) can lie just far enough (laterally) away from the original source of the dominant intrusive body as not to have undergone strength-building via high-grade pressure-temperature reworking to schistose gneiss. Locate the nearest mapped pluton or stock and then draw a straight line perpendicular from its contact zone, out to your site. If you are far enough distant to have received a mere baking, the original material may have been damaged by not enough temperature/pressure conditions and may have been adversely affected by hydrothermal fluids.

New Mexico (Northwestern): Cretaceous-Aged Mancos Shale

Mancos Shale represents the prime weak rock of Four-Corners area of the southwest. Here is a region that generally is deficient in precipitation yet the presence of the rumpled hillsides of mass wastage of fine-grained clay-rich strata underlying cap rocks belie its troublesome nature.

In the late 1970s, the Public Service Company of New Mexico financed an exhaustive feasibility study of the ability of Mancos rocks to host the power generation hall of the proposed Seboyeta pumped storage electric generation plant (Stone & Webster Engineering Corporation, 1980). Slightly earlier, in the late 1970s, the Omaha District, U.S. Army Corps of Engineers made a similar study of the feasibility of construction of the Garrison County (South Dakota) Pumped Storage Project using Missouri River waters with the generation cavern to be placed in Niobrara Chalk. Both projects indicated technical feasibility but were negatively affected by the national response to President Carter's call for individual citizen actions to conserve electricity.

Lessons Learned: Even though a weak-rock unit is regionally known for its poor engineering characteristics, it should be possible to design and construct underground facilities in such rock. Layouts must consider the joint-influenced structural integrity and openings must be geometrically designed to provide sufficient lateral confinement. Measures must be taken to control the degree of saturation.

Texas (Central): Upper Cretaceous Eagle Ford Group

Eagle Ford rocks are perhaps the classic weak-rock unit of Texas. Eagle Ford rocks have a general reputation among civil engineers, contractors, and public works and grading code enforcers such that there is a common-knowledge standard of what constitutes a minimal site investigation. The most dangerous property of the Eagle Ford assemblage is its swelling nature.

Lessons Learned: The state of knowledge relating to its nature is high enough that there is no basis for ignorance among engineering geologists and geotechnical engineers.

SUMMARY

Now more than 80 years after introduction of the term, there is little excuse for not recognizing and appropriately dealing with weak rock. There are many varieties of weak rock and many circumstances under which engineering geologists encounter such materials. It is imperative that we know the indicators that suggest its presence. Once discovered in the project area, it is mandatory to make the observations, collect the necessary data and samples, test to verify the presence or absence of such material and then describe the degree to which weak rock may affect the project and to serve the basis dealing with its undesirable characteristics and behavior.

ACKNOWLEDGMENTS

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the grade" at the time. The author thanks the two anonymous reviewers for their helpful suggestions, and especially co-editor Paul M. Santi for his in-depth review .

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The Effects of Weathering on Rock Masses

ROBERT J WATTERS
Department of Geological Sciences, University of Nevada-Reno, Reno, NV 89557

ABSTRACT

The weathering process, which is dependent on climate and the chemical composition of the rock, affects the engineering behavior of the rock mass by inducing changes in the physical nature, strength, deformability, and permeability. Weathering induces structural, textural and mineralogical changes in rocks which fundamentally affect their engineering properties. Chemical, physical and biotic weathering processes produce these changes to both the rock material and rock mass. Chemical weathering depends on the presence of water, which causes volume and textural changes through solutioning, formation of acids, and hydration and dehydration. Physical weathering processes include differential thermal expansion and insolation, freeze-thaw action, wet-dry expansion, and erosion and crystal expansion. These processes produce intergranular and rock mass disintegration with the formation of new fractures and increase in rock volume. Biological weathering is mainly organic in nature and is confined to one to two meters below the ground surface.

Particular rock types are often associated with specific and dominant types of weathering and two different rocks in juxtaposition with different mineralogy and structure may exhibit vastly different weathering characteristics. At the rock material scale the rock becomes more porous, loses bond strength between grains, and new minerals may be deposited in pores and along fractures. At the rock mass scale, new fractures are formed and extended, discontinuity wall rock strength is reduced, and relic corestones of stronger rock survive as a result of inward weathering from discontinuity joint sets. Regardless of scale a weaker rock mass is generally produced which requires great care in characterization. The weathered rock can exhibit a range of properties, from mechanical behavior controlled by relict structure, to situations where the rock mass must be treated as a soil. Cross-sections of weathered rock masses generally show increasing strength and fracture spacing, and decreasing porosity, permeability, and fissility with depth. Exceptions however do occur to this idealized picture. The recognition of weathering from observational and simple strength index tests is generally possible on rock outcrops, surface excavations and through borehole logging and correlation.

INTRODUCTION

Weathering is a result of the alteration and breakdown of the rock mass by physical, chemical and biotic means near the earth's surface. Weathering has very important engineering consequences. Strong and resistant rock is changed into a state of lower strength and greater permeability. Weathering is a major component in the first stage of the process of erosion, and it is the first step in soil formation and in the release or accumulation of iron oxides, alumina, and silica within the soil or weathered rock mass.

Chesworth (1992) comments that the word "weathering" infers that climate is the principal agent of weathering and consequently rainfall and mean temperature are fundamental in causing the change. Weathering has been shown (Tardy and Nahon, 1985) to be entirely dependent on the water cycle which is driven by climate. Hence, geologically stable regions of the earth's surface in the recent past produce soil and weathering zones which are broadly coincident with climatic zones (Strakhov, 1967). This simple picture must be modified when Quaternary glaciation and tectonic forces have taken place as climatic factors alone are insufficient in unraveling the changes to the rock mass caused by weathering. Furthermore, the style of weathering and the character of the weathering products is also strongly affected by lithology. Given similar lithologies weathering in the tropics produces a rock mass very different from one that weathers in a temperate environment, and different rocks can react very differently to the same climatic conditions.

It is hardly surprising therefore that the study of the weathering phenomena by different disciplines and by investigators in different parts of the world has produced a large range of approaches, terminology, and conclusions. For engineering geologists, the description and assessment of a weathered rock mass has to convey the distribution of the weathering states, their engineering properties, and the significance to a proposed design. A survey of active researchers and practicing engineers and geologists worldwide indicated what weathering processes were significant in their region, and how these processes were described and classified (Anon, 1995). The major points of agreement were:

* Different lithologies respond differently within the same weathering regime and may produce very different engineering properties.

* Mineralogical banding or penetrative "master" joints can produce complex weathering profiles.

* Classification of weathered rocks is difficult unless the fresh end members are seen especially with weak rocks.

* The application of the same weathering classifications in all situations regardless, of the rock type(s), can produce erroneous results.

* Weathered rocks are often classified without defining the reference source for the terms used.

* The use of similar terminology in different classification systems where the meaning of the terminology is used differently.

This paper utilizes this consensus of opinions and conclusions to interpret the engineering aspects of weathering.

WEATHERING PROCESS

Physical, chemical, and biotic processes are the mechanisms which produce a weathered rock mass. They commonly act together, though the individual contribution they make to the weathered product varies directly with climatic conditions and the distance below the earth's surface. Excellent reviews of the weathering process are given by Ollier (1984) and Selby (1993). Physical weathering is generally characterized by the physical breakdown of rock material into progressively smaller and smaller fragments without marked changes in the nature of the mineral content. This disintegration process produces residual material essentially unchanged from the original rock. The main processes by which physical weathering occurs include differential thermal expansion and insulation, wet-dry expansion, freeze-thaw action, and crystallization expansion. Chemical weathering differs markedly in that the original minerals are replaced or altered into new mineral species, usually clay minerals. The major process in chemical weathering is chemical alteration, and associated volume and textural change. Physical and chemical weathering are often interdependent, as physical weathering increases the surface area available for chemical weathering to act on.

Biological weathering is largely confined to the upper few meters of the earth's surface. The major processes in biological weathering are the breakdown of rock material by the action of roots and burrowing animals, the effects of soil bacteria on mineral weathering, and the production of organic acids and other compounds derived from vegetation. Biological weathering is typically much less intense than chemical and physical weathering. Schematic illustrations of the more significant processes of chemical and physical weathering that lead to the disintegration and alteration of the rock mass are shown in Figures 1 and 2.

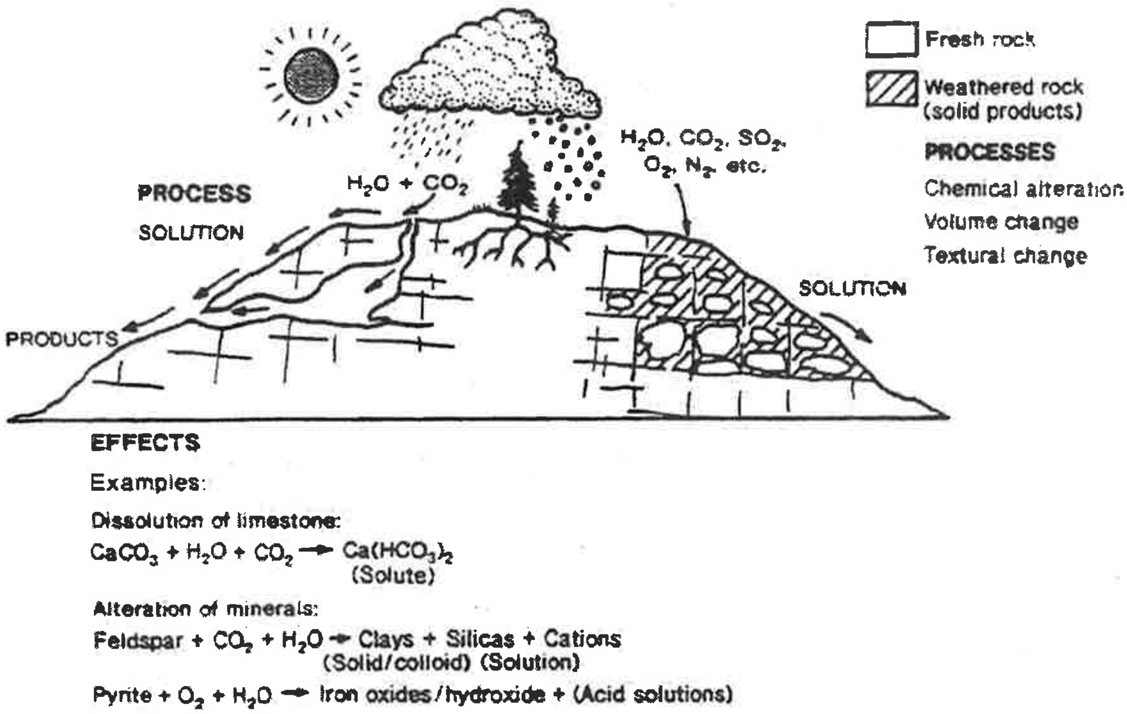

Figure 1. Chemical weathering processes (modified from Anon. 1995).

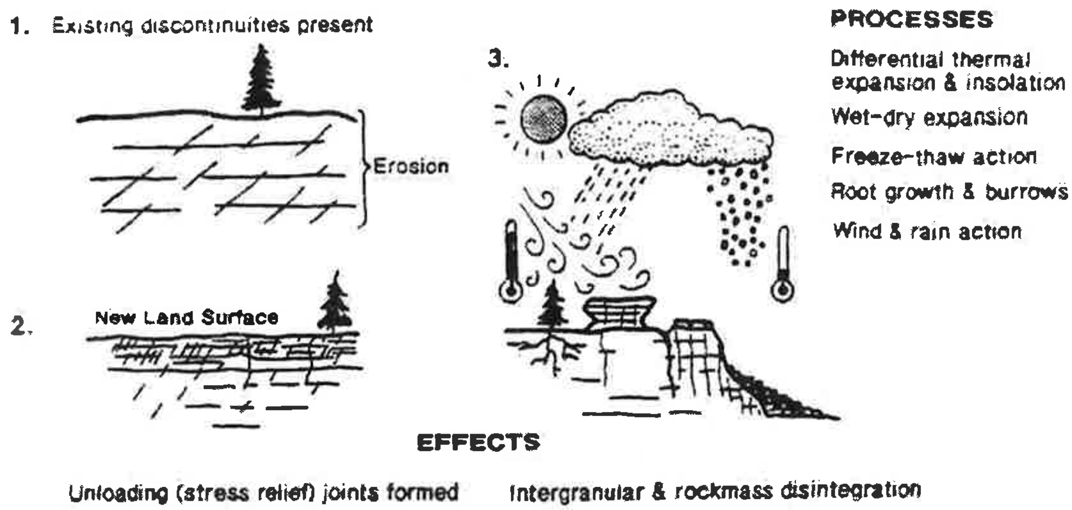

Figure 2. Physical weathering processes (modified from Anon, 1995).

A process whose products are very similar to surface chemical weathering is hydrothermal rock alteration (Guilbert and Park, 1986). This process differs in one important respect in that it is independent of the surface weathering process. Hydrothermal rock alteration can be vertically and laterally widespread (Figure 3). It develops in response to hydrothermal fluids affecting the parent rock through a combination of chemical, temperature, and pressure effects (Watters and Delahaut, 1995).

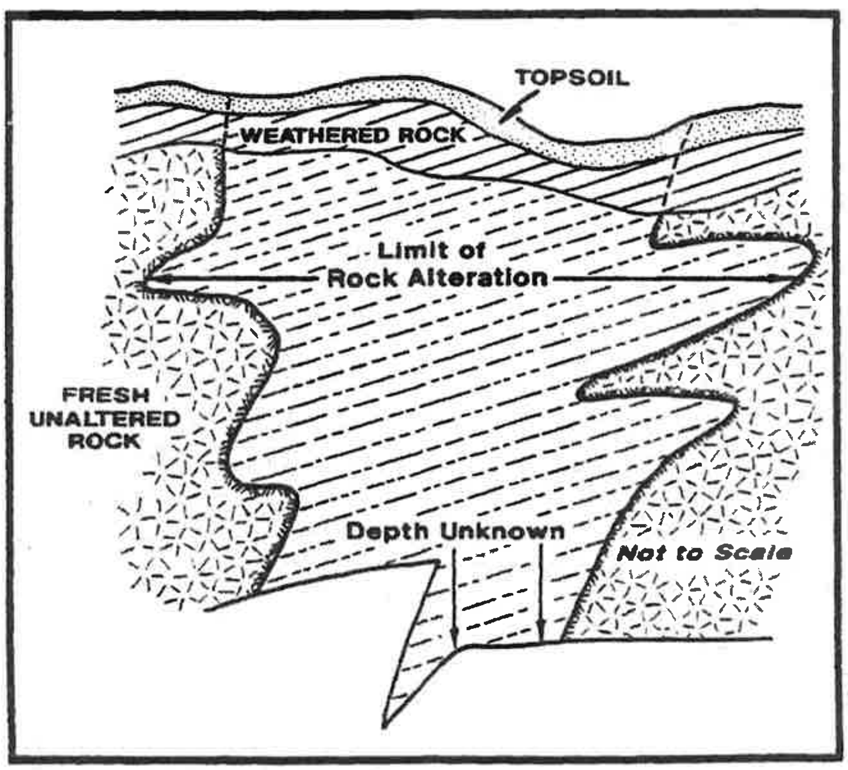

Figure 3. Cross-section comparing extent of surface weathering and hydrothermal alteration (Watters and Delahaut, 1995).

FACTORS AFFECTING WEATHERING

The physical and chemical composition of the unaltered rock together with climate are of prime importance in discussing the many factors which affect the type of weathering and the weathering rate. The weathering process and weathering rate are affected by the spacing, nature, persistence and aperture of discontinuities together with the hardness, permeability, degree of cementation, mineralogy and particle size (Figure 4).

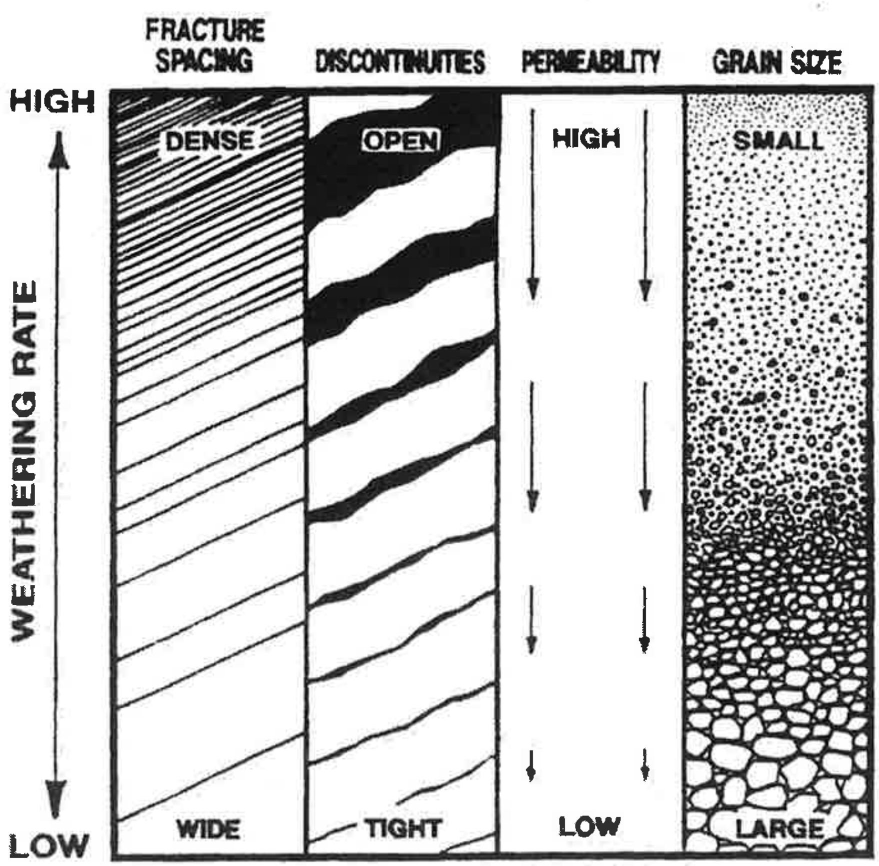

Figure 4. Rock mass conditions affecting the weathering rate.

For example particle size affects chemical weathering as the surface area of the particle or grain reacts with chemical solutions. Hence, the larger the total surface area of the particles comprising the rock the faster is the reaction: e.g. silt size particles would weather faster than sand size. Equally, a calcareous cemented sandstone will weather faster than a siliceous cemented assuming comparable permeability values. The weathering characteristics and products of different rock types can therefore be quite unlike.

The influence of climate is that the local temperature and annual precipitation will determine whether chemical or physical weathering dominates the process, which consequently determines the type of weathering products. In conditions of low rainfall, physical weathering takes place with breakdown of the parent rock into smaller fragments with minimal mineralogical alteration. Increasing precipitation enhances chemical reaction activity and produces an increase in dissolved and altered minerals. Eventually, chemical weathering dominates over physical, especially if the increased precipitation is combined with a temperature increase. An increase in 10 degrees centigrade may double or treble the chemical reaction rate. In northern latitudes with conditions of mean low temperatures of less than 5 degrees centigrade and annual precipitation lower than 30 centimeters, physical weathering dominates with low rates of weathering and weathering profile of one to two meters in depth. In equatorial latitudes which may experience high precipitation in excess of 200 centimeters and mean temperatures greater than 25 degrees centigrade, chemical weathering is dominant. Weathering profiles up to tens of meters in depth, occasionally up to 100 meters, and rapid weathering rates are typical.

WEATHERING MODES OF THE MAJOR ROCK TYPES

Particular rock types are often associated with specific and dominant types of weathering. Igneous, sedimentary, and metamorphic rocks behave differently to weathering processes depending on their chemical composition and the nature and degree of fracturing (mass characteristics). Closely fractured rock with a high mass permeability may weather far more rapidly than one with few discontinuities whose weathering profiles generally developing over geological rather than engineering timescales. This is particularly true in cases of deep weathering. Two different rock units in juxtaposition with significantly different mineralogy and structure may show vastly different weathering characteristics with corresponding changes in engineering behavior.

Igneous rocks

Substantial research has been performed in describing the weathering of plutonic rocks as a result of their uniform mineralogy, rock structure, weathering profiles and world wide distribution (Ruxton and Berry, 1957; Raj, 1985; Krank and Watters, 1983). The majority of the research has been on granitic plutons, gabbros, and on finer grained varieties including basalt. Grain size has been found to play an important role in the weathering of igneous rocks. The coarser grained plutonic rocks of granite and gabbro tend to weather more rapidly than the finer grained intrusions within the pluton. The apparent contradiction from the previous section that smaller grains weather faster than larger grains is that the larger mineral grains contain microfractures, open cleavages, and interconnected pores, which subdivide the grains into smaller particles, permitting chemical solutions to penetrate into and between the minerals. However, Higgs (1976) describes an exception with slaking basalts as a result of the presence of montmorillonite.

The spatial relationship of the discontinuities strongly influences the weathering profile and process. For igneous rocks the original discontinuity pattern is usually relatively simple, with sets perpendicular and parallel to the linear flow direction as determined by preferred mineral orientation or growth, a diagonal set at an angle of approximately 45 degrees to the trend of the flow lines, and a flat lying set. On weathering, an additional set develops parallel to the ground surface as a result of stress relief.

Granitoid rocks dramatically show the effects of climate on their weathered products. There is a general consensus that weathered products in temperate areas are generally "arenaceous" in character while those from tropical regions are "argillaceous". The engineering behavior of the weathered rock mass therefore is strongly affected by the type of weathered product. Clay assemblages in particular are seen to differ between arid, temperate and tropical areas, due to climatic influences upon the degree of leaching (ion removal) and hydrolysis (ion availability) (Keller, 1964). In arid regions smectites (montmorillonite) and chlorite are the dominant alteration products, while illite is dominant in dry temperate regions. Very wet temperate areas can be dominated by kaolinite, which is often the most commonly reported product of tropical weathering (Power and Smith, 1994). Igneous rocks often produce corestones on weathering in which weathering can clearly be seen progressing into rock blocks from surrounding joint sets (Figure 5).

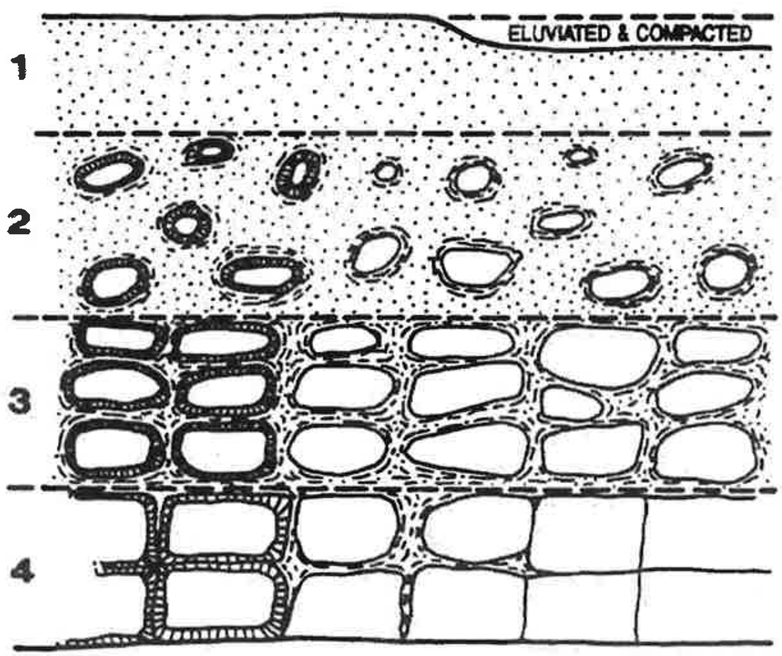

Figure 5. Mature granite weathering profile (Ruxton and Berry, 1957).

Sedimentary Rocks

Compared to igneous and volcanic rocks, sedimentary rocks are relatively more stable under ambient climatic conditions and most do not weather as rapidly. Sandstone, carbonate, and argillaceous (shale, mudstone etc.) rocks comprise the majority of sedimentary rocks which have been addressed by researchers (Price, 1995). The effects of the weathering process are markedly different for the different rock types as a result of the wide range of mineralogies and rock mass characteristics. Sedimentary rocks generally develop discontinuity sets, or bedding planes at the interface between different phases of deposition, though this pattern can be complicated by the development of cleavage or foliations imposed by later tectonic movement.

Sandstone weathering consists mainly on the breakdown and removal of the cementing agent of the sand grains comprising the rock. Calcite sandstones are affected by solution and often weather like limestone. Iron oxide cements generally hydrate to hydroxides with migration of iron within the sandstone to form iron rich accumulations. Given the high porosity and the different types of cementing agents, color banding is common. On weathering, sandstone often crumbles or forms blocks and on occasions exfoliation is observed.

The two major carbonate rock types are limestone and dolomite. Carbonate rocks are composed mainly of the minerals calcite and dolomite, which have two important physical properties: they can be soft and they are soluble in water. Carbonate rocks are distinct from other rocks in their higher solubility, which gives rise to the well known and distinctive "karst" topography and engineering problems associated with sinkhole development, collapse structures, caves and smaller openings underground, and residual clay. The type of weathering depends on the variety of carbonate, its density, impurities, porosity, and fracture density. The residual soil developed over carbonate rocks is the insoluble portion of the original rock, mainly quartz, chert, iron and manganese oxides, and clay minerals (Figure 6). The residual soil is usually clayey but can be sandy and pebbly (Dearman, 1981).

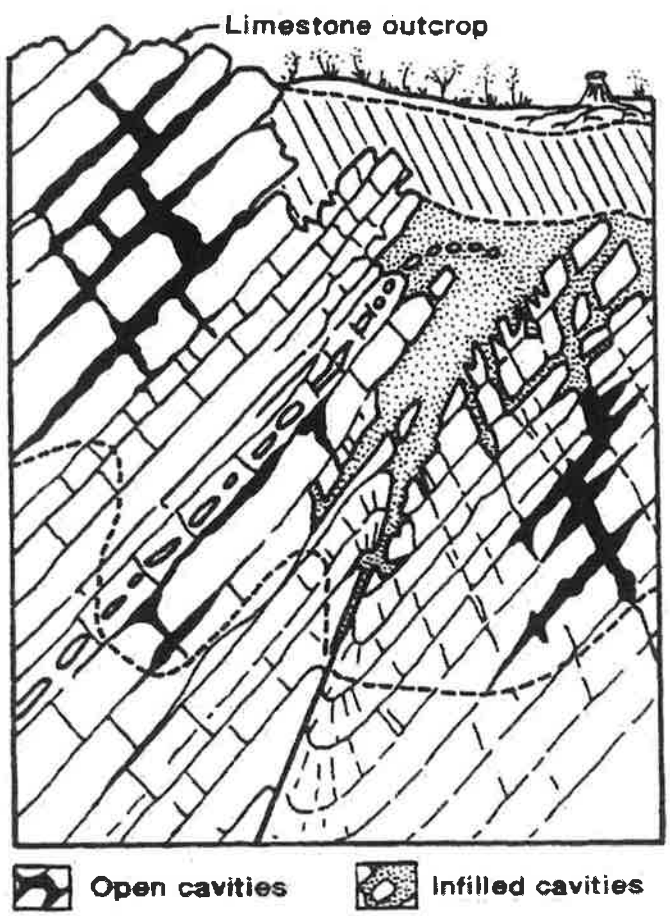

Figure 6. Typical weathering profile for carbonate rocks (Dearman, 1981).

The clay rich sedimentary rocks, shales, mudstones etc. have been extensively studied as a result of their widespread distribution. They are composed of clay and silt sized particles generally with strong bedding or laminations which often permits the penetration of water. Stress release cracks, cracks produced from wet dry cycles, and swelling following water absorption are some of the more common signs of weathering (Wetzel and Einsele, 1991). Weathering in clay rich rocks can be detected by color changes produced by oxidation. The changes can be obvious, but are often subtle, going from gray to brown or color variations of the two.

Metamorphic Rocks

Metamorphic rocks are often mineralogically, texturally and structurally more complex than other rocks as a result of mineralogical anisotropy at both the small and large scale. The anisotropy produced by concentrated accumulations of minerals in phyllitic, schistose, and gneissose rocks often results in the preferential weathering of these minerals, especially when the minerals are biotite, muscovite, and hornblende (Dobereiner et al, 1993). Frost weathering can rapidly break up a schist which contains very strong schistosity. Amphibolite, consisting mainly of hornblende, weathers in a similar style to basalt producing a deep weathering profile and the production of clay, both of which have major engineering implications. One of the most resistant metamorphic rocks to weathering is quartzite. If pure, it is highly resistant to chemical weathering but can be affected by physical weathering.

RECOGNITION AND DESCRIPTION OF WEATHERING IN THE ROCK MASS

A precise description of the changes occurring to the rock is required to assess the effects of weathering and the development and characterization of the weathering profile. The accurate description of the stages observed in weathering from a unweathered rock to a residual soil is the first step in the generation of engineering classifications for weathering. Weathering affects both the rock material and the rock mass. Weathering descriptions have been made by numerous authors for describing the rock material (Moye, 1955; Newberry, 1971) and the rock mass (Ruxton and Berry, 1957; Gamon, 1983). The majority of the descriptions are for igneous and volcanic rocks and rock masses, though a few researchers have attempted to encompass all major rock groups (Dearman, 1976 and 1986).

Many factors are recognized in the weathering process which are utilized in establishing weathering grades within the weathered profile. Lee and de Freitas (1989) proposed that these factors can be classified into three main groups for granite, though their grouping is applicable to most rock types. Their three divisions are based upon visual identification of particular geological characteristics, index mechanical tests, and combining visual recognition and index testing.

Visual characteristics include noting the discoloration of rock material, the proportion of decomposition of the dominant minerals within the rock, percentage of physical disintegration, preservation of original texture, presence of roots, degree of discoloration along a joint plane, angularity of corestones, and rock to soil ratio. Mechanical characteristics incorporate the degree of penetration of knife or pick, method of excavation, friability, NX core recovery, RQD, relative permeability, and slakability characteristics.

Ruxton and Berry (1957) were among the first to discuss the weathering of granite in Hong Kong in both geological and pedological terms, and their granitic weathering profile has been used as a starting point for many other descriptive weathering profiles developed for other rock types. Their four weathering zones (shown on Figure 5) comprise a mature weathering profile resulting from chemical alteration accompanied by consequent physical disintegration. Zoning encompasses residual soil development (zone 1) at the ground surface, to partially weathered rock at depth (zone 4), with intermediate zones based on percentage of corestones and debris. The profile by Ruxton and Berry was developed more fully by Dearman (1986) to show both rock mass weathering and rock material weathering (Figure 7). The figure illustrates Dearman's concept showing a mature chemical weathering profile for the rock mass and rock material. The rock material is shown as the weathering would develop around a corestone. Each of the four zones has unique chemical and mechanical properties and associated engineering behavior. Weathering grades of I to VI have been superimposed on the figure with grade VI comparable to zone 1, grades V and IV to zone 2, grade III to zone 3, and grade II to zone 4.

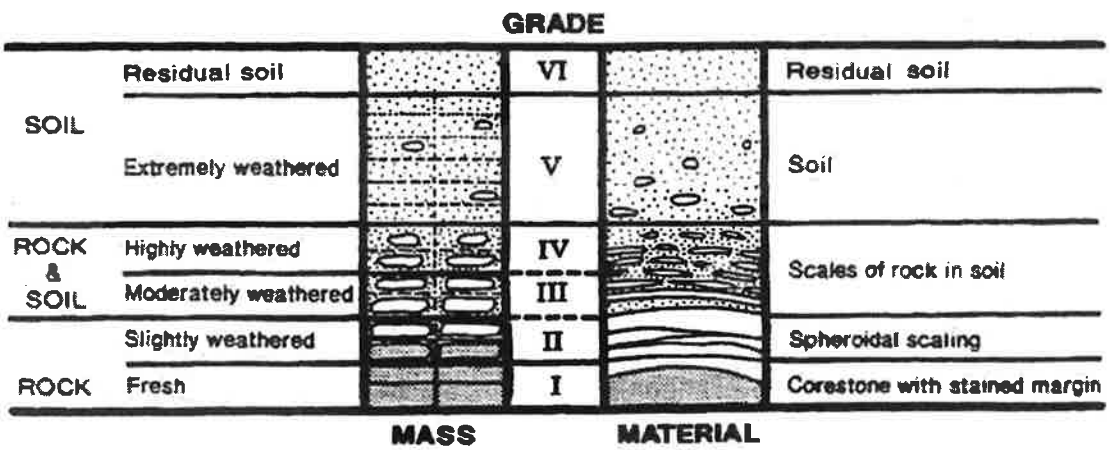

Figure 7. Comparison of weathering profile in both a granite mass and in rock material (Dearman, 1986).

CHANGES IN PHYSICAL AND ENGINEERING PROPERTIES

Since weathering affects both the rock material and the discontinuities, which are two independent variables from the standpoint of engineering properties, the distribution of weathering effects is mainly controlled by the arrangement of the discontinuities. The orientation, frequency, and persistence of discontinuities will primarily govern the ability of chemical fluids and physical stresses to act on the rock mass. Dearman (1995) produced a flow chart which illustrates the stages in the weathering of the rock mass incorporating both rock mass and a joint bounded block of rock material within the rock mass (Figure 8). In this schematic portrayal of rock mass weathering the following terms are defined. "Fresh" indicates no visible signs of weathering with "discolored" indicating that the original fresh rock color is changed. The "disintegrated" term shows that the rock is weathered to the condition of a soil with original rock fabric intact and mineral grains mainly unaltered, whereas "decomposed" differs in that most mineral grains are decomposed.

It is readily apparent that a rock mass having descriptions of fresh and discolored will behave as a rock whereas disintegrated and decomposed will reflect essentially soil behavior characteristics. Figure 8 illustrates that as penetration of weathering goes inwards from discontinuities the dimensions of the original block are important in determining the engineering behavior of the rock mass at any one stage in the weathering process. The larger the dimensions (block size) the longer the rock mass will portray rock-like behavior as the ratio of soil to rock remains low.

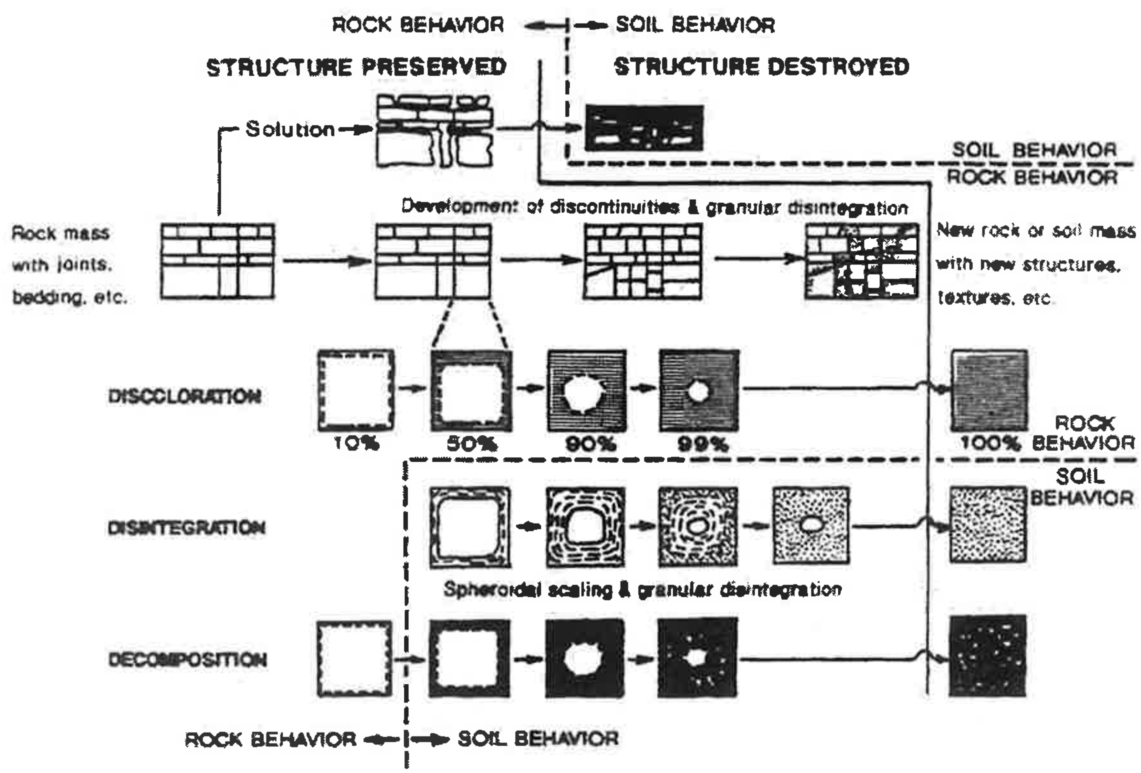

Figure 8. Idealized weathering stages of a rock mass (modified from Dearman, 1995).

The idealized diagram in Figure 1 negates the distribution of distinct rock types within the overall rock mass. Different rock types may be present as a result of geologic structures and other geologic processes of rock formation and deposition. As has been discussed, different rock types often respond differently to weathering, one rock type may exhibit minimal alteration whereas another may show extensive alteration.

Non-uniform weathering of a rock mass is often the norm for most weathering profiles as a result of mineralogical arrangements, structure, and rock type grouping within the rock mass. Weathering profiles, therefore, can be very complex, with highly irregular contacts between bedrock and the weathering material and an unpredictable grading from one weathered zone to the next. For a relatively unweathered rock mass the rock structure is generally more important than the rock material in understanding the influence of weathering on rock mass behavior. As the rock mass weathers, the difference in shear strength between the intact material and the discontinuities becomes less significant and failure planes develop which are controlled partly by the relict structure and partly by the presence of weaker weathered materials. Ultimately in a residual soil where all traces of relict structure have been lost, the failure characteristics are controlled by the engineering behavior of the weathered material alone. Generally rock will lose strength, have lower modulus values, become more deformable, and change its permeability depending upon the nature of the rock and type of weathering products. The degree of weathering may be reflected by changes in index properties such as dry density, void ratio, clay content, and seismic velocity. Figure 9 illustrates changes in index properties on weathering for Sierra Nevada granodiorite.

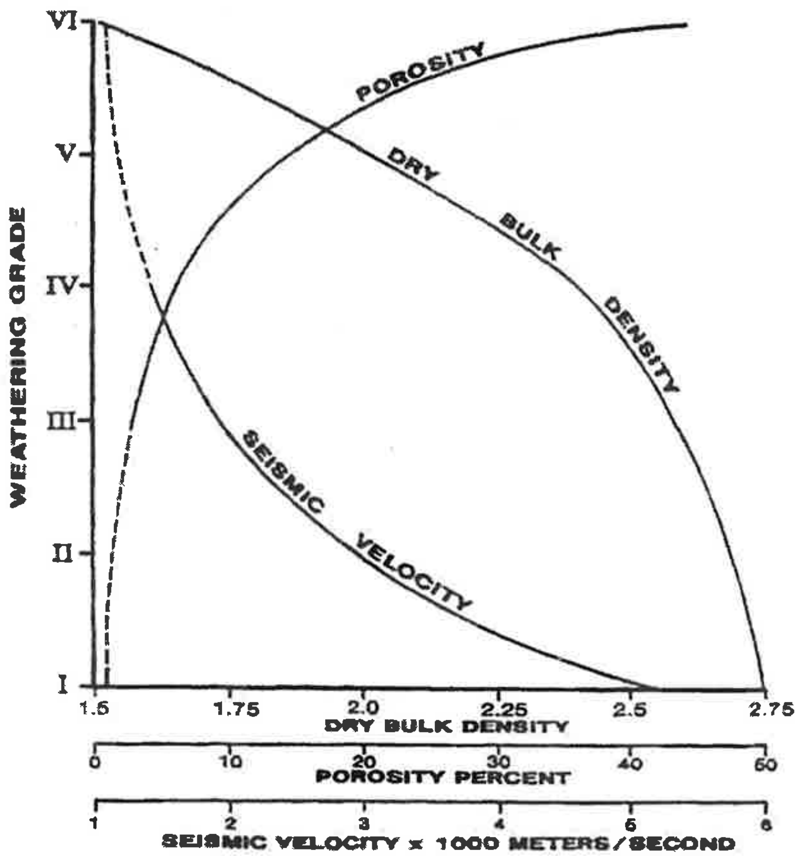

Figure 9. Changes in index properties of weathered Sierra Nevada granodiorite.

Baynes and Dearman (1978) describe voids developing in granite both between and within mineral grains as weathering develops, and Taylor (1988) found similar behavior for mudrocks. The voids can be partially or completely filled with weathering products such as clay or these products may be removed by leaching. In exceptional cases more than half the original mass of material may be lost through decomposition and ensuing removal of weathering products without changing the original relative positions of relict mineral grains within the weathered rock. This produces metastable conditions within the "rock mass", which on application of load leads to collapse. These types of weathered rocks showing a stiff response to load at low stress levels and a marked increase in deformability at high stress levels. All rock types do not show this response.

The high void ratios seen in many soils derived from weathered rocks often exhibit significant apparent cohesion as a result of suction when partially saturated, though this apparent strength is lost on saturation. Permeability changes with rock weathering depend on the growth of voids, the framework of relict minerals, and the possible leaching of weathering products and redistribution or infilling of fractures and voids by these products. An increase in permeability may be seen in the early stages of weathering which can be reversed at a later stage as flow paths become infilled by weathering products (Anon, 1995).

The weathering of argillaceous rocks produces changes in the arrangement of particles, fabric, cement and pore space. In slightly weathered materials the shear resistance and failure pattern are strongly controlled by bedding and fabric. On weathering the fabric starts to break down and preferred particle orientation is destroyed. This loss of orientation often results in an increase in shear strength. Hence, for many sedimentary rocks with a well developed fabric, the start of weathering can lead to a strength increase rather than a strength reduction especially for weathering in tropical climates (Fan et al, 1994). As weathering continues, the pore space is enlarged, the fabric disappears and the material can be regarded to be in a remolded state. The result is a considerable reduction in strength.

CONCLUSIONS

Climate and rock lithology and mass characteristics are of prime importance in assessing the effects of weathering. Chemical and physical weathering are the main processes, with each producing distinctive weathering characteristics and engineering behavior. Chemical weathering effects are manifest by the formation of new minerals, particularly clay minerals, associated with the decomposition of the rock. Physical weathering breaks down the rock mass into smaller and smaller fragments without changing the mineralogy. Igneous, sedimentary, and metamorphic rocks respond differently to the same weathering processes depending on their chemical and mass characteristics. The depth of weathering profiles are highly variable, measuring one to two meters in arid arctic conditions to extreme cases of over 100 meters for tropical weathering.

In slightly to moderately weathered rock the shear strength of the rock mass is governed by the distribution and orientation of discontinuities. As weathering intensifies the difference in shear strength between intact material and discontinuities becomes significantly less as failure can just as easily propagate through the weathered rock material. Void ratio and clay content generally increase on weathering whereas dry density, seismic velocity, strength, and modulus all decrease.

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Changes in Engineering Properties of Weak and Weathered Rock with Time

PETER P. HUDEC

Department of Earth Sciences, University of Windsor, Windsor, ON N9B 3P4

ABSTRACT

Some rocks are inherently weak, i.e., are 'born' weak, others become weak due to tectonic fracturing, or weathering of their components. The changes in their engineering properties with time and exposure are therefore dependent on their origin, and on their lack of physical and chemical equilibrium with their current environment. The rate of response to this disequilibrium determines their durability. Various indexes have been proposed to forecast rock durability. The petrologic indexes relate ratios of sound to unsound minerals and unfavourable textures. As these ratios change with time and weathering, so do the indexes. Statistical correlations of various engineering tests, such as Los Angeles Abrasion, sulphate soundness, point load, uniaxial compressive strength, slaking resistance, freeze-thaw resistance, etc. have resulted in empirical equations called 'Rock Durability Indicators' (RDI) that predict the changes of engineering properties of weak rocks with time and exposure conditions. In the last analysis, it is the weathering potential and weathering rate of a weak rock that will determine its engineering behavior with time.

INTRODUCTION

The term 'weak rock' is an engineering evaluation of the state of rock based principally on its behaviour in cuts, openings, embankments, structures, and foundations. Rock is 'weak' because it has lower strength than other rocks, or has a tendency to become weak or deteriorate in relatively short time while in service.

Geologists do not recognize 'weak' rocks. In their eyes, these are the rocks that are weathered or can easily and rapidly weather. The origin, mineralogy, and the tectonic state of weak rocks dictate their behaviour, and a geologic evaluation is made on the basis of long-term exposure to weathering. On the other hand, engineers evaluate the rock based on its response to established, standardized tests and to length of service.

Many of the engineering tests are designed to accelerate the normal weathering response of the rock under controlled laboratory conditions. However, not all the rocks weather in the same way, by the same process, and for the same reasons. The tests are designed for the 'average' rock, and often give inappropriate results for any one given rock type. Although the tests simulate accelerated weathering, they do not by themselves give a rate of weathering. Some 'rate of disintegration or failure' data can be obtained from the standard tests, but usually only the end result is given.

No single test can be expected to define the rock's behaviour. Most specifications for acceptance of the rock for engineering use are therefore based on acceptable behaviour in more than one test. With the wide availability of computers and statistical software, the emphasis is shifting from evaluation based on single or a multiple test result to a multivariate statistical evaluation of the test results.

ROCK WEATHERING AND WEATHERING RATES

Physical (mechanical) and chemical weathering of rocks can be considered as two end members of a continuous series, dictated by the type and severity of the climate. Figure 1 is a diagram after Peltier (1950) which illustrates the type and severity of weathering as a function of temperature and rainfall. The 'severity' can be translated into weathering rates. In the temperate

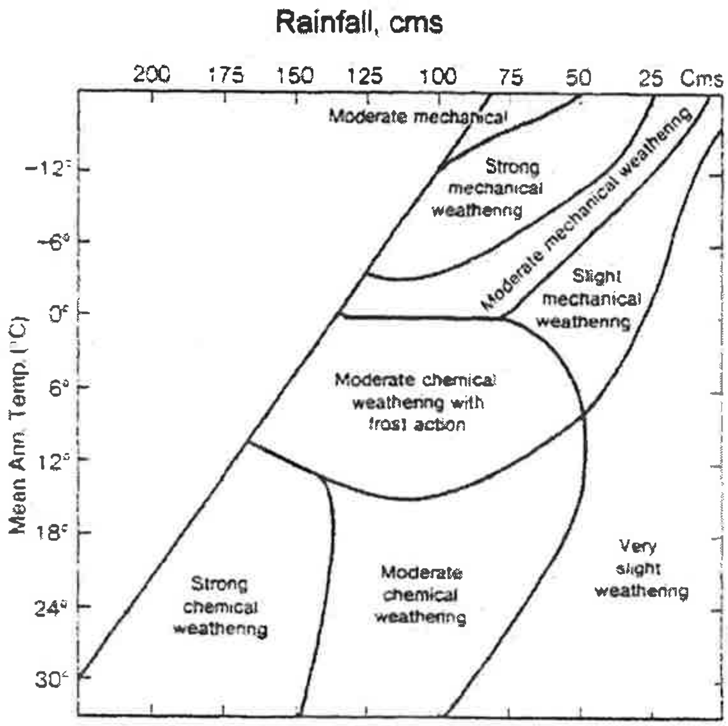

Figure 1. Rock weathering in diverse climatic zones. (After Peltier, 1950)

climate of much of North America, the weathering would fall in the middle of the diagram - moderate chemical weathering with frost action. The diagram is a general one, and specific rock types, especially the weak rocks, may behave differently, depending on the mineral composition and porosity and pore size distribution. Mineral composition would dictate the chemical weathering rates, and the pore characteristics would dictate the physical weathering rates.

MINERAL COMPOSITION AND CHEMICAL WEATHERING

All first year students of geology are exposed to Bowen's reaction series. For those of us who have gone far beyond the introductory concepts and are somewhat unsure of the Bowen's reaction series, the diagram of the series is given in Figure 2, but in inverse order and related to

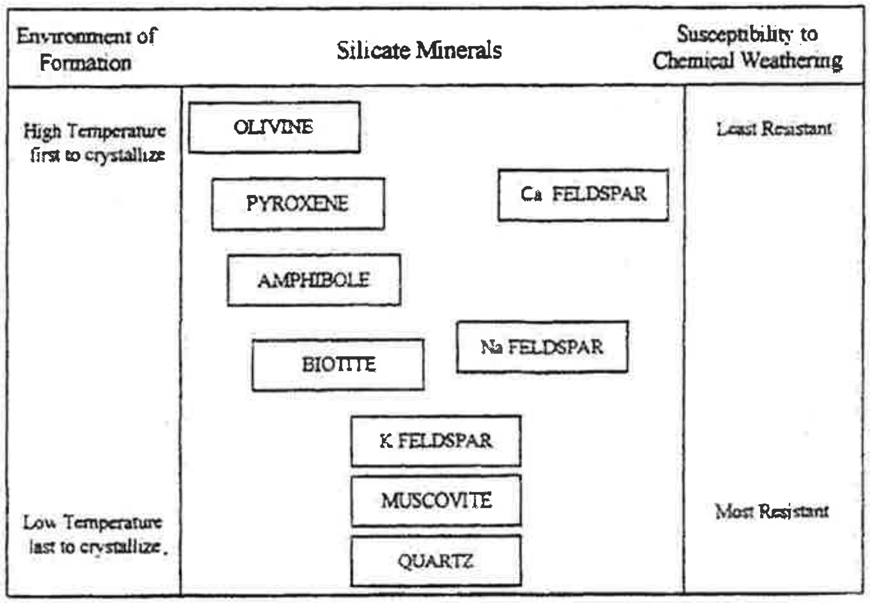

Figure 2. Chemical weathering as function of crystallization temperature of igneous rocks.

weathering susceptibility and rates. A well-known Le Chatelier's principle of chemical reaction states that any system in disequilibrium will react to restore equilibrium. Thus rocks formed at elevated temperature and pressure are at disequilibrium at earth's surface, and therefore subject to reaction, i.e., weathering. The higher the temperature of mineral crystallization, the greater the disequilibrium. The trend is to form minerals with lower free energy. The weathering of an anorthite ($CaAl_2Si_2O_8$) to clay and dissolved ions, for instance, results in the decrease of 1.32 kJ g atom^{-1} of standard free energy. The thermodynamics of weathering is reviewed by Curtis (1976).

Dissolution, oxidation and hydration are relatively rapid chemical reactions of weathering, generally effective in non-silicate minerals and rocks. Hydrolysis can be considered as the principal weathering process of certain silicate minerals into clays.

Figure 3 (modified after Curtis, 1976), shows the environmental factors which directly influence the nature, extent and rate of chemical weathering. The chief among these is abundant water supply which not only facilitates reaction, but removes reaction products so that the reaction can proceed. Dissolved acids in the water and warm climate accelerate the reaction.

PORE PROPERTIES AND ROCK WEATHERING

Figure 4 is a plot of cube strength of basalt cubes versus water absorption (Hudec, 1995). The data represents two basal flows, Bulhary and Camovce, of somewhat different composition. The variation in strength is strictly due to the difference in the degree of weathering. Weathering increases porosity, thus absorption, and decreases strength. The two basalts have different, but parallel behaviour patterns; the strength property can be predicted directly from the absorption

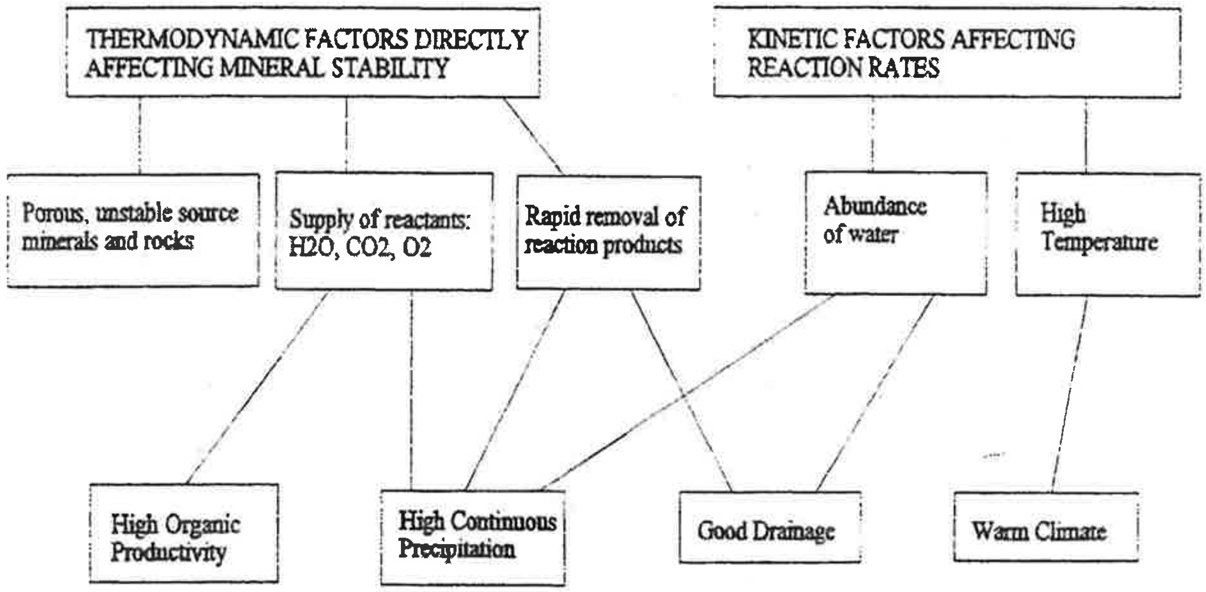

Figure 3. Environmental factors that affect the rate of weathering. (Modified after Curtis, 1976).

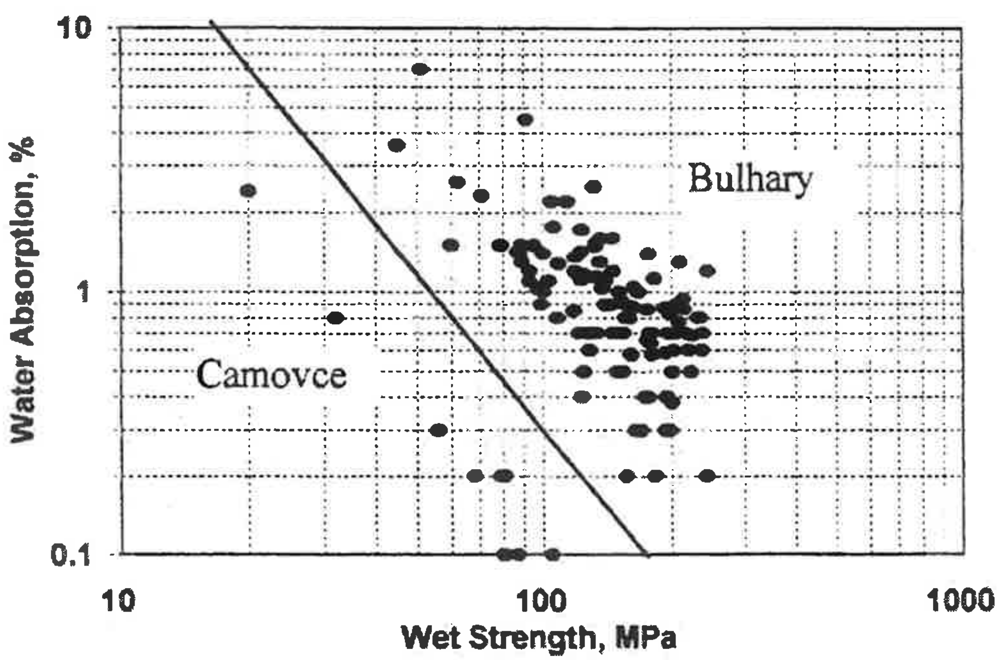

Figure 4. Relationship of uniaxial strength of two basalts (Camovce and Bulhary) relative to their absorption (weathering). (Hudec, 1995).

parameters. As the basalt weathers, absorption increases, and the strength decreases. Figure 5 (Hudec, 1997) illustrate the relationship between bulk density and water absorption among a variety of Ontario rock types, including carbonates, granites, and crushed gravels of various lithic types. Superimposed over the plotted points is the observed service record of the crushed stone used in cement and bituminous concrete pavements. Number 1 indicates excellent resistance to weathering and abrasion, whereas number 3 represents a poor response. The service index factors have been obtained by visual examination of the pavements. One hundred aggregate sources are represented in the survey. It can be seen, that as the bulk density of the original source material decreases and its water of absorption increases, the aggregate has an increasingly poorer record in performance. The poor performance record reflects the increased rate of weathering under conditions of use.

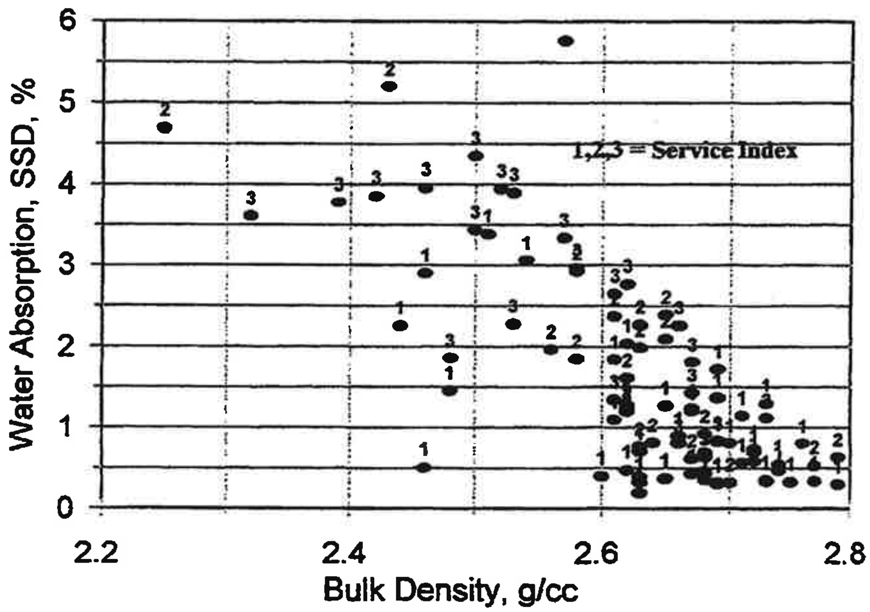

Figure 5. Service record of aggregates as a function of their density and absorption characteristics. 1= Good, 3 = Poor. Ontario Rock Set, crushed carbonates and gravels. (Hudec, 1997)

PETROGRAPHIC INDEXES IN WEATHERING

Figure 6 shows the same rock set as in Figure 5 in which a petrographically determined quantity, the Petrographic Number, is related to water absorption. Petrographic Number (PN) has been used by the Ministry of Transportation, Ontario (MTO) as means of estimating the engineering properties of rock. It is based on the mineralogy, density, softness, and the degree of weathering evident in the rock fragments. The rock pieces are grouped according to the above criteria, and their percent weight determined. Each group is assigned a weatherability or durability factor of 1 (excellent), 3, 6, or 10 (deleterious), based on its expected performance. The PN is the sum of the products of percent weight times the durability factor. It is seen in Figure 6 that as absorption increases, the PN also increases. When the service records are

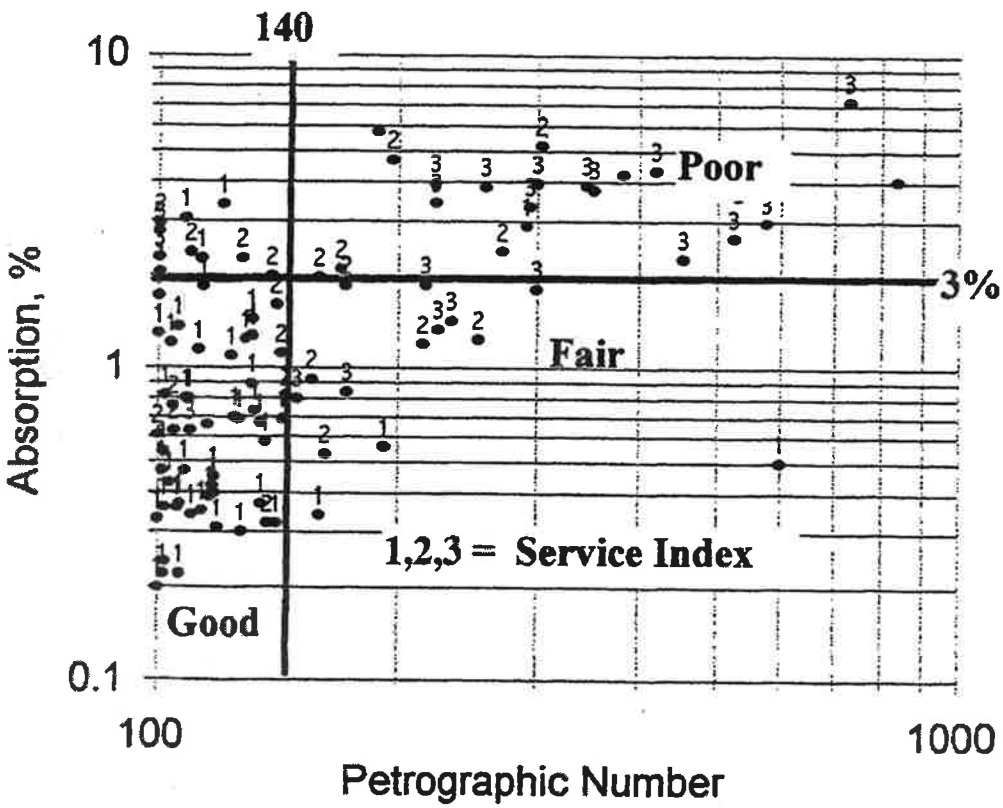

Figure 6. Quantitative Petrographic Number versus water absorption. Service index of 1 = excellent, 3 = poor.

plotted, and the acceptance limits for absorption and PN applied, the rocks can be divided into good, fair, or poor categories in terms of their expected behaviour upon weathering.

Relative hardness of the particle is one criterion that determines to which petrographic group it is assigned during petrographic analysis. A simple scratch test with a knife, or the degree of rounding the particle experiences during handling determine the 'softness' of the particle. The particles are soft because they are weathered, or inherently weak. Physical tests such as the microDeval abrasion test work on the same principle. Other observations, such as quick drying of the surface of the rock particle after momentary immersion in water suggest the presence of microporosity that draws in water by capillary action. The quick drying rocks are generally weak or weathered.

Other petrographic indices have been proposed by other researchers (Lumb, 1962, Mendes et al, 1966). The simplest index was that of Irfan and Dearman (1978). IP (index petrographic) is a ratio of volume percent of primary minerals in a thin section to volume percent of secondary minerals derived by weathering, plus pores and cracks.

$$IP = \frac{\{\%\text{Primary (sound) minerals}\}}{\{\%\text{ Unsound (secondary) minerals + voids + cracks}\}}$$

A secondary mineral rating R_{sm} by Cole and Sandy (1980) is somewhat similar to the MTO Petrographic number in that it is a sum of the products of percentage of secondary minerals times their stability rating, modified by textural rating:

$$R_{sm} = \sum[(P,M)]TR$$

where: P = % Secondary Minerals
M = Stability rating
TR = Textural rating

The secondary minerals in both of the above cases reflect mostly the weathering products of the rock; occasionally they may represent hydrothermal (deuteric) alteration products. Thus, the above indices are applicable to mostly igneous and some metamorphic rocks. Stability rating is the weatherability of the mineral under ambient conditions; olivine and anorthite, for instance, would have low stability. Texture refers principally to grain size.

Martin (1986) has an extensive compilation of index texts that can be applied to determine not only the degree of weathering present, but also the potential of the rock to weather. Most of these tests apply to a specific rock type, generally igneous or metamorphic bodies. The development or proportion of secondary minerals is usually used in some form, similar to the equations above. Other weatherability tests are based on the sonic velocity ratios in weathered to unweathered states (Iliev, 1967, applied to monzonite), and a quick water absorption index (Hamrol, 1961, and Pender, 1971) applied to granite and greywacke, respectively.

The weatherability and durability of basaltic rocks has been recently studied by Kuhnel et al (1994). They recognize two major forces that contribute to rock degradation and decay: (1) inner strain which is caused by shrinkage during rapid cooling, and (2) phase transformations which are accompanied by volume changes. Basalts are known to develop a local 'illness' called 'sunburn' when aged in air or in water. The 'illness' shows up as lighter brown spots and fractures that grow progressively with time. The degradation of basalts in dikes in Holland due to this 'illness' has caused serious problems. Kuhnel et al have developed a number of rapid tests to predict the basalt degradation. All tests involve a combination of grinding and/or 'punching' of the basalt, and rehydration of the stressed rock and powder in water or in a high temperature autoclave and then studying the mineralogical changes that result by the XRD method. The basalts that produce abundant secondary minerals (clays) are considered as rapidly degradable under normal weathering conditions.

The determinant factor of durability of Paleozoic carbonates was shown to be their Al_2O_3 content which reflects the amount of clay present (Hudec, 1980). The clay content influenced the degree of saturation achieved by the rocks, and their freeze-thaw breakdown. A direct, positive

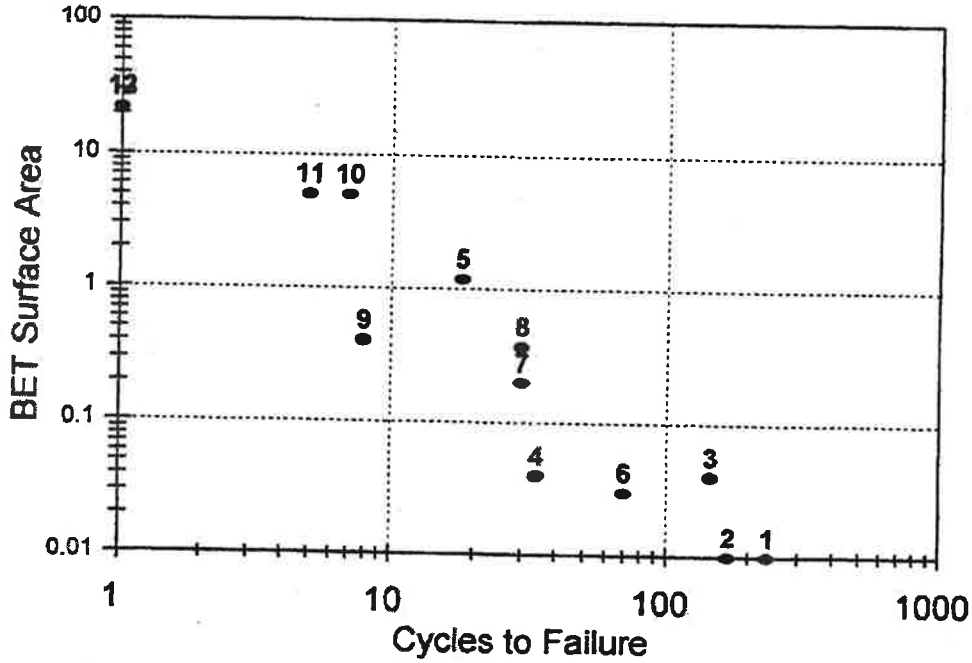

Figure 8. Effect of internal surface area on frost damage of diverse porous materials (after Blaine et al, 1953). Numbers represent materials in the table.

1	Georgia Granite	8	Red shale brick B
2	George marble	9	Red shale brick C
3	Carrara Ls	10	1:3 mortar
4	Beford Ls	11	Mortar, w/c 0.5
5	McDermott ss.	12	Concrete
6	Brick, fire clay	13	Concrete, 14 day, w/c .5
7	Red shale brick A		

List of materials tested in Figure 8.

Figure 9 (Ondrasik, 1996) shows the relative proportion of unfilled voids, bulk water, and adsorbed water in carbonate rock sequences used as construction material in Ontario.

The proportion of adsorbed water was measured after 72 hours of exposure to 95% relative humidity, bulk water was measured after 24 hour saturation, and void space was calculated as additional absorption after boiling. The rock cores from the sample set were resaturated for 24 hours, and then frozen at -20°C in a quantitative differential thermal apparatus (QDTA) which allowed the measurement of the amount of water freezing in the rock pores. The diagram indicates that the fine-grained, fine-pored rocks containing mostly adsorbed water showed no freezing; this same group also showed most deterioration in testing. No water froze in rocks containing more than 35% of their water in adsorbed form.

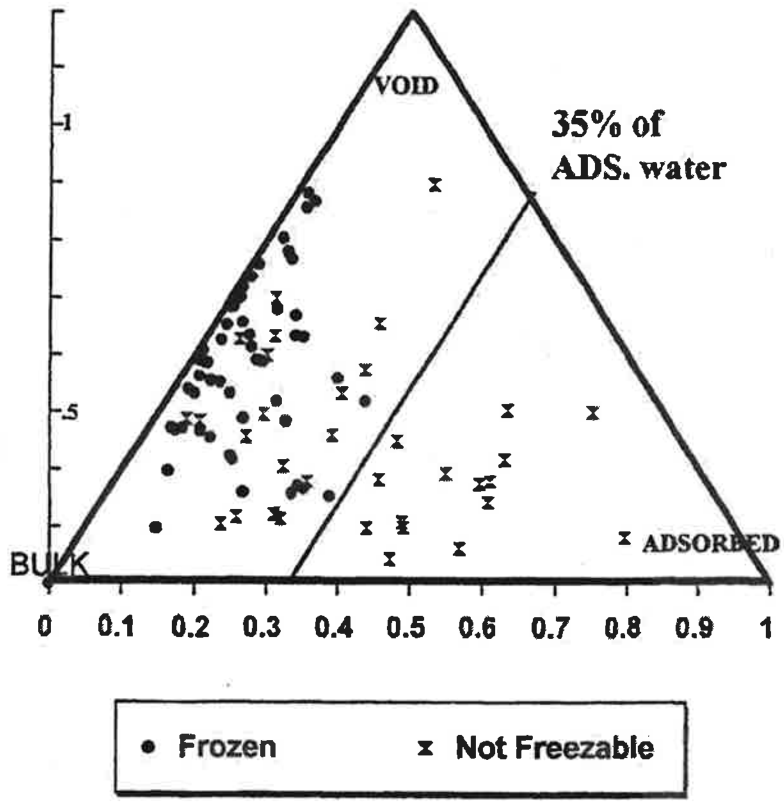

Figure 9. Ternary diagram of pore-water characteristics as they relate to freezing of pore water. (Ondrasik, 1996)

The osmotic pressure that develops in a rock pore is a function of its radius. It is the same force that causes water to rise in a capillary against gravity, and can be expressed by the formula:

$$P\,V = RT \ln\,(p / p_o)$$

where : P = Pressure
V = Molar volume
R = Avogadro's No.
T = Absolute Temperature
p/p_o = Relative vapour pressure

Since V, R, T are constant for given conditions, the pressure varies as the relative vapour pressure of water in pores. This relationship is graphed in Figure 10. The osmotic pressures that can theoretically develop are quite significant; they are even more destructive since they act against the tensile strength of the rock.

Under internal osmotic stress, the rock expands until either its tensile elastic limit is reached, or until the meniscii in the pores are removed by saturation or drying. The expansion as a function of the adsorbed is shown in Figure 11 (Hudec, 1993), where isothermal expansion is

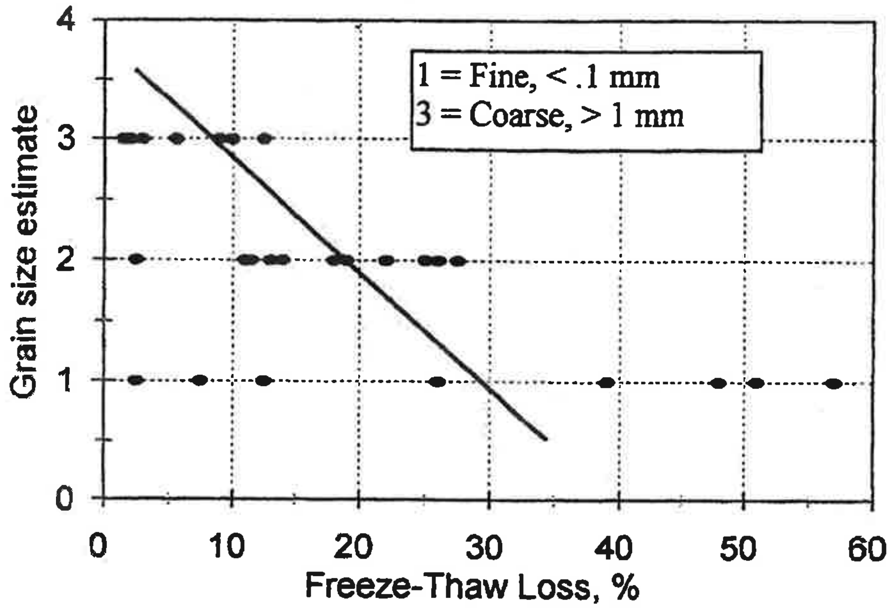

Figure 13. Effect of pore size estimate from grain size on freeze-thaw durability of rock aggregate. (Hudec, 1991).

which is osmotically active, especially if it contains dissolved salts. The rocks fail during the freezing process because any freezing occurring within the few larger pores or cracks in the rock draws the unfrozen water to the freezing sites. The ice in the freezing sites therefore continues to grow and exert pressure.

DISCUSSION

Weathering and durability of rocks can be studied from many points of view, as the review of literature above suggests. The approach taken depends on the rock type studied, and on the purpose of the study. Most of the sustained research has been in the durability of rock aggregates used in concrete, and in the durability of concrete. The purpose of these studies has been to identify and perhaps eliminate the proportion of 'weak rock' in the source rock for aggregate, and thus improve its overall quality. Summaries of some of the approaches taken are given below.

Density and Water Absorption as Indices of Weathering

Density, water absorption, and strength are related, as shown in Figures 4 and 5, and the higher absorption rocks are also seen to have poorer performance in durability tests and in service. For a given rock type such as basalt, water absorption or density are a direct indication of its weathering state and weatherability. The more highly weathered the rock is, the more porous and less dense it is. The density is also decreased because of the formation of secondary minerals of lower specific gravity, such as clays. The clays, in turn, are highly sorptive, and absorb more water into the rock.

It is more difficult to determine the degree of weathering for diverse or multiple rock types. However, a ratio of absorption or density of weathered rock density or absorption of unweathered rock can provide some idea of the degree of weathering present.

Petrographic Weathering Indexes

Petrographic indexes can be used to estimate both the weatherability and the degree of weathering. Weatherability is determined by the expected response or reaction of the minerals contained in the rock to the environment. High temperature minerals and glasses would be most unstable. But even so, the chemical reaction rates of minerals such as olivine or volcanic glass are slow in human terms. What is more important, the presence of these minerals may indicate that the rock may already contain secondary minerals produced by chemical weathering reaction which may break down rapidly in human terms. The susceptibility of rock to the more rapid physical weathering due to the secondary minerals is determined by considering the ratios of secondary to primary minerals present in the rock.

The more weathered the rock is, the more susceptible it is to weathering. This observation is used in classifying rocks through quantitative petrographic analysis tests such as those used for rock aggregates in many Canadian provinces and some US states. A weathered rock particle of any rock type is given a lower rating than an equivalent unweathered particle. Empirical tests suggest that, depending on the degree of weathering, the weathered rock may be three to six times less durable than the equivalent unweathered rock.

Some rocks such as shales are composed entirely of products of chemical weathering of the protolith rock. Some shales are more indurated, cemented, lithified, or metamorphosed than others. Their response to weathering can be measured by their response to drying and wetting - the most common weathering cycle in the environment. Slake tests measure both the wetting-drying response, and the abrasion response. The results of the tests can classify the rock in engineering terms into rock-like, and at the other extreme as soil-like, in their behaviour to weathering.

Thermodynamic Principles of Rapid Physical Weathering

The physical breakdown of weak rocks is both the function of freezing and thawing and the forces associated with ice formation, as well as a function of the osmotic pore water pressures that pry apart the shale particles. All fine grained, fine pored rocks are subject to these pressures. The water in small pores can be considered as an osmotic fluid with lower vapour pressure. Normal water with higher vapour pressure is osmotically enticed to enter the already filled small pores, creating pressure within the pore, expansion of the rock, and the ultimate fracturing of the weak tensile bonds that bind together the clay particles in shales and other weathered rocks. During freezing of these rocks, the water in the small pores does not freeze; however, water in larger pores and cracks does freeze. The ice in the larger pores has lower vapour pressure than the non-frozen water in the small pores, and the ice continues to grow by vapour transfer, exerting pressure on the pore walls. Presence of dissolved salts (such as de-icing salts) increases the osmotic disequilibrium between the water in various pore sizes. Repeated cycles of wetting and drying, or freezing and thawing that constitute the normal processes of weathering continually change the thermodynamic equilibrium within the pore system of the rock, causing expansion and contraction of the system, eventually fatiguing the bonds that bind the minerals. This causes what we perceive as physical weathering.

An example of the relationship of freeze-thaw deterioration and decreasing compressive strength of porous materials can be found in the standard ASTM test C666 which measures the resistance of concrete to freezing and thawing. The compressive strength is not measured directly, but is related to the dynamic modulus of elasticity. The dynamic modulus is determined by the transverse sonic frequency method. During the test, a concrete specimen is subjected to rapid freezing and thawing cycles, and its transverse frequency is measured at stated cycle intervals. The relative dynamic modulus of elasticity P_c at any point in the test is expressed by:

$$P_c = [\, n_1^2 / n^2 \,] \times 100$$

where n_1 = fundamental transverse frequency after c cycles of freezing and thawing
n = fundamental transverse frequency at 0 cycles of freezing and thawing

Normally, a reduction of 60% of the dynamic modulus is considered a failure. Similar testing could be applied to rock, or to rock fragments embedded in a cementing medium.

Selected Geotechnical and Geologic Tests to Identify Degree of Weathering and Proportion of Weathered Rock

Water Absorption and Density are the simplest tests that can be used to identify weak rocks. Normally, rocks with high water absorption and low density would indicate advanced weathering. However, it is not an absolute test. Rocks with naturally high open porosity, such as sandstones, and some porous carbonates would also have high absorption and low bulk density without being weathered and weak.

Petrographic Analysis is a geologist's tool to visually identify the weathered rock by the presence of secondary minerals. It is also used to identify rocks which are weak, but not necessarily weathered, such as shales that are prone to rapid weathering. Petrographic analysis can be done on hand specimens, thin section, or rock aggregates. A large number of rock fragments representing the entire rock mass is perhaps the preferred method of determining the weathering or weatherability of rock rather than single sample analysis. The best approach is quantifying one which gives ratios, percentage or weighted percentage of rock fragments with different degrees of weathering.

In carbonates, the proportion of insoluble residue, which reflects mostly clay content, was found to relate directly to the rapidity of weathering.

Abrasion, such as the microDeval abrasion test has been shown to be have a good relationship to the expected performance of the rock when exposed to the weathering environment. Weak rock abrades more easily than strong rock, so its presence and proportion in the sample is easily detected in the test.

Slaking or Wetting/drying test is suitable if the sample being tested consists mostly of weak rock such as shale. The precision of the test suffers when the sample consists mostly of strong rock fragments.

Freezing and Thawing test, especially in an accelerator solution of 3% NaCl, is a relatively rapid and inexpensive way to determine the susceptibility of rock to breakdown both due to freezing and wetting and drying. It is suitable for somewhat stronger rock, or a mixture of strong and moderately strong rock. Weak rock disintegrates within very few cycles.

Hardness of the rock is often indicative of the presence of secondary weathering minerals, and the weakened bonds due to weathering. It is the main tool in bulk sample quantitative petrographic analysis.

Pore Size may be indicative of the rock's proclivity to weathering. It may also indicate the presence of fine grained minerals such as clay in the sample. Clay-rich rocks often have finer pore sizes. However, the pore size as an indicator of weathered or weatherable rock must be considered together with hardness and petrographic analysis.

CONCLUSIONS

The weathering of weak rock can be considered as ongoing process. In most cases, the rock is weak because it is weathered, or contains products of earlier weathering. The weathering products of chemical weathering, mostly phyllosilicates (various clays), then advance or accelerate the process of physical weathering. The prediction of the weatherability of weak rock is based on identifying and quantifying the presence of secondary or weathered minerals in the rock, and on the results of physical tests of weatherability, such as direct freezing and thawing and abrasion.

Although it is difficult to assign a weathering rate to a rock in terms of years, the various tests outlined above are able to distinguish weak rocks that are likely to deteriorate in few years of exposure from the stronger rocks that may last for generations in human terms. Of course, the environment of exposure as well as the nature of the rock determine how the rock will behave with time.

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Laboratory Measurement of Weak Rock Strength

BRIAN H. GREENE
ANDREW SCHAFFER
U.S. Army Corps of Engineers, Pittsburgh, PA 15222

ABSTRACT

Laboratory test methods that have been most commonly used in measuring the compressive strength of weak rocks are the unconfined compression test and point load test. It has been established that there is a close correlation between the point load index and unconfined compressive strength. It is recommended that in applying the point load test to shales and other weak sedimentary rocks possessing strong bedding anisotropy, that the test be conducted axially rather than diametrally. Shear strength testing of rock and weak rock is commonly determined by either the direct shear test or the triaxial compression test. Either test procedure can be conducted on intact rock core samples or oriented tests can be run on samples with discontinuities. Problems with unacceptably high degree of variability of direct shear test results on similar rock samples have been reported. This variability is primarily due to differences in test apparatus. Weak rocks are notorious for problems due to poor sample preservation. Moisture loss associated with the drying of clay rich rock samples results in slaking of the material and poor survivability of the samples prior to delivery to the lab. Special care must be taken to prevent moisture loss and in some cases provide radial and axial confinement. There is a strong correlation between moisture content and unconfined compressive strength for mudrocks.

INTRODUCTION

The weak rocks most commonly encountered in the construction of civil engineering projects are argillaceous sedimentary rocks, commonly termed mudrocks. The term mudrock is used as a general classification of all weak, fine-grained sedimentary rocks containing more than 50% clastic grains smaller than 60 microns in diameter (Blatt, 1982; Dick and Shakoor, 1992). They are a lithologically diverse group of rocks that include: claystones, shales, mudstones, siltstones and argillites. Mudrocks are the most common type of sedimentary rock comprising about two-thirds of the entire geologic column (Blatt, 1982).

Mudrocks are unique in terms of their engineering properties. Upon exposure in an open rock excavation, the release of overburden confining pressure and changes in natural moisture content often cause many mudrocks to deteriorate physically. This physical breakdown or slaking behavior presents a serious durability problem. The problem is compounded by the fact that the durability of freshly exposed mudrocks is not always readily apparent. Mudrocks are very sensitive to changes in moisture content. Loss of moisture often leads to degradation of physical properties, including strength, due to slaking. Certain mudrocks will deteriorate quite rapidly upon exposure, whereas others can survive years of exposure before their true durability behavior becomes apparent (Dick, 1992). The strength of a mudrock will generally decrease with increasing exposure time due to loss of moisture.

UNIQUE GEOLOGICAL AND ENGINEERING PROPERTIES OF WEAK ROCK

Certain engineering properties are of paramount importance in the design of slopes and foundations in weak rocks. In particular the swelling, slaking, shear and compressive strength properties are critical to design and construction. The swelling properties of mudrocks and related damages due to swelling are recognized as a widespread phenomenon throughout the world. Because of the multitude of problems involving different types of structures, durability has been a major focus of mudrock research.

Similar to the widespread problem of expansive soils, the swelling property of mudrocks is dictated by the nature of clay minerals present. The highly variable nature of mudrocks has been a significant deterrent in the development of the complete understanding of swelling behavior of these materials (Sarman, 1991). Further, the low strength and durability of certain mudrocks has been responsible for countless slope instability problems, foundation failures, mine failures and shale embankment failures (Strohm, 1980; Dick and Shakoor, 1992, Goodman, 1993). There is likely a relationship between durability and the compressive strength of mudrocks, however additional reasearch in this area is needed.

TRADITIONAL STRENGTH TESTS

Traditional engineering geologic test procedures developed for rocks in general are also used for characterization of weak rock materials. Laboratory tests are usually performed in conjunction with field testing to determine the strength properties of weak rock. Laboratory testing of rock can be subdivided into strength tests, deformability tests, characterization tests and durability tests. After performing characterization (or index property) testing for purposes of identification and correlation, strength testing is conducted to provide values needed for design of foundation and cut slopes. Table 1 summarizes the commonly performed laboratory tests that have been employed in the field of rock mechanics and rock foundation engineering (U.S. Army Corps of Engineers, 1994). Individual laboratory strength test procedures are discussed below:

Unconfined Compression Test

The unconfined compression test, in which rock specimens are compressed perpendicular to their longitudinal axes, is one of the most convenient and useful ways for determining the strength and deformational properties of rock. The ends of the specimens are sawn and ground flat in order to obtain near perfect right circular cylinders with a length to diameter ratio of 2.0 to 2.5. The specimens are then placed in a compression testing machine and a hydraulic ram compresses the specimen to failure. The load is measured by either a gauge connected to the hydraulic system supplying pressure to the ram or by a load cell placed in between the specimen and the ram; the

Table 1. Summary of rock test procedures (after USACE, 1994).

Purpose of Test	Type of Test
Strength	Uniaxial Compression Point Load Direct Shear Triaxial Compression Direct Tension Brazilian Tensile Strength
Deformability	Uniaxial Compression Triaxial Compression Swelling - Free Swell - Swelling Pressure - Three Dimensional Swelling test
Characterization	Water Content Density (Unit Weight) Atterberg Limits Absorption Pore Size Distribution Abrasion Resistance Sonic Velocity
Durability	Slake Durability Test Jar Slake Test Slake Differential Test Modified Jar Slake Test

NOTES:
1. Table modified from that published in U.S. Army Corps of Engineers, Engineering Manual 1110-2908, Rock Foundations, 1994.

2. The point load test is also frequently performed in the field.

latter being the more accurate method. The stress required to fail the specimen is called the unconfined compressive strength (UCS). Underwood (1967) found that the compressive strength of shale ranges from less than 25 psi for weaker compaction (or soil like) shales to more than 25,000 psi for well cemented shales.

If the deformational properties of the rock are desired, measurements of axial and radial or circumferential strain are made on the specimen. This can be accomplished by using either electrical resistance strain gauges, cantilever strain gauges, or LVDT's (linear variable differential transformers). From these measurements, Young's modulus and Poisson's ratio may be determined at the stress level of interest.

Point-Load Test

The point-load test was originally described by Reichmuth (1963) and by Broch and Franklin (1972). It is widely used to obtain a strength index which can be employed to estimate the UCS. The point-load test provides a rapid method, which can be used in the field or laboratory, to determine the unconfined compressive strength of a rock material.

Broch and Franklin (1972) found that diametral point-load tests of relatively high strength, isotropic rocks produced constant values when a length to diameter ratio of 1.4 was used. Their recommended procedure requires that 20 pieces of 50 mm diameter rock core be loaded diametrally. Broch and Franklin suggested that UCS / Is = 24; where Is is the point load strength index (Figure 1). Forster (1983) and others have determined that no UCS / Is ratio can be applied universally, and that each rock type (or unit) must be tested to establish a material specific UCS / Is ratio for use in subsequent strength estimates. Application of the standard point load test developed by Brock and Franklin (1972) for weak rock requires modification to the test procedure. Shales and other rocks have strongly defined bedding or layering which results in strong anisotropy in physical properties including point load strength. Recommended changes to the point load test as applied to weak rocks is discussed later in the paper.

Direct Shear Test

The laboratory direct shear test has been used with minor modification by the Corps of Engineers since the 1930's. Figure 2 presents the essential elements of the laboratory direct shear apparatus. The test specimen is generally a 4 or 6-inch diameter rock core about 4 inches in length. It is first cemented in a lower box. The cement used is Hydrostone, a gypsum compound. 1/16 to 1/4-inch spacers are installed between the lower and upper boxes to create a shear plane.

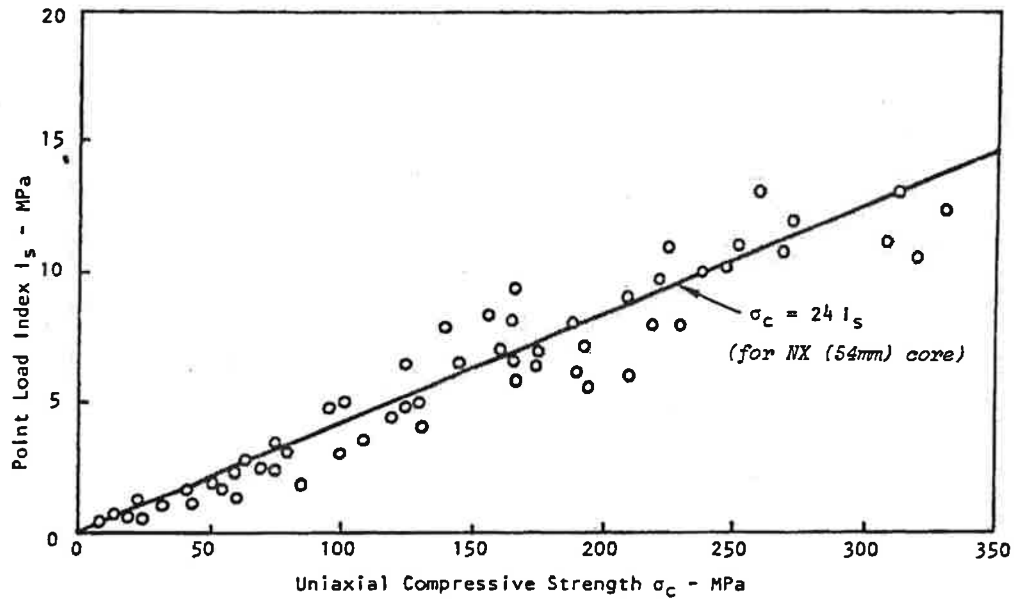

Figure 1. Relationship between point-load strength and unconfined compressive strength for NX (54 mm) size core samples (after Hoek and Bray, 1981).

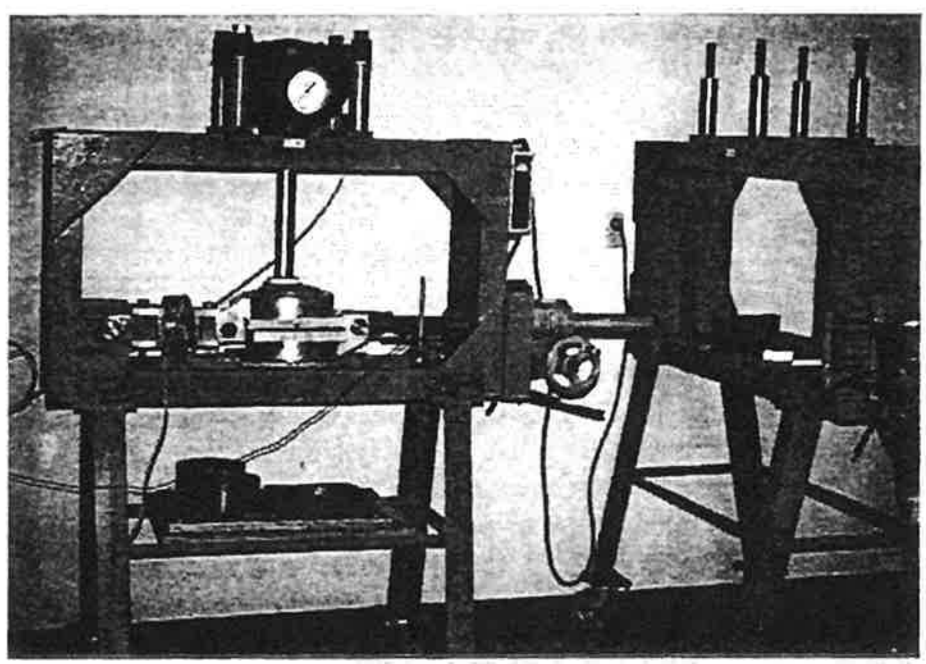

Figure 2. Direct shear test apparatus.

The other half of the sample is then cemented into the upper box. After the Hydrostone has hardened, the spacers are removed. A normal load is applied across the upper shear box, often using a hydraulic jack, with a uniform load applied to the specimen. The shear load is then applied at a slow, constant rate. Laboratory direct shear tests are commonly made at three different normal loads with a separate test specimen required for each normal load (Mellinger, 1966). The test results are reported with reference to the shearing load at failure versus the appropriate normal load. Depending on the uniformity of test conditions and the characteristics of the rock, the shear strength versus normal load can be plotted to define a shear envelope for the material as shown on Figure 3.

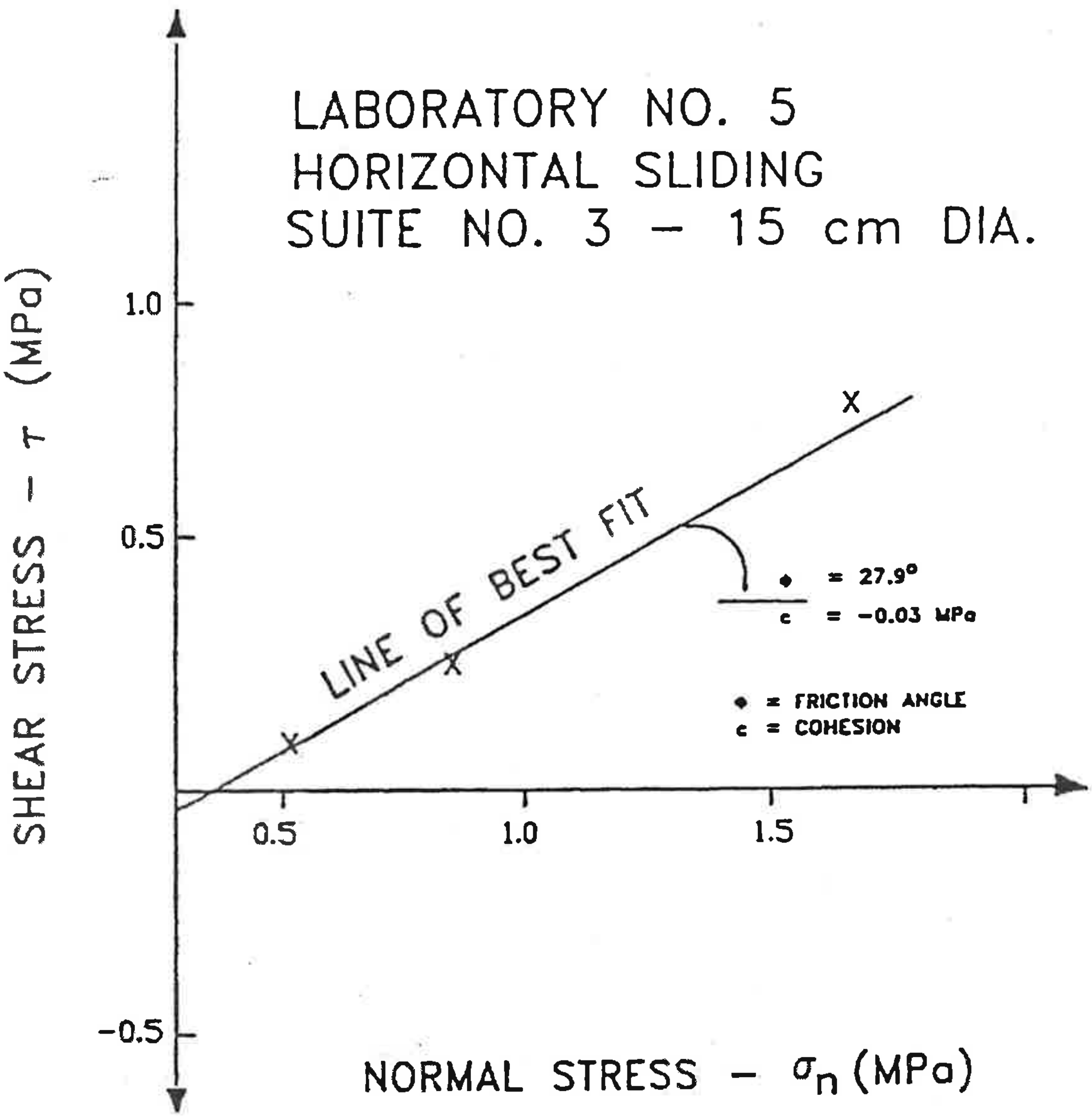

Figure 3. Typical plot of shear stress versus normal stress with failure envelope (after Nicholson, 1994).

All direct shear devices are somewhat similar in that they provide a means for applying and measuring a shear force and normal force parallel and perpendicular, respectively, to a chosen shear surface (Nicholson, 1994). There are numerous designs for direct shear testing apparatus. In general the differences lie in details of construction dealing with the manner in which shear and normal loads are applied and measured.

The direct shear test has been used widely for testing shales and other weak rocks because of the ability to orient the applied loads with respect to the sample anisotropy (i.e. bedding, seams, shear zones, and other rock discontinuities). Nicholson (1983) provides an interesting summary of direct shear testing on small diameter samples in the laboratory and makes a comparison with results of in situ shear devices.

Triaxial Compression

The triaxial compression test is the most useful test for determining the mechanical properties of rock over a wide range of stresses and at different temperatures that are representative of in-situ conditions at depth. Specimens of rock are made in the same way as in the unconfined compression test. The specimen is enveloped in a rubber jacket, placed in a triaxial cell, essentially a steel vessel, and then subjected to a lateral confining pressure by means of hydraulic fluid. The specimen is then axially loaded to failure by a compression testing machine. The test is run on several samples at different confining pressures and a shear strength envelope is obtained which is similar to the direct shear test. Joints and weak seams oriented 45 to 65 degrees from the horizontal can also be tested triaxially.

If the triaxial cell is equipped with the appropriate connections, deformability measurements can be made internally with the same instrumentation devices used in the unconfined compression test. The specimens can also be heated either externally by wrapping band heaters around the triaxial cell or internally if the cell has such a provision. Many cells also have pore pressure ports, however very few intact rock specimens (and especially fine grained, weak rocks) have permeabilities high enough to yield a pore pressure measurement.

Other Test Procedures

In addition to those standard laboratory strength tests summarized on Table 1, there are tests that are unique to the evaluation of weak rock. One test in particular is the slake durability test originally developed by Franklin and Chandra (1972) with details of the test method reported in ISRM (1979). In addition to the traditional index tests (i.e., specific gravity, bulk density, moisture content, absorption, etc.), other tests are often conducted and are applicable to weak rock. These include cyclic wet-dry, jar slake, and rate of slaking. Also, Atterberg Limits is a test originally designed for soils but has often been used for characterizing clay rich weak and weathered rock. Santi (1995) reports that existing classification and testing methods for weak rock do not adequately characterize these materials for engineering use. In his research on Colorado shales, Santi (1995) addresses problems unique to the testing of weak rocks and proposes a Slake Differential test which can be correlated to percent swell, jar slake and percent clay. There is a need for additional work in the important area of relating durability to index properties.

PROBLEMS ASSOCIATED WITH STRENGTH TESTING OF WEAK ROCKS

Sample Preservation

Two methods of sample preservation have been commonly used by the Pittsburgh District of the U.S. Army Corps of Engineers (USACE). The first method involves the use of a hot wax mixture of 50 percent paraffin and 50 percent microcrystalline wax. The sample is first wrapped in cellophane and then wrapped in cheesecloth. Next, the sample is placed in the molten wax and then removed until the wax cools and hardens. The sample must then be rotated and immersed several more times until it is completely covered with wax. The entire process is very tedious. It is also difficult for the laboratory to remove the layers of material and extract the sample. In addition, a concern was raised about the effect of heat from the hot wax on the natural moisture content of the sample. Consequently, this method is not favored by the District.

The second method involves first wrapping the sample in parafilm followed by wrapping the sample in heavy gauge plastic and sealing the ends with duct tape. The sample can easily be removed by cutting the plastic at one end. Parafilm is kept on the sample during sample preparation. If the rock is fragile, duct tape is wrapped around the sample to prevent breakage. Based on the authors' experience, this method of core preservation has proven very effective in preserving weak rock core and improving sample survivability.

Preber (1984) describes a sample preservation method that is very suitable for highly fissile shales. The sample is first wrapped in cellophane and aluminum foil, then placed in a slightly undersized PVC tube with a slit cut along one side. Metal hose clamps are placed around the tube to provide radial confinement. Wood disks and blocks are then placed on the evened ends and two threaded rods are subsequently tightened to provide axial confinement. An illustration of this sample container is shown in Figure 4.

Application of the Point Load Test to Weak Rock

There is a problem in the application of the point-load test to weak rocks such as shales and other rocks which exhibit a strong bedding anisotropy. Data from a study by Bauer (1984) indicate that diametral point-load testing on highly anisotropic rocks cannot be used to estimate the UCS because failure of the core sample occurs parallel to bedding planes and not across the fabric of the rock. For this reason, it is suggested that shales and other weak sedimentary rocks possessing strong bedding anisotropy be tested axially or normal to bedding planes. ISRM (1989) recommends that a length to diameter ratio of 0.3 - 1.0 be used for axial tests.

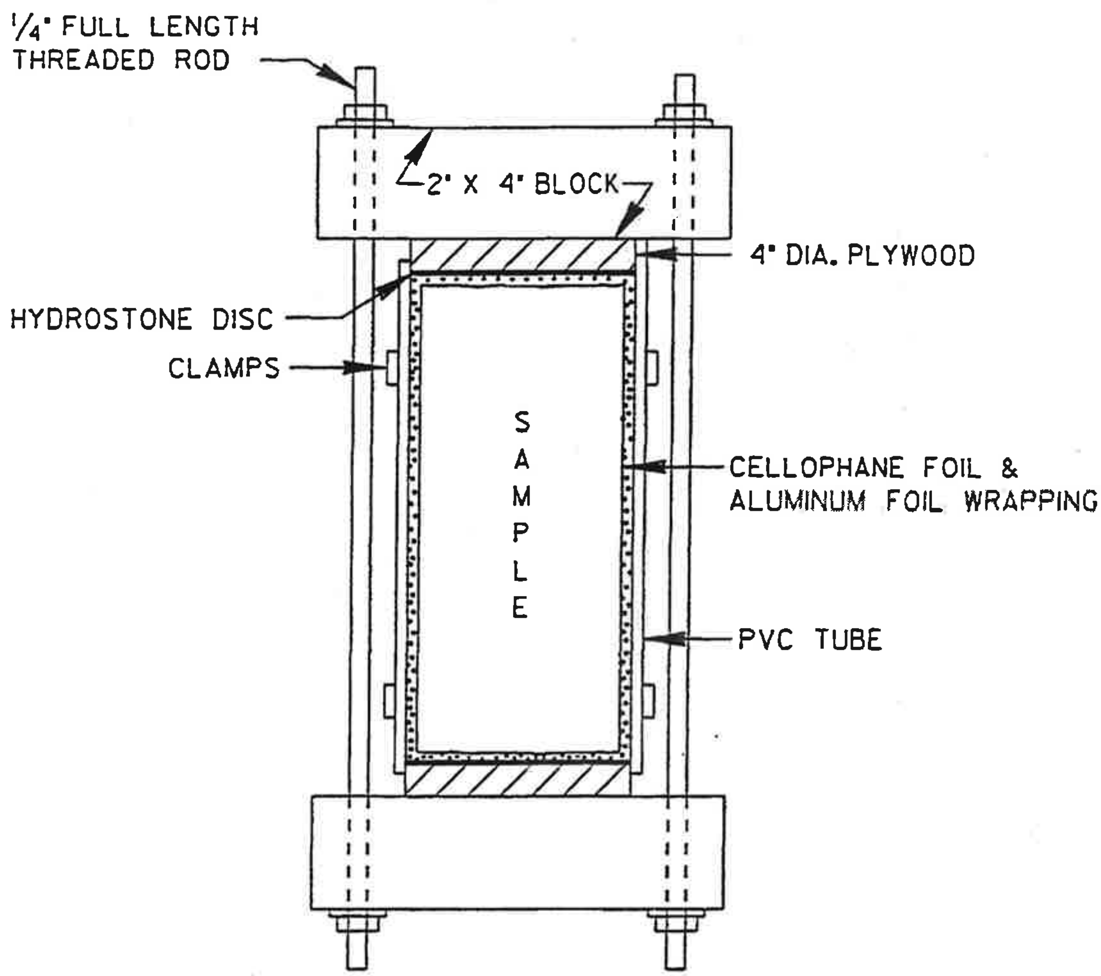

Figure 4. Sample container section view (after Preber, 1984).
Application of the Point Load Test to Weak Rock

Problems Associated with Direct Shear Testing Within the Corps of Engineers

The direct shear test is the laboratory procedure most commonly utilized within the U. S. Army Corps of Engineers (USACE) to determine the shear strength of rock. Shear strength parameters applied in the design and stability analyses of most USACE structures founded within or on top of rock are obtained from direct shear tests on small diameter rock core coupled with empirical application of index tests, and intuition based on engineering judgment. Very commonly, the geological designer thinks in terms of "selecting" design shear strengths rather than precisely determining them from either laboratory or in-situ testing. The selection process relies heavily on an understanding of the many factors influencing the strength of a rock mass as well as engineering judgment tempered with experience (Nicholson, 1994). Although the selection process involves the judgment and experience of the designer, the process is primarily based on test results from laboratory direct shear tests. The tests are performed on rock core specimens carefully chosen to reflect the upper and lower bounds of the actual in situ strengths deemed to be representative for the likely failure plane or mode of failure. The available shear strength of a rock mass lies between the peak and residual strength of all its component parts along some failure surface throughout that rock mass (Simmons and Swartz, 1988). Direct shear testing is normally performed on carefully selected samples of intact rock as well as on representative samples of discontinuities (bedding planes, joints, clay seams, etc.).

In the course of testing samples of similar rock materials at different USACE laboratories, geological designers have observed seemingly large variations in direct shear test results. These large variations were observed in the routine testing of all types of rock, both competent and weak. These variations were reported within the Corps geotechnical community and resulted in a study that was performed by the agency's Waterways Experiment Station (WES). WES undertook a comparative testing program to investigate the cause or causes of variation in direct shear test results. A uniform rock material, the Berea Sandstone, was selected for the testing program. Cylindrical specimens consisted of two sizes, 10 cm and 15 cm nominal diameters. For both sample sizes, three types of standard surfaces were prepared for shear testing. These were a smooth pre-sawn and lapped surface cut parallel to the direction of shear stress (horizontal sliding); a smooth pre-sawn surface inclined upward 4.0 degrees with respect to the direction of shear stress; and a similar surface inclined 4.0 degrees downward. All the tests were performed in strict accordance with U.S. Army Corps of Engineers standards as reported in the Rock Testing Handbook (USACE, 1990). The model specimens were sent to five USACE laboratories for comparative testing.

The study indicated that direct shear test results from the five USACE laboratories varied widely. Figure 5 presents a plot of the comparative direct shear test results on the uniform Berea Sandstone. For smooth shear surfaces, theoretically there should be no variation between test results between 10 cm and 15 cm diameter samples. However, the testing program indicated that horizontal sliding friction values for 15 cm diameter samples were, on an average, 4.5 degrees higher than values for 10 cm diameter samples. Further, a comparison of all results from the labs indicated that friction values for horizontal and downslope sliding varied by 12.5 degrees; whereas, upslope sliding varied by 10.5 degrees.

In his evaluation of results, Nicholson (1994) states that one of the most important provisions of any direct shear device used to test natural discontinuities is the ability to maintain a constant normal load. Most normal load systems apply the load by means of a hydraulic jack. The jack must be able to track with the specimen as it dilates or compresses. The system must overcome a certain amount of inertia which in most cases requires an almost constant adjustment of hydraulic pressure applied. This adjustment, particularly applicable to upslope and downslope sliding, leads to the potential for variability in the results. The results of Nicholson's study led to the standardization of the direct shear device, which incorporates a gas over water diaphragm that supplies pressure to the hydraulic jack. This provides for a soft normal loading system that is able to maintain constant normal load while the sample is being sheared.

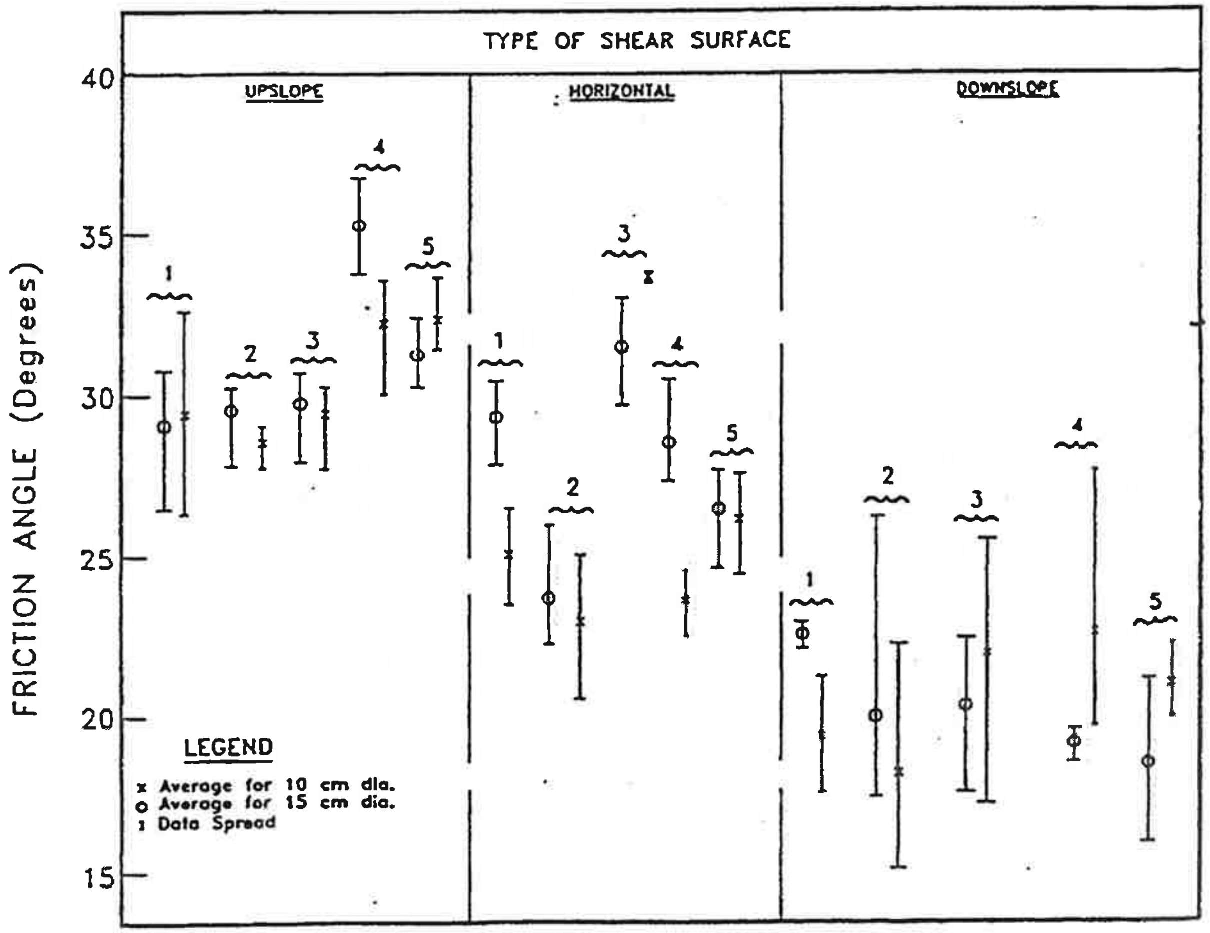

Figure 5. Plot of the distribution of direct shear test data by type of shear surface and by laboratory, numbered 1 through 5 (after Nicholson, 1994).

RELATIONSHIP BETWEEN MOISTURE CONTENT AND STRENGTH

It has long been known that there is an inversely proportional relationship between a weak sedimentary rock's moisture content and its UCS. Bauer (1984) reports a logarithmic relationship between moisture content and the UCS. The results of testing by the Pittsburgh District have revealed that different types of curves such as a power or exponential curve often give a better fit to the data. Figure 6 illustrates that a negative exponential relationship yielded the best fit to four siltshale units from two sites on the Monongahela River, near Pittsburgh, PA. Although this data set has a correlation coefficient (r) of 0.71, higher degrees of correlation have been obtained when the data set has been restricted to one rock unit. It has also been noticed that an inversely proportional relationship exists between a sample's moisture content and its bulk unit weight; hence a proportional relationship between UCS and unit weight exists. It is therefore recommended that the bulk unit weight be determined for each UCS sample prior to testing.

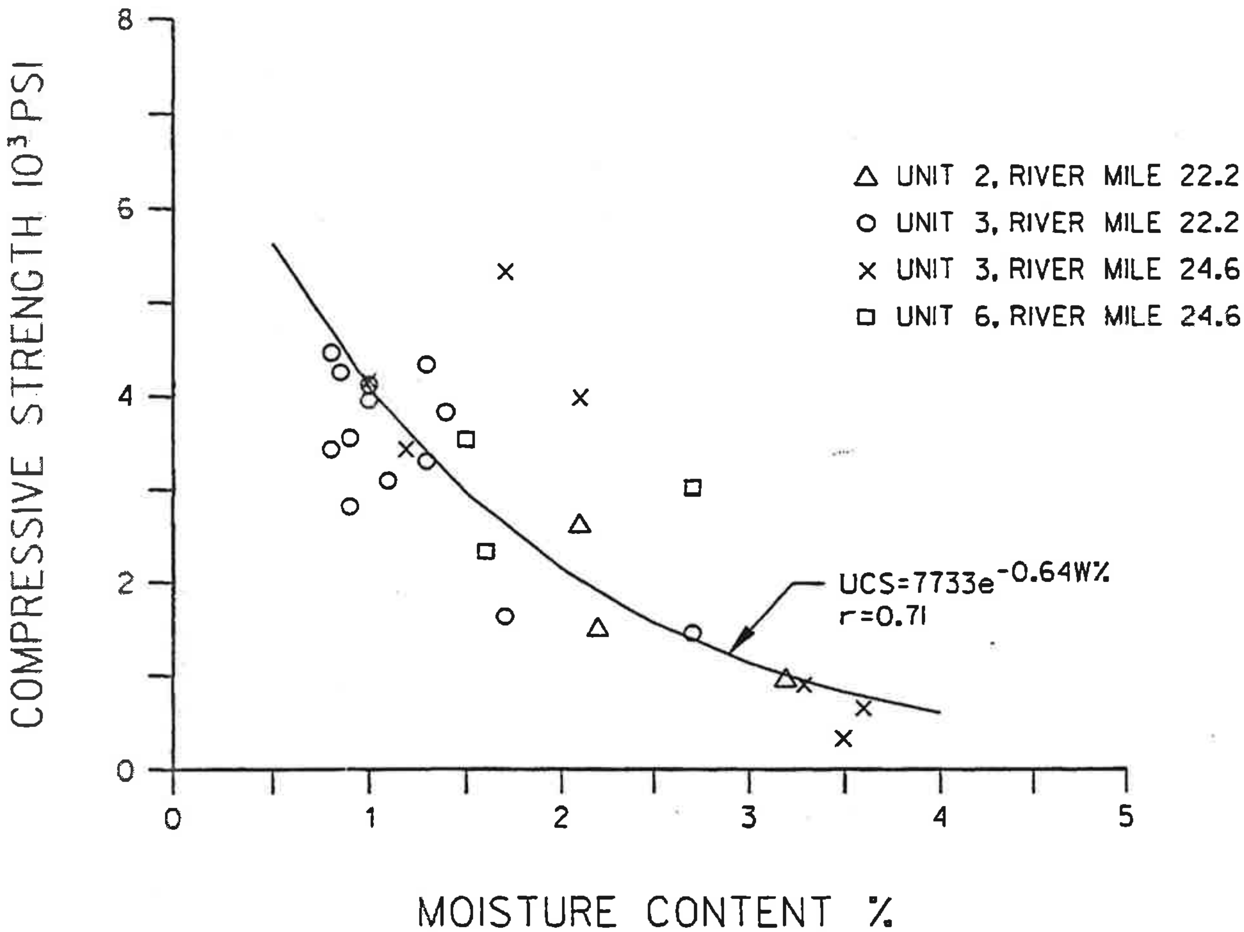

Figure 6. Moisture content versus compressive strength for Monongahela River siltshales.

CONCLUSIONS

Laboratory test methods that have been most commonly used in measuring the strength of weak rocks are the direct shear, unconfined compression and the point load tests. It has been established that there is a close correlation between point load index and unconfined compressive strength. The point load test has been used both in the field and in the lab, but has certain limitations in its use with soft weak rocks. Whereas the point load test was originally developed for diametral testing of strong isotropic rock, studies by Bauer (1984) and other researchers have shown that this type of test procedure cannot be used to accurately estimate unconfined compressive strength of weak rock. Instead it is recommended that, when applying the point load test to shales and other weak sedimentary rock possessing strong bedding anisotropy, the test be run axially.

Shear testing of rock is commonly determined by either the direct shear test or the triaxial compression test. Either test procedure can be conducted on intact rock core samples,or oriented tests can be run on samples with discontinuities. Problems with direct shear testing have been experienced within the U.S. Army Corps of Engineers laboratory facilities. In studies conducted by the agency (Nicholson, 1994), unacceptably high degree of variability of direct shear test results on similar rock samples has been reported. Primarily due to differences in test apparatus, the results of direct shear testing of uniform rock at five agency labs indicated a maximum variation of 12.5 degrees in friction angle. In taking necessary corrective measures, the agency has standardized the direct shear device used for all testing and has restricted testing to two of its several laboratories.

Weak rock materials are notorious for problems due to poor sample preservation. Moisture loss associated with the drying of clay rich rock samples results in slaking of the material and poor survivability of the samples prior to delivery to the lab. Special care must be taken to prevent moisture loss and in some cases provide axial and radial confinement. It is clear from studies presented in this paper, that there is a strong correlation between moisture content and unconfined compressive strength for mudrocks.

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Predicting the Durability of Mudrocks from Geological Characteristics

JEFFREY C. DICK
Department of Geology, Youngstown State University, Youngstown, OH 44555

ABDUL SHAKOOR
Department of Geology, Kent State University, Kent, OH 44242

ABSTRACT

Mudrock is the collective term used for fine-grained, argillaceous sedimentary rocks (claystones, shales, mudstones, and siltstones). They constitute the majority of rocks classified as soft or weak, and are widely distributed throughout the world. These rocks present many problems in engineering construction, most of which can be attributed to their low durability (disintegration upon changes in water content). The engineering problems associated with mudrocks are further exacerbated by the extreme variation in their geological characteristics, which influence durability behavior. It is important to recognize the significance of geological characteristics and their variability in order to understand the durability behavior of mudrocks. This paper presents the results of a comprehensive study conducted at Kent State University aimed at investigating the role of geological characteristics in predicting the durability behavior of mudrocks.

The relationships between second-cycle slake durability index, geological characteristics (clay content, clay mineral composition, texture, and micro-fracture frequency), and various index properties related to geological characteristics (absorption, adsorption, void ratio, dry density, and plasticity characteristics) were investigated for 49 mudrock samples from the United States and portions of Canada. The mudrocks were categorized as claystones, shales, and mudstones on the basis of clay-size material (< 0.004 mm) and the presence or absence of laminations. A series of statistical analyses were performed to quantify the relationships between durability and certain geological characteristics (including geologically controlled index properties), and to develop methods of durability prediction.

The results of the study show that the durability of claystones, shales, and mudstones correlates with specific geological characteristics. Overall, the durability of claystones is low and the percentage of expandable clay minerals is the primary geological characteristic controlling durability. Shales and mudstones both exhibit wide ranges of durability. The durability of shales best correlates with the state of consolidation and presence of expandable clay minerals as represented by absorption, whereas the durability of mudstones is closely related to the frequency of micro-fractures. Based on these relationships between geological characteristics and durability, the mudrocks were categorized into groups of low durability, medium durability, and high durability. The three classes of durability were used to evaluate the potential for instability problems that affect mudrock slopes.

INTRODUCTION

Mudrocks are the most common type of sedimentary rock. Representing nearly two-thirds of the stratigraphic column (Blatt, 1982) and exposed over one-third of the land area (Franklin, 1983), mudrocks are frequently encountered in all types of engineering construction. The combination of their high clay mineral content and poor state of induration causes many mudrocks to physically deteriorate in response to changes in moisture content and release of overburden stress. This nondurable behavior of mudrocks has resulted in countless slope instability problems and has earned mudrocks the reputation as a problematic material. In many instances this reputation is well deserved; however, many mudrocks are quite durable.

The combined works of a large number of researchers over the past 25 years demonstrates that mudrock durability is controlled by variations of clay content, proportion of expandable clay minerals, rock fabric, sedimentary structures (laminae, slickensides, and micro-fractures), and cementation. Spears and Taylor (1972) found that the ratio of quartz to clay minerals has an important bearing on the residual strength of weathered mudrocks. In studying the Queenston Formation and the Georgian Bay Formation of Ontario, Canada, Russell (1981) discovered that slake durability index values were influenced by clay mineral content, calcite content, and the presence of microcracks. Smart and others (1982) found significant relationships between the quartz content and the cohesive and uniaxial compressive strengths, as well as the swelling characteristics of British Coal Measures mudrocks. Grainger's work (1983) on the mudrocks of the Crackington Formation of southwest England demonstrated that composition and fabric anisotropy influence durability. Steward and Cripps (1983) concluded that the residual strength of pyritic shale is sensitive to alterations of pore water chemistry and mineralogical changes that accompany weathering. In studying the relationships between fissility, composition, and engineering properties of shales from northeastern Ohio, USA, Shakoor and Brock (1987) concluded that durability is influenced by the degree of lithification. Huppert (1988) studied the influence of microfabric on the geomechanical behavior of mudrocks from Central North Island, New Zealand. Taylor (1988) concluded that the weathering of British Coal Measures mudrocks is governed by the presence of sedimentary structures such as laminae, slickensides, stress fractures, and the amount of expandable mixed-layer clay minerals. In a comprehensive approach to establishing relationships between geological properties and mudrock durability, Dick (1992) and Dick and Shakoor (1992) found the amount of expandable clay minerals, degree of induration, and the frequency of micro-fractures to be the primary factors controlling mudrock durability. Finally, Moon and Beattie (1995) found that in kaolinite rich mudrocks of the Waikato Coal Measures, New Zealand, the amount of clay-sized material and microstuctural features control durability.

A natural outcome of mudrock durability research is the development of mudrock durability classification systems used to better understand durability behavior. Gamble (1971) developed a durability-plasticity classification in which six classes of durability were recognized on the basis of slake durability index (Id) test and plasticity index values. Morgenstern and Eigenbrod (1974) devised a durability classification based on liquid limit and rate of slaking values. Wood and Deo (1975) proposed a system for classifying the durability of certain Indiana shales for use in compacted highway embankments based on the simple slake test, the slake durability index test, and the modified soundness test. Finally, Taylor (1988) developed a mudrock durability classification for British Coal Measures mudrocks based on composition, fabric, unconfined compressive strength, and third-cycle slake-durability index (Id_3) values. These durability classification systems have limited application primarily because they do not adequately represent the geological diversity of mudrocks.

The underlying premise of the research presented in this paper is that durability behavior of mudrocks is a product of their geological makeup and in order for a durability classification system to be broadly applicable, it must take into account the extreme variation of geological characteristics of mudrocks. The objective of the research was to establish relationships between the geological characteristics and durability behavior of a large variety of mudrocks and use these relationships as the basis for a mudrock durability classification system.

METHODS

Geological Classification of Mudrocks

The first step in developing a comprehensive mudrock durability classification system is to classify the rocks according to geological characteristics that distinguish them from both geological and engineering behavior perspectives. A modified version of Blatt's classification (1982) accomplishes this distinction and is presented in Table 1. This classification recognizes six classes of mudrocks based on the percentage of clay-sized particles and the presence or absence of a laminated sedimentary structure. The textural division between mudstones and claystones was placed at 50 percent clay (< 0.004 mm diameter) rather than 66 percent clay (as suggested by Blatt) to reflect a marked change in the durability behavior of the mudrocks at this clay content.

Table 1. Geological classification of mudrocks (modified from Blatt, 1982).

	PERCENT CLAY-SIZED PARTICLES		
	0 - 32%	33 - 49%	50 - 100%
NONLAMINATED	SILTSTONE	MUDSTONE	CLAYSTONE
LAMINATED	SILTSHALE	MUDSHALE	CLAYSHALE

Sampling

The work presented here is based on 49 fresh mudrock samples collected from excavations, highway cuts, surface mines, and natural exposures throughout much of United States and portions of southern Ontario, Canada. The samples were selected on the basis of published information about geology and engineering properties as well as on the basis of durability behavior. A listing of the sampled mudrocks according to formation name, geologic age, location, and geologic classification is presented in Table 2. Ten claystones, seventeen shales and twenty-two mudstones constitute the sample population. The samples range in age from Ordovician to Cretaceous and have widely differing mineral composition.

Table 2. Mudrocks sampled.

FORMATION	GEOLOGIC AGE	LOCATION	GEOLOGIC CLASSIFICATION
Kope Formation	Ordovician	Hamilton Co., Ohio	Claystone
Pierre Shale	Cretaceous	Huerfano Co., Colorado	Claystone
Kirtland Formation	Cretaceous	Garfield Co., Colorado	Claystone
Morrison Formation	Jurassic	Grand Co., Utah	Claystone
Mowry Shale	Cretaceous	Uintah Co., Utah	Claystone
Pierre Shale	Cretaceous	Jefferson Co., Colorado	Claystone
Dakota Group	Cretaceous	Ellsworth Co., Kansas	Claystone
Chanute Formation	Pennsylvanian	Johnson Co., Kansas	Claystone
Conemaugh Group	Pennsylvanian	Allegheny Co., Pennsylvania	Claystone
Georgian Bay Formation	Ordovician	Southern Ontario, Canada	Claystone
Conemaugh Group	Pennsylvanian	Athens Co., Ohio	Clayshale
Chagrin Shale	Devonian	Lake Co., Ohio	Mudshale
Chagrin Shale	Devonian	Lake Co., Ohio	Mudshale
Chagrin Shale	Devonian	Lake Co., Ohio	Mudshale
Chagrin Shale	Devonian	Lake Co., Ohio	Mudshale
Allegheny Group	Pennsylvanian	Lawrence Co., Pennsylvania	Mudshale
Georgian Bay Formation	Ordovician	Southern Ontario, Canada	Mudshale
Benton Formation	Cretaceous	Eagle Co., Colorado	Mudshale
Olentangy Shale	Devonian	Franklin Co., Ohio	Mudshale
Olentangy Shale	Devonian	Franklin Co., Ohio	Mudshale
Conemaugh Group	Pennsylvanian	Athens Co., Ohio	Mudshale
Dunkard Group	Permian	Wood Co., West Virginia	Siltshale
Chagrin Shale	Devonian	Lake Co., Ohio	Siltshale
Allegheny Group	Pennsylvanian	Lawrence Co., Pennsylvania	Siltshale
Allegheny Group	Pennsylvanian	Lawrence Co., Pennsylvania	Siltshale
Allegheny Group	Pennsylvanian	Lawrence Co., Pennsylvania	Siltshale
Conemaugh Group	Pennsylvanian	Noble Co., Ohio	Siltshale
Fairview Formation	Ordovician	Kenton Co., Kentucky	Mudstone
Fairview Formation	Ordovician	Kenton Co., Kentucky	Mudstone
Kope Formation	Ordovician	Hamilton Co., Ohio	Mudstone
Conemaugh Group	Pennsylvanian	Noble Co., Ohio	Mudstone
Monongahela Group	Pennsylvanian	Galia Co., Ohio	Mudstone
Conemaugh Group	Pennsylvanian	Athens Co., Ohio	Mudstone
Conemaugh Group	Pennsylvanian	Allegheny Co., Pennsylvania	Mudstone
Dunkard Group	Permian	Wood Co., West Virginia	Mudstone
Dunkard Group	Permian	Wood Co., West Virginia	Mudstone
Queenston Shale	Ordovician	Southern Ontario, Canada	Mudstone
Queenston Shale	Ordovician	Southern Ontario, Canada	Mudstone
Mancos Shale	Cretaceous	Garfield Co., Colorado	Mudstone
Mancos Shale	Cretaceous	Garfield Co., Colorado	Mudstone
Lewis Formation	Cretaceous	Grand Co., Utah	Mudstone
Dakota Group	Cretaceous	Ellsworth Co., Kansas	Mudstone
Morrison Formation	Cretaceous	Cimarron Co., Oklahoma	Mudstone
Bedford Shale	Mississippian	Cuyahoga Co., Ohio	Mudstone
Queenston Shale	Ordovician	Southern Ontario, Canada	Mudstone
Dunkard Group	Permian	Wood Co., West Virginia	Mudstone
Conemaugh Group	Pennsylvanian	Athens Co., Ohio	Mudstone
Conemaugh Group	Pennsylvanian	Noble Co., Ohio	Mudstone

Laboratory Investigations

Laboratory investigations consisted of determinations of durability, geological characteristics, and index properties controlled by geological characteristics. The geological characteristics of interest are the degree of induration, grain size distribution, clay mineral composition, and micro-fractures. Mudrock durability was characterized using the second-cycle slake-durability index test (Id_2) according to ASTM method D4644 (ASTM, 1990). A minimum of two replicate tests were performed on each mudrock to obtain average Id_2 values.

Mudrocks are indurated by consolidation and cementation which results in reduction of void space. Since the dominant mineral constituents of mudrocks have similar specific gravities, void ratio and dry density can be considered valid measures of induration (Hudec, 1982). Void ratio was calculated from the measured values of dry density, water content, and specific gravity of solids. Absorption, which is dependent on the volume of voids as well as the presence of expandable clay minerals was also considered a useful measure of induration. Water content was determined according to ASTM method D2216 (ASTM, 1990). Bulk specific gravity of solids was determined using disaggregated samples according to ASTM method D854 (ASTM, 1990). Absorption and dry density were determined following a modified version of ASTM method C97 (Dick and others, 1994). Absorption and void ratio tests were performed on at least three replicate samples of each mudrock to obtain average values.

Grain size distribution analysis was performed for the purposes of geologically classifying the mudrocks, preparing the samples for X-ray analysis, calculating clay mineral content, and investigating relationships between grain size and durability. Grain size distribution analysis of disaggregated mudrock samples was performed according to ASTM method D422 (ASTM, 1990). Approximately 50 g of sample was disaggregated by alternate cycles of wetting and drying accompanied by gentle agitation (Dick and others, 1994). Clay mineralogical analyses were performed using standard X-ray diffraction methods on oriented ceramic tile mounts prepared from 0.002 mm and smaller clay fractions. The percentage of individual clay minerals was estimated using the method described by Schultz (1964). The estimated percentage of each clay mineral was multiplied by the weight percent of 0.002 mm particles to determine the weight percent of each clay mineral with respect to the whole rock. Clay minerals of particular interest are the expandable varieties, namely, montmorillonite and mixed layer illite-smectite. The weight percent of each of these minerals was summed to represent percent expandable clay.

The use of Atterberg limits (liquid limit and plastic limit) and water adsorption tests as indicators of expandable clay mineral content was investigated. Liquid limit and plastic limit were determined according to ASTM methods D423 and D424, respectively (ASTM, 1990). The Atterberg limits are sensitive to clay mineral content and the proportion of clay sized particles. The tests were performed on the particle fraction finer than 0.425 mm (#40 sieve). Adsorption is sensitive to the relative quantity of clay minerals present in mudrocks as well as the relative proportion of expandable clay minerals. Adsorption was measured on samples exposed to 95 percent relative humidity for 96 hours in a humidity and temperature controlled chamber (Dick and others, 1994).

Micro-fractures are a common feature of mudstones and claystones and have a demonstrated influence on durability (Olivier 1980; Russell 1981; Grainger 1983; Dick and

Shakoor 1992). A method of quantifying the degree of micro-fracturing, termed the "micro-fracture frequency index" (Imf) was devised by Dick and Shakoor (1992). The method consists of dry-cutting mudrock samples into roughly orthogonal shapes and counting the number of micro-fractures along a series of randomly drawn linear traverses. The micro-fracture frequency index is calculated by dividing the number of micro-fractures along each traverse by the traverse length in centimeters. The micro-fracture frequency index is easily determined for most mudstones and claystones. Shales also possess micro-fractures, however, their laminated structure overshadows the effects of micro-fractures and renders the measurement of micro-fracture frequency practically impossible (Dick and Shakoor, 1992).

Data Analysis

The geological characteristics and index properties of the mudrock samples were correlated with the corresponding values of the second-cycle slake-durability index (Id_2) using bivariate and multiple regression analyses. The regression analyses were performed with the objective of establishing relationships between geological characteristics and durability that could, in turn, provide the framework for a durability classification system. As an initial step in the regression analysis, the data were statistically analyzed to evaluate normality and covariance of the data sets.

RESULTS

Durability

The results of durability testing for the claystones, shales, and mudstones are presented in Figures 1, 2, and 3, respectively. The durability of the claystones is very low having a maximum Id_2 of 50 percent and a mean Id_2 of only 16.9 percent. The shales are generally the most durable of the three lithologies, having a mean Id_2 of 77.4 percent. The durability of the mudstones is quite variable with Id_2 values ranging from 3 to 93 percent.

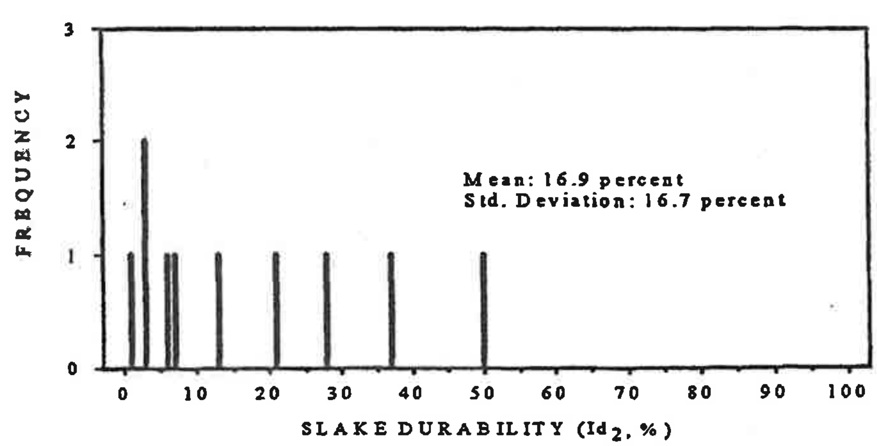

Figure 1. Frequency histogram of slake durability for claystones.

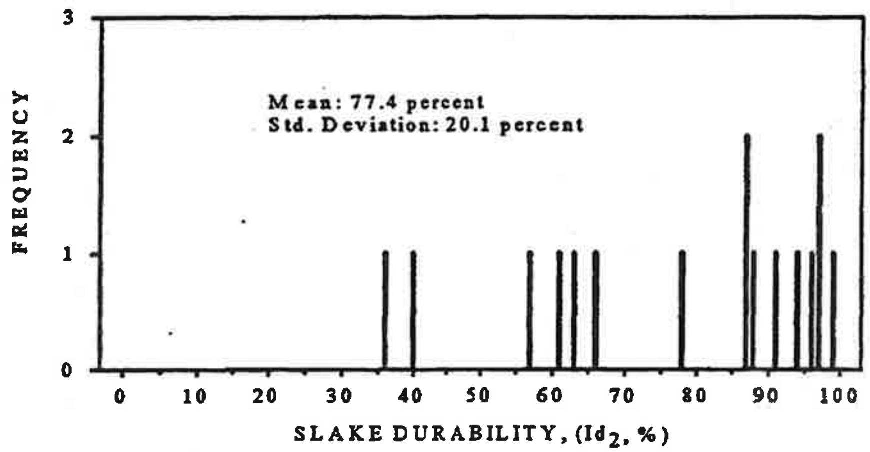

Figure 2. Frequency histogram of slake durability for shales.

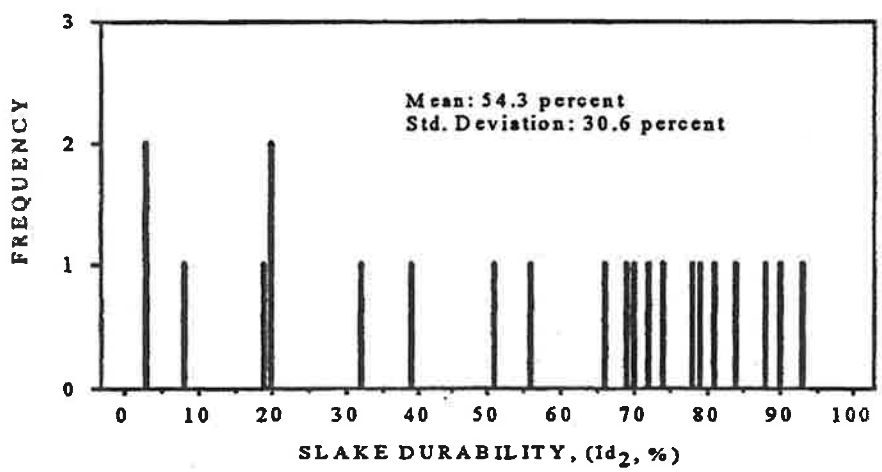

Figure 3. Frequency histogram of slake durability for mudstones.

Geological Characteristics and Index Properties

The results of the geologic analyses and index property tests are summarized in Table 3. The claystones are distinguished by their high mean values of expandable clay mineral content, plasticity index, adsorption, absorption, and void ratio. The strong affinity for water resulting from the high expandable clay content is reflected in the comparatively high mean values of plasticity index and adsorption. The poor state of induration of the claystones is reflected in the comparatively high mean void ratio and low dry density.

Shales are distinguished from other mudrocks by their laminated structure. Laminations are largely the result of parallel to subparallel alignment of mineral grains. This ordered texture is reflected in the comparatively low mean values of absorption and void ratio and the high mean value of dry density. The low absorption values may also be influenced by the relatively low expandable clay mineral content of the shales (mean = 4.8 percent).

Mudstones are intermediate between shales and claystones in all the properties measured except plasticity index. The void ratio, and dry density values suggest a higher state of induration than the claystones, but a less ordered structure than the shales. These results are supported by SEM analysis (Dick, 1992) that indicates a tighter packing of mineral grains for mudstones than claystones. SEM analysis also shows the relatively coarse-grained mudstones having a degree of

Table 3. Results of geologic analyses and index property tests.

PROPERTY	N	MEAN	STD. DEVIATION	RANGE
CLAYSTONES				
Void Ratio	10	0.183	0.054	0.10 - 0.26
Dry Density (g/cc)	10	2.25	0.154	2.03 - 2.48
Percent Absorption	10	35.41	41.22	10.62 - 142
Percent Clay (<0.004 mm)	10	66.5	14.8	51 - 91
Percent Expandable Clay	10	28.4	25.5	2 - 85
Percent Adsorption	10	8.60	4.89	3.60 - 20.78
Plasticity Index	10	29.4	18.6	11 - 72
Micro-Fracture Frequency (#/cm)	6	1.22	0.480	0.47 - 1.85
SHALES				
Void Ratio	17	0.081	0.035	0.04 - 0.16
Dry Density (g/cc)	17	2.52	0.121	2.23 - 2.62
Percent Absorption	17	6.01	2.25	3.64 - 11.0
Percent Clay (<0.004 mm)	17	35.5	8.4	20 - 52
Percent Expandable Clay	17	4.8	5.1	1 - 24
Percent Adsorption	17	3.16	1.29	1.99 - 6.23
Plasticity Index	17	11.0	3.2	6 - 16
MUDSTONES				
Void Ratio	22	0.12	0.04	0.05 - 0.23
Dry Density (g/cc)	22	2.42	0.135	2.06 - 2.60
Percent Absorption	20	9.99	3.98	4.0 - 20.8
Percent Clay (<0.004 mm)	22	40.6	6.4	33 - 49
Percent Expandable Clay	22	7.1	5.3	3 - 19
Percent Adsorption	22	3.72	0.72	1.82 - 5.21
Plasticity Index	22	10.8	2.9	7 - 17
Micro-Fracture Frequency (#/cm)	18	0.863	0.560	0.40 - 2.25

cementation comparable to that of siltshales. Plasticity index and adsorption for the mudstones is comparable to that of the shales reflecting similar clay-size particle content and clay mineralogy. Compared to the claystones, the micro-fracture frequency index of the mudstones is low. Furthermore, a significant number of the mudstone micro-fractures occur as slickensides (Dick and Shakoor, 1992).

RELATIONSHIPS BETWEEN DURABILITY AND GEOLOGICAL CHARACTERISTICS

In order to identify geologic variables or combinations of variables that could best predict mudrock durability, discriminant analyses were performed on the data from all the mudrock samples combined. The results showed that higher durability was associated with a higher degree of induration as indicated by high dry density and low void ratio values and that low durability was related to the amount of expandable clay minerals as indicated by high adsorption values (Dick, 1992). Although these results were too general to be useful in predicting durability, they did support the findings of Spears and Taylor (1972), Russell (1981), Shakoor and Brock (1987), Taylor (1988), Dick (1992), Dick and Shakoor (1992), and Moon and Beattie (1995) that durability of mudrocks is controlled, in part, by degree of induration and clay mineral content.

The geological characteristics of the individual mudrock classes were correlated with second-cycle slake-durability index values using bivariate and multiple regression analyses. The bivariate regression analyses yielded several statistically significant relationships ($r \geq |0.80|$) between Id_2 and geological characteristics. Combinations of variables in multiple regression analysis failed to significantly improve the bivariate analysis results. The correlation coefficients for the bivariate regressions are shown in Table 4.

Table 4. Correlation coefficients for regression of slake durability index and geological characteristics and index properties.

	Correlation Coefficient (r)		
	Claystones	Shales	Mudstones
Void Ratio	-0.47	-0.76	0.21
Dry Density (g/cc)	0.68	0.57	-0.16
Percent Absorption	-0.53	-0.93*	-0.06
Percent Clay (<0.004 mm)	-0.68	-0.44	-0.18
Percent Expandable Clay (log)	-0.98*	-0.61	0.00
Percent Adsorption	-0.83*	-0.73	-0.06
Plasticity Index	-0.69	-0.55	0.00
Micro-Fracture Frequency (#/cm) (log)	NA	NA	-0.92*

* statistically significant ($r \geq |0.80|$)

The dominant geological characteristic of claystones is their high proportion of clay. The predominance of clay has a pronounced effect on durability as indicated by the strong correlation ($r = -0.98$) between Id_2 and the log of the percent expandable clay. A plot of the relationship between Id_2 and log of percent expandable clay is shown in Figure 4. The equation for the regression line is:

$$Id_2 = 57.3 - 32.3 \log (C) \quad (1)$$

where C is the percentage of expandable clay minerals.

The strength of this correlation suggests that the percent expandable clay mineral content is the primary geological factor controlling claystone durability and that it can be used as a predictor of durability for claystones. Residuals for the regression are shown in Figure 5. The maximum difference between the observed and predicted values of Id_2 is 7, which is an acceptable error in many situations.

Plasticity index and percent adsorption, which are strongly influenced by expandable clays, also correlate with Id_2 (Table 4). Although the correlations (Id_2 versus plasticity index: $r = -0.69$ and Id_2 versus percent adsorption: $r = -0.83$) are not sufficiently strong to serve as predictors of durability, they do support the general relationship between durability and percent expandable clay.

The only geological characteristic or geologically controlled index property of shales that exhibits a strong correlation with Id_2 (Table 4) is percent absorption, having a r value of -0.93. Absorption is dependent on both the state of induration and the amount of expandable clay minerals. Separately, void ratio (indicator of induration) and percent adsorption (indicator of expandable clay) show comparatively weaker correlations with Id_2 ($r = -0.76$ and $r = -0.73$, respectively) for shales. The combined effect of void ratio and adsorption on durability was investigated using multiple regression analysis. Due to a high degree of covariance between variables, the multiple regression analysis did not result in any improvement over the bivariate relationship between percent absorption and Id_2 for shales. These results suggest that the durability of shales is dependent on the state of induration and the percent expandable clay minerals as indicated by percent absorption, and that percent absorption may be a useful predictor of shale durability.

A plot of the bivariate relationship between Id_2 and absorption for the shales is shown in Figure 6. The equation for the regression line is:

$$Id_2 = 126.0 - 7.5 \times \text{percent absorption} \quad (2)$$

A plot of the residuals for the regression (Figure 7) shows that the difference between the observed and predicted Id_2 values can be quite large, exceeding 9 percent for shales having absorption values above 6 percent. This departure of predicted values from observed values can be attributed to experimental error. The less durable shales tended to part along laminations during absorption testing affecting the measured absorption values.

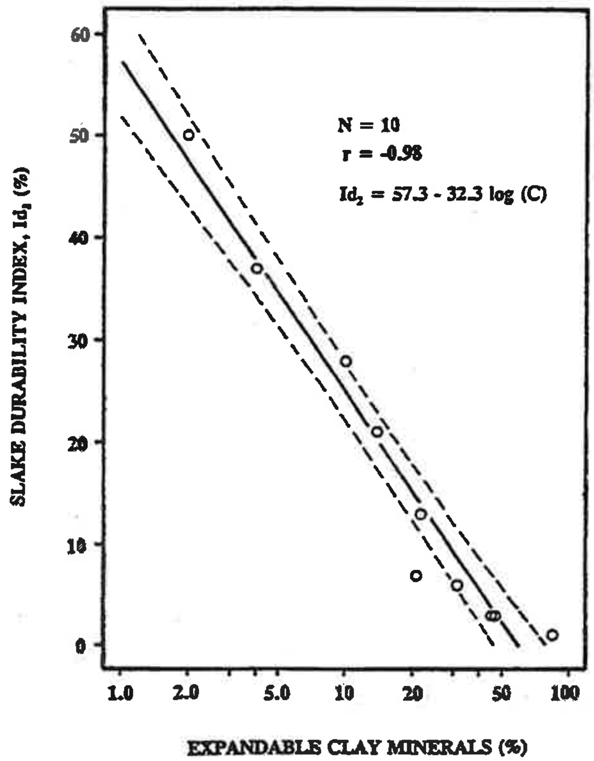

Figure 4. Regression of Id_2 and log percent expandable clay for claystones. Dashed lines represent 95 percent confidence limits.

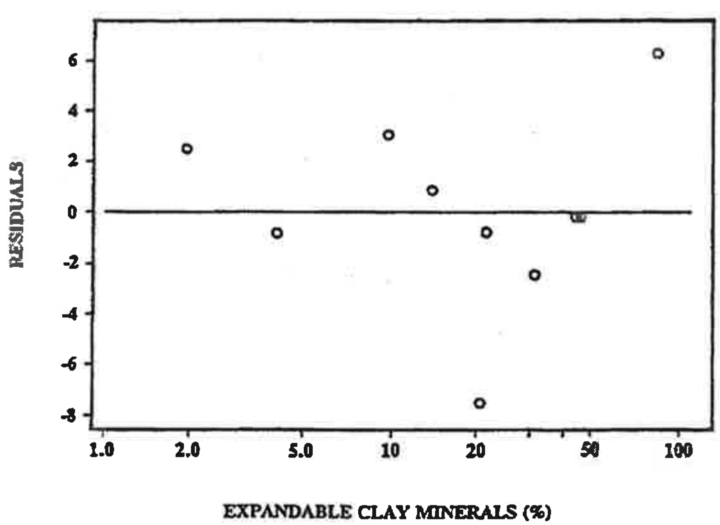

Figure 5. Residuals for the regression of Id_2 and log percent expandable clay minerals.

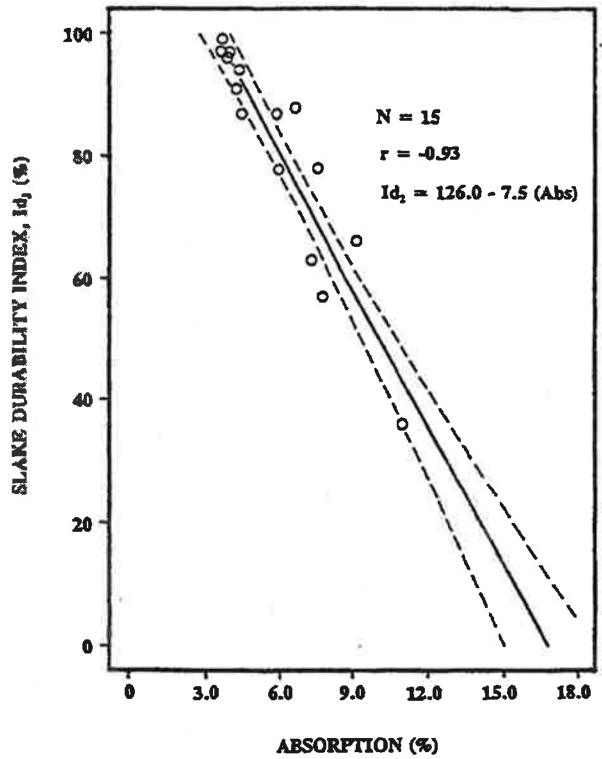

Figure 6. Regression of Id_2 and absorption for the shales. Dashed lines represent 95% confidence limits.

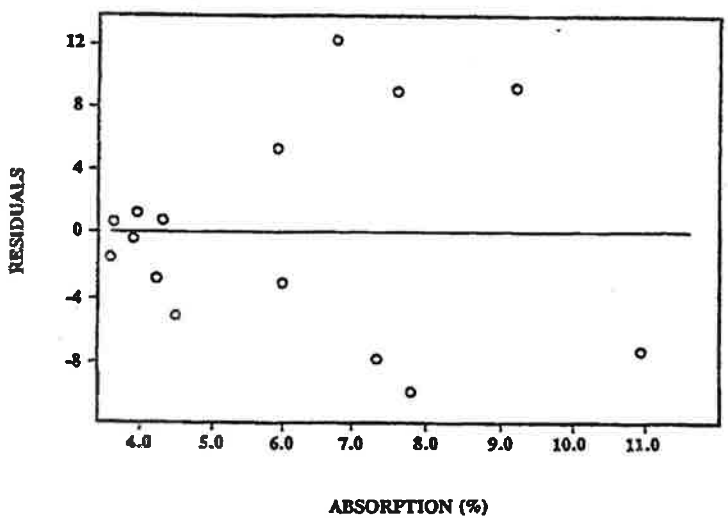

Figure 7. Residuals for the regression between Id_2 and absorption for the shales.

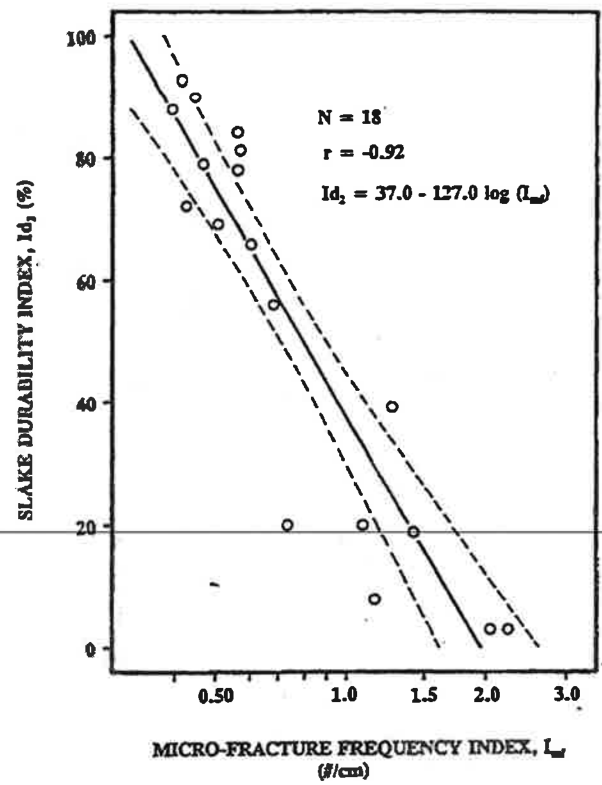

Figure 8. Regression of Id_2 and log (Imf) for mudstones. Dashed lines represent 95% confidence limits.

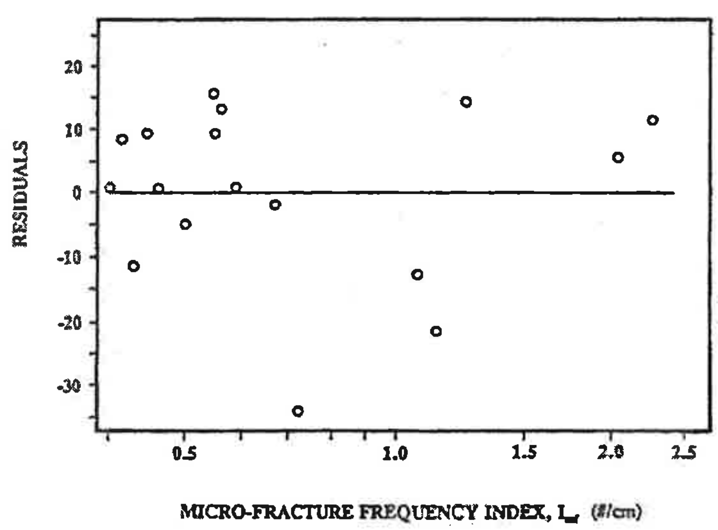

Figure 9. Residuals for the regression of Id_2 and log (Imf) for mudstones.

The durability of mudstones is controlled by the degree of micro-fracturing as suggested by the statistically significant correlation between Id_2 and log of micro-fracture frequency index (Imf), having a r value of -0.92 (Table 4). No other geological characteristics or index properties show any meaningful correlations. The equation for the regression of Id_2 and log Imf is:

$$Id_2 = 37.0 - 127.0 \log (Imf) \qquad (3)$$

A plot of the relationship between Id_2 and Imf is shown in Figure 8. A plot of the residuals (Figure 9) shows a departure of predicted values from observed values in excess of 10 over most of the Imf range. The greatest amount of deviation occurs above Imf values of 0.7. Six of the seven mudstones falling into this range exhibited slickensided micro-fractures and had Id_2 values of 20 percent or less. The presence of slickenside structures corresponds to comparatively high amounts of expandable clays in five of the mudstones (Dick, 1992), suggesting that the presence of expandable clay minerals may be influencing the behavior of the less durable mudstones.

MUDROCK DURABILITY CLASSIFICATION

The relationships between durability and geological characteristics of the claystones, shales, and mudstones were used to develop a classification of mudrock durability (Table 5). Three classes of durability are recognized, namely, *High Durability* mudrocks, *Medium Durability* mudrocks, and *Low Durability* mudrocks.

Table 5. Classification of mudrock durability in relation to geological characteristics.

DURABILITY RATING	Id_2 (percent)	GEOLOGICAL CHARACTERISTIC			
		CLAYSTONE % Expandable Clay	SHALE % Absorption	MUDSTONE Slickensided	MUDSTONE Imf (#/cm)
HIGH	> 85	NA	< 5.5	NA	< 0.4
MEDIUM	50 - 85	NA	10.0 - 5.5	NA	0.8 - 0.4
LOW	< 50	ALL	> 10.0	ALL	> 0.8

As can be seen from Table 5, the durability of claystones is based on the weight percentage of expandable clay minerals. The only applicable class of durability for claystones is *Low* since all the claystones in this study had Id_2 values less than 50 percent. The durability of shales ranges from *High* to *Low*. Shale durability is based on absorption which is an indicator of state of induration and the amount of expandable clay minerals. *High Durability* shales are predominantly siltshales and tend to be well cemented. The durability ranking of the shales tends to decrease with increasing clay content and as such, the mudshales tend to have *Medium Durability* to *Low Durability*. The durability of the mudstones ranges from *High* to *Low* and is based on the frequency of micro-fractures. The presence of slickensides in the mudstones relates to durability. The mudstones that exhibited slickenside structures had Id_2 values of 20 percent or less. Hence, slickensided mudstones are classified as *Low Durability*.

This durability classification system is directly applicable to slope stability problems. All of the mudrock specimens were collected from natural and man-made slopes. As part of the sampling procedure, a general assessment of slope stability, rock mass properties, and field durability performance was made at each site. The durability classification system based on laboratory analyses (Table 5) was combined with observed field behavior for each of the sampled formations to develop a scheme for evaluating potential slope stability problems (Dick and Shakoor, 1995). The intent is not to provide mudrock slope design criteria, but to alert engineers and geologists to potential durability-related instability problems.

A tentative scheme for evaluating mudrock slope instability based on durability is presented in Table 6. The three classes of durability (high, medium, and low) are compared to the likelihood (unlikely, potential, and probable) of each of four common types of mudrock slope instability: excessive erosion, slump failures, debris flows, and undercutting-induced failures. All mudrocks are susceptible to undercutting. The projected rates of undercutting , as listed in Table 6, are based on the findings of Shakoor and Rodgers (1992). High-durability mudrocks are unlikely to result in excessive erosion, slumps, or debris flows. It is unlikely that medium-durability mudrocks will produce excessive erosion problems; however, there is potential for slumping and debris flow instability. Low-durability mudrocks are prone to all four modes of slope instability.

Table 6. Application of proposed durability classification to evaluation of slope instability.

DURABILITY	TYPE OF SLOPE INSTABILITY			
	EXCESSIVE EROSION	SLUMP	DEBRIS FLOW	UNDER-CUTTING (cm/yr.)
HIGH	Unlikely	Unlikely	Unlikely	2 - 3
MEDIUM	Unlikely	Potential	Potential	3 - 5
LOW	Probable	Probable	Probable	5 - 10

CONCLUSIONS

The key to understanding mudrock durability is understanding the geological characteristics of mudrocks. The detailed study of the geological characteristics and durability behavior of 49 mudrocks from North America shows that durability and geological characteristics of the mudrock group as a whole are highly variable and that no single geologic characteristic controls durability. Geologically classifying the mudrocks as claystones, shales, and mudstones, however, effectively categorizes the geological variation and allows for the establishment of quantitative relationships between durability and different geological characteristics for each type of mudrock.

The durability of claystones is closely related to the quantity of expandable clay minerals present in the rock. All the claystones used in this study have low durability ($Id_2 < 50$ percent). Shales exhibit a higher degree of induration than the claystones and mudstones. Consequently, shales tend to be more durable. Absorption, which is influenced by the state of induration and the presence of expandable clay minerals exhibits a strong correlation with the durability of shales. The durability of the shales ranges from low ($Id_2 < 50$ percent) to high ($Id_2 > 85$ percent). The durability of the mudstones is controlled by the amount and type of micro-fractures present in the rock. The durability of the mudstones ranged from low to high, but all the mudstones possessing slickenside structures had low durability.

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Expansive Behavior of Shale Rock Masses: A Case Study

JERRY D. HIGGINS, Ph.D., P.G.
Department of Geology and Geological Engineering, Colorado School of Mines, Golden, CO 80401

ABSTRACT

Deformation of the ground surface associated with expansion of bedrock has caused millions of dollars of damage to buildings and infrastructure along a narrow belt of steeply dipping sedimentary units that parallels the Front Range foothills west of Denver, Colorado. These units are dominated by thinly bedded claystones and shales. Each bed of the rock sequence displays a different potential to expand due to varying clay mineralogy and content. Bedrock units are highly weathered near the ground surface and those containing expansive clays swell in the presence of water, which can result in the formation of heave ridges at the surface that can exceed one foot in vertical displacement.

Until recently, the heaving bedrock problem was not differentiated from common expansive soil problems, and the state of practice for site investigations did not adequately determine where potential problems for differential bedrock heave might occur. Foundation design for expansive soils environments (drilled piers) has performed poorly in heaving bedrock zones.

This paper presents an overview of the geologic setting, damage patterns, and material properties typical of heaving bedrock areas. Models of bedrock deformation based on field observations are also presented.

INTRODUCTION

Heaving bedrock is defined as bedrock containing expansive clay minerals that tend to expand with the addition of moisture and "heave" the ground surface. Heaving bedrock caused ground surface deformation has resulted in millions of dollars of damage each year to buildings, roads, sidewalks, and utilities along a narrow belt of steeply dipping sedimentary units paralleling the Front Range of the Rocky Mountains west of Denver, Colorado. These units are dominated by thinly bedded claystones and shales that are highly weathered near the surface. Each bed of the rock sequence displays various swell potentials due to changes in clay mineralogy and percent clay content, and the steep dip allows several different units to be exposed under the foundations of structures. As a result, differential movement at or near the ground surface creates elongate ridges that may range in width from a few inches up to 15 feet or more and in height to greater than one foot.

Until recently, expansive bedrock has not usually been differentiated from expansive soils by engineers in the Denver area. Heaving bedrock and expansive soils both undergo volume changes due to the influence of water, but heaving bedrock is a more complex geologic hazard. Expansive soils are usually associated with flat-lying deposits that swell and shrink within the active soil-moisture zone, whereas heaving bedrock occurs within steeply dipping units that swell

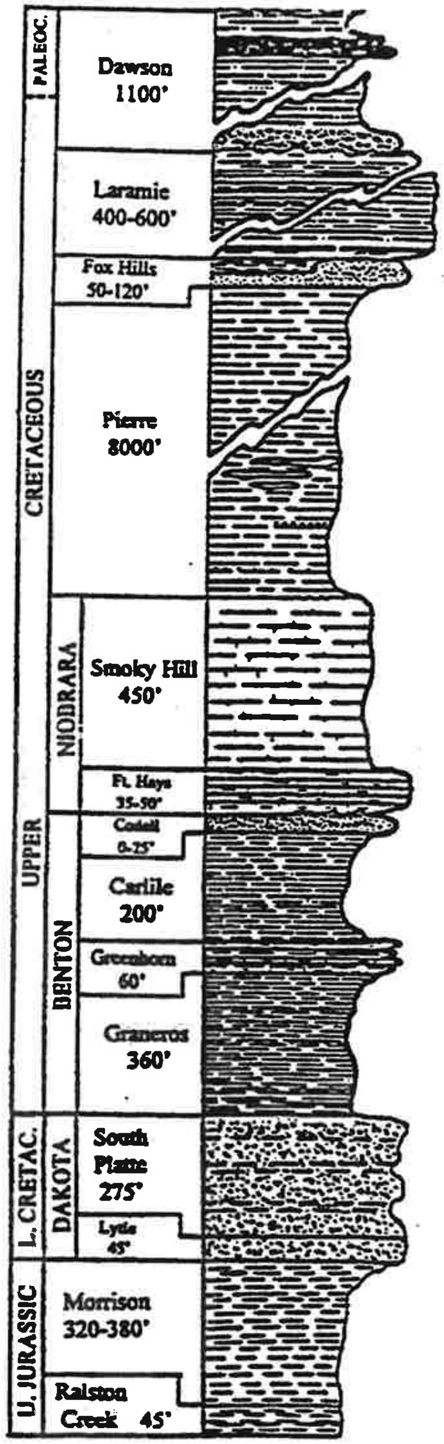

Figure 2. Stratigraphic column of bedrock units in the Denver, Colorado area that may be subject to bedrock heave. (From Dodson, 1996)

formation. Airborne pyroclastics settled into the interior seaway during volcanic eruptions, also. Major eruptions produced discrete layers of volcanic ash with little detrital material, which have subsequently altered to bentonite beds (nearly pure smectite clays). As volcanic eruptions became smaller and/or less frequent, the deposition rate of the air-fall sediment was slow which allowed mixing with detrital quartz. The resulting shales and claystones are bentonitic as opposed to discrete bentonite beds (Schultz et al., 1980 and Schultz, 1978).

DAMAGE FROM BEDROCK HEAVE

Since the Denver area is a well known location for expansive soils, the typical site investigation is designed to detect, sample, and test expansive soils. The typical foundation design for those soils utilizes drilled piers to establish foundation bearing in bedrock or at a depth below a zone where soil moisture variation is small between seasons (Figure 3). Standard design strategies for expansive clay soils assume that bedrock is stable and is located below the active moisture zone. Building on the steeply dipping Pierre Shale and associated units in the Denver area used these assumptions for foundation design between 1973 and 1996, which has resulted in significant damage to structures.

To illustrate heaving bedrock problems, this section will describe the damage and associated geologic conditions for four of the many subdivision sites that have been studied. Based on the experience gained from these discussions, conceptual models of the hazard will be described.

Site A

Construction began in 1973 on the subdivision at Site A (Figure 1), and by the late 1970s, a number of houses had experienced damage ranging from minor to major in extent. Many streets, sidewalks, curbs and gutters, etc. had also been damaged, and Jefferson County was replacing roads much more often than normal. The most obvious ground surface deformation resulted from the formation of nearly parallel ridges under structures. The ridges typically ranged in width from a few feet to 15 feet or more and ranged in height up to one foot or more.

Figure 4 shows a road running nearly perpendicular to strike of the Pierre Shale. Construction of the road and subdivision removed most of the thin soils down to weathered, steeply dipping claystones of the Pierre Shale. The road surface shown has been replaced within the past four years to remove the heave ridges that had developed; however, the displacements are visible in the curbs, yards and sidewalks, and the road surface is beginning to deform once again. Typically where roadways or similar rigid flat-work crosses heaving bedrock zones, similar ridges develop. Figure 5 shows one such ridge crossing a sidewalk and cutting through a yard. Figure 6 shows a home that has had at least one heave ridge form under it. Note the damaged concrete driveway, leaning exterior walls, and out-of-square garage doors and window frames. This damage occurred even though the house rests on a relatively deep drilled pier foundation. This home and several others in the neighborhood have not been repaired. Many others have had minor to extensive foundation repair or replacement, many of which continue to be deformed and damaged.

weathered claystone walls on both sides of the excavation and is nearly parallel to bedding. There is no obvious thick, steeply dipping bentonite bed in the excavation, only a few very thin beds. Some of the homes damaged in the steeply dipping strata have had the pier foundations sheared, which may be consistent with shear zones such as the one observed here.

Site D

Construction began on a subdivision located at Site D (Figure 1) in 1986 and continues at present. The thin soils were regraded, and buildings were constructed with drilled pier foundations placed 14 feet into weathered, steeply dipping Pierre Shale bedrock. Once structures were built and lawn watering began, heave ridges began forming. Within the first nine years the neighborhood had experienced damage from bedrock heave as severe as any other area studied to date (as much as 12 inches of vertical displacement has been observed).

A trench was excavated into the shale at this location, which provided an excellent view of the near-vertical bedrock (Figure 10). Noe (1995a) carefully mapped and monitored the trench over a several month period. Bentonite layers of 1 to 12 inches thick were exposed. Also exposed were several local shear zones that showed thrust-type movement with displacements between approximately 2 and 20 inches. During a relatively dry three-month period, little or no active movement was observed in the trench. However, after a two-inch rainfall, over a dozen bedding and fault features in the trench showed heaving movement (Figure 11). Several bentonite beds and zones of high bentonite content heaved up to an inch. Many of the shear zones were activated causing heaves between one and three inches in 24 hours.

Gill and others (1996) studied the geologic controls of the damage at Site D. They mapped the linear heave features through the subdivision by correlation with severely deformed pavements and bentonites exposed in the trench. Thirty-seven different bentonite layers in the excavation were mapped. Most were less than one-inch thick, although there was one 12-inch, one 8-inch and several 3- to 4-inch thick beds. The thicker bentonite beds and zones of numerous thin bentonite beds correlated well with damage. Typically, isolated thin beds of less than one-inch thickness did not correlate with damage. They found that bentonite beds correlating with the worst damage were approximately 60 percent smectite (by weight) compared with approximately 20 percent smectite for the adjacent silty claystone strata. Laboratory test results (Table 1) show that the bentonite beds have a much higher affinity for water and swell potential than the silty claystone strata. Their conclusion was that differential swell potential resulting from stratigraphically controlled differences in clay mineralogy and grain-size distribution was the primary factor controlling the extreme damage at this site.

CONCEPTUAL MODELS OF BEDROCK HEAVE

Based on observation of case examples such as those described above, Noe (1995b) has proposed preliminary conceptual models of the bedrock heave mechanisms. Joint research between Colorado School of Mines and the Colorado Geological Survey is continuing, and as more sites are investigated and lab tests completed, these models may be modified.

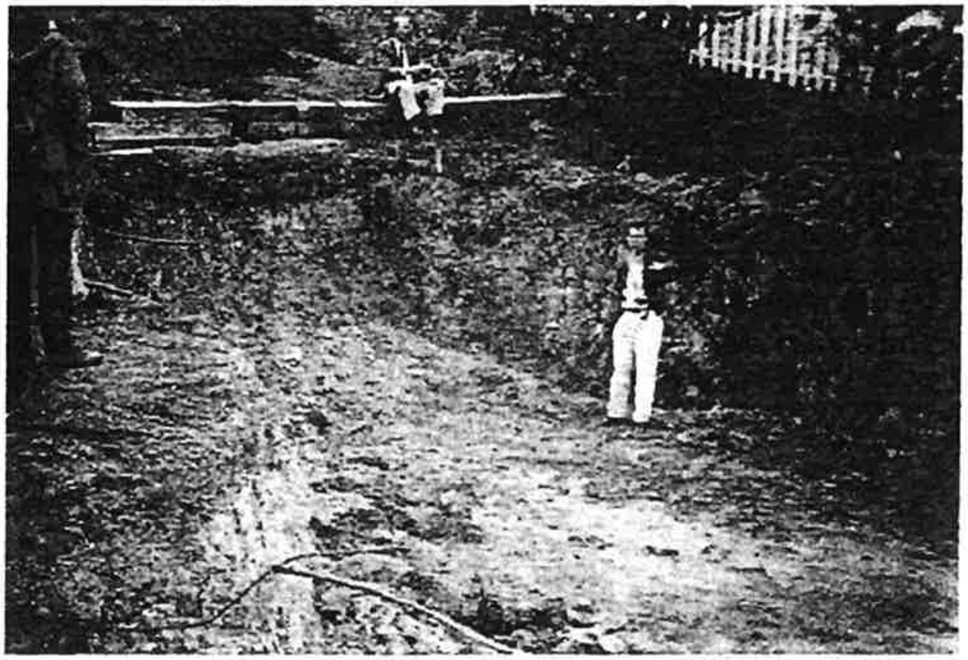

Figure 9. Shear zone in excavation that is along strike of ridge shown in figure 8 and is thought to be responsible for major damage to a house that once occupied this site.

Figure 10. Exposure of near-vertical, weathered claystone and bentonite beds (light color) of the Pierre Shale. (Photo by David Noe, Colorado Geological Survey)

Table 1. Laboratory Test Results (Gill et al., 1996). () number of samples tested.

	Silty Claystone		Bentonite	
Equilibrium Air-Dried Moisture Content (%)	3.6-5.2	(2)	13.7-15.1	(2)
Atterberg Limits				
Liquid Limits	33-52	(10)	95-103	(2)
Plasticity Index	28-47	(10)	57-65	(2)
Activity Index	0.55-0.95	(12)	0.97-1.10	(2)
Swell Index (Remolded)				
% swell on saturation (800 psf surcharge)	2-8	(2)	39-43	(2)
Cation Exchange Capacity (meq/100 g)	12.7	(1)	31.8	(1)

Summary of Geologic Observations

A conceptual model of bedrock heave must be constructed on the basis of field observations aided by laboratory testing results. The following geologic attributes have been noted at bedrock heave sites.

1) Bedrock is dipping greater than about 30 degrees. Similar damage has been reported at gently dipping sites in Colorado and South Dakota; however, the sites either have not been studied in detail or movement has been attributed to other mechanisms.
2) The bedrock is primarily shale/claystones, and bentonite is present as discrete beds or as a dispersed component of other beds.
3) Individual beds vary greatly in clay mineralogy and swell potential.
4) Bedrock is overconsolidated, i.e. under less loading than in the past.
5) Bedrock is highly weathered and has abundant fractures to a depth of 70 to 75 feet.
6) Ground-water occurrence and flow is controlled by fracturing and bedding, which may cause irregular and compartmentalized flow.
7) Bedrock has low natural moisture content, which allows maximum expansion if water is added to the environment.
8) Thick bentonite beds may form ridges by simply swelling, or bedrock blocks may move along shear surfaces to form asymmetrical heave ridges.
9) Heave damage is greatest where bedrock is near the surface.

Conceptual Models

Figure 12A is a block diagram depicting steeply dipping beds and heave ridges formed by expansion of some beds or zones. The composition of the beds varies greatly, with some having a high bentonite content dispersed throughout the claystone/shale beds and some are nearly all bentonite or contain many thin bentonite beds within a zone. The heaves form linear ridges parallel to strike and are similar to those reported by Gill and others (1996) at Site D.

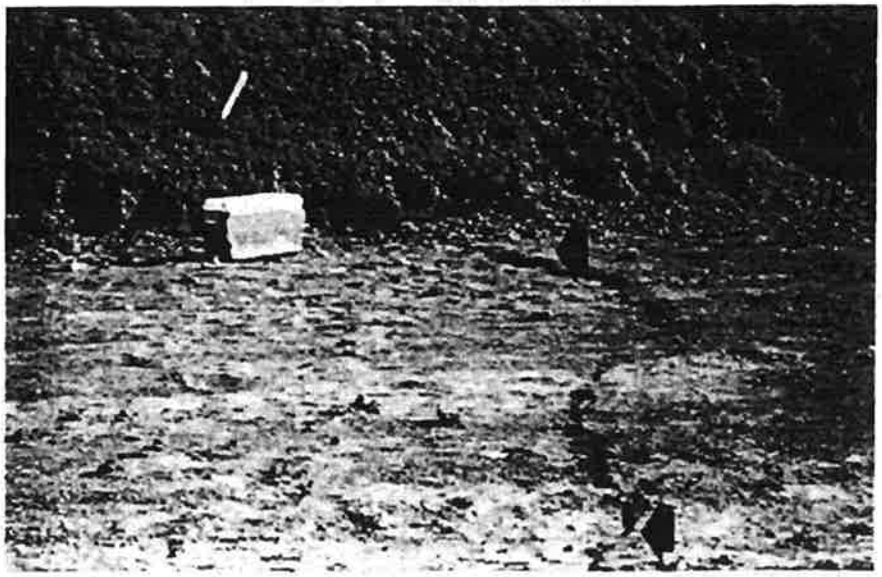

Figure 11. Swell of a steeply dipping bentonite bed in trench floor following a 2-inch rainfall. (Photo by David Noe, Colorado Geological Survey)

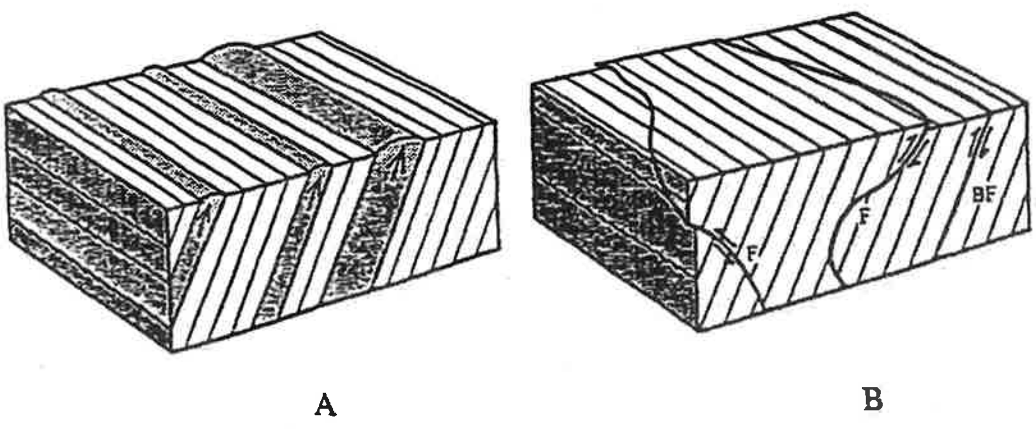

Figure 12. Block diagrams depicting conceptual models of formation of bedrock heave ridges (after Noe, 1995). Model A shows heave ridge formation by swelling, and model B shows ridge formation by swelling induced faulting.

Chugh and Missavage, 1981; Chugh et al., 1981, Huang et al., 1986; Seedsman, 1986; Sarman et al., 1994), and durability or strength characteristics (Olivier, 1979; Sarman et al., 1994).

Although swelling of mudrocks is a significant problem, and although extensive research on the subject has been conducted over the last three decades, there is a general lack of detailed characterization of mudrocks with respect to their swelling behavior. How to accurately predict the swelling potential of mudrocks still remains a major challenge. Previous researchers (Lee and Klym, 1978; Olivier, 1979; Kojima et al., 1981; Huang et al., 1986) have proposed a few classifications to predict the amount of swelling in mudrocks, but they are based on either a limited variety of mudrocks tested or they consider only one or two of the relevant properties. This greatly restricts these classifications from application to many other mudrocks encountered in engineering practice. Any method used to predict the swelling behavior of mudrocks must take into account the geologic diversity of mudrocks. The purpose of this paper is to present a systematic approach to predict the swelling potential of a wide variety of mudrocks, taking into account the relevant properties that influence swelling potential.

METHODOLOGY

Sample Collection

Forty-two samples of fresh mudrocks were collected for this study from across the United States (Figure 1). A swelling potential map of the United States, prepared by Snethen and others (1975), and other available information on swelling type rocks were used to select the sampling sites. All samples were properly stored to prevent slaking. Thirty-six of the 42 samples were used to develop equations for predicting the amount of swelling in mudrocks whereas the remaining six samples (A through F) were used to validate the prediction equations.

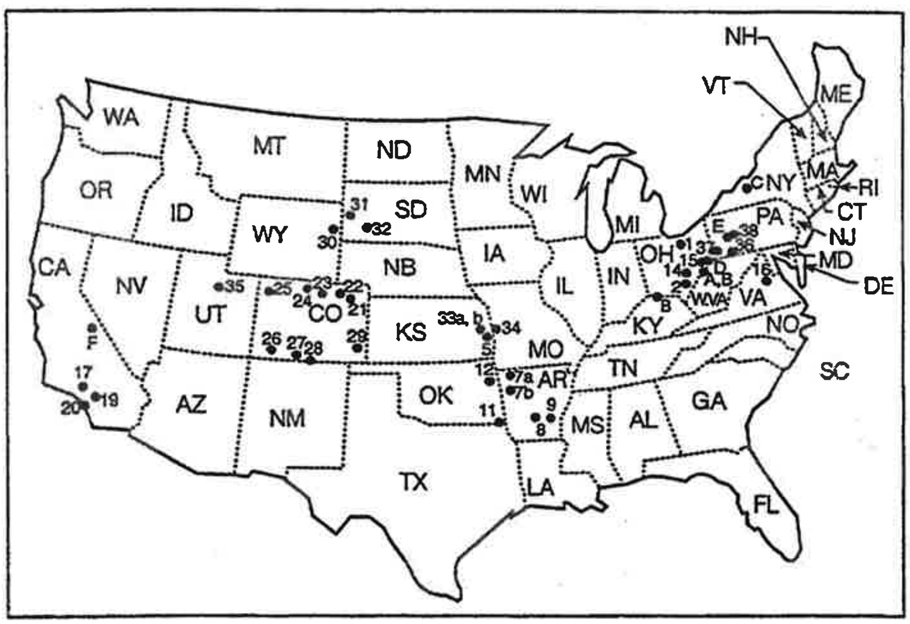

Figure 1. Location of sampling sites.

Determination of Geologic Characteristics and Engineering Properties

A series of geologic analyses and engineering tests were performed on the samples collected to determine grain-size distribution, clay content, clay mineralogy, texture and structure, absorption, adsorption, Atterberg limits, specific gravity of solids, bulk density, void ratio, uniaxial compressive strength, slake durability index, volumetric increase, swelling pressure, and pore-size distribution. Material for grain-size distribution and Atterberg limits was prepared by disintegrating mudrock samples through alternate cycles of wetting and drying. The amount of clay-size (.005mm) material was determined by performing hydrometer analysis in accordance with the American Society for Testing and Materials procedure D422 (ASTM, 1987). X-ray diffraction analysis was performed to identify clay minerals present. A scanning electron microscope (SEM) was used to study mudrock texture and structure (mineral alignment, size and abundance of voids, degree of microfracturing) at various magnifications.

Absorption was determined using ASTM procedure C-97. Adsorption was measured by placing oven-dried samples of each mudrock in a humidity chamber preset at 25°C and 100 percent relative humidity. Adsorption was computed as the ratio of the weight increase after 72 hours to the initial dry weight, expressed as a percentage. Atterberg limits (liquid limit, plastic limit, plasticity index) were determined using ASTM procedures D423 and D424.

Specific gravity of solids was determined using ASTM procedure D854, whereas ASTM procedure C97 was used for density determinations. The samples for density determination were oven-dried and coated with a thin layer of Krylon to prevent disintegration when soaked in water. Void ratios of mudrocks were calculated from specific gravity and density values, using phase relations (Holtz and Kovacs, 1981):

Point load test, as described by Broch and Franklin (1972), was used to determine uniaxial compressive strength. For each mudrock sample, 10-20 irregular lumps were tested and the average values were taken. Second-cycle slake durability index (Id_2) values of the mudrocks studied were determined in accordance with the ASTM procedure D4644.

The apparatus shown in Figure 2, designed in accordance with the specifications set by the International Society for Rock Mechanics (1979), was used to measure volumetric increase (amount of swelling) for each mudrock sample. Three 2.5-cm cubical samples were tested for each mudrock. The cubes were oven-dried at 105°C for 24 hours, cooled to room temperature in a desiccator, and loaded in the apparatus with all three gauges set at zero. The dial gauges were capable of measuring strain to 0.0001 inch, or 0.0025 mm, accuracy. The apparatus allowed the samples to expand in three mutually perpendicular directions. Once the sample was placed in the apparatus and water was allowed to enter the chamber, dial gauge readings were taken at the pre-determined time intervals for a period of 24 hours. The change along each axis was calculated as percentage of the initial dimension along that axis. The total volumetric increase was then calculated as the percentage of the initial volume of the cube.

Swelling pressure exerted by mudrock samples was measured using a modified version of the equipment used for measuring swelling pressure of soils during saturation. The modification allowed testing of cubical, instead of circular, samples. The samples were confined on all four sides, as well as the bottom, so that they could expand in the vertical direction only.

The swelling exerted pressure on a proving ring located above the sample. The deformation of the proving ring, as indicated by the dial gauge, was taken as a measure of the swelling pressure. The changes in the dial gauge readings were recorded at the same predetermined time intervals as for the volumetric increase.

Pore-size distribution was determined for 12 representative samples, using mercury intrusion porosimetry according to the method described by Kaneuji (1978).

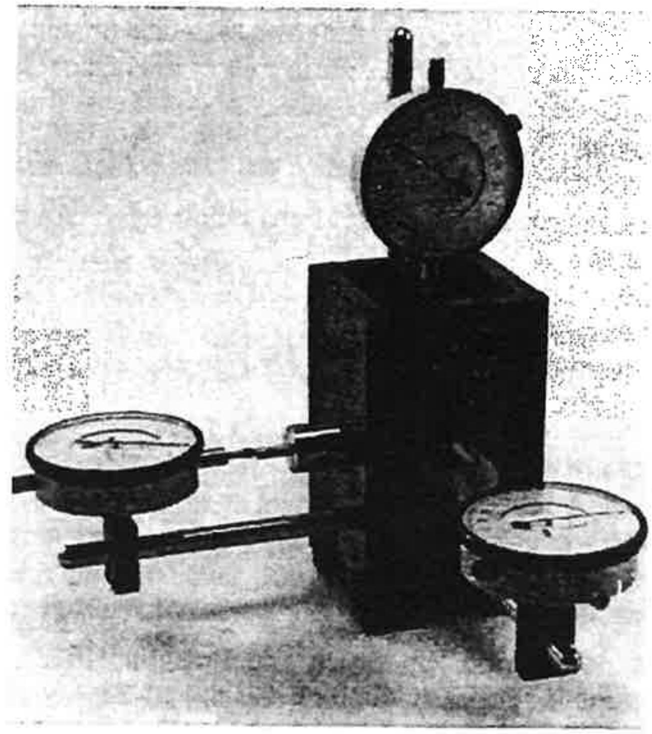

Figure 2. Apparatus used for measuring volumetric increase.

Data Analysis

The volumetric increase for 30 of the 42 mudrock samples was correlated with all other properties using bivariate and multiple regression analyses. The purpose was to identify those properties which could be used to predict volumetric increase. Values of some properties were transformed using square root and logarithmic transformations. In order to take into account the effect of geological factors on regression analyses, the samples were first divided into laminated and non-laminated categories, resulting in 14 laminated and 22 non-laminated samples. Next, the samples were divided into four groups using the classification suggested by Lundegard and Samuels (1980). The classification is based on two important geological characteristics of mudrocks: the amount of clay size (0.005 mm) material and the presence or absence of laminations. The classification resulted in 7 claystones (>66% clay), 7 mudstones (33-66% clay), 11 mudshales (33-66% clay; laminated), and 11 siltstones (<33% clay). For multiple regression analysis, all possible subset options were used to obtain correlation coefficients for different combinations of properties. The best combination of properties was taken to be the one for which further increase in the number of properties did not significantly increase the correlation coefficient or reduce the residuals (difference between measured and predicted values of volumetric increase). The equations developed on the basis of regression analysis were verified using relevant properties of the remaining 6 of the 42 samples. Finally, the results of the study were used to categorize mudrocks studied on the basis of their swelling potential.

RESULTS AND ANALYSIS

The results of engineering tests and geologic analyses for the 36 mudrock samples are presented in Table 1. The amount of 0.005 mm clay varies from 4 percent to 77 percent. Illite and kaolinite are the most commonly occurring clay minerals in the rocks studied, followed by smectite, chlorite, and vermiculite in decreasing order.

Percent absorption (w_{ab}) varies from 2.5 to 86.5, with claystones exhibiting the highest values and siltstones the lowest (except sample 22 which has abundant microfractures). Percent adsorption (w_{ad}) ranges from 0.11 to 9.57. Claystones exhibit the highest adsorption values of the rock types tested because of their lower degree of induration, higher clay content, and abundance of expandable clay minerals. Mudshales have the lowest adsorption values even though some of them contain expandable clay minerals. This is because the tightly packed texture of mudshales, as indicated by the SEM work performed, does not allow easy access of water to the clay surfaces.

The liquid limit (LL) and plasticity index (PI) values (Table 1), when plotted on Casagrande's plasticity chart, indicate that the plasticity characteristics of mudrocks are highly variable, ranging from ML to CL to CH. Generally speaking, mudrocks with higher plasticity index values are considered to exhibit higher swelling potential (Ola, 1982). In this study, however, not all mudrocks characterized by high plasticity show high volumetric increase (Table 1), suggesting that factors other than clay mineralogy, such as texture and structure, also affect swelling in mudrocks.

Specific gravity of solids (G_s) varies from 2.27 to 2.89 and dry density (ρ) from 1.46 g/cm^3 to 2.63 g/cm^3. Void ratio, as calculated from specific gravity and density values, lies between 0.05 and 0.78. In general, mudstones and mudshales exhibit lower void ratios than claystones and siltstones. Since the void ratio indicates the total volume of pore space in a mudrock that can be intruded by water, it is an important property for predicting the swelling potential of mudrocks.

Second-cycle slake durability index (Id_2) varies between 0.2 and 98.9 percent, with all claystones characterized by low durability. The compressive strength (σ_c) of mudrocks studied ranges from 13 MPa to 214 MPa. The large variation in compressive strength is attributed to the presence of fractures and laminations in some samples as well as to the overall diversity of mudrocks investigated.

Table 1 also lists the volumetric increase (ΔV) and swelling pressure (P_s) values for the mudrocks studied. Volumetric increase ranges from 0.1 percent to 68.9 percent whereas swelling pressure varies from 10 KPa to 8242 KPa, again indicating the large variety of

Table 1: Engineering properties of mudrocks studied.

Sample No.	Mudrock Type	5μm (%)	ω_{ab} (%)	ω_{ad} (%)	Id_2 (%)	σ_c MPa	G_s	ρ g/cm^3	e	LL	PI	ΔV (%)	P_s KPa	Clay Minerals*
1	Mudshale	42	5.5	0.20	91.0	30	2.73	2.59	0.05	27.6	10.1	0.2	10	C (34), I (100), K (35)
2	Mudstone	42	4.0	3.05	4.0	28	2.86	2.54	0.13	29.3	10.5	54.0	4531	C (38), I-S (60), K (19)
5	Mudshale	41	19.0	0.98	51.0	72	2.73	2.70	0.15	36.7	16.9	15.3	701	V-I (56), I (91), K (92)
6	Mudstone	50	20.5	1.49	4.8	63	2.72	2.22	0.23	35.5	8.9	11.4	910	C (40), I (100), K (67)
7a	Siltstone	18	8.3	0.77	6.1	43	2.71	2.44	0.11	25.2	6.9	0.8	156	V-I (75), I-V (57), K (18)
7b	Mudshale	37	15.2	2.25	66.7	30	2.68	2.26	0.19	35.3	13.2	4.3	227	C (31), I (100), K (25)
8	Mudshale	36	11.7	0.37	92.8	42	2.72	2.49	0.09	39.7	10.9	2.8	1106	V (24), I (74), K (37)
9	Siltstone	21	4.4	0.33	98.3	90	2.76	2.45	0.13	26.2	6.5	0.6	427	C (16), I (58), K (8)
10	Siltstone	12	5.1	0.65	98.7	116	2.70	2.57	0.05	30.7	7.0	1.8	206	S-I(29), C-S(29), I(96), K(24)
11	Mudshale	34	6.8	0.22	97.8	64	2.89	2.51	0.15	32.0	7.4	2.5	209	S (22), I (100), K (25)
12	Siltstone	4	2.5	0.15	97.8	138	2.50	2.33	0.07	27.8	7.2	1.1	280	I (26), K (5)
14	Siltstone	16	8.1	1.60	90.3	96	2.74	2.33	0.18	32.6	18.9	1.6	916	S-I (56), I-S (91), K (92)
15	Siltstone	21	4.8	0.46	92.5	79	2.75	2.58	0.07	28.5	9.4	4.8	958	S-I (65), I-S (100), K (35)
16	Siltstone	–	3.4	0.11	98.9	214	2.78	2.57	0.08	28.5	8.2	0.3	232	C (45), I (60), K (19)
17	Mudshale	38	39.0	5.31	79.0	71	2.60	1.46	0.78	64.7	24.5	10.2	2072	S (100), I-S (24), K (24)
19	Mudstone	34	17.8	1.12	19.0	15	2.80	2.08	0.35	36.9	11.4	6.5	442	K (100)
20	Siltstone	23	17.0	2.99	78.0	35	2.66	2.04	0.30	39.3	11.6	10.6	1799	S (100), I-S (34), K (18)
21	Mudshale	41	21.0	1.89	0.2	15	2.75	1.97	0.40	35.8	13.7	13.0	1101	C (75), I (91), K (92)
22	Siltstone	28	21.6	1.93	16.1	65	2.75	2.43	0.13	26.1	6.0	17.8	1818	S-I (100), I-S (45), K (13)
23	Mudshale	44	36.8	4.28	0.3	13	2.72	2.38	0.14	37.4	15.8	11.6	840	S-I (100), I-S (64), K (8)
24	Mudstone	56	24.1	0.68	90.5	76	2.74	2.39	0.15	34.2	14.5	8.5	1753	C (35), I (100), K (44)
25	Claystone	67	34.0	6.58	0.2	70	2.66	2.15	0.24	48.6	26.0	54.2	8242	S-I (100), I-S (28), K (22)
26	Mudshale	34	6.3	0.54	90.4	49	2.63	2.36	0.11	25.7	7.4	0.9	41	I-S (100), K (38)
27	Mudshale	41	13.7	8.69	7.5	69	2.69	2.22	0.21	36.8	10.4	25.8	2574	S-I (100), K (48)
28	Claystone	72	20.7	6.80	2.9	32	2.75	2.10	0.31	53.4	28.3	30.1	3674	S-I (100), I-S (62), K (92)
29	Siltstone	13	9.8	3.13	95.7	41	2.81	2.24	0.25	31.0	11.7	1.0	433	I (100), K (13)
30	Claystone	68	15.7	5.69	1.1	48	2.76	2.29	0.21	40.9	13.5	37.0	4555	S-I (35), I (100), K (20)
31	Claystone	72	29.7	7.80	5.9	15	2.27	1.80	0.26	51.4	20.9	32.6	4012	S-I (100)
32	Claystone	67	24.6	9.57	59.0	31	2.65	1.98	0.34	52.5	23.4	11.6	1939	S-I (100), I (31), K (17)
33a	Siltstone	28	14.3	5.30	95.4	81	2.35	1.71	0.37	36.4	14.4	4.7	2720	C (30), I (100), K (25)
33b	Mudshale	42	12.7	5.45	73.9	49	2.76	2.18	0.27	35.6	15.4	5.6	604	C (25), I-S (100), K (19)
34	Claystone	68	16.6	7.11	24.7	37	2.72	2.21	0.23	41.7	21.4	17.8	30	C (44), I (100), K (70)
35	Claystone	77	86.5	9.25	0.5	62	2.80	2.31	0.21	53.4	37.9	68.9	6323	S (82), I (39), K (93)
36	Mudstone	50	20.4	2.12	70.5	113	2.76	2.63	0.05	32.5	16.0	9.5	5115	C (32), I (100), K (54)
37	Mudstone	44	52.0	3.21	16.4	27	2.75	2.58	0.07	27.7	10.6	54.2	5751	C (27), I (100), K (22)
38	Mudstone	42	7.0	1.80	84.3	94	2.74	2.57	0.07	32.6	12.1	0.1	51	C (39), I (100), K (79)

LEGEND:

5μm = 5 micron clay; ω_{ab} = absorption; ω_{ad} = adsorption; Id_2 = second-cycle slake durability index; σ_c = uniaxial compressive strength; G_s = specific gravity; ρ = density; e = void ratio; LL = liquid limit; PI = plasticity index; ΔV = volumetric increase; P_s = swelling pressure

* C = Chlorite; I = Illite; K = Kaolinite; S = Smectite; V = Vermiculite
Figures in parentheses refer to relative peak intensities

mudrocks tested. Samples containing large amounts of smectite (samples 22, 25, 27, 28, 31, and 35), as indicated by the relative peak intensities in x-ray diffractograms, exhibit large volumetric increase. However, large volumetric increase is also exhibited by samples containing a little or no smectite (samples 2, 5, 34, and 37). On the other hand, some samples with more smectite than sample 2 (samples 27, 28, and 31) have lower volumetric increase. The reason for this unexpected behavior is that swelling of mudrocks depends on not only clay mineralogy but also on texture as pointed out previously. SEM work indicated that samples 2, 5, 34, and 37 were all characterized by the presence of open, inter-connected microfractures which allowed easy ingress of water whereas samples 27, 28, and 31 were massive with few microfractures (Sarman, 1991).

Another indicator of the texture of mudrocks is their pore-size distribution. Table 2 shows the total pore volume, average pore diameter, and volumetric increase data for 12 selected mudrock samples. The data in Table 2 indicate that mudrocks exhibiting large volumetric increase are generally characterized by a relatively large volume of small size (<0.05 microns) pores (e.g., sample 35). An explanation for this behavior is that large pore volume allows more water to enter the mudrock and small size pores provided more surface area for exposure to moisture. Small size pores also help in water absorption and adsorption through capillary action as is indicated by the fact that sample 35 has the highest value of percent absorption and the second highest value of percent adsorption. Sample 17 has the largest pore volume (0.310 cm^3/g) consisting of relatively large average size (0.300 micron) pores, indicating fewer pores and smaller surface area exposed to moisture. That is why sample 17 exhibits an intermediate value of volumetric increase (10.2 percent, Table 2). These observations suggest that texture, as represented by pore characteristics, significantly influences the swelling potential of mudrocks.

TABLE 2. Pore-volume, pore-diameter, and volumetric increase data for selected mudrock samples arranged in order of decreasing volume change.

Sample No.	Rock Type	Total Pore Volume (cm^3/g)	Average Pore Diameter (microns)	$\Delta V(\%)$
35	Claystone	0.047	0.006	68.9
37	Mudstone	0.017	0.035	54.2
2	Mudstone	0.025	0.010	54.0
30	Claystone	0.054	0.010	37.0
28	Claystone	0.107	0.040	30.1
27	Mudshale	0.082	0.040	25.8
22	Siltstone	0.026	0.030	17.8
5	Mudshale	0.063	0.025	13.3
17	Mudshale	0.310	0.300	10.2
36	Mudstone	0.007	1.000	9.5
15	Siltstone	0.020	0.035	4.8
1	Mudshale	0.016	5.000	0.2

The importance of structure in influencing the swelling potential of mudrocks is indicated by the fact that the amount of swelling in the cubical samples used in this study was found to be higher perpendicular to the plane of laminations than in the other two directions as water can enter along the planes of lamination with greater ease than through the small pores elsewhere.

Bivariate Regression Analysis

The bivariate regression analysis was performed between volumetric increase and untransformed and transformed values of all other engineering properties shown in Table 1. The strength of correlations was evaluated using statistical parameters such as correlation coefficient (r), the standard error of estimate, the F-statistic, the t-statistic, and the residuals (difference between the measured and the predicted values of volumetric increase). A minimum r value of 0.87, which explains at least 75 percent of the variance in the data, and a maximum residual of 10 percent were considered as the acceptable limits for this study.

The bivariate regression analysis did not indicate a strong correlation between any of the engineering properties and the volumetric increase, even when square root and log transformations were applied. Percent absorption exhibited the best correlation ($r = 0.82$) but the residuals remained high (>10 percent). This leads to the conclusion that a single property cannot be used to predict the volumetric increase for all types of mudrocks without large errors (residuals >10 percent). Subdivision of mudrocks into laminated and nonlaminated groups did not improve the correlations. A slight improvement did occur when the mudrocks were subdivided into claystones, mudstones, mudshales, and siltstones, especially with respect to percent adsorption, which showed the highest correlation coefficients, but the residuals (-8 to +15) still exceeded the 10 percent limit set for this study.

Multiple Regression Analysis

Since bivariate regression analysis did not indicate a significant correlation with volumetric increase, a multiple regression analysis was performed to identify a group of properties that could be used to predict volumetric increase. In this analysis, volumetric increase was considered as the dependent variable and all other engineering properties (percent clay, absorption, adsorption, second-cycle slake durability index, unconfined compressive strength, specific gravity, density, void ratio, liquid limit, and plasticity index), along with their logarithmic and square root transformations, were considered as the independent variables.

The multiple regression analysis resulted in the development of the following equations which indicate the role of geological characteristics in influencing the swelling potential of mudrocks:

All Mudrocks ($R = 0.89$):

$$\Delta V = 8.1 + 0.7\,\omega_{ab} + 1.5\omega_{ad} - 0.1\,Id_2 - 30.2\,e + 0.04\,\sigma_c \qquad \text{Eq. 1}$$

Claystones (R = 0.98):

$$\Delta V = 42.3 + 0.4\ \omega_{ab} - 14.8 \log(Id_2) - 40.6\ e - 0.1\ \sigma_c \qquad \text{Eq. 2}$$

Mudstones (R = 0.98):

$$\Delta V = 52.5 + 0.5\ \omega_{ab} - 17.5 \log(Id_2) - 137.1\ e - 0.4\ \sigma_c + 1.9\ PI \qquad \text{Eq. 3}$$

Claystones plus Mudstones (R = 0.98):

$$\Delta V = 61.7 + 0.4\ \omega_{ab} - 15.5 \log(Id_2) - 136.1\ e - 0.3\ \sigma_c + 0.8\ PI \qquad \text{Eq. 4}$$

Mudshales (R = 0.95):

$$\Delta V = 4.6 - 0.3\ \omega_{ab} - 0.2\ Id_2 - 7.9\ e + 0.2\ \sigma_c + 0.4\ LL \qquad \text{Eq. 5}$$

Siltstones (R = 0.96):

$$\Delta V = -8.8 + 1.2\ \omega_{ab} + 4.0 \log(Id_2) - 33.0\ e \qquad \text{Eq. 6}$$

In these equations:

ΔV = volumetric increase, %
ω_{ab} = absorption, %
ω_{ad} = adsorption, %
Id_2 = second-cycle slake durability index, %
σ_c = unconfined compressive strength, MPa
e = void-ratio
LL = liquid limit, %
PI = plasticity index, %

The fact that the correlation coefficient for all mudrocks treated as one group (Eq.1) is less than the correlation coefficient values for individual classes of mudrocks suggests that it is important to geologically classify mudrocks into individual groups before evaluating their swelling potential. It is also evident from Equations 2 through 6 that different combinations of properties reflect the swelling behavior of different types of mudrocks. The high R values (>0.95) clearly indicate the improved nature of correlations over the bivariate regression analysis and, hence, the need for using a set of properties for predicting the swelling potential. The regression analysis shows that the same set of properties is needed to predict the volumetric increase of either claystones or mudstones, except for the use of PI in the case of mudstones. Therefore, claystones and mudstones can be grouped together and Equation 4 can be used to predict the volumetric increase of both. The reason why different properties influence the swelling behavior of different types of mudrock can be attributed to the distinct differences in their mineral composition, texture, and structure.

Figure 3 shows a plot of predicted versus measured values of volumetric increase for all mudrocks combined. The plot shows that the use of Equation 1 for predicting volumetric

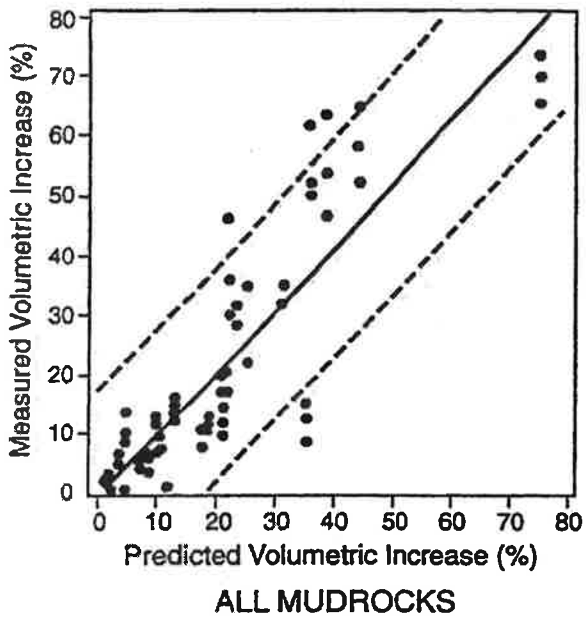

Figure 3. Plot of predicted versus measured values of volumetric increase for all mudrocks combined. Dashed lines indicate 95 percent confidence band.

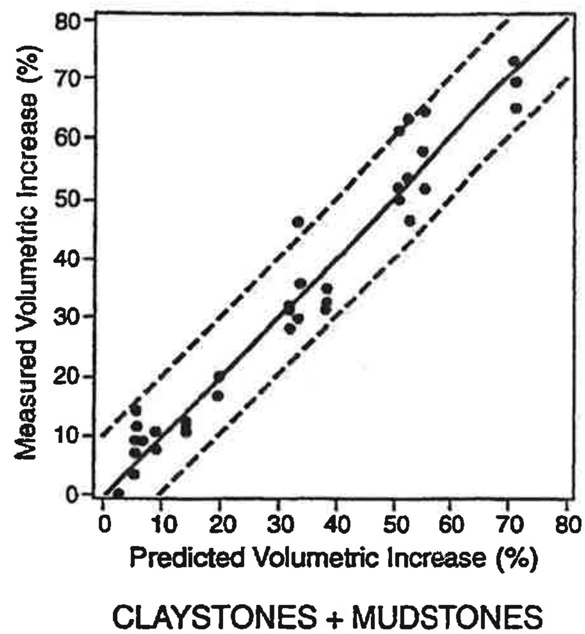

Figure 4. Plot of predicted versus measured values of volumetric increase for claystones plus mudstones. Dashed lines indicate 95 percent confidence band.

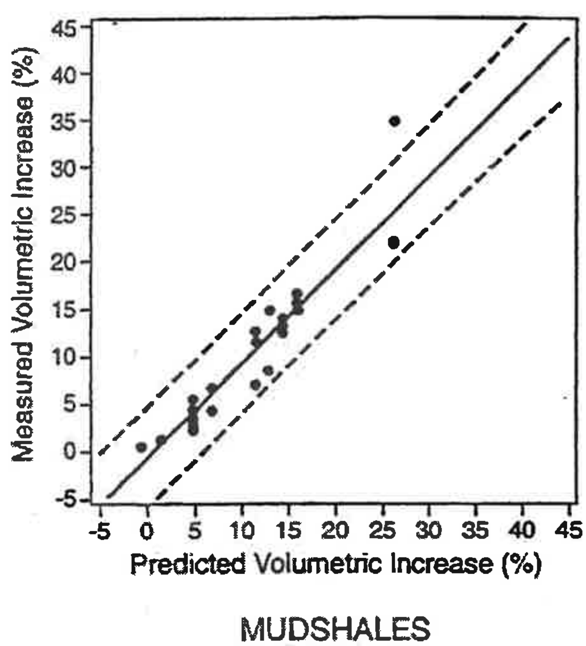

Figure 5. Plot of predicted versus measured values of volumetric increase for mudshales. Dashed lines indicate 95 percent confidence band.

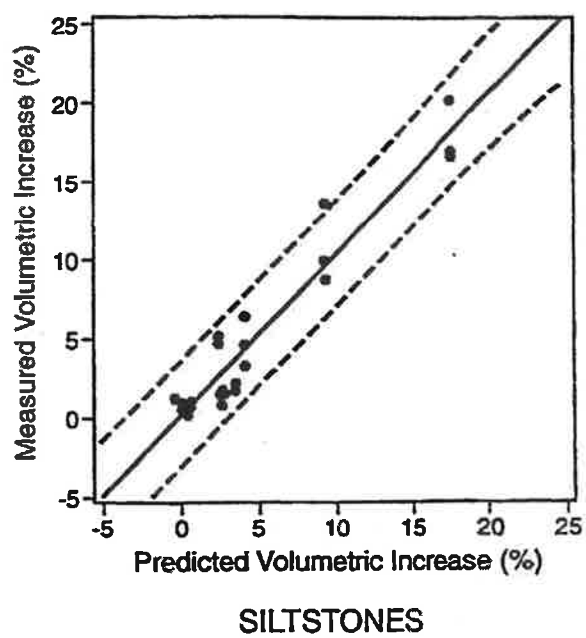

Figure 6. Plot of predicted versus measured values of volumetric increase for siltstones. Dashed lines indicate 95 percent confidence band.

increase for all types of mudrocks can result in rather large errors (residuals approaching ± 27 percent). Figures 4-6 show plots of predicted versus measured values of volumetric increase for claystones plus mudstones, mudshales, and siltstones, respectively. The residuals in all cases are less than ± 9 percent, and in most cases less than ± 5 percent, which indicates that Equations 4, 5, and 6 can be used to predict the volumetric increase with a reasonable degree of confidence. Figures 4-6 also show that classifying mudrocks into individual groups on the basis of geological characteristics is essential for adequate evaluation of their swelling potentials. Since clayshales and mudshales were not included in the rocks studied, clayshales can be included in the mudshale group and siltshales in the siltstone group for prediction purposes.

Verification and Limitation of Prediction Equations

In order to verify equations (4), (5), and (6), six additional samples (2 claystones, 1 mudstone, 1 mudshale, and 2 siltstones) were tested and the relevant properties were used to calculate volumetric increase. Table 3 lists the measured and calculated values of volumetric increase for the six samples as well as the differences between the two sets of values (error of estimation or residuals). Although the error of estimation is rather large for samples D and E (5.28% and 6.19%, respectively), it is still within acceptable limits (<10%). Thus, equations (4), (5), and (6) can be used to predict volumetric increase with a reasonable degree of accuracy. Table 3 also shows that the percent error generally increases with increasing ΔV ($R^2 = 0.59$).

It should be pointed out that several of the predictive tests (σ_c, w_{ab}, w_{ad}, LL) may be just as difficult to perform as the volumetric increase test, which may limit the application of the proposed equations for predicting the swelling potential of mudrocks. However, even if the equations were not used for predictive purposes, they do verify the importance of geological classification and show which properties influence swelling the most. An additional limitation of the proposed equations may be that, even though 42 samples from across the United States were used to develop the equations, they still may not be representative of all types of mudrocks.

Table 3: Comparison between measured and estimated values of volumetric increase for the six additional samples.

Sample No.	Measured ΔV (%)	Estimated ΔV (%)	Error of Estimation (%)
A	9.50	8.77	0.73
B	7.70	6.66	1.04
C	2.50	-1.20	3.70
D	30.30	35.58	-5.28
E	24.40	30.59	-6.19
F	0.27	-0.65	0.92

SWELLING POTENTIAL CLASSIFICATION

A swelling potential classification can be meaningful and practical only if it relates to the pressures that result from swelling. This is because the extent of damage caused by swelling depends upon the swelling pressure. Information about swelling pressure is also essential for proper design of structures built on expansive soils or rocks. The swelling pressure values for the thirty-six mudrock samples were correlated with the corresponding values of volumetric increase, yielding a correlation coefficient of 0.84. The results were used to develop a swelling potential classification that takes into account both the volumetric increase and the swelling pressure. The classification is shown in Table 4. The boundaries between various classes were chosen on the basis of observed structural damage caused to buildings founded on some of the mudrocks studied as well as information provided in literature by Olivier (1979) and Huang et al. (1986).

According to the classification shown in Table 4, mudrocks are divided into five categories based on their volumetric increase and the associated swelling pressure. The "very low swelling" mudrocks are not likely to present any major threat to structures whereas "very high swelling" mudrocks may cause extensive damage to even heavy structures if foundations are not designed to cope with high pressures. In order to use the classification proposed in Table 4, the given mudrock must first be classified as a claystone, mudstone, mudshale, siltstone, etc. The sample should be tested for the relevant properties which should then be used to estimate the expected volumetric increase using equations (4), (5), and (6). Once the volumetric increase is known, Table 4 can be used to estimate the anticipated swelling pressure and classify the rock accordingly.

Table 4. Swelling potential classification of mudrocks.

Volumetric Increase (%)	Swelling Potential	Expected Swelling Pressure (kPa)
0-4.5	very low	70-1,000
4.5-13	low	1,000-2,000
13-23	medium	2,000-3,000
23-50	high	3,000-5,000
50-100	very high	5,000-10,000

CONCLUSIONS

Based on the results of this study, it can be concluded that the swelling potential of mudrocks is greatly affected by their geological characteristics (type and amount of clay material, presence or absence of laminations, microfractures, and texture). This is confirmed by the fact that subdividing mudrocks into different groups based on their geological characteristics, improves the correlation between volumetric increase and other engineering properties.

However, since swelling of mudrocks is a complex process, one single property cannot be used to predict the swelling potential of all types of mudrocks. On the other hand, excessive swelling in mudrocks is not restricted to mudrocks containing highly expansive clay minerals. Texture, as indicated by total pore volume and average pore diameter, plays an equally important role. Absorption, adsorption, slake durability index, void ratio, compressive strength, liquid limit, and plasticity index, in various combinations, can all be used to predict the swelling potential of different types of mudrocks. These engineering properties are a reflection of the geological characteristics of mudrocks. For example, absorption, adsorption, and liquid limit are all closely related to the type and amount of clay material present, slake durability and compressive strength are influenced by both the clay mineralogy and the structure (laminations, microfractures), and void ratio is a good indicator of the degree of induration of mudrocks.

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Comparison of Weak and Weathered Rock Classification Systems

PAUL M. SANTI
Department of Geological and Petroleum Engineering, University of Missouri-Rolla, Rolla, MO 65409

ABSTRACT

The goal of weak and weathered rock classification, like rock and soil classification, is to provide a consistent description of geologic and engineering properties for comparison to other similar materials. Unlike soil and rock, weak and weathered rock depend on both mass and matrix properties and have strength and durability characteristics which change over time. Furthermore, the magnitude of many of the strength properties falls in the range between typical soil and rock values. Because of these differences, classification systems for weak and weathered rock must provide additional information over soil or rock systems.

Most weak and weathered rock classification systems have tended to focus on site-specific or material-specific parameters, while few systems have attempted global classification of weak and weathered rock. This paper presents an overview of many of these classification systems, summarizes the originally intended breadth of application of the system, and notes strengths and drawbacks for each one. The effectiveness of each system is evaluated, and potential applicability beyond the limits specified by the original author is hypothesized. Finally, the most useful systems are judged, and modifications to enhance their applicability are proposed.

INTRODUCTION

Classification systems for soil and rock help us to recognize that, although materials may have drastically different appearances, they may share similar genetic, mineralogical, and engineering properties. Because weak and weathered rock materials display properties characteristic of both soil and rock, they are typically classified based on the background of the observer. Geologists tend to note the residual rock fabric and rock mass properties, and soils engineers tend to characterize the particle sizes, density, and durability. Numerous workers have called for a universal classification system which includes both rock and soil characteristics, and considers the intended use of the material (Deen, 1981; Franklin, 1981; Chapman, et al, 1976; Wood and Deo, 1975). The goal of this paper is to evaluate and compare several popular classification systems for weak and weathered rock, and to propose modifications to enhance the applicability of some of the more useful systems.

According to Aufmuth (1974) and Franklin (1970), the preferred diagnostic tests in any classification system, particularly early in the classification process, are simple, inexpensive, repeatable, give a broad range of values, and provide a logical indication of the appropriate follow-up tests. For weak and weathered rock, Santi and Rice (1991) suggest that the diagnostic tests should allow for values to fall into a continuum between soil and unweathered rock. Santi

(1994) provides an overview of both field and laboratory diagnostic tests which have been used in previous classification systems for weak and weathered rock, as well as a summary of previous research which has evaluated the effectiveness of many of the diagnostic tests.

The multitude of diagnostic tests have been combined in a variety of ways to form a frustrating, diverse number of classification systems. Chapman and others (1976) suggest that these systems are either 1) developed by geologists and consist primarily of genetic and qualitative information, 2) developed by engineers and consist primarily of quantitative information based on a variety of laboratory tests, or 3) developed by a specific agency for its own use and are typically limited regionally or occupationally.

The ideal classification system for earth materials should, at a minimum, provide a preliminary estimate of genetic, mineralogical, and engineering properties. Information obtained in the field should be sufficient to apply the ideal system, and information obtained in the laboratory should confirm the classification and provide more data for design. The system should evaluate material strength, durability (or tendency to slake or disaggregate when wet), degree of weathering, and mass properties influenced by fractures, corestones, and zones of weakness.

The systems compared herein by no means represent a comprehensive list of those available, but rather were selected because of their popularity, their accessibility to the typical practitioner, or because of their novel or unique contributions toward understanding weak and weathered rock masses. Each system is summarized according to a rough grouping regarding applicability, followed by a comparison of the set of systems, evaluation of the most useful systems, and proposed modifications to enhance the applicability of the most useful systems.

SYSTEMS DESIGNED FOR WEATHERED ROCK

The classification systems described below focus on weathered rock and the continuum of engineering properties between intact rock and soil derived from that rock.

Systems Based on Compressive Strength

The simplest method of classification is based solely on compressive strength. Many such systems have been proposed: several are summarized in Afrouz (1992), in a diagram reproduced in Figure 1. In general, these systems recognize "hard" or "strong" rock with a compressive strength exceeding 20-80 MPa, and "soil" with a compressive strength less than approximately 1 MPa. Weak or weathered rock has compressive strength between these two ranges.

Geological Society Engineering Group System

One of the most thorough and comprehensive classification systems for weathered rocks was proposed recently by the Engineering Group of the Geological Society (1995). This system, which is outlined in Figure 2, focuses on field classification of the full range of weathered rock, including weak materials such as mudstones. The system was developed based as much as

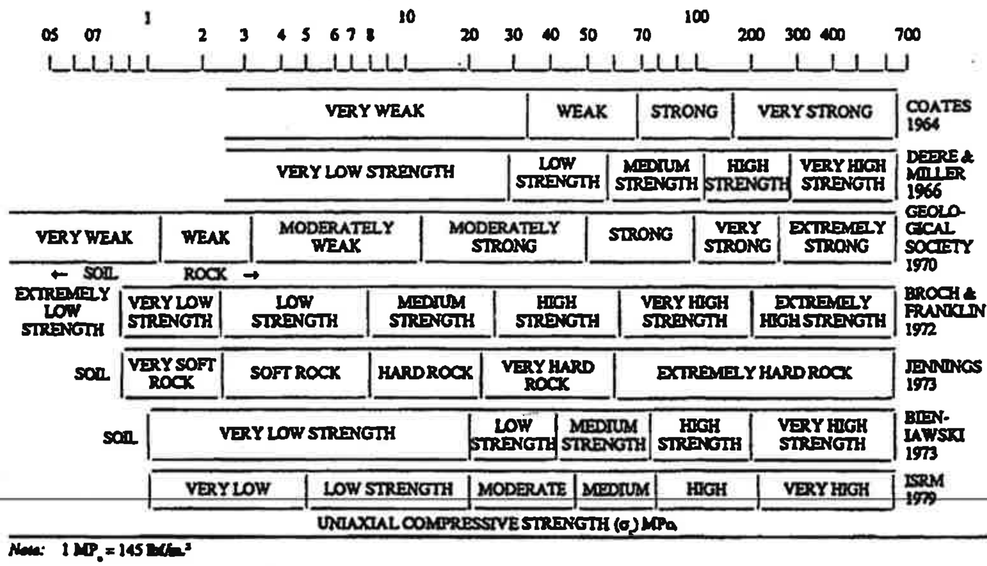


Figure 1. Comparison of classifications based on rock strength (from Afrouz, 1992). Strength values are in MPa.

possible on good, existing systems to prevent developing yet another new system in the effort to develop a better system.

In general, the Geological Society System consists of five "approaches," which are shown on Figure 2. Approach 1 is a standard factual description of the material, and should be completed for all samples. Following the flowchart on Figure 2, the user then applies one of the remaining four approaches to add detail to the classification. Approach 2 is for uniform weathered materials such as igneous and metamorphic rocks which show weakening and susceptibility to slaking upon weathering. Approach 3 is for heterogeneous masses containing corestones within a weaker matrix, a situation expected where significant weathering has occurred along fracture sets. Materials addressed by Approach 3 might include weathered interbedded sedimentary rocks or complex volcanic sequences comprised of both weak and strong rocks. Approach 4 is for materials which incorporate both matrix and mass features. For these materials, weathering is enhanced along fractures, but a significant degree of weakening occurs near the surface. Materials included in Approach 4 would be overconsolidated clays, shales and mudstones. Approach 5 is a catch-all category for special cases, such as karst limestone or evaporites.

This system has the advantage that it was developed by a committee of experienced practitioners, who then conducted a day-long field exercise for 50 meeting delegates to test the system.

APPROACH 1 : FACTUAL DESCRIPTION OF WEATHERING (MANDATORY)

Standard descriptions should always include comments on the degree, extent and nature of any weathering effects at material or mass scales. This may allow subsequent classification and provides information for separating rock into zones of like character. Typical indications of weathering include:

- Changes in colour
- Reduction in strength
- Changes in fracture state
- Presence, character and extent of weathering products

These features should be described using standard terminology, quantified as appropriate, together with non-standard 'English' descriptors as necessary to describe the results of weathering. At the mass scale, the distribution and proportions of the variously weathered materials (e.g. corestones vs matrix) should be recorded.

Can classification be applied unambiguously ? — No — DO NOT CLASSIFY

Yes

Rock is moderately strong or stronger in fresh state

APPROACH 2 : CLASSIFICATION FOR UNIFORM MATERIALS

Grade	Classifier	Typical Characteristics
I	Fresh	• Unchanged from original state
II	Slightly Weathered	• Slight discolouration, slight weakening
III	Moderately Weathered	• Considerably weakened, penetrative discolouration • Large pieces cannot be broken by hand
IV	Highly Weathered	• Large pieces can be broken by hand • Does not readily disaggregate (slake) when dry sample immersed in water
V	Completely Weathered	• Considerably weakened • Slakes • Original texture apparent
VI	Residual Soil	• Soil derived by in situ weathering but retaining none of original texture or fabric

Is a zonal classification appropriate and is there enough information available? — No — Use Rock Mass Classification if appropriate

Yes

APPROACH 3 : CLASSIFICATION FOR HETEROGENOUS MASSES

ZONE	PROPORTIONS OF MATERIAL GRADES	TYPICAL CHARACTERISTICS
1	100% G I - III (not necessarily all fresh rock)	Behaves as rock; apply rock mechanics principles to mass assessment and design
2	>90% G I - III <10% G IV - VI	Weak materials along discontinuities. Shear strength, stiffness and permeability affected
3	50 to 90% G I - III 10 to 50% G IV - VI	Rock framework still locked and controls strength and stiffness; matrix controls permeability
4	30 to 50% G I - III 50 to 70% G IV - VI	Rock framework contributes to strength; matrix or weathering products control stiffness and permeability
5	<30% G I - III >70% G IV - VI	Weak grades will control behaviour. Corestones may be significant for investigation and construction
6	100% G IV - VI (not necessarily all residual soil)	May behave as soil although relict fabric may be still be significant

Rock is moderately weak or weaker in fresh state

APPROACH 4 : CLASSIFICATION INCORPORATING MATERIAL AND MASS FEATURES

Class	Classifier	Typical Characteristics
A	Unweathered	Original strength, colour, fracture spacing
B	Partially Weathered	Slightly reduced strength, slightly closer fracture spacing, weathering penetrating in from fractures, brown oxidation
C	Distinctly Weathered	Further weakened, much closer fracture spacing, grey reduction
D	Destructured	Greatly weakened, mottled, ordered lithorelicts in matrix becoming weakened and disordered, bedding disturbed
E	Residual or Reworked	Matrix with occasional altered random or "apparent" lithorelicts, bedding destroyed. Classed as reworked when foreign inclusions are present as a result of transportation

APPROACH 5 : SPECIAL CASES

For rocks whose weathering state does not follow the other patterns indicated here, such as karst in carbonates and the particular effect of arid climates

Figure 2. Weak and weathered rock classification proposed by Geological Society Engineering Group Working Party (1995).

Weathering Grade

In the 1970's, Dearman developed a classification system which recognized the weathering grades of granite (Dearman, 1976). The system used the following six categories:

VI Residual Soil
V Completely Weathered
IV Highly Weathered
III Moderately Weathered
II Slightly Weathered
I Unweathered Fresh Rock

Dearman (1976) provides short descriptions of each category, and Bell (1992) provides more detailed descriptions. Dearman and others (1978) summarize engineering and rock mass properties expected for each category, as shown on Table 1. Lee and de Freitas (1989) summarize geotechnical properties for each category, including point load strength, Schmidt hammer rebound, and uniaxial compressive strength. Krank and Watters (1983) summarize slope stability, foundation, building material, and excavation characteristics for each category.

Strength, Change in Strength, and Discontinuity Spacing

Palicki (1997) suggests modifying Dearman's system to assess additional factors besides weathering grade, as shown on Figure 3. Absolute strength, which is measured using field estimates or laboratory values, is added because materials with similar weathering grade could have very different strengths (for example, compare "fresh" granite to "fresh" gypsum). Discontinuity spacing is added to gauge the rock mass behavior, since mass strength and resistance to weathering increase with increasing discontinuity spacing. The "rate of change of strength" is added to include the effects of short-term reaction to water.

FIELD SYSTEMS DESIGNED FOR SHALE

As the most common type of weak rock and as a material which weathers rapidly, shales deserve special attention during field classification. From an engineering standpoint, it is important to identify grain sizes, fissility, and bonding or cementing agents. It is also helpful to distinguish among terms such as "stone," "shale," "mud," "clay," etc. The systems described below were selected on the basis that they intend to imply engineering significance in addition to geologic characteristics.

Maturity of Induration

Gamble (1971) presents a simple breakdown of shale types based on particle size, fissility, and degree of induration, shown in Figure 4 (this classification is based on Twenhofel, 1937 and Underwood, 1967). Although very basic, this system recognizes a progression from sediment, to indurated stone (or shale, if fissile), to metamorphosed rock. No engineering parameters are given, but the user can estimate durability and fissility from the rock name.

Table 1. Properties of weathered granites and gneisses (modified from Dearman, et al, 1978).

Engineering Property	Fresh, I	Slightly Weathered, II	Moderately Weathered, III	Highly Weathered, IV	Completely Weathered, V	Residual Soil, VI
Foundation conditions	Suitable for concrete and earthfill dams	Suitable for concrete and earthfill dams	Suitable for small concrete structures, earthfill dams	Suitable for earthfill dams	Suitable for low earthfill dams	Generally unsuitable
Excavatability	In general, blasting necessary	In general, blasting necessary	Generally blasting needed, but ripping may be possible depending upon the jointing intensity	Generally ripping and/or scraping necessary	Scraping	Scraping
Slope design	1/4:1 H:V	1/2:1 to 1:1 H:V	1:1 H:V	1:1 to 1.5:1 H:V	1.5:1 to 2:1 H:V	1.5:1 to 2:1 H:V
	Benches and surface protection structures are advisable, particularly for more highly weathered material. The presence of through-going adversely oriented structures is not taken into account.					
Tunnel support	Not required unless joints are closely spaced or adversely oriented	Not required unless joints are closely spaced or adversely oriented	Light steel sets on 0.6- to 1.2-m centers	Steel sets, partial lagging, 0.6- to 0.9-m centers	Heavy steel sets, complete lagging on 0.6- to 0.9-m centers. If tunneling below water table, possibility of soil flow into tunnel	Heavy steel sets, complete lagging on 0.6- to 0.9-m centers. If tunneling below water table, possibility of soil flow into tunnel
Drilling RQD (%)	75, usually 90	75, usually 90	50-75	0-50	0 or does not apply	0 or does not apply
Core recovery (%)(NX)	90	90	90	Up to 70, if a high % of core stones, as low as 15 if not	15 as sand	15 as sand
Drilling Rates (m/hr)						
(Diamond NX)	2-4	2-4	8-10	8-10	10-13	10-13
2 1/2 " percussion	5-7	8	12-15	12-15	17	17
Permeability	Low to medium	Medium to high	Medium to high	High	Medium	Low
Seismic velocity (m/sec)	3050-5500	2500-4000	1500-3000	1000-2000	500-1000	500-1000
Resistivity (ohm-m)	340	240-540	180-240	180-240	180	180
	Tends to be determined by joint openness and water-table depth					

Note: Weathering grades (shown here as column headings) are based on Dearman (1976)

STRENGTH SCALE

S-Value	Term	Field Estimation	Unconfined Compressive Strength (MPa)
0	Very Soft	Soil extrudes between fingers when squeezed in hand	<0.04*
1	Soft	Soil easily molded with fingers	0.04 - 0.08
2	Firm	Soil can be molded only by strong pressure of fingers	0.08 - 0.15
3	Stiff	Soil cannot be molded by fingers	0.15 - 0.30
4	Very Stiff	Soil can be indented by finger nail	0.30 - 0.60
5	Very weak rock or hard soil	Brittle or tough, may be broken in hand with difficulty	0.60 - 1.25
6	Weak	Very soft rock-- material crumbles under firm blows with the end of a geological pick	1.25 - 5.0
7	Moderately Weak	Too hard to cut by hand into a triaxial specimen	5.0 - 12.5
8	Moderately Strong	Soft rock-- 5mm indentions with sharp end of pick	12.5 - 50
9	Strong	Hard rock-- hand held specimen can be broken with single blow of geological hammer	50 - 100
10	Very Strong	Very hard Rock-- more than one blow of geological hammer required to break specimen	>100

** The compressive strengths for soils given above are double the unconfined shear strengths.*
For systems composed of two components, use portion composing 30% or more of rock mass.

DISCONTINUITY SIGNIFICANCE SCALE

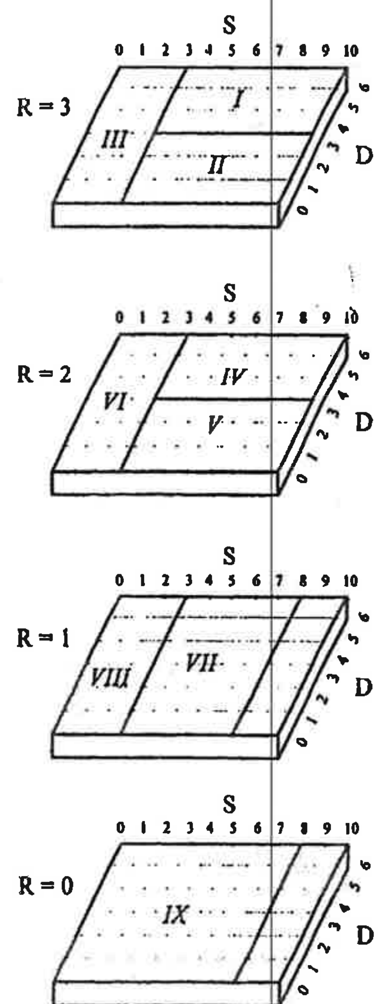
	Spacing		Aperture		Roughness
D-Value	Term	Spacing	Term	Aperture or Thickness	Term
0	Very Narrow	<6mm	Wide	> 200 mm	Polished
1	Narrow	6 - 20 mm	Moderately Wide	60 - 200 mm	Slickensided
2	Moderately Narrow	20 - 60 mm	Moderately Narrow	20 - 60 mm	Smooth
3	Moderately Wide	60 - 200 mm	Narrow	6 - 20 mm	Rough
4	Wide	200 - 600 mm	Very Narrow	2 - 6 mm	Defined Ridges
5	Very Wide	600 mm - 2 m	Extremely Narrow	> 0 - 2 mm	Small Steps
6	Extremely Wide	>2 m	Tight	Zero	Very Rough

Select lowest D-value from the 3 factors.

RATE OF CHANGE IN STRENGTH

R-Value	Term	Description
0	Mud Flakes	Degrades to a mud-like consistency; Sample totally reduced to chips. Original outline of sample not discernible
1	Chips Fractures	Chips of material fall from sides of the sample. Sample may also be fractured. Original outline of the sample is barely discernible; Sample fractures throughout, creating a chunky appearance
2	Slake	Sample parts along a few planar surfaces
3	No Reaction	No discernible effect

30 minute Modified Jar Slake Test
Place sample in jar of water for 30 minutes; choose term which best describes material after soaking



Region I
Strength controls properties

Region II
Fractures control properties

Region III
Soil-like properties

Region IV
Strength controls properties-- may experience slightly accelerated weathering

Region V
Fractures control properties-- may experience slightly accelerated weathering

Region VI
Soil-like properties, susceptable to minor reduction in strength

Region VII
Significant loss in strength may occur, remain aware of original strength and fracturing

Region VIII
Soil-like properties, susceptable to a reduction in strength with time

Region IX
Readily slakes to soil-like material

Figure 3. Classification system proposed by Palicki (1997).

Unindurated	Indurated		After Incipient Metamorphism	Metamorphic Equivalents
	Massive	Fissile		
Silt	Siltstone	Silty Shale		
Mud (unspecified amounts of silt and clay) →	Mudstone	Shale →	Argillite →	Slate, Phyllite or Schist
Clay	Claystone	Clayey Shale		

Figure 4. Field classification for argillaceous (clayey) rock (from Gamble, 1971).

Fissility and Grain Size

Folk (1965) used the system shown in Table 2 to subdivide shales based on fissile characteristics and clay content. Classification using this system generally requires disaggregation of the sample by crushing or by alternate wetting and drying in order to determine clay content.

Table 2. Classification system based on fissility and clay content (from Folk, 1965).

	<33% clay	33-66% clay	>66% clay
Non-fissile:	siltstone	mudstone	claystone
Fissile:	siltshale	mudshale	clayshale

Bonding or Cementing Agents

Underwood (1967) modifies a system proposed by Mead (1936), shown in Figure 5, which classifies shale based on the type of particle bonding, grain size, and cement type. Underwood suggests that "soil-like" shales can be excavated with modern earth-moving equipment (1967-vintage) and that they slake rapidly when subjected to wetting and drying cycles. "Rock-like" shales, on the other hand, might possibly be excavated with conventional equipment, although it is usually more economical to blast them. The "rock-like" shales also "maintain their essential character" when subjected to wetting and drying.

LABORATORY SYSTEMS DESIGNED FOR SHALE AND SLAKING ROCK

On the basis of laboratory tests, many classification systems have been devised for shale and similar rock, most of which recognize the importance of slaking as an indicator of long-term behavior.

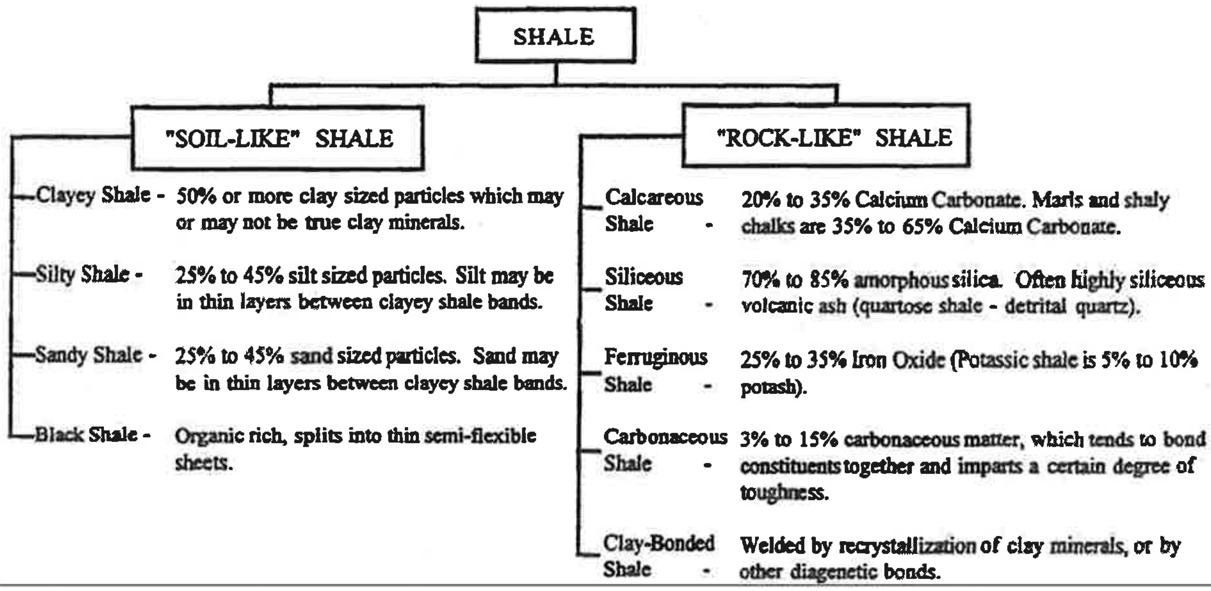


Figure 5. Field classification of shales based on bonding or cementing agents (from Mead, 1936).

Multiple Engineering Properties

Underwood (1967) presents an evaluation scheme based on engineering properties, unrelated to the geologic properties (Table 3). Material is judged as "favorable" or "unfavorable" based on ranges of several engineering properties, and specific engineering problems are identified for each unfavorable property.

Shale Rating System

A system proposed by Franklin (1981) is reproduced as Figure 6. This classification system relies on the slake durability test, followed by the Atterberg limits test for non-durable materials and the point load test for durable materials. It has the advantage of a continuous scale, and provides correlations between the shale rating, R, and construction parameters such as allowable lift thickness, embankment height, slope angle, angle of internal friction, and cohesion.

Office of Surface Mining System

Welsh and others (1991) proposed a durability classification system based on point load strength and free-swell index, shown on Figure 7. This system identifies suitability for rock drains or rock fills based on three rock classes:

Class I - Non-durable and weak rock - This material slakes rapidly, splits apart under sample preparation, has low strength, or exhibits high swelling.

Table 3. Engineering evaluation of shales (modified from Underwood, 1967 and Wood and Deo, 1975).

Physical Properties			Probable in-situ behavior ("X" indicates expected problems)						
Laboratory Tests and In Situ Observations	Unfavorable Range of Values	Favorable Range of Values	High Pore Pressure	Low Bearing Capacity	Tendency to Rebound	Slope Stability Problems	Rapid Slaking	Rapid Erosion	Tunnel Support Problems
Compressive strength (psi)	50 to 300 (0.3 to 2 MPa)	300 to 5,000 (2 to 34 MPa)	X	X					
Modulus of elasticity (psi)	20,000 to 200,000 (140 to 1400 MPa)	200,000 to $2X10^6$ (1400 to 14000 MPa)		X					X
Cohesive strength (psi)	5 to 100 (0.03 to 0.7 MPa)	100 to >1,500 (0.7 to >10 MPa)			X	X			X
Angle of internal friction (degrees)	10 to 20	20 to 65			X	X			X
Dry density (pcf)	70 to 110 (1.1 to 1.8 g/cm^3)	110 to 160 (1.8 to 2.6 g/cm^3)	X					X(?)	
Potential swell (%)	3 to 15	1 to 3			X	X		X	X
Natural moisture content (%)	20 to 35	5 to 15	X			X			
Coefficient of permeability (cm/sec)	10^{-5} to 10^{-10} ($3x10^{-7}$ to $3x10^{-12}$ ft/sec)	$>10^{-5}$ ($>3x10^{-7}$ ft/sec)	X			X	X		
Predominant clay minerals	Montmorillonite or illite	Kaolinite or chlorite	X			X			
Activity ratio = (Plasticity index / Clay content)	0.75 to >2.0	0.35 to 0.75				X			
Wetting and drying cycles	Reduces to grain sizes	Reduces to flakes					X	X	
Spacing of rock defects	Closely spaced	Widely spaced		X		X		X(?)	X
Orientation of rock defects	Adversely	Favorable		X		X			X
State of stress	>Existing overburden load	=Overburden load			X	X			X

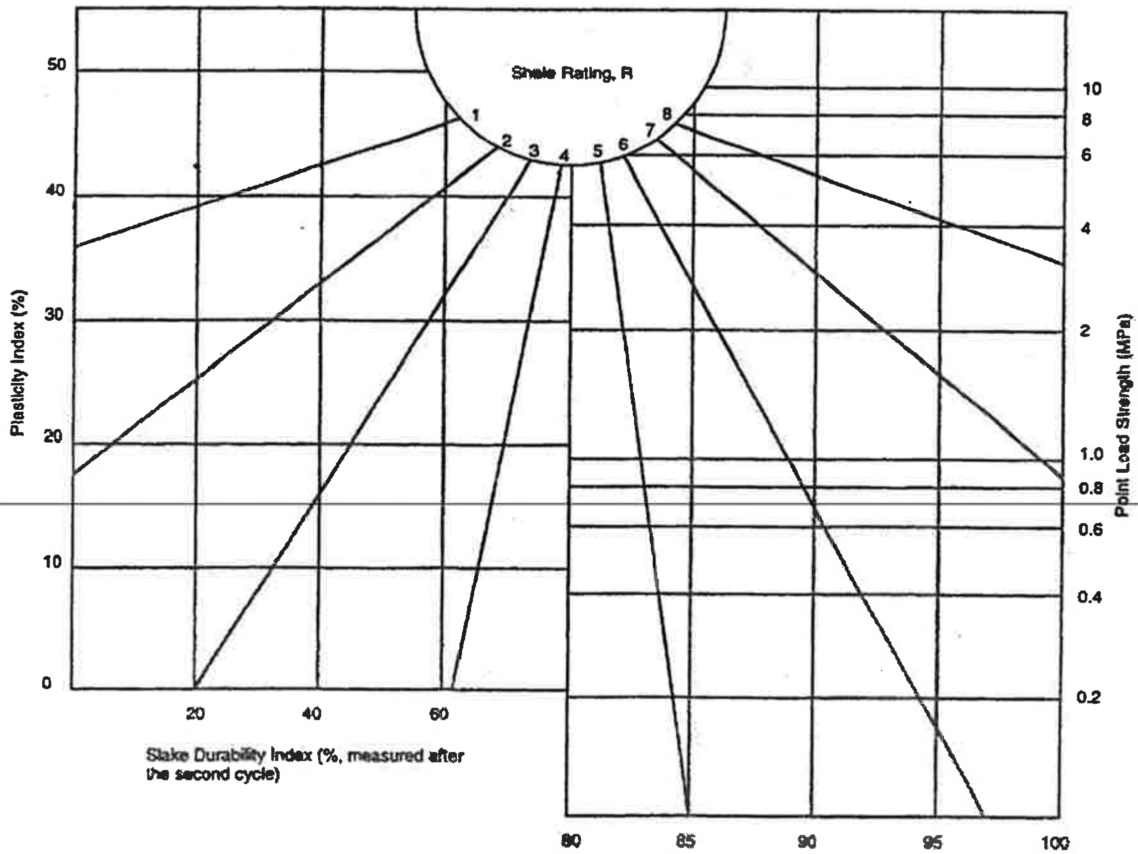


Figure 6. Franklin's (1981) system for classification of shales for embankment construction, based on the slake durability test, followed by the plasticity index for non-durable materials and the point load strength test for durable materials.

Class II - Conditionally suitable for drains - This material is suitable for drains only if it is sandstone. It shows moderate strength and low swelling.

Class III - Durable and strong - This material is suitable for drains and fills.

Slake Index System

Santi (1995a) proposed a durability classification system based on one- and five-cycle slake index values, shown on Figure 8. One-cycle slake index values represent the current soil:rock ratio, and five-cycle values represent long-term durability behavior. Santi (1995b) outlines supplemental testing and ideal mitigation methods for each classification category.

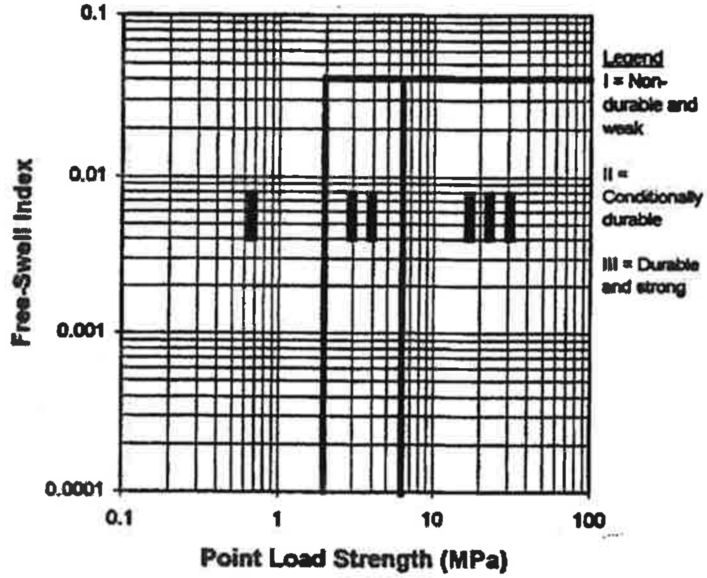


Figure 7. Office of Surface Mining classification system for rock fills and rock drains (from Welsh, et al., 1991).

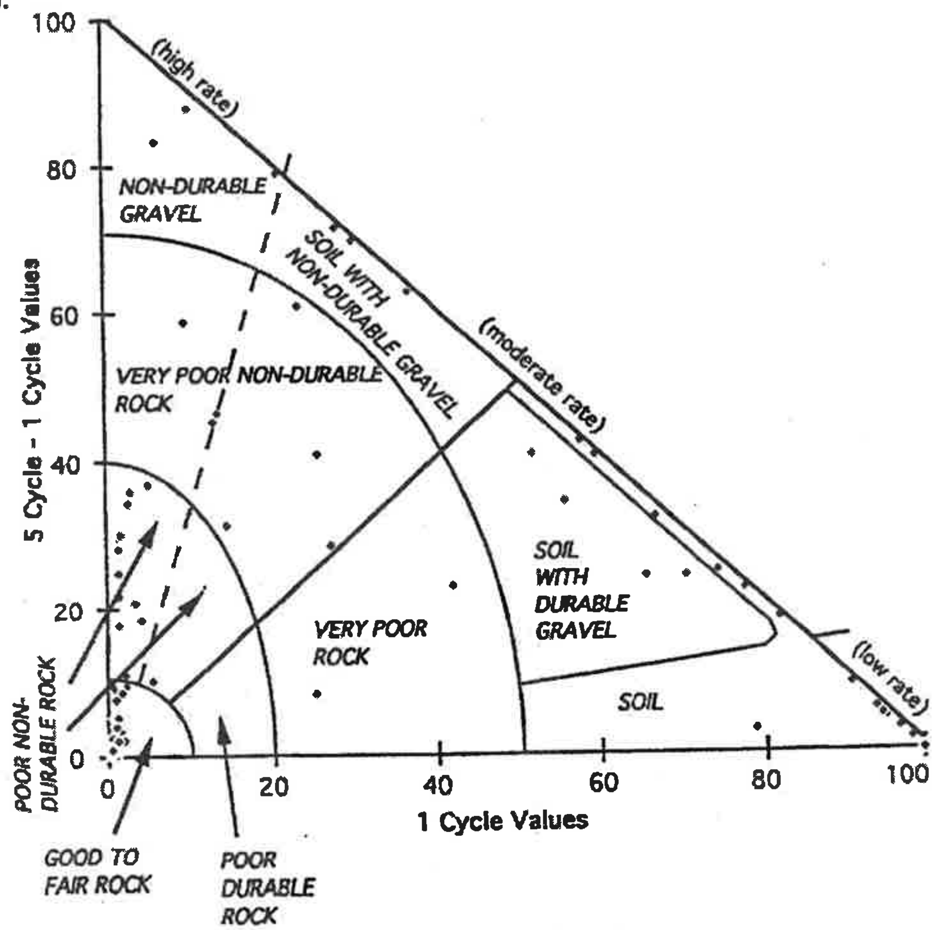


Figure 8. Classification system based on one- and five-cycle slake index test results (from Santi, 1995a).

Other Laboratory-Based Systems

Gamble (1971) proposes a system based on plasticity index and second-cycle slake durability, shown on Figure 9. Wood and Deo (1975) propose a system, shown on Figure 10, based on jar slake, slake durability and modified soundness. Morgenstern and Eigenbrod (1974) propose a system, shown on Figure 11, based on undrained shear strength, softened shear strength after immersion, rate of slaking, and amount of slaking.

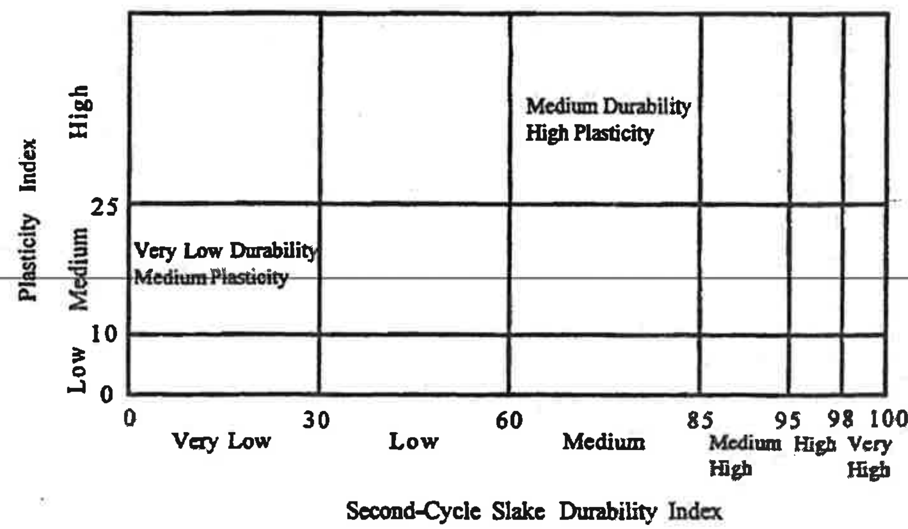


Figure 9. Durability-plasticity classification system based on laboratory tests (from Gamble, 1971).

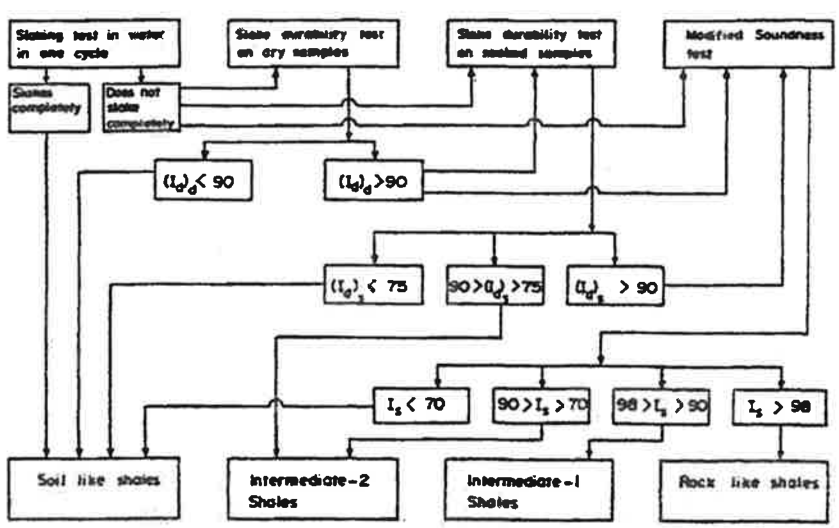
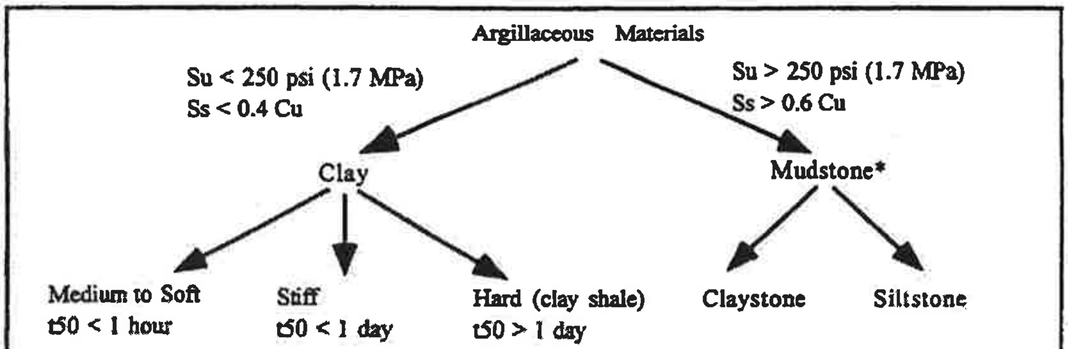


Figure 10. Classification of Indiana shales for embankment construction (from Wood and Deo, 1975).



Engineering Classification of Argillaceous Materials

Amount of Slaking

Rate of Slaking	very low $W_L<20$	low $20<W_L<50$	medium $50<W_L<90$	high $90<W_L<140$	very high $W_L>140$
slow: $\Delta I_L < 0.75$					
fast: $0.75 < \Delta I_L < 1.25$				high fast	
very fast: $\Delta I_L > 1.25$		low very fast			

Classification Based on Slaking Characteristics

Where:
* shale if fissile
Su = undrained shear strength
Ss = softened shear strength after immersion
t50 = time for 50% strength loss
W_L = liquid limit
ΔI_L = change in liquidity index following a 2 hour slaking cycle

Figure 11. Classifications based on strength, softening, and slaking characteristics (from Morgenstern and Eigenbrod, 1974).

COMPARISON OF CLASSIFICATION SYSTEMS

The classification systems summarized above are compared in Table 4. The purpose of this comparison is to evaluate the ease of application, breadth of application, and the usefulness of the engineering properties indicated by each system. A discussion of each issue evaluated in Table 4 is presented below.

A. Can the system be applied in the field without any special equipment?

A true field classification should not require special equipment. Normal field equipment would include items such as a rock hammer, hand lens, acid bottle, ruler, and color charts.

Table 4. Comparison of the classification systems evaluated in this paper.

	Weathered Rock Systems				Field Systems for Shale			Laboratory Systems for Shale and Slaking Rocks						
Classification System:	1. Afrouz	2. Geological Society	3. Dearman	4. Palicki	5. Gamble (field)	6. Folk	7. Mead	8. Underwood	9. Franklin	10. Welsh	11. Santi	12. Gample (lab)	13. Wood and Deo	14. Morgenstern and Eigenbrod
Figure or Table in this paper:	Figure 1	Figure 2	Table 1	Figure 3	Figure 4	Table 2	Figure 5	Table 3	Figure 6	Figure 7	Figure 8	Figure 9	Figure 10	Figure 11
A. Can it be applied in the field <u>without</u> special equipment?	X	X	X	X	X	X	X							
B. Does it require laboratory work?								X	X	X	X	X	X	X
C. Is the classification confirmed and enhanced with laboratory data?	X		X	X		X	X	X	X	X	X	X	X	X
D. Applies to <u>weak</u> rock	X	X		X	X	X	X	X	X	X	X	X	X	X
E. Applies to <u>weathered</u> rock		X	X	X							X			
F. Applies to <u>rock masses</u>		X		X							X			
G. Evaluates Strength	X		X	X				X	X	X				
H. Evaluates Durability								X	X	X	X	X	X	X
I. Suggests a range of <u>quantitative</u> engineering properties	X		X					X	X					
J. Suggests a range of <u>qualitative</u> engineering properties		X		X						X	X	X	X	X

Atypical, but acceptable, equipment for field classification systems might also include items such as a Schmidt Hammer, pocket penetrometer, Point Load testing apparatus, and beakers and triple-beam balances. All of the weathered rock and field systems meet this criterion, while none of the laboratory systems meet it.

B. Does the system require laboratory testing to apply it?

Unusual, heavy, externally powered, or sensitive equipment cannot be used in the field. Similarly, long test time would preclude field use of some simple equipment, such as slake index testing materials. All of the laboratory systems and none of the field or weathered rock systems meet this criterion.

C. Is the classification confirmed and enhanced with laboratory data?

The usefulness of a classification system is greatly expanded if there is a provision to refine and improve the confidence of the classification with laboratory tests. This quality also verifies that the field classification attempts to gauge the same properties that are deemed important for laboratory classification. Most systems provide for incorporation of laboratory data. Those systems proposed by the Geological Society and by Morganstern and Eigenbrod do not.

D. Does the system apply to weak rock?

An affirmative answer to this question indicates that the system applies to shales and mudstones, and perhaps other homogeneous, yet weak material (less than approximately 20 MPa unconfined compressive strength). These systems tend to focus on slaking characteristics and estimates of low compressive strengths. All systems except Dearman have some provision for addressing weak rock.

E. Does the system apply to weathered rock?

These systems recognize the range of strength and behavior properties for continuum from fresh rock through a weathering profile to soil. These systems show less emphasis on slaking characteristics than those directed solely at weak rock. Only a few of the evaluated systems have a provision for weathered rock: Dearman, Palicki, Santi, and that proposed by the Geological Society.

F. Does the system apply to rock masses?

These systems address the influence of both intact, unweathered blocks, and the weaker, weathered matrix between the blocks. This requires a larger scale view than a single sample or even limited outcrop exposure. These systems are the same as those which address weathered rock, with the exception that Dearman has no mechanism to assess rock masses.

G. Does the system evaluate rock strength?

These systems either require point load testing or require an estimate of rock strength. In all cases, detailed laboratory testing results (such as unconfined compression tests) can be used in the system as well. The systems which evaluate rock strength are Afrouz, Dearman, Palicki, Underwood, Franklin, and Welsh.

H. Does the system evaluate rock durability?

All of the laboratory systems address rock durability, using jar slake, slake index, slake durability, free swell, and rate of slaking tests. Of the field tests, only Palicki incorporates durability, in the form of a modified jar slake test. None of the weathered rock systems evaluate durability.

I. Does the system suggest a range of quantitative engineering properties?

These systems either intrinsically provide quantitative estimates of compressive strength, durability, and other factors, or they have later been linked to databases containing these values.

Systems proposed by Afrouz, Dearman, Underwood, and Franklin were judged to meet this criteria.

J. Does the system suggest a range of qualitative engineering properties?

Although less desirable than quantitative values, some systems categorize materials in strength and durability ranges, often using quantitative values as boundaries for the ranges. Systems which meet this provision include those proposed by the Geological Society and by Palicki, and all the laboratory systems excluding Franklin, which was judged to the quantitative. No field systems were considered to provide either quantitative or qualitative engineering information, although with experience, the user should be able to use the field systems to estimate engineering behavior.

MOST VERSATILE SYSTEMS

The most desirable properties for a classification system include the applicability to weak and weathered rock, as well as rock masses, the incorporation of genetic and descriptive information, the ability to use in the field without special equipment, the ability to insert laboratory values into the system to improve confidence, and the correlation to quantitative engineering properties. The engineering properties assessed should include strength of blocks, estimate of mass strength based on discontinuity characteristics, durability, and identification of other adverse engineering characteristics.

Dearman's system meets many of these criteria, but it cannot be applied to weak rocks or rock masses. Palicki addresses this shortcoming, but by dropping the description of weathering grade (as in Dearman), he loses the advantages of the correlation to quantitative values established for Dearman. Palicki's system could benefit from using standard jar slake categories, and by including correlation between jar slake and more quantitative laboratory slake tests.

The system proposed by the Geological Society is an excellent descriptive method, although it lacks the mechanism to incorporate laboratory data or to link it to quantitative values.

The remaining systems either lack applicability to weathered rock or rock masses, neglect to evaluate either strength or durability, or focus too heavily on laboratory work to be suitable for field use.

PROPOSED MODIFICATIONS TO ENHANCE APPLICABILITY

At the risk of overcomplicating the field geologist's job, the most thorough method to classify weak and weathered rock materials is probably a combination of the systems presented above, with some modifications to ensure completeness. If the practitioner recognizes the inherent complexity, variability, and temporal nature of the properties of weak and weathered rock, the careful attention in the field seems more a matter of prudence than overkill.

To provide the greatest quantity of the most useful information for the least work, the author recommends that the user classify weak and weathered rock material as described below.

1. Apply the Geological Society system, with some modifications so that it will correspond more closely to Dearman. First, Approach 2, which uses identical categories as Dearman, should be viewed as equivalent to Dearman, so that the qualitative and quantitative engineering information from Dearman and others (1978), Lee and deFreitas (1989), and Krank and Watters (1983) may be used.

Second, Approach 4 should be modified to better correspond to Dearman. Recognizing that Approach 4 will be used for traditionally weak rocks, the categories in Approach 4 should match with higher categories in Dearman. For example, Class A in Approach 4 should be considered the engineering equivalent to Category III (Moderately Weathered) in Dearman. Class B and C should be combined, since they show little difference, and considered equivalent to Category IV in Dearman. Class D should match with Category V, and Class E should match with Category VI. This proposed matching is summarized in Table 5:

Table 5. Proposed correlation between Dearman (1976) and Geological Society (1995) systems

Dearman Category	Geological Society Grade Approach 2 (weathered rock)	Geological Society Class Approach 4 (weak rock)
VI	VI	E
V	V	D
IV	IV	B or C
III	III	A
II	II	--
I	I	--

For materials composed of both blocks and matrix, or showing a range of degrees of weathering, Approach 3 should be added to either Approach 2 or 4 to account for estimated percentages within each grade or class. In this way, an assessment of rock mass properties and the influence of weathered matrix material can be inferred.

2. Apply Palicki's system, revised to measure "rate of change of strength" using standard jar slake categories. A modified diagram is shown as Table 6, which includes correlations to laboratory tests of slake durability and slake index from Santi and Higgins (in review) and Santi (1995b), respectively. The information from this system will enhance the Geological Society classification by providing quantitative information on strength, influence of discontinuities, and reaction to water.

Table 6. Proposed modifications to Palicki's (1997) "Rate of Change in Strength" table.

R Value	Term	Description	Slake Durability*	Slake Index**	Long Term Treatment***
1	Mud	Degrades to a mud-like consistency.	0-15	75-100	Soil
2	Many Chips, Flakes	Sample totally reduced to chips. Original outline of sample not discernible.	15-25	40-90	Soil
3	Few Chips	Chips of material fall from the sides of the sample. Sample may also be fractured. Original outline of sample is barely discernible.	25-40	25-70	Very Poor Rock
4	Many Fractures	Sample fractures throughout, creating a chunky appearance.	40-55	5-30	Poor, Durable Rock
5	Few Fractures, Slabs	Sample parts along a few relatively planar surfaces.	55-70	5-15	Good to Fair Rock
6	No Reaction	No discernible effect.	70-100	0-10	Good to Fair Rock

* based on Santi and Higgins (in review)

** based on Santi (1995b)

***based on Santi (1995b). "Long Term Treatment" is the expected behavior of the material following long term exposure to weathering.

By combining revised versions of these two systems, the user will produce a thorough descriptive classification which incorporates simple field tests for strength and durability. This classification also predicts approximate strength and durability values, as well as behavior in many engineering situations. Laboratory tests will improve the accuracy of the classification.

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Clay-Rich Joints And Fractures In Tectonically Disturbed Columbia River Basalts Perform As Weak Rock During TBM Mining Of Tri-Met's Westside Light Rail Project Tunnel In Portland, Oregon

MICHAEL L. CUMMINGS
Department of Geology, Portland State University, P. O. Box 751, Portland, OR 97207

KEN WALSH
Parsons Brinckerhoff Quade & Douglas, Inc., 2140 SW Jefferson St., Suite 200, Portland, OR 97201

REKA K. GABOR
Department of Geology, Portland State University, P. O. Box 751, Portland, OR 97207

ABSTRACT

Cooling joints and tectonic fractures in flows of the Columbia River Basalt Group, especially where lined by smectite clays, resulted in unexpected problems during mining of twin tunnels by a tunnel boring machine (TBM) through the Tualatin Mountains, Portland, Oregon. The intraflow transition from widely spaced joints in the colonnade to irregular closely spaced joints in the overlying entablature was particularly problematical where infilled with clay. This combination produces extreme contrast in mass properties between the thin clay seams and the hard, dense basalt. The TBM dislodged blocks of solid basalt from the weak clay and these blocks rolled around on the cutting heads producing excessive wear and reducing rates of excavation. In some tectonic crush zones where altered basalt is surrounded by a smectite clay-rich matrix the TBM performed as expected for rock with weak mass properties. Alteration of the basalt reduces the contrast in mass properties between the basalt and clay-rich matrix.

INTRODUCTION

Excavation of twin 3-mile long tunnels, 21.29 feet in diameter, through the Tualatin Mountains was the largest and most complex aspect of the 18-mile expansion of the light rail system (MAX) in Portland, Oregon (Figure 1). A 278-foot-long tunnel boring machine (TBM) nicknamed Bore-Regard was utilized in construction of approximately sixty percent of the tunnel lengths. Three quarters of the tunnel is within flows of the Columbia River Basalt Group (CRBG) where zones of fractured and weathered basalt greatly reduced penetration rates and caused unexpected wear to cutting surfaces of the TBM during mining of the west-bound tunnel.

Altered basalt in tectonic crush zones and weathered basalt approximate "weak rock" characteristics and resulted in reduced rates of penetration and excessive caving of the ceiling. In addition, other troublesome zones contained dense, hard basalt blocks surrounded by clay mineral-rich seams. In these settings, the basalt blocks were dislodged by the cutting heads and rolled around the cutting surface, reducing penetration rates and increasing wear rates. Although the basalt is hard and dense and should have been handled by the TBM, the clay-rich seams dramatically altered the physical characteristics of the rock mass. Due to these construction problems, the clay mineralogy was examined to assess the impact of clay mineralogy and its distribution on the progress of the TBM.

In this paper, we present a brief overview of the tunnel project, describe the pre-construction rock mass characterization, examine progress of the TBM from construction records under different rock mass conditions, present the clay mineralogy of selected sites within the east-bound tunnel, and discuss the conditions that lead to the difficulties encountered during construction.

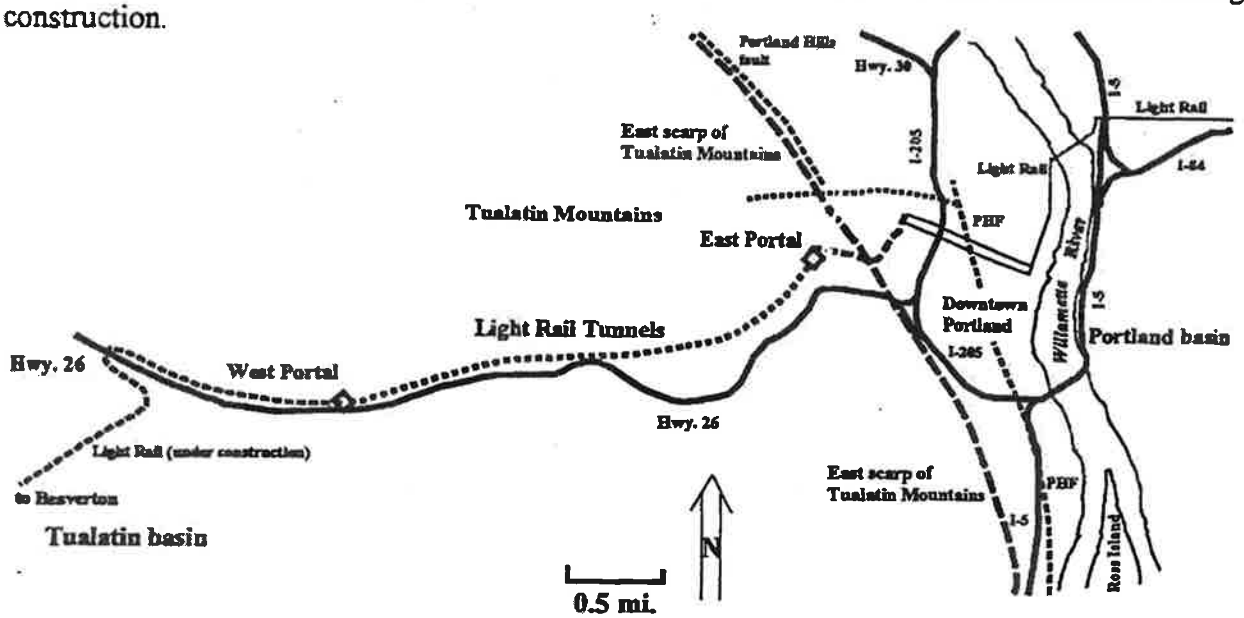

Figure 1: Location map of the light rail tunnels through the Tualatin Mountains, Portland, Oregon.

OVERVIEW OF THE PROJECT

The $944 million expansion of the light rail system in Portland, Oregon is 75 percent Federally funded. The remaining 25 percent is funded by local bond measures, Oregon lottery funds, and contributions by local governments. The operator of the system, the Tri-County Metropolitan Transportation District of Oregon (TRI-MET), expects to start service on the 18-mile expansion in the Fall of 1998. This addition completes a total of 33 miles of light rail in the Portland metropolitan area. The general contractor is a joint venture by Frontier-Kemper and Taylor Brothers. Design and construction management was performed by Parsons Brinckerhoff Quade & Douglas, Inc. and Parson Brinckerhoff Construction Services.

In July, 1993 construction of twin tunnels through the Tualatin Mountains inaugurated the expansion of the light rail system. Work started from the east portal of the west-bound tunnel. After mechanical excavation of approximately 200 feet of rocks with Q-rating generally poor to fair, the TBM was introduced into the tunnel. On December 29, 1995 the TBM "holed-through" into the mile of the west-bound tunnel that had been excavated from the west by conventional mining methods. After four months of repairs and redesign, the TBM started operating in the east-bound tunnel on April 5, 1996 and "holed-through" on August 13, 1996. The TBM excavated approximately 4 miles of the 6 total miles of tunnel.

The average daily rate of penetration for the west-bound tunnel was 45 feet per day (three-eight hour shifts) and was 76 feet per day in the western half and 99 feet per day in the eastern half of the TBM drive of the east-bound tunnel. The maximum penetration was 151 feet in a single day in the west-bound tunnel. A maximum of just over 200 feet in one day was excavated in the east-bound tunnel with several days over 180 feet. Under the worst conditions, the penetration in a single day fell to zero.

GEOLOGY OF THE TUNNEL SITE

The Tualatin Mountains form the western margin of the Portland basin and separate the Portland basin (Figure 1) from the larger Tualatin basin to the west. Based on topography, water-well logs, and geophysical data, the boundary between the Tualatin Mountains and the Portland basin is the Portland Hills fault, which is not exposed in the Portland area.

The Tualatin Mountains are an asymmetrical anticline that has been greatly modified by faulting. Transverse thrust faults, developed early in the growth of the Tualatin Mountains, initiated within low-amplitude transverse anticlines and synclines that were growing concurrently with eruption of flows of the CRBG during middle Miocene time. The thrust faults continued to deform in post-middle Miocene time along the limbs of the folds. Younger northwest and northeast striking sets of normal faults displace the thrust faults. Beeson and Tolan (1990), Beeson and others (1991), Yelin and Patton (1991), and Blakely and others (1995) interpret the Tualatin Mountains as part of a larger northwest-trending, dextral wrench fault zone.

The stratigraphy of the Tualatin Mountains is dominated by the Grande Ronde Basalt and Frenchman Springs Member of the Wanapum Basalt of the CRBG. CRBG flows have physical characteristics that, in combination with geochemical and paleomagnetic properties, are used to identify individual flows, formational members, and formations. Flows of the CRBG display rock mass characteristics that vary with stratigraphic position and flow zone within individual flows. Each basalt flow is subdivided on the basis of textures and jointing patterns into flow zones. In general, a basalt flow has three main zones. An upper vesiculated zone that may also be brecciated (flow-top breccia), a dense flow interior zone where vesicles are relatively rare and cooling joints are prominent, and a basal zone that is vesiculated, locally brecciated, and may show considerable variation in thickness. The textures and characteristics of these zones vary as a function of eruption, emplacement, and cooling conditions.

The degree of subaerial weathering of CRBG flows varies considerably in western Oregon. In the most extreme cases, the flows have been bauxitized (Cummings and Fassio, 1990). Although the flow top zone and the upper part of the flow interior zone of individual flows may show variable degree of weathering, two widespread weathering horizons are recognized in western Oregon. Between the Grande Ronde Basalt and Wanapum Basalt is the deeply weathered Vantage horizon, a locally significant unconformity that developed during a hiatus in basalt eruptions after about 15.6 Ma. Weathering of the basalt is also noted at the unconformity that separates flows of the CRBG from younger interlayered Boring Lava and siltstones/mudstones.

The Boring Lava erupted locally from vents throughout the Portland Metropolitan Area. The Boring Lava is light-gray to gray and contains abundant fine, irregularly-shaped gas cavities between lath-shaped crystals of plagioclase feldspar. Olivine is present as a phenocryst. The flows in the vicinity of the tunnel are believed to be Pleistocene in age (Beeson and others, 1991). Immediately above the CRBG in the tunnel area, mudstone, the dominant lithology, is interlayered with near-source volcanic deposits. Upward, lava flows become the dominant lithology. Due to the high rock mass quality of the Boring Lava, the tunnel alignment, mined by conventional methods, was moved to pass through Boring Lava flows.

PRE-CONSTRUCTION ROCK MASS CHARACTERIZATION

The alignment for the tunnels was investigated using a combination of borings, field mapping, and geophysical methods. Engineering properties of samples collected from borings and outcrops were determined and reported as part of the package of materials provided to contractors during the bid process (Geotechnical Interpretation Report, 1993). In this section, pre-construction information on rock mass properties of the portion of the tunnel excavated by TBM is described.

As part of site characterization, eighty-seven borings were drilled along the tunnel alignment. The location of borings was constrained, in part, by land access and in some cases borings were offset as much as 350 feet from the tunnel alignment and gaps between borings were as great as 850 feet in some areas. Although the pre-construction alignment characterization was found to be accurate, several high-angle faults and tectonic crush zones were not detected and the construction problems produced by clay-rich seams in cooling joints were not anticipated. An array of tests were conducted on core materials recovered from the borings. The following data are summarized from the pre-bid Geotechnical Interpretive Report (1993).

Engineering Properties

Table 1 contains a summary of engineering properties for the Grande Ronde Basalt and, for comparison purposes, Frenchman Springs Member of the CRBG along the tunnel alignment. The greater number of tests for Grande Ronde basalt reflects its importance in the alignment. The generally higher values for the Grande Ronde basalt relative to the Frenchman Springs basalt relates to the textures of these units. The Grande Ronde basalt is very fine grained and relatively uniform in texture in the flow interior zones. The Frenchman Springs basalt is coarser grained with abundant small, irregularly shaped cavities between crystal grains and glass. The glass content in Frenchman Springs flows is higher than in Grande Ronde flows. In addition to these differences, the Frenchman Springs flows are commonly more weathered than Grande Ronde flows.

Also noted in Table 1 are the differences in engineering properties between the flow interior zone and the flow-top breccia zone. The data are for Grande Ronde flows and indicate lower values for engineering properties in all categories in flow-top breccia zones relative to flow interior zones.

Table 1: Summary of engineering properties of flows of the Columbia River Basalt Group along the alignment through the Tualatin Mountains, Portland, Oregon.

Engineering Property	Frenchman Springs (flow interior)	Frenchman Springs (flow-top breccia)	Grande Ronde (flow interior)	Grande Ronde (flow-top breccia)
Unit Weight (pcf)				
Average	No data	No data	174.0	125.9
Range	No data	No data	166.4-179.1	123.2-127.7
No. of tests	0	0	12	3
Unconfined Strength (psi)				
Average	16,600	No data	23,750	1,050
Range	16,600	No data	6,800-49,650	400-2,180
No. of tests	1	0	23	5
Point Load (psi)				
Average	7,420	No data	20,350	12,430
Range	2,880-12,120	No data	1,350-40,560	70-25,010
No. of tests	3	0	199	9
Young's Modulus (10^6 psi)				
Intact				
Laboratory	5.3	No data	4.9	0.1
Average	5.3	No data	0.7-8.2	0.1
Range	1	0	7	1
No. of tests				
RQD (%)				
Average	30	0	31	33
Median	38	0	17	18
Range	0-50	0	0-100	0-100
No. of runs	17	3	624	132
Typical Weathering	Moderate to very severe	Moderate to very severe	Fresh to moderate	Moderate to very severe
Typical Discontinuity Characteristics				
Orientation	Random	Random	Random	Random
Spacing	<2 to 8 inches	<2 to 8 inches .	<2 to 8 inches	<2 to 8 inches
Width	Up to several inches	Up to several inches	Up to several inches	Up to several inches
Roughness	Slightly rough to rough	Slightly rough to rough	Slightly rough to rough	Slightly rough to rough
Planarity	Wavy to undulating	Wavy to undulating	Wavy to undulating	Wavy to undulating
Weathering	Fresh to moderate	Fresh to complete	Fresh to moderate	Fresh to complete
Infilling	Clean to clay and mineral fill	Clean to clay and mineral fill	Clean to clay and mineral fill	Clean to clay and mineral fill

Terzaghi, Q, and RQD Classification

Frequency plots for length of core classified under the Terzaghi (1946) ground classification, rock quality designation (RQD), Barton's Q classification, joint set number (Jn), and joint roughness number (Jr) for Grande Ronde flow interiors and flow-top breccia zones are presented in Figures 2 to 6 (Geotechnical Interpretive Report, 1993). The Terzaghi classification indicates the Grande Ronde Basalt is moderately jointed to moderately blocky and seamy along much of the tunnel alignment (Figure 2). Barton's joint set number (Jn) was applied only where discrete tectonic joint sets could be clearly differentiated from cooling joints (Geotechnical Interpretive Report, 1993). Since cooling joints are abundant, particularly in the entablature of

CRBG flows, the Jn values under estimate the extent of jointing in the basalt. Where applied, Barton's joint set number (Jn) (Figure 3) indicates one joint set plus random (Jn = 2), two joint sets (Jn = 4), and two joint sets plus random (Jn = 6) predominate in flow interior zones, but that some areas of three joint sets plus random (Jn = 12) and four or more sets, random, heavily jointed "sugar cube" patterns (Jn = 15) are present.

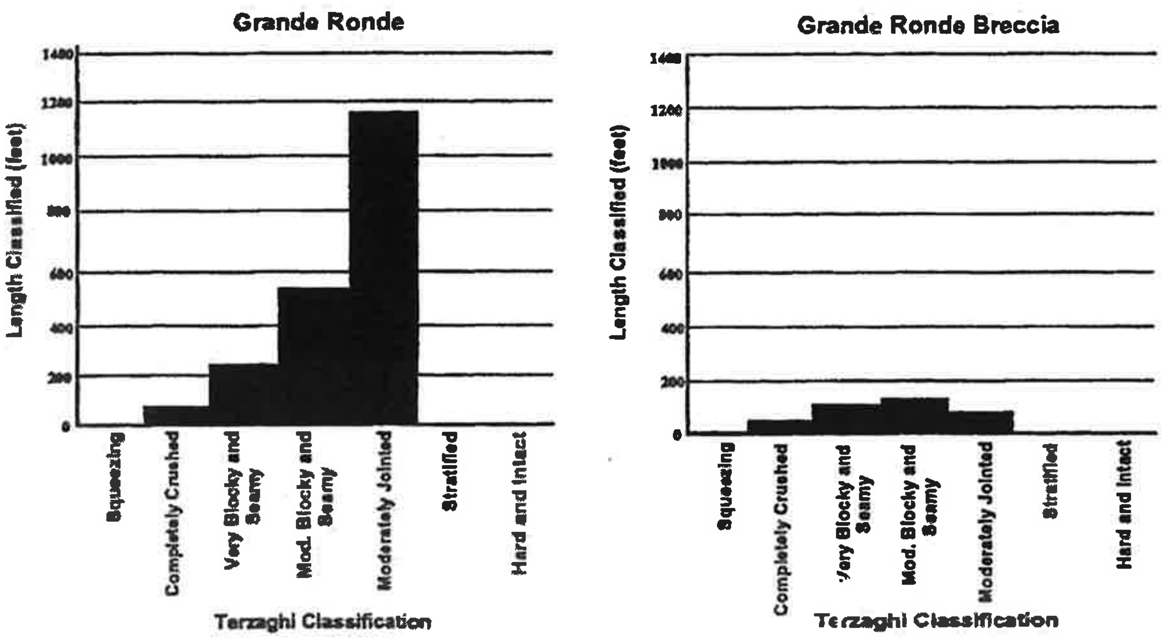

Figure 2: Terzaghi classification for flow interior (left) and flow-top breccia (right) zones of Grande Ronde Basalt (Technical Interpretive Report, 1993).

The roughness of the joints was determined for each core run. The Barton joint roughness number (Jr) (Figure 4) indicate most joints are rough, irregular, or undulating (Jr=3). However, significant lengths of core contain joints that were classified with Jr = 1.5 (slickensided, undulating, or rough or irregular, planar) and Jr = 1.0 (zone containing clay minerals thick enough to prevent rock wall contact). The Barton alteration number (Ja) indicates the walls of joints in flow interior zones are unaltered to slightly altered (Ja = 1 and 2). The mineral coatings that may be present are non-softening. A second grouping of joints have non-softening clay fractions that are less than or equal to 0.04-0.08 inches thick (Ja = 3 and 4). A third population of joints have Ja values of 6 and 8. In these cases the filling is less than 0.2 inches thick and the fillings are softening clay mineral fillings (Ja = 8) or non-softening clay mineral fillings (Ja = 6).

Barton's rock mass classification system (Q) is shown in Figure 5. Approximately 50 percent of the core was rated as fair to extremely good with fair and good the dominant designations. Poor to very poor Q values were assigned to approximately 40 percent of the core. Approximately 70 percent of the core was assigned a rock quality designation (RQD) of very poor to poor (Figure 6).

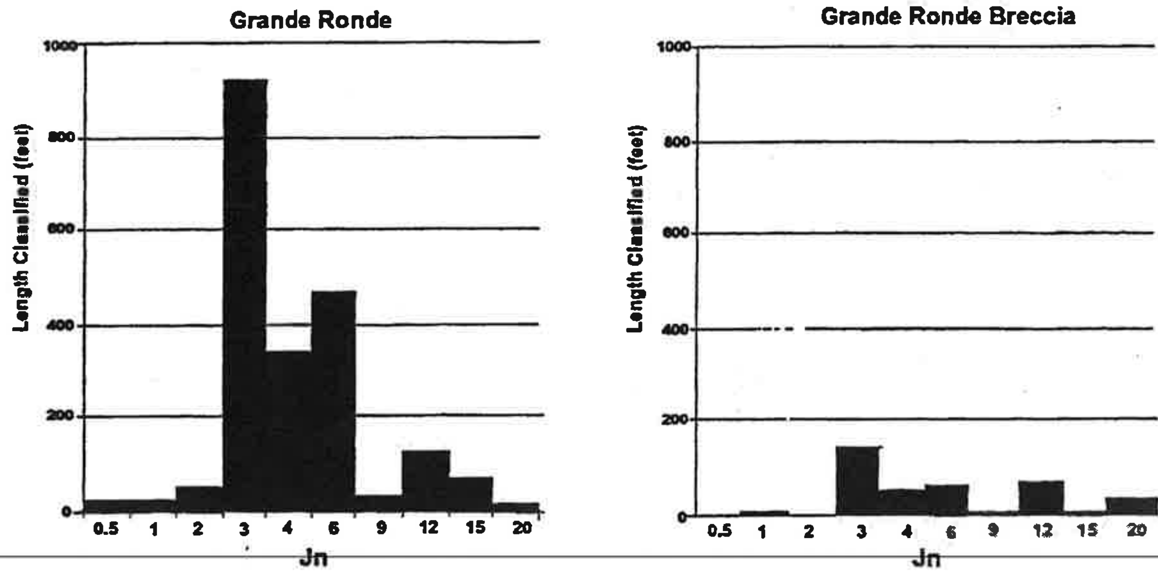

Figure 3: Barton's joint number classification (Jn) for flow interior (left) and flow-top breccia (right) zones of Grande Ronde Basalt.

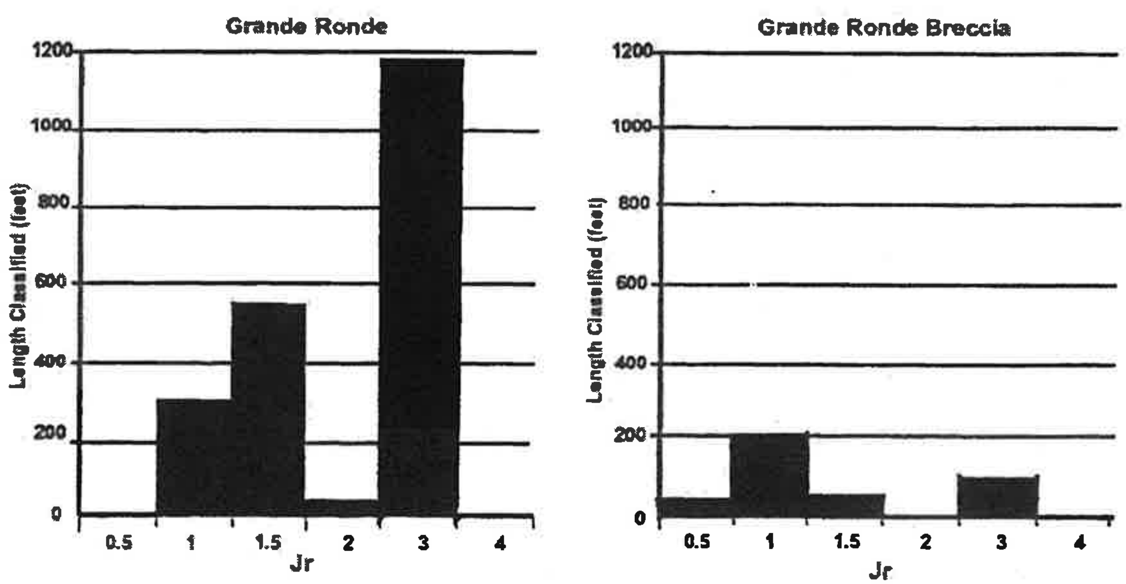

Figure 4: Barton's joint roughness number (Jr) for flow interior and flow top breccia zones of Grande Ronde Basalt (Geotechnical Interpretive Report, 1993).

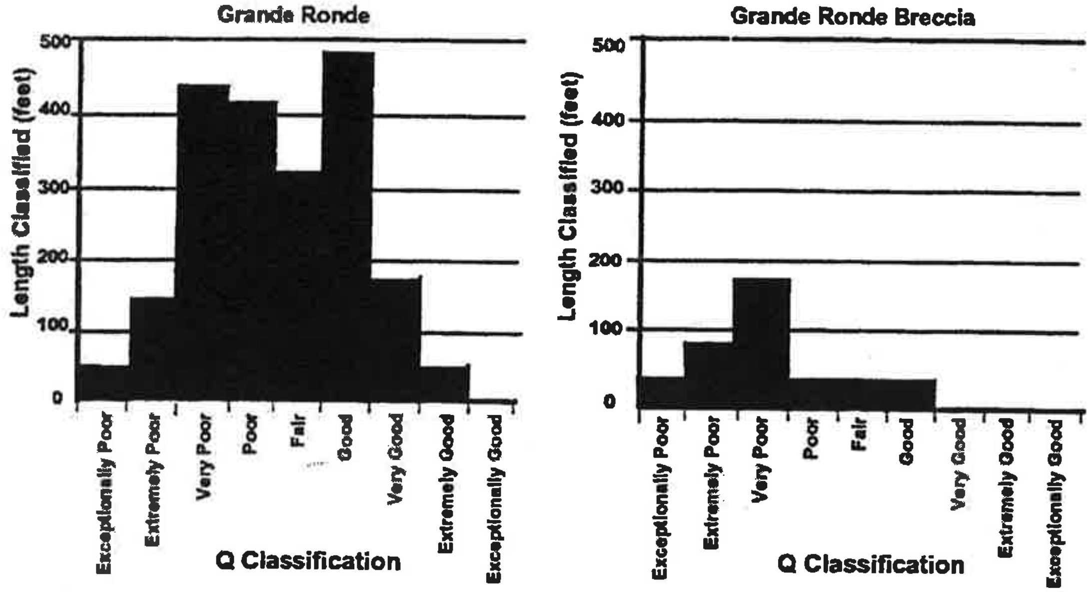

Figure 5: Barton's Q classification for flow interior (left) and flow-top breccia (right) zones of Grande Ronde Basalt (Geotechnical Interpretive Report, 1993).

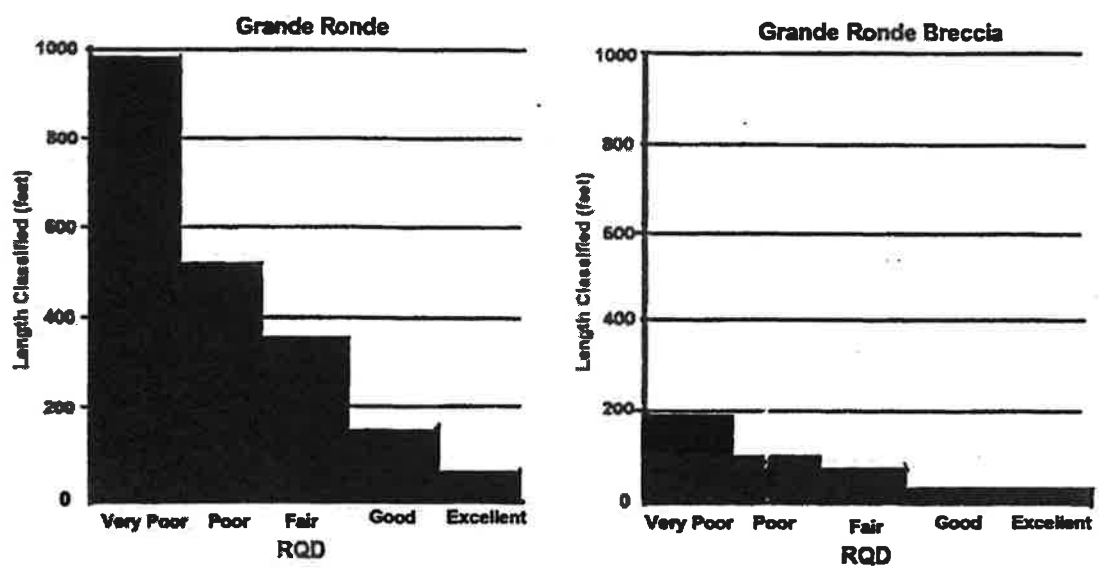

Figure 6: Rock quality designation (RQD) for flow interior (left) and flow-top breccia (right) zones of Grande Ronde Basalt (Geotechnical Interpretive Report, 1993).

Boring Logs

Data presented in Figures 2 to 6 provide a synthesis of rock mass qualities from all borings. Detailed engineering properties at the level of the tunnel are provided by data from individual borings. A few examples of rock mass classification in log format are shown in Figures 7 to 10 to illustrate the information available before the start of mining. In the next section, we examine the performance of the TBM in a selection of these zones.

Figures 7 to 10 contain pictorial logs of a selection of borings from near the clay sample locations. The pictorial logs in Figures 8 to 10 present the Terzaghi ground conditions, rock mass quality, and Q-rating for each boring from approximately 60 feet above to 20 feet below the level of the rails.

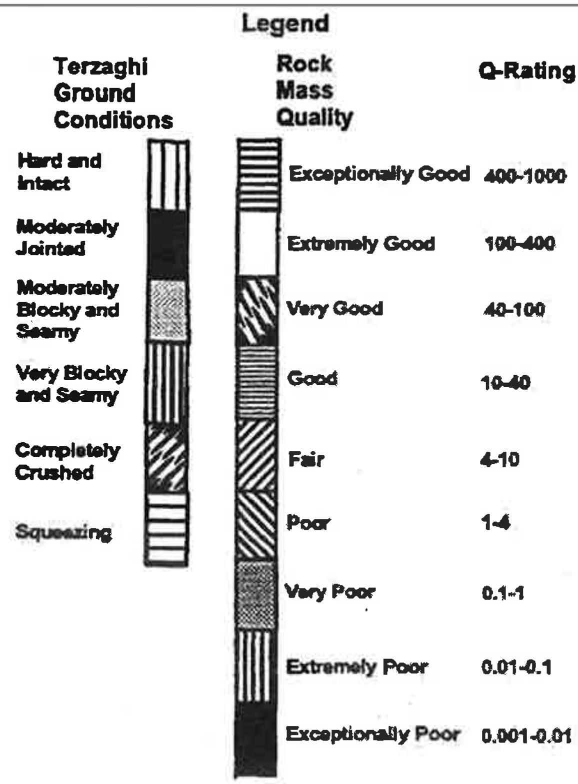

Figure 7: Legend for symbols used in Figures 8 to 11 for characterization of drill cores from a selection of borings (Geotechnical Interpretive Report, 1993).

Boring B-566 (831+89.3) (Figure 8; key to symbols is in Figure 7) indicates a crush zone where clay-mineral sample 832+10 was collected. The boring and clay sample are located in a portion of the tunnel mined by conventional methods. This boring intersected a thrust fault

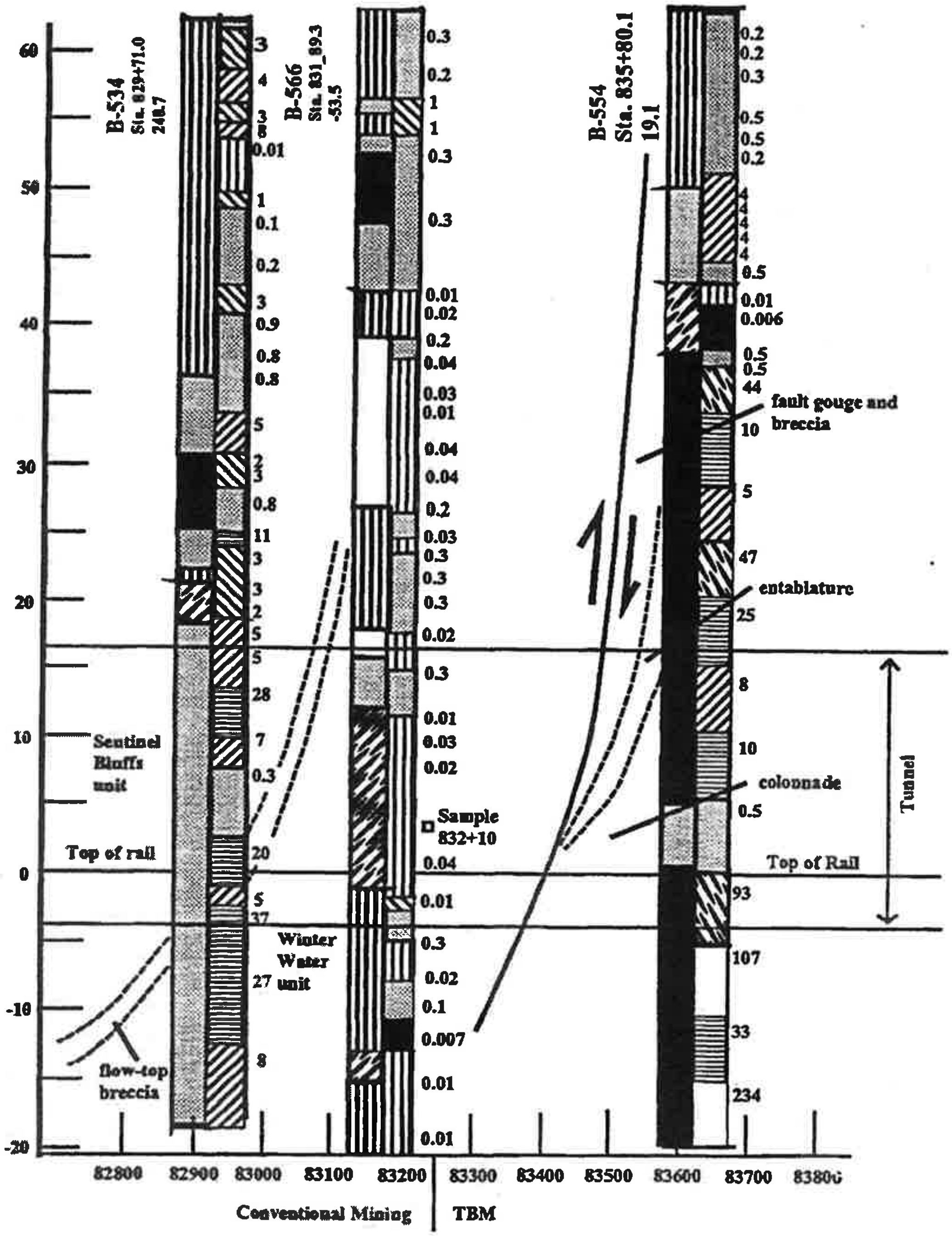

Figure 8: Rock classification for boring B-534, B566, and B554. The left column is the Terzaghi classification, the center column contains rock mass quality and the numbers are Q-ratings (see Figure 7 for legend).

affecting the Grande Ronde and Wanapum Basalt of the CRBG. The Grande Ronde basalt flows are deeply weathered at this site. Rock properties at the tunnel level range from completely crushed to very blocky and seamy (Terzaghi). The rock mass classification ranges from extremely poor to very poor and the Q-ratings are between 0.007 and 1. The rock properties improve greatly a short distance either side of boring B-566 (Figure 8, borings B-534 and B-554.

Boring B-533 (840+10.6; Figure 9), in Grande Ronde Basalt, is near the clay-mineral sample collected at 840+97. Here, the contact between the entablature and colonnade of the lowest flow of the Sentinel Bluffs unit lies near the crown and the flow-top breccia at the top of the Winter Water unit lies within the tunnel. This boring is also near the axis of an anticlinal

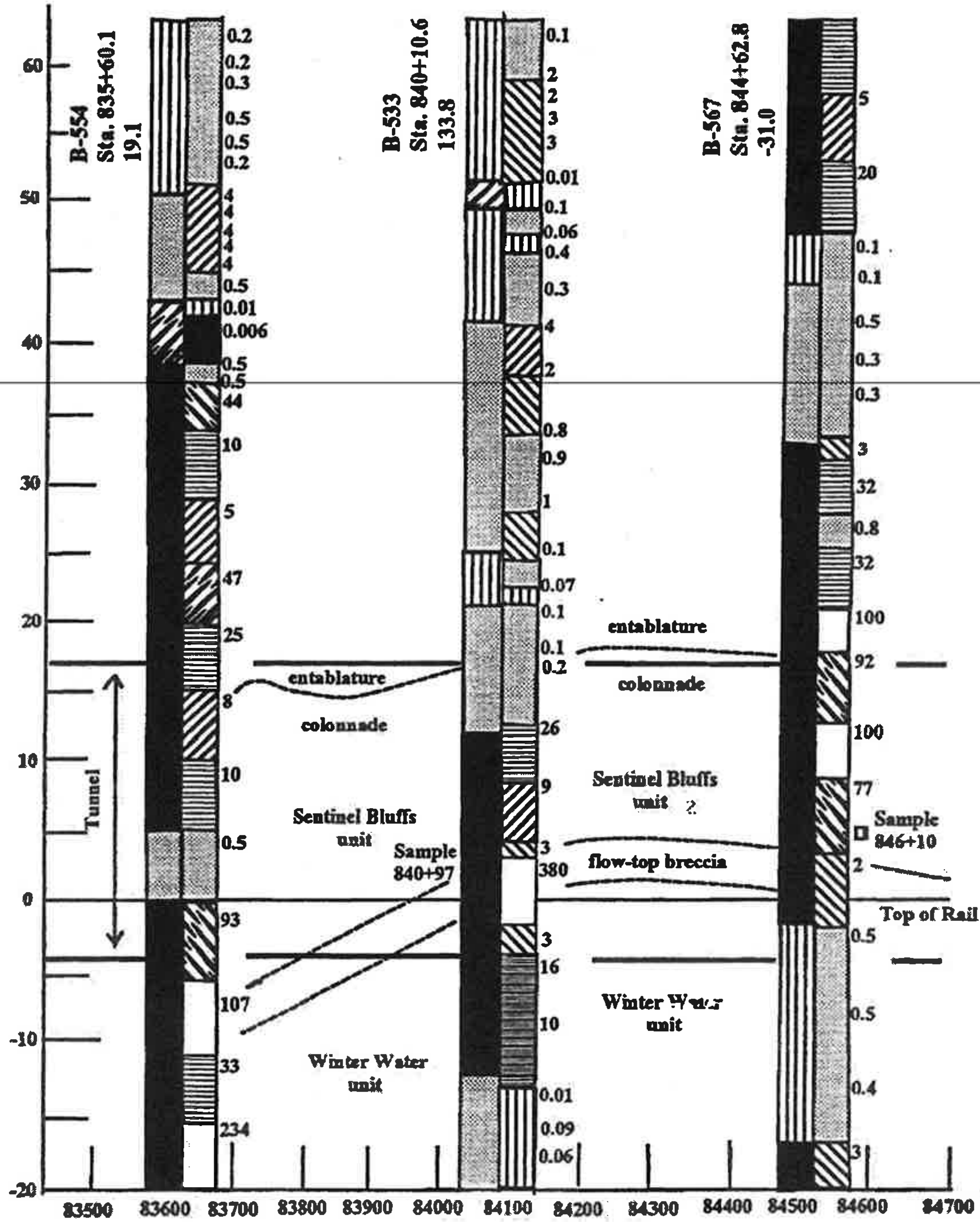

Figure 9: Rock mass classification according to the Terzaghi (left column), rock mass quality (center) and Q-rating systems (right column) for boring B-554, B-533, and B-567 (Geotechnical Interpretive Report, 1993).

flexure that is cut by numerous shear zones. At the level of the tunnel the Terzaghi classification indicates moderately jointed and moderately blocky and seamy rock with lesser very blocky and seamy zones. The rock mass quality indicates considerable variability over short distances from very poor to extremely good. Likewise, the Q-rating varies from 0.007 for the poorest rock to 380 for rocks rated as extremely good.

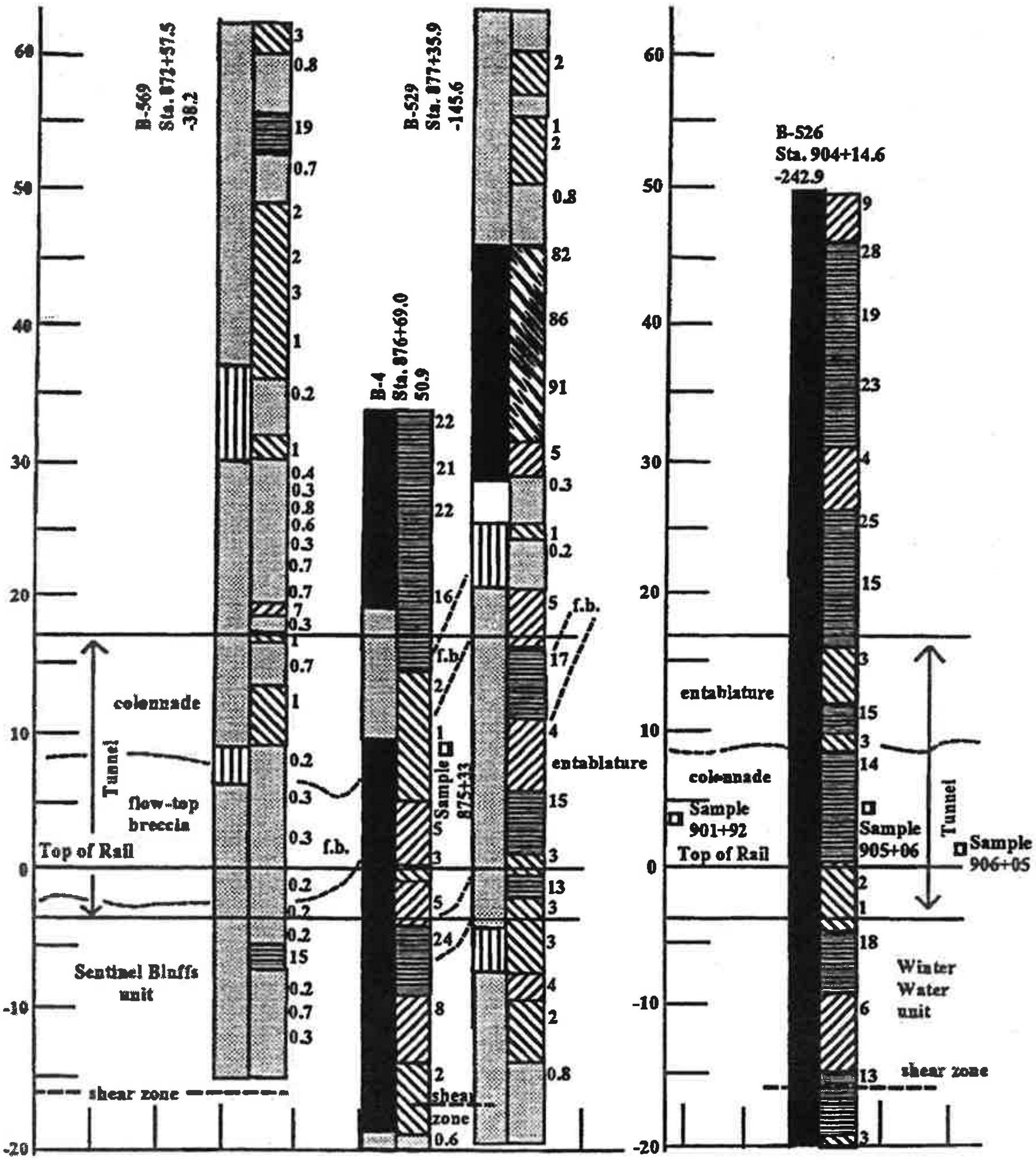

Figure 10: Rock mass classification according to the Terzaghi (left column), rock mass quality (center) and Q-rating systems (right column) for borings B-569, B-4, and B-529. B-526 at station 904+14.6 is shown on the right side of the figure (Geotechnical Interpretive Report, 1993).

The boring nearest the clay mineral sample collected at station 875+33 is B-4 (876+69.0) (Figure 10). This boring was one of eight borings drilled during the first phase of characterization of the tunnel alignment. The flow unit is the Sentinel Bluffs unit of the Grande Ronde Basalt. The Terzaghi classification indicates moderately jointed to moderately blocky and seamy basalt that has rock mass quality ranging from poor to good. The Q-rating ranges from 1 to 22.

Boring B-526 (904+14.6) is the nearest boring to clay-mineral samples collected at stations 899+42, 900+00, 901+92, 905+06, and 906+05. This flow unit is also the Winter Water unit of the Grande Ronde Basalt. The Terzaghi classification (Figure 10) indicates uniform moderately jointed rock at the tunnel level. The rock mass quality is good to very good, but sections of poor rock are present. The Q-rating ranges from 1 to 25.

GEOLOGY OF SAMPLING SITES

Table 2 summarizes the geology in the immediate vicinity of the nine samples collected for clay mineral analysis. Nearly all samples were collected in the Grande Ronde Basalt. Table 2 is based on observations made in the tunnel during construction. Descriptive terminology used in Table 2 is presented in Table 3.

Table 2: Geology determined in the vicinity of samples collected for clay-mineral analysis. Clay mineralogy for these samples is presented in Table 4.

Clay-mineral sample number	East-bound tunnel	West-bound tunnel
840+97 Adjacent to a crushed zone. Area contains joints with clay fill up to 1/2 inch wide. Shear zone/crushed colonnade	**Sentinel Bluffs unit** Strength: to R4/R5 Joint Spacing: vc-mc Joint fill: to 1/2 inch (few to 1 inch)	 R3-R5 vc-mc to 1/2 inch
846+10 Shear zone oblique to boring: intersects west-bound tunnel at about 846+50	**Sentinel Bluffs unit** Shear zone/entablature over crush colonnade Strength: R4/R5 (altered colonnade zone R2/R3 Joint spacing: vc-c Joint fill: to 1/4 inch	 R4/R5 vc-mc to 1/4 inch, local 1 inch
875+33 At the edge of a major fault zone. Flow top present and perched water in zone. Colonnade: mostly stands/some fallout	**Sentinel Bluffs unit** Strength: R3/R4 (dense) Joint spacing: w Joint filling: tight to 1/4 inch	 R3/R4 c 1/4 inch
899+42 In 30-foot wide shear at the west edge of a 90-foot wide zone of altered rock (R1/R2). Shear zone crushed adjacent to altered zone. Stands well	**Winter Water unit** Rock strength: R1/R3 Joint spacing: vc/c Joint filling: to 1 inch	 R2/R3 vc/c tight to 1/8 inch

Table 2: Continued.

900+00 Within altered zone with cream colored hydrothermal alteration. May be a crush zone with clay-rich matrix.	**Winter Water unit** Strength: R1/R2 (dense to slightly vesicular) Joint spacing: vc/c Joint filling: to 1 inch	 R3 vc/c to 1/4 inch
901+92 Narrow to 60-foot wide zone of crushing of colonnade	**Winter Water unit** Strength: R3? Joint spacing: vc/c Joint filling: to 1/4 inch	 R3 vc/c to 1/4 inch
905+06 Ravine fault with entablature over blocky colonnade.	**Winter Water unit** Strength: R4/R5 (dense) Joint spacing: c-mc Joint filling: to 1 inch	 R4/R5 c-mc to 1 inch
906+05 Ravine fault zone	**Winter Water unit** Strength: R4 Joint spacing: c-mc Joint filling: 1/2 inch to 1 inch	 R4 c-mc 1/2 inch

CLAY MINERALOGY

Nine clay-rich samples were collected from sites in the east-bound tunnel. The location of each sample and a general description of their characteristics are presented in Table 4. Table 4 also contains the clay mineralogy determined for each sample.

Sample preparation procedures followed standard methods described by Moore and Reynolds (1989). X-ray diffraction analyses were conducted on a Norelco X-ray diffractometer. The diffractograms were analyzed for peak location, intensity, shape, and width for the magnesium-treated samples, and changes in location, intensity, shape, and width for the glycerol treated samples. Clay minerals were identified by analysis of these data.

Smectite, the group name of the swelling clay minerals, comprises the clay suite in each sample, however, in three samples the characteristics of the smectite require further discussion. The samples collected at stations 832+10 and 900+00 are similar in that they contain feldspar in the less than 2μ size fraction. Since it is unlikely that the grinding process used to prepare samples for analysis produced feldspar grains of this size, natural very fine grained feldspar is present in these samples. The clay suite from station 832+10 is approximately 100 percent smectite; halloysite may be present as a minor, but distinct mineral. The sample from 900+00 contains 100 percent expandable mixed layer clay of halloysite interlayered with smectite. Halloysite may also be present as a distinct phase as well.

The sample collected from station 840+97 also contains 100 percent smectite, however, this smectite is distinct from the smectite in the other samples. During sample preparation this clay tended to form a gel and the (001) lattice spacing for the oriented, natural state sample mount is 13Å. The gelling characteristics and basal spacing (001) are consistent with Na-smectite.

Table 3: Terminology used to describe rock samples during construction of the light rail tunnels through the Tualatin Mountains. Table 3a is the scale used for rock strength. Table 3b presents descriptive terminology for joint spacing.

Table 3a: Scale of Rock Strength.

Descriptive Terminology	Strength Designation	Approximate Range of Unconfined Compressive Strength, psi	Field Identification
Very low strength	R1	100 - 1,000	Crumbles under firm blows with point of geology pick can be peeled by a pocket knife, breakable by finger pressure with difficulty
Low strength	R2	1,000 - 4,000	Can be peeled by a pocket knife with difficulty, shallow indentation made by firm blows of geology pick, craters under point impact.
Moderate strength	R3	4,000 - 8,000	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single firm blow of geology hammer, and will develop a smooth pit or dent under point impact.
Medium-high strength	R4	8,000 - 16,000	Specimen requires more than one blow with a geology hammer to fracture it; explosively dents or pits under geology hammer point impact.
High strength	R5	16,000 - 32,000	Specimen requires many blows of geology hammer to fracture it, does not indent.
Very high strength	R6	> 32,000	Specimen can only be chipped with geology pick.

Table 3b: Descriptive terminology used for joint spacing.

Spacing of Joints	Descriptive Term
Less than 2 inches	very close, vc
2 inches to 1 foot	close, c
1 foot to 3 feet	moderately close, mc
3 feet to 10 feet	wide, w
Greater than 10 feet	very wide (massive), vw

Table 4: Clay mineralogy and field characteristics of analyzed samples.

Station number	Sample characteristics	Clay mineralogy
832+10*	Clay gauge from thrust fault zone.	100% smectite; possibly distinct halloysite
840+97	Prominent, up to 2 inches wide dark green infilling of tectonic fracture/fault(?).	100% Na-smectite
846+10	Clay-mineral rich matrix between altered angular fragments of basalt in a tectonic crush zone.	100% smectite
875+33	Clay infilling in fractures near a fault zone.	100% smectite
899+42	Clay-rich joint filling at edge of a crush zone.	100% smectite
900+00*	Clay-rich joint filling within a zone of altered basalt.	100% expandable mixed layer with halloysite; possibly distinct halloysite
901+92	Clay matrix from a narrow crush zone in fresh rock.	100% smectite
905+06	Clay matrix within a fault zone cutting fresh rock.	100% smectite
906+05	Clay matrix within a fault zone cutting fresh rock.	100% smectite

* These samples contain feldspar in the less than 2μ size fraction.

CONDITIONS ENCOUNTERED BY TBM

The design criteria for the TBM were established in the Geotechnical Design Report (1993) and the TBM, Bore-Regard, was constructed to meet these design criteria. After difficulties that resulted in excessive wear were encountered in construction of the west-bound tunnel, the TBM was repaired and modified before construction of the east-bound tunnel.

The TBM was used during construction of 9,603 feet of the west-bound and 8,460 feet of the east-bound tunnels. In general, the highest rates of penetration were in the flow interior zones of the Winter Water unit of the Grande Ronde Basalt and the lowest rates were in flow-top breccia zones, tectonic crush zones, and areas where open cooling joints were infilled by clay-mineral-rich seams.

Two parameters from construction records, "crown conditions" and "distance advanced per day" are summarized in pictorial form in Figures 11 and 12 for selected intervals. The advancement by the TBM in each figure is the gross progress that includes all delays whether directly related to geology or not. "Crown conditions" document deflection of the wire lagging supported by ring steel set on four foot centers. The deflection of the lagging reflects varying load due to raveling ground. The lowest horizontal dashed line indicates no deflection; the second dashed line indicates less than 2 inches of deflection; the third indicates heavy loading producing 2 to 4 inches of deflection; the top line indicates areas where deflection due to loading was great enough to require installation of support in addition to the ring steel. The locations of samples collected for clay-mineral analysis are also indicated for the east-bound tunnel.

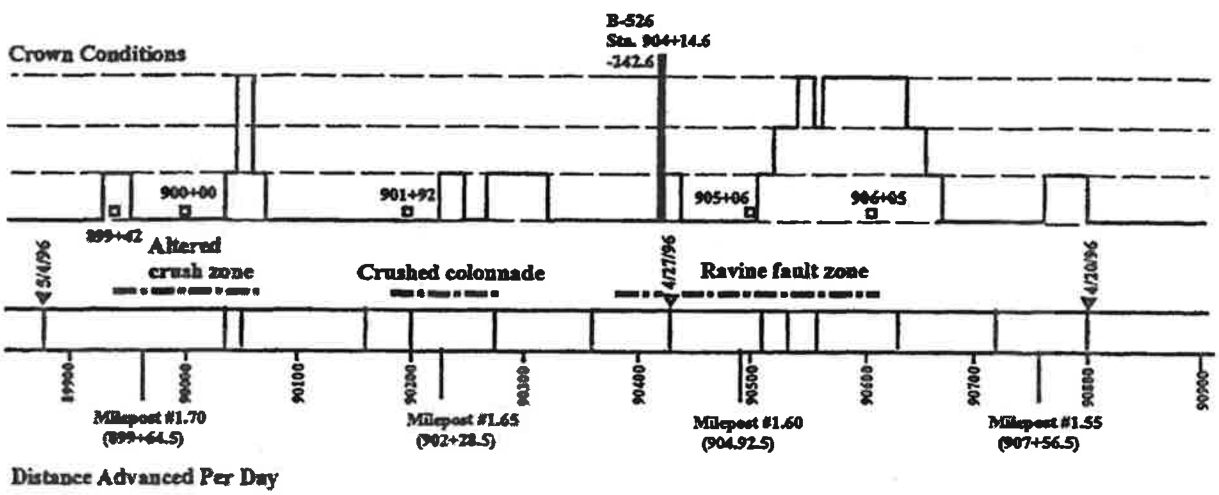

Figure 11: Crown conditions, distance advanced per day, and general geologic features for the east-bound tunnel between stations 898+50 and 909+00. The offset of borings relative to the center line of the east-bound tunnel is indicated (negative values indicate offset to the North).

Figure 11 illustrates patterns from approximately station 898+50 to 909+00 for the east-bound tunnel. The locations of the five clay-mineral samples collected from this interval are also indicated. The "Crown conditions" panel indicates that considerable additional support was required for the wire lagging installed in the crown of the tunnel. Slowing of the rate of advancement is also indicated in this area. In the interval shown, the tunnel crosses the intraflow boundary between the colonnade and entablature of the Winter Water unit of the Grande Ronde Basalt near a faulted zone where cooling joints are lined by clay minerals. Only one boring, B-526, was drilled in this troublesome area. From this boring, the rock mass qualities were indicated to be "moderately jointed" (Terzaghi) and dominantly "good" rock mass quality, with thin zones where quality is "poor". The difficulties in this area were not anticipated from the pre-construction information.

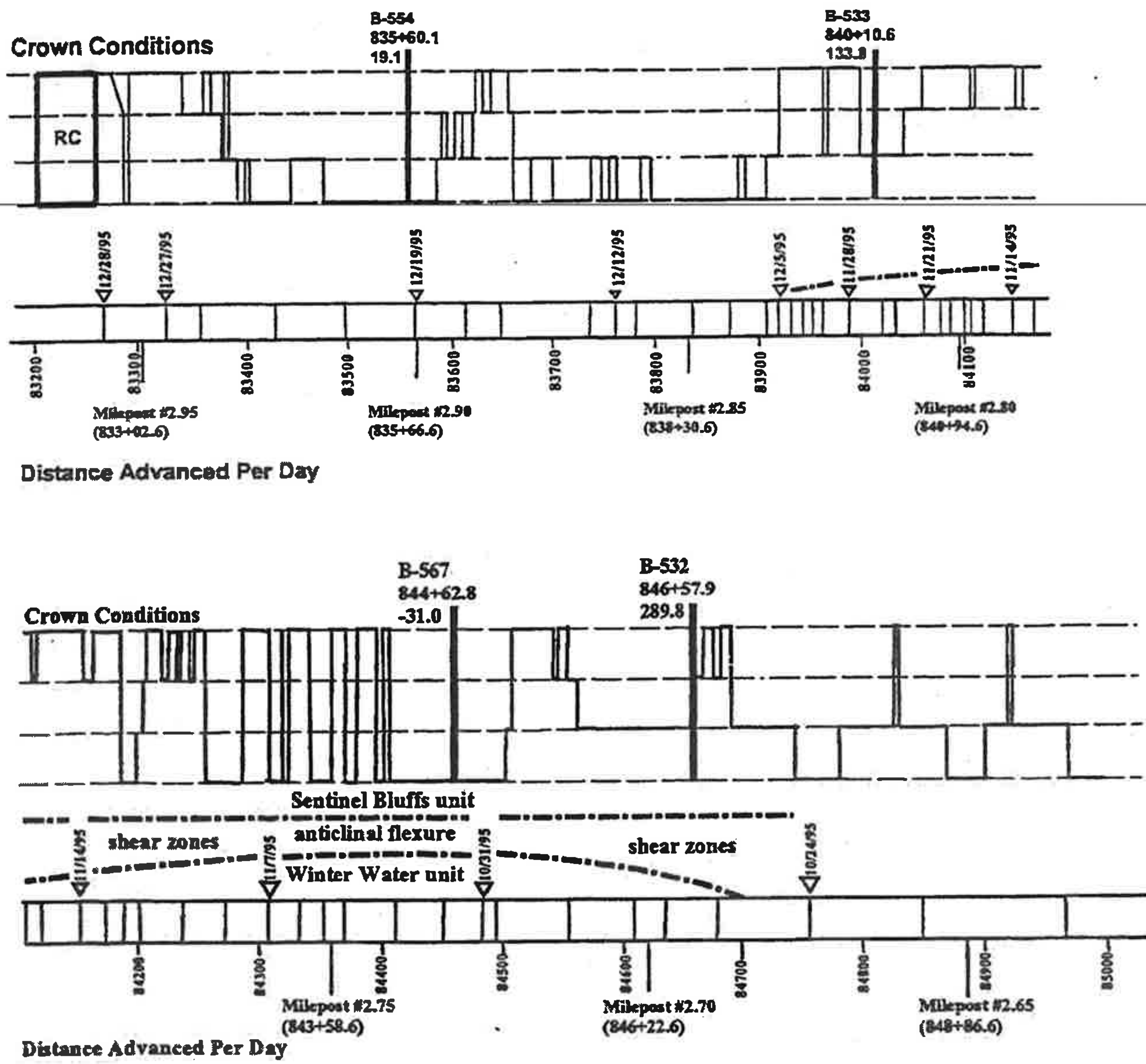

Figure 12: Crown conditions, distance advanced per day, and general geologic features for the west-bound tunnel between stations 833+00 and 850+00. The receiving chamber (RC) is located west of station 832+00.

Figure 12 illustrates the same construction parameters for the interval between stations 832+00 and 850+00 for the west-bound tunnel. The upper panel in this figure indicates that added support was necessary for the wire lagging. Difficulties are also indicated by the rate of advancement of the TBM. Figures 8 and 9, pre-construction borings, indicate variable rock quality east of the fault zone intersected in boring B-566. The Terzaghi classification for this area determined from cores indicates "moderately jointed" to "moderately blocky and seamy" rocks. The rock mass quality ranges from "extremely good" to "very poor". The area east of the thrust fault intersected in B-566 is near the boundary between the colonnade and entablature of the lowest flow in the Sentinel Bluffs unit. A flexure exposes the flow-top breccia of the underlying Winter Water unit and base of the lowest flow of the Sentinel Bluffs unit in the tunnel. Clay-mineral lined cooling joints and tectonic fractures are common in this area.

Field mapping of the west-bound tunnel classified 3,240 of the 9,603 feet mined by the TBM as disturbed ground (34%). Deflection of lagging indicating heavy loading and areas requiring additional support (third and fourth dashed lines in Figures 11 and 12) were present in 2,716 feet (28%). In 2,850 feet (30%), the disturbed ground was intraflow and interflow contacts. In addition, the thickness of clay fillings in cooling joints and tectonic fractures were found to be consistent with ratings of 8 to 20 for Barton's joint alteration number (Ja), significantly higher than estimated from pre-construction evaluation of borings (Ja = 1 to 8). A higher value for Ja would lower the rock mass quality (Q) value for the rocks in the west-bound tunnel.

DISCUSSION

Contrasting engineering properties between joint- and fracture-filling clays and the basalt blocks they surround produced unexpected difficulty during excavation of the light rail tunnel through the Tualatin Mountains. However, in tectonic crush zones where the mineralogy of the basalt blocks was altered and the rock mass characteristics were more consistent with the definition of "weak rock", performance of the TBM measured by rate of penetration was at expected levels. In this discussion we will examine the influence of jointing patterns and the character and distribution of clay minerals in basalt flows on the performance of the TBM.

The distinct cooling joint patterns and flows zones in CRBG flows produce considerable variation in rock mass quality that must be considered during site characterization. Cooling joints develop by contraction as the basalt transforms from a liquid to a solid. The joints can be widely spaced and very regular as in a well-developed colonnade to closely spaced and irregular as in the entablature of the flow interior zone. In addition to the prominent high-angle cooling joints, low angle joints may produce thin plates of basalt, particularly in the basal flow zone, as well as in the flow interior zone of some flows.

The jointing patterns in flows of the CRBG are closely related to the environment of emplacement and the constancy of conditions during cooling. Thus, although joint patterns are relatively distinct in a CRBG flow, the aerial extent of jointing patterns in a flow reflects the extent of a particular set of conditions in the emplacement environment and persistence of conditions during cooling. The jointing patterns in different flows or within different zones of the same flow may vary considerably.

The boundary between joint patterns characteristic of the colonnade and those of the entablature in a CRBG flow can be sharp to gradational over a few feet. Long and Wood (1986) argued that the joints in the colonnade developed as the flow cooled upward from the base while the jointing in the entablature developed by downward cooling from the flow top. The change in jointing patterns across this intraflow boundary can be dramatic. Figure 10 illustrates one case where the tunnel crossed the boundary between the colonnade and entablature of the Sentinel Bluffs unit of the Grande Ronde Basalt. In this area, penetration rates decreased and additional support was required for the wire lagging.

Normally, in CRBG flows the cooling joint surfaces are free of secondary mineral precipitates. However, where these joints have been opened in zones of tectonic disturbance, the joints may become infilled by clays and other secondary mineral precipitates. In the Tualatin Mountains, folding and faulting have opened the cooling joints and generated fractures that are locally infilled by clay minerals. Development of secondary minerals, particularly clay minerals, in basalt may occur at the time of emplacement in the depositional environment or during subsequent weathering, low-temperature interactions with ground water, or circulation of heated ground water within geothermal systems.

These interactions, known as fluid-rock reactions, occur over a range of temperatures and fluid:rock mass ratios. Clay minerals are sensitive indicators of the intensity of fluid-rock interactions. Smectite clays, such as nontronite, develop at relatively low fluid-rock ratios and low temperatures. An increase in the fluid-rock ratio produces changes in the smectite that may include development of Na-smectite and/or development of mixed layer expandable clays where smectite is interlayered with a non-expandable clay mineral such as halloysite. As the intensity of alteration increases, other clay minerals such as halloysite develop as separate phases distinct from the mixed layer types.

In basalt flows, fluids may migrate along open cooling joints and precipitate thin layers of clay minerals with little impact on the rock mass quality ratings of the basalt. Such patterns might evolve where fluid:rock mass ratios and ground water temperatures are low. On the other hand, where fluid:rock mass ratios are high and/or water temperatures are somewhat elevated, thick layers of clay minerals are precipitated and the rock mass quality is severely degraded as secondary minerals replace the primary phases of the basalt. Such conditions are most likely in tectonic crush zones and along faults.

Both patterns are present in the tunnel. The first pattern, clay fillings developed along cooling joints separating blocks of otherwise high rock mass quality basalt, produced problems during mining with the TBM. Clay minerals from cooling joints and separating blocks of dense basalt were collected from several stations (Table 4). In these samples the clay mineral is 100 percent smectite. This clay mineralogy associated with unaltered blocks of basalt suggest low fluid:rock ratios during clay mineral deposition.

The second pattern, altered blocks of basalt in a clay-rich matrix, produced fewer problems and approximated the characteristics of "weak rock". Samples collected at stations 832+10 and 900+00 contain feldspar, less than 2μ, suggesting considerable crushing of the rock in these zones of faulting and alteration. The clay at station 900+00 is 100 percent expandable mixed layer clay where halloysite, the non-expanding clay, is interlayered with smectite. These patterns are consistent with higher fluid:rock ratios during alteration and may indicate circulation of low temperature geothermal fluids.

In general, unexpected problems arose at changes in intraflow cooling joint characteristics (colonnade to entablature). Tectonic flexuring had opened these joints, and clay-minerals had infilled the open joints. The extreme contrast in physical properties of these materials and the ability of the TBM to dislodge large blocks from the face and ceiling without effectively grinding them resulted in considerable delay and equipment wear. The rate of penetration decreased and the wear to the cutting surfaces of the TBM increased. Figures 11 and 12 illustrate sections where these patterns were present. Added support was required for the wire lagging and the penetration rate decreased in both the east- and west-bound tunnels.

Although pre-construction site characterization indicated areas of problems along the tunnel alignment, additional unexpected problems arose during construction. The location and spacing of borings was significantly controlled by land access and in places (i.e. near station 900+00), borings could not be spaced closely enough to intersect zones of poor rock quality. Pre-construction interpretation of the geology of the alignment was found to be in agreement with actual conditions observed during construction. However, the scale of tectonic fracture zones and crush zones, variations in cooling joints patterns, and the distribution of clay-rich linings in fractures were beyond the resolution provided by exploration methods.

CONCLUSIONS

From experience gained during construction of the light rail tunnels through the Tualatin Mountains, three geologic variables were found to be of greatest importance where a TBM is to be used for tunnel excavation. These are 1) stratigraphy, 2) tectonic deformation, and 3) weathering and alteration. Stratigraphy includes all the physical properties imparted to the flows from the emplacement and cooling environment. Properties may vary extremely within a single flow both vertically and laterally, and vary from flow to flow in a stacked sequence of flows. The intraflow transition from the colonnade to the entablature is particularly important in the flow interior zone. Tectonic faulting and folding of the basalt flows also influences rock mass properties. Where broad flexures produced opening of cooling joints and where faults produced shearing and brecciation, migration of water lead to deposition of smectite clays without significant alteration of the basalt. This combination produced most of the unexpected mining problems for the TBM. Where the basalt between fractures had been altered and weathered, fewer problems were encountered and the rock approximated "weak rock" conditions.

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Implementation of a Zoning Ordinance for Shale Slopes in Huntsville, Alabama

STANLEY J. VITTON
Department of Civil & Environmental Engineering, Michigan Technological University, Houghton, MI 49931

MARCIA G. BJØRNERUD
Geology Department, Lawrence University Appleton, WI 54912

BENJAMIN A. FERRILL
Planning Division, City of Huntsville Huntsville, AL 35804

ABSTRACT

Recent landslides within the city of Huntsville, Alabama, which occur in the weathered shales located in the city's upland areas, have resulted in the implementation of a series of zoning ordinances to provide for the safe development of these areas. However, an analysis of the zoning regulations indicates that they may not adequately provide protection for development or may in turn be too restrictive. While the recent landslides have not yet caused injury or loss of property, the potential for such loss is significant. In addition, larger paleolandslides have been identified within the city limits and reoccurrence of such slides poses far greater potential for loss of life and property. However, little is known concerning the frequency and triggering mechanism for the generation of these landslides. Also, the unknown effects from possible seismicity complicate an analysis of slope stability. Consequently, current zoning regulations for the upland areas have essentially stopped development in various designated portions of the slopes. To aid in the assessment of the stability of the region an integrated approach is suggested which would 1) allow a better understanding of slope instability and 2) to provide a means by which the true cost of development can be assessed with respect to geologic hazards.

INTRODUCTION

Since the early 1980's, there has been growing awareness of the potential for damaging landslides in western Appalachia (Brabb, 1989, Fleming & Taylor, 1980). As in many cities in the western Appalachian region, development of steep mountain slopes in Huntsville, Alabama is accelerating in spite of incomplete understanding of the geologic hazards associated with urbanization of these previously uninhabited areas. Huntsville is a growing city on the western escarpment of the Cumberland Plateau, within the basin of the Tennessee River, in an area of steep hillsides (Figure 1a). Potential geologic hazards in upland terrain include sinkholes, rock falls and topples, soil creep, debris flows and deep seated landslides. A US Geological Survey report (Pomeroy & Thomas, 1985) identified several major prehistoric landslide deposits that extend well onto the floor of the valley in which Huntsville lies (Figure 2).

In response to the USGS report and to a series of slope failures in newly developed areas of Huntsville in the 1980's, the city took the politically unpopular step of enacting the Mountainside Development District Ordinance to guide future city planning (City of Huntsville, 1991). The ordinance, later amended (City of Huntsville, 1996) on account of opposition from development advocates, requires stability analyses prior to application for permits to build on steep slopes, and it includes specifications for construction of enhanced foundations and retaining walls. In addition, the ordinance designates areas underlain by particular geologic units - the Mississippian-age shales of the Pennington and Upper Bangor Formations -- as special "hazard zones." A stratigraphic section illustrating the relationship between these units and the general stratigraphy in the area is shown in Figure 3. A cross-section through Monte Sano Mountain (northern most upland area within Huntsville), illustrates the relationship between these stratigraphic units, recent earth flows, and topography (Figure 4). Some researchers consider the Pennington formation to be part of the Upper Bangor formation and therefore no longer use the term Pennington. However, the identification of the Pennington is widely used in the Huntsville area and will be used in this paper. The shales of the Pennington and Upper Bangor Unit are the strata that weather to weaker residual soils and result in the development of colluvium that is implicated in most of the slope failures in the area.

Obtaining a permit to build within a hazardous zone requires that an engineer certify the proposed development site as safe after analyzing not only the site itself but also areas immediately upslope and downslope from it. The effect of the ordinance has been to stop development on areas designated as hazard zones and to limit new building on the remaining upland areas (those not named as hazardous zones but with slopes greater than 15%). It has been estimated by the City of Huntsville's Planning Department that the ordinance has added significantly to the cost of development within the upland areas of Huntsville (e.g., the cost of a single building permit has risen from $2,000 to $10,000 since implementation of the ordinances). Few engineers will certify a development within the hazardous zones because of liability considerations; at present, there is no meaningful way to assess the safety of the slopes.

While the ordinance does reflect an awareness of possible risks associated with hillslope development, it is based on a very limited understanding of the nature of geologic hazards in the area. Indeed, an integrated risk assessment, which would be required, is not feasible at present because the processes contributing to these geologic hazards are not well understood. In fact, the Mountainside Ordinance may give city residents a false sense of protection from natural hazards. Although it is clear that recent landslide activity generally occurs in the residual soils of the Pennington-Upper Bangor unit, little is known about the cause(s), age, or frequency of the considerably larger paleolandslides identified in the USGS report by Pomeroy and Thomas (1985). With thousands of people now living at the base of the mountain slopes, recurrence of slope failures of this magnitude would be disastrous.

The purpose of this paper is to review the geologic characteristics of the Huntsville area and the potential geological hazards that have influenced the implementation of the zoning ordinances for the City of Huntsville and to suggest that a more regional, integrated approach to the analysis of slope instability be undertaken.

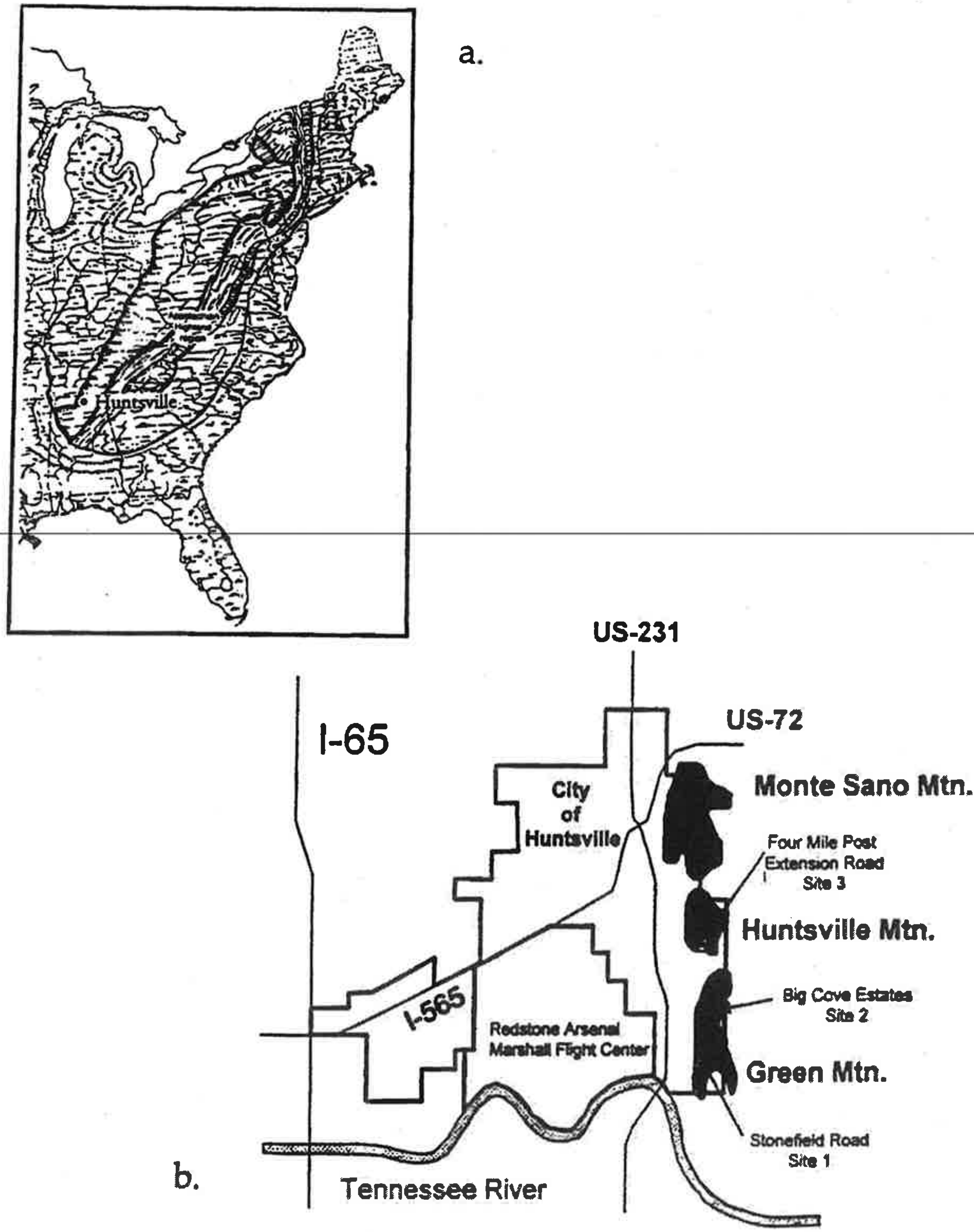

Figure 1. Huntsville, Alabama location maps. A) location of Huntsville within the Appalachian region, B) map of Huntsville metropolitan area showing locations of paleolandslides (sites 1, 2, and 3) within the city limits.

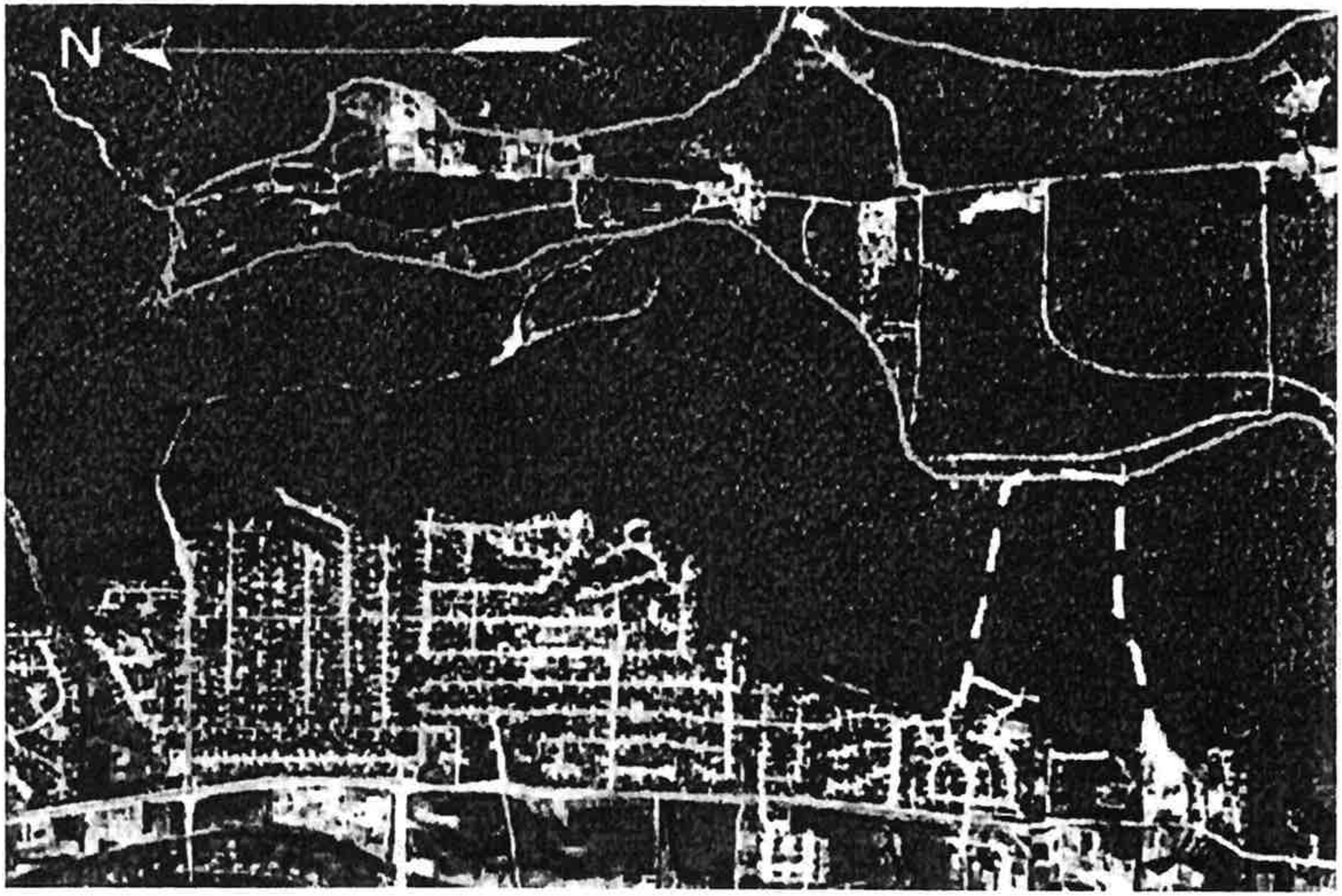

Figure 2. NAPP photography (originally color infrared) of the west flank of Green Mountain in 1985, showing the increasing urbanization of Huntsville's upland areas. Visible in the lower right (dashed line) is one of the large paleolandslides, identified by Pomeroy and Thomas (1985), on which part of a housing development now sits.

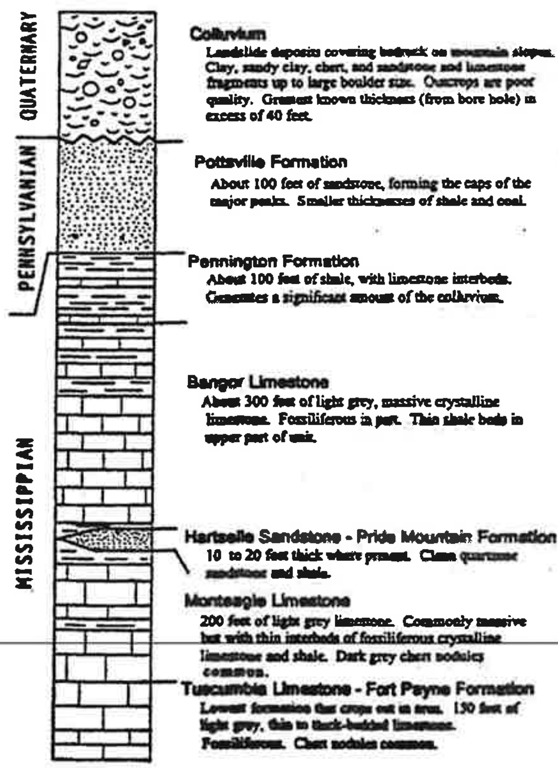

Figure 3. Stratigraphic section (From Bodden, 1992).

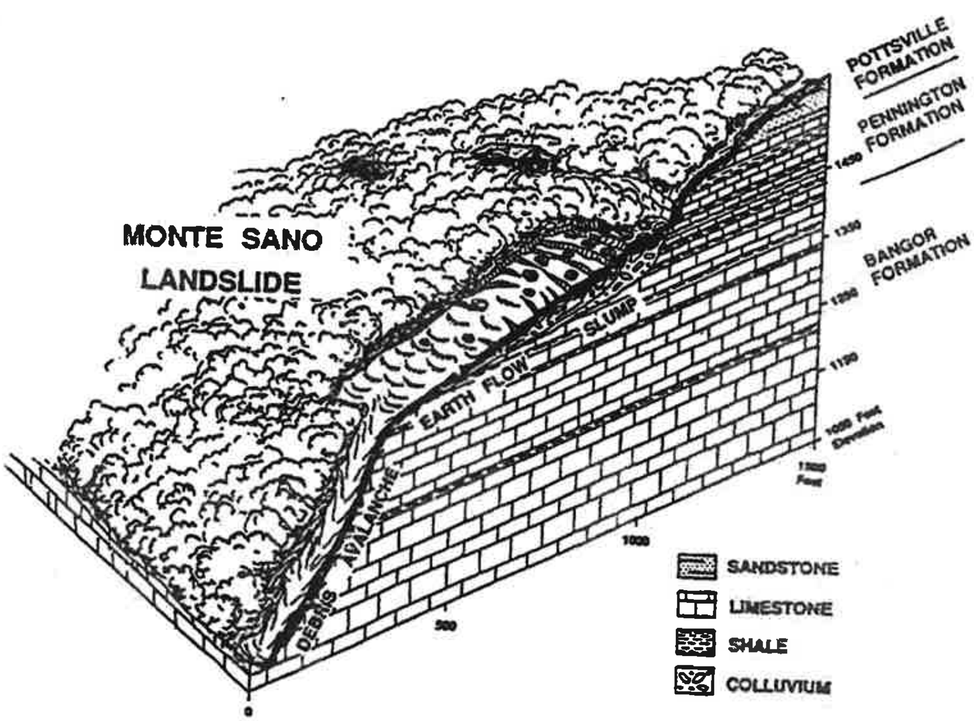

Figure 4. Geologic cross-section through Monte Sano Mountain, showing relationship between stratigraphic units, recent earth flows, and topography (From Ferrill, 1993).

GEOLOGIC CHARACTER OF HUNTSVILLE SLOPES

Bedrock geology

As in other parts of the Cumberland Plateau, ridges in the Huntsville area are capped by the relatively resistant Pottsville sandstone, a Pennsylvanian unit representing coarse sediment deposited in foreland basins during the final phase of Appalachian mountain building. The distribution of Pottsville boulders near the ridgetops indicates that the principal modes of slope failure in the Pottsville are rock falls and topples (Varnes, 1978). Underlying the Pottsville Formation are interbedded shallow marine shales and limestones of the Mississippian Pennington and Bangor Formations, with a total thickness of about 125 m (Thomas, 1972). Most of the slope failures, including the recent mudflows and older translational slides, are initiated within the Pennington and Bangor units (Pomeroy and Thomas, 1985). Both the shales and limestones are potential contributors to slope failure, as discussed below.

Geomorphology

Few studies have addressed the landscape evolution of northern Alabama. Geomorphic studies in adjacent areas (central and eastern Alabama, Georgia) indicate that the landscapes of the region were largely shaped in the mid to late Pleistocene, when the climate was more strongly seasonal and prone to long periods of drought (Delcourt, 1980; Markewich and Markewich, 1994). The gravelly fluvial deposits of the Mobile River basin in central Alabama are all interpreted to be of Pleistocene age; no evidence for Tertiary deposits has been found (Markewich and Christopher, 1982). Many of the Pleistocene streams appear to have been braided, consistent with a climate characterized by extremes in both rainfall and aridity. Modern patterns of precipitation and river behavior were established by at least 7500 years BP (Markewich and Christopher, 1982; Markewich and Markewich, 1994). Although the evolution of hill slopes in both the Mobile and Tennessee River basins was undoubtedly related to fluvial processes, little is known about the ages of the upland geomorphic surfaces.

One of the notable aspects of Huntsville-area slopes is the relative lack of gullying by small streams, whose base level should be linked with the Tennessee River (see Fig. 1, 2). This suggests either that the surface runoff is limited, perhaps diverted by underground drainage systems (see section on Karst features, below), and/or that gullies are continuously filled in by colluvium. Another distinctive characteristic of the flat-topped mountains is the cuspate edges of the ridgetops. The rims trace open arcs that contrast markedly with the narrow hollows formed by streams (contrast, for example, the north and south sides of Monte Sano Mountain). Figure 5 illustrates a slope image of the Huntsville area that was generated by a US Geological Survey digital elevation model (DEM). Variations in white and black represent various slope levels, with black indicating the steepest slopes and white the flattest slopes. Note the long narrow features of the upland areas along with the solid black band just below the flat top of the mountains. This black band indicates the location of the Pennington-Upper Bangor unit. The north and south sides of Monte Sano Mountain are located in the upper right-hand side of Figure 5. This morphology again suggests that the hillslopes have been shaped by processes other than channelized flow.

Figure 5. Slope image map of the Huntsville area generated from USGS DEM data, with black indicating the steepest slope and white indicating the flattest slope.

Residual soils

When the Mississippian shales are exposed at the surface, they are rapidly weathered to residual soils. Undisplaced soils from Huntsville-area slopes have not been extensively studied. However, limited work by Vitton (1993) determined the grain-size, plasticity, and soil mineralogy using x-ray diffraction. While no strength testing was conducted due to the difficulty of testing the residual soils on such highly vegetated slopes, strength parameters were estimated from a back analysis using a recent flow failure in the residual soil. Results indicate that the residual soil is a low plasticity clayey silt (ML-CL) composed primarily of chlorite and illite with a liquid limit of about 26 and a plasticity index of approximately 3. The back analysis indicated that the soil had a higher strength than would be expected for a ML-CL soil, with an effective friction angle ranging from 43° to 52° assuming an effective cohesion of 0. It is speculated that the higher strength may be the result of 1) wetting and drying cycles that have caused overconsolidation of the soil, 2) root reinforcement by vegetation and/or 3) remnant microstructure surviving from the shale or interbedded limestone that did not weather. Interestingly, the downslope colluvial material, which is derived from the residual soil, has significantly higher plasticity (PI ranging from 13 to 37), higher clay content, and a distinctly different color (the dark greenish gray residual soils become reddish within days of a flow failure). These contrasts indicate that mineralogical and mechanical changes occur in the shale-derived materials over time.

Colluvial Soils

The colluvium consists primarily of clay and numerous sandstone and limestone rock fragments ranging from clay size to large boulders, with approximately 50 percent of the material passing the number 200 mesh. The extent of colluvium on the upland areas was mapped by Brewer (1991) and Bodden (1993). According to Bodden, the colluvium is generally absent from the crests of topographic highs, where significant thicknesses cannot accumulate. Colluvium is also absent for long stretches from many stream beds on the mountain slopes, where the streams have eroded the colluvium down to bedrock. The thickest occurrences of colluvial clay occur on the upper slopes of the mountain, especially where there is a hollow or concavity in the topography. Most of the colluvium appears to have been generated in a belt formed by the Upper Bangor, Pennington, and lower Pottsville Formations. Lesser amounts of colluvium have been generated by shales within and below the Pride Mountain Formation. A main feature of Brewer and Bodden's mapping was the interpretation of a "colluvium line" which represents the downslope limit of the larger accumulations of colluvium. Below this line, occurrences of colluvium are generally only noted along terraces on the underlying bedrock, which results in a stair-step pattern of alternating colluvium and bedrock outcrops on the slope.

While considerable development is occurring on the colluvium on the Huntsville area slopes, only limited information regarding the strength and properties of the colluvium is available (Ground Engineering and Testing Service, Inc., 1992; Lockett, 1986; and Vitton, 1993). According to the Ground Engineering and Testing Service, Inc. study, the amount of soil passing the #200 sieve for the soils tested was approximately 60%. Six Atterberg limit tests were listed with an average Atterberg Limit of LL=45 and PI=25, which would then classify the soil as a low plasticity clay (CL) (USCS). However, individual soils tested ranged between a high plasticity clay (CH) and a low plasticity clay (CL) with one sample being classified as a clayey silty sand (SC). Lockett, conducting an investigation for the Alabama Department of Transportation, indicated the colluvium was a borderline CH/CL with a PI ranging from 21 to 29. Testing by Vitton (1993) indicated that the colluvial material ranged from a SC to a CH with plasticity ranging from 8 to 37, with an average of 13.

Karst features

Huntsville has long been renowned for its limestone caverns and is home to the headquarters of the National Speleological Society. Most of the caves are developed in massive limestones of the Lower Bangor Formation, but laterally extensive caverns are also common in the thinner limestone strata that alternate with shales in the Upper Bangor and Pennington Formations (Bossong, 1989). Where these dissolution features underlie colluvium, sinkholes and seeps are common (Bodden, 1993). An excavation for a new home located on Pennington shale on Monte Sano Mountain (begun prior to enactment of the slope ordinance) intersected a 0.5 m-high, 10-m long cavity between shale layers with significant water flow (Vitton & Ferrill, field notes). Drilling data obtained for a feasibility study for a road between Huntsville and Green Mountain similarly revealed laterally extensive caverns, some with mud- and rubble-filling, within the Bangor Formation (Ground Engineering, 1992). A drilling log in the Bangor formation indicating mud-filled caverns is provided in Figure 6. It is possible that sudden collapse of limestone caverns (caverns within the interbedded Pennington-Upper Bangor unit and not filled with mud), is one mechanism triggering deep-seated failures in the Huntsville area.

Presumably, when cavities reach a critical size, they can no longer support the load of the overlying strata. Collapse of Pottsville sandstone layers into carbonate caverns has been documented elsewhere in Alabama (Nielson and Bearce, 1993). Such collapse may also be an explanation for the cuspate shape of the ridgetops in the Huntsville and other parts of the Appalachian Plateau. Lateral dissolution of the thin layers may be hastened when the outlets of limestone channels are blocked by small soil movement within the residual soil, which forms at the exposed face of the Upper Bangor-Pennington Formation, causing groundwater to back up (Vitton, 1993). At the same time, pore pressure may rise in the soil leading to larger flow failures.

Seismicity

Recent analyses of seismic hazards in the central United States (Wheeler et al, 1994; Powell et al., 1994) indicate that the Huntsville area is at significant risk from both the New Madrid and Eastern Tennessee Seismic Zones. In a new map of probable Mercalli ground motion intensities that would result from a magnitude M_s=7.6 earthquake in the New Madrid zone, Huntsville lies within the intensity VII region (Wheeler et al., 1994). It is possible that earthquake activity may have triggered some of the ancient landslides in the Huntsville area. The New Madrid earthquakes provide some limited data concerning the effects of ground motion in the eastern United States. Jibson and Keefer (1988) have used this information to estimate the distance in which landslides may have resulted from the New Madrid earthquakes. This study indicates that the Huntsville area is well within the zone of influence.

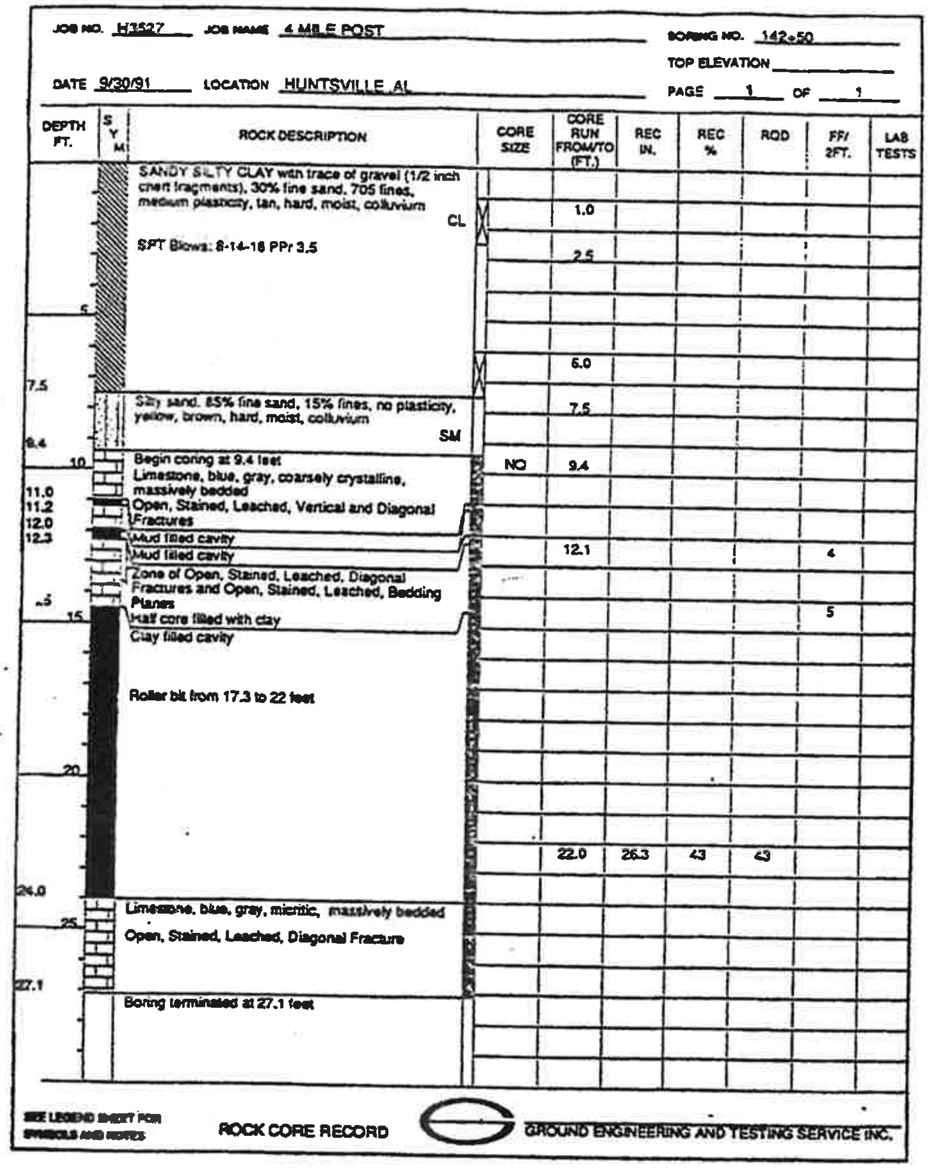

Figure 6. Drilling log for the Four Mile Post Extension road study between Huntsville and Green Mountain (Ground Engineering, 1992).

Given the narrow topography of the upland areas it is also possible that topographic effects contributed to the initiation of the landslides during seismic events. It has been observed that structures located on steep, narrow hilltops during earthquakes are more heavily damaged than structures closer to the epicenter but located on flatter and generally wider landform (Faccioli 1991). The narrow ridge shape, for example, of Green Mountain (Figure 5) may make it more susceptible to landslides from seismic events than the other mountains. A classic example of this occurred in 1978 near Stuttgart, Germany where a medieval castle on top of a 300 m conical hill was heavily damaged during an earthquake, while surrounding villages were not damaged (Borm, 1990). Jibson (1987), of the USGS, has summarized the research on the effects of topographic amplification of earthquake shaking on slope stability. Research by Griffiths and Bollinger (1979) in the Valley and Ridge Province of Appalachia found that the motion in the mountain tops were amplified 1.7 to 3.4 times with respect to the motion in the valley floor, with the amount of seismic amplification depending on the response spectra of the mountains. Narrow, steep mountains will amplify more than wider, less steep mountains. Given this information, there are at least two predominant mechanisms that could induce slope failure. The first mechanism occurs when the natural frequency of the mountain and the earthquake are similar, possibly resulting in intense, resonant shaking. For example, Jibson and Keefer (1988, 1993) speculated that the New Madrid earthquakes produced body waves with magnitudes as high as 7.4 and surface waves with magnitudes as high as 8.8 on the Richter scale. Although ground motion attenuation would occur between the New Madrid fault zone and the Huntsville area, it is possible that significant ground shaking could occur. Thus, coupling seismic amplification with weakened rock structure from rock weathering may account for the deep-seated landslides found in the Huntsville area.

A second mechanism discussed by Krinitzsky (1993) is the possibility that failure is caused by fatigue. This could result from repeated shaking or the extensive duration of an earthquake. During the period of December 1811 to March 1812, extensive seismic activity occurred in the New Madrid Fault and surrounding area. In Louisville, Kentucky 2000 earthquakes were recorded during this period (Schultz and Southward, 1989). It is possible the Huntsville area may have also experience several seismic events during the same period.

Recent slope failures

In the past decade, potentially serious slope failures have occurred in the residual soils on the mountainsides within the Huntsville City limits. In September 1989, for example, a mudflow occurred on an undeveloped eastern slope of Monte Sano Mountain (Fig. 1) only 100 m below picnic grounds at a state park. The flow uprooted large trees in a swath approximately 30 m wide over a distance of 500 m, and created levees as high as 1.5 m at its margins. Although the timing of the slope failure can be bracketed only within a week, the morphology of the flow material indicates that it occurred at a catastrophic rate (Ground Engineering and Testing Services, Inc., 1989). In a similar event in 1991, a major earth flow occurred on the eastern flank of Green Mountain, immediately downslope from a residential lot. This flow, which was approximately 20 m wide, 800 m long and 3 m thick, toppled large trees and left a prominent scarp at its head. The triggering mechanism is unknown for both of these recent slides; both occurred after relatively wet summers, but neither was associated with a particular precipitation event.

Paleolandslides

Both of the recent slope failures could have caused serious injury and property damage if they had occurred on developed slopes, but they are small in comparison to the recognized paleolandslides in the area. The regional study by the U.S. Geological Survey and subsequent mapping by the City of Huntsville documented Holocene landslides as large as 1 km in width and 2 km in length on the slopes within the city (Pomeroy and Thomas, 1985; Bodden, 1993). A subdivision has been built on the toe of one of the paleolandslides. At present no attempt has been made to constrain the age or causes of these massive, apparently deep-seated, slope failures.

SLOPE ORDINANCE

Geologic Hazards

Five modes of slope movement have been identified in the Huntsville area. These are summarized in Figure 7, which illustrates both their modes and locations on the slope (Vitton, 1993). Table 1 shows the frequency of occurrence of slope movements along with an estimate of seriousness of occurrence. From this table it can be seen that the deep-seated landslides are the most serious. However, there is no record of this type of slide occurring within historical times. The mud flows from the weathering Pennington and Upper Bangor shales represent the next level of severity. These recent flows are what prompted the development of zoning ordinances to protect the public from landslides. However, the additional issues of rock falls, sinkhole assessment, and seismic potential are not addressed by the zoning ordinances.

The initial slope ordinance adopted in 1991 was referred to as the Mountainside Development Ordinance. However, various aspects of this ordinance were opposed by developers. Changes were then made to the ordinance in 1996 and reissued as the Slope Development District Regulations. The following sections discuss some of the main points of the ordinances.

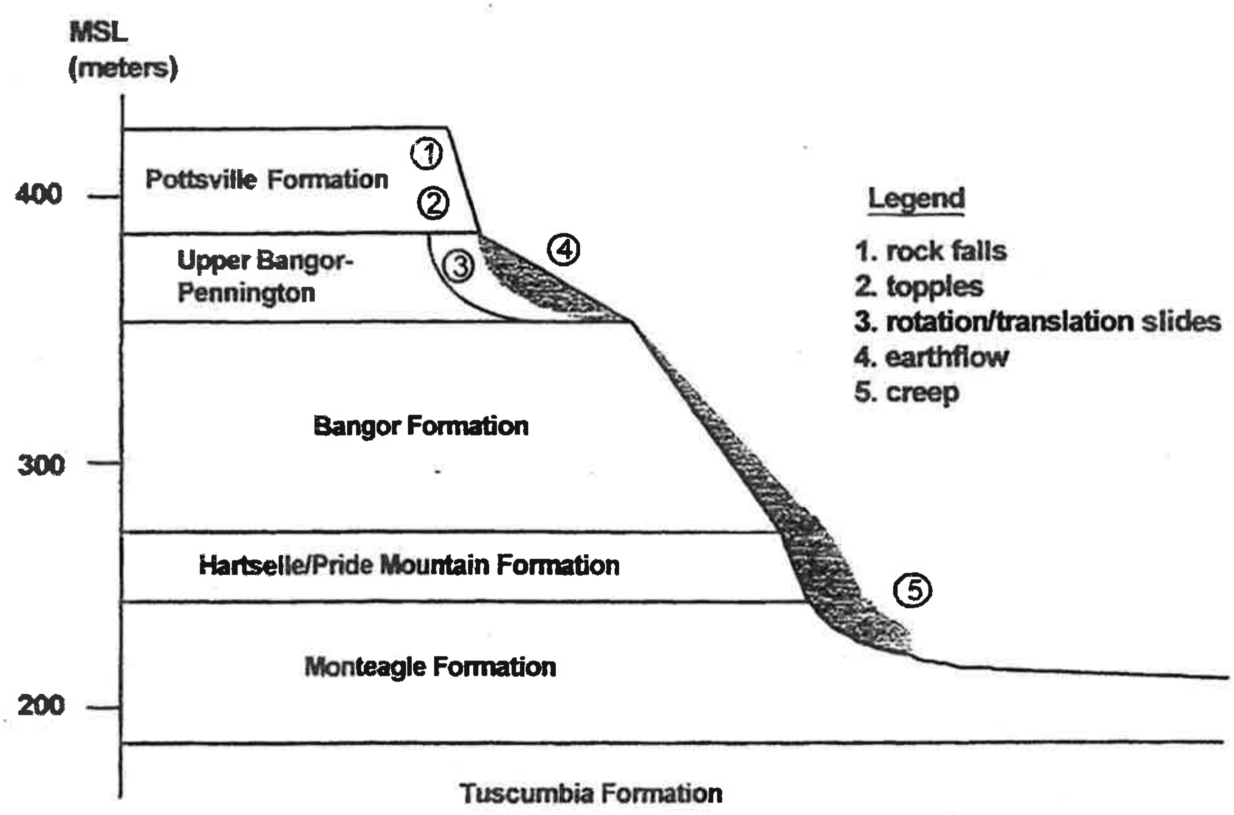

Figure 7. Examples of slope failure modes

Table 1. Estimated frequency of occurrence and of seriousness of slope movements in the Huntsville area.

Geologic Formation	Slope Movement Mode	Frequency of Occurrence	Degree of Seriousness
Pottsville	Rock Falls Topples	Common - 1 yr Common - 1 yr	Medium Low - Medium
Pennington Upper- Bangor	Rock Falls Topples Deep seated slides Flows	Common - 1 yr Common - 1 yr Over Geologic Time Continuous - 5 yr	Medium Low - Medium High Medium - High
Bangor	Limited erosion	Over Geologic Time	Low
Hartselle/ Pride Mountain	Rock Falls Topples Deep seated slides Flows	Common - 1 yr Common - 1 yr Over Geologic Time Continuous - 5 yr	Medium Low - Medium Medium Medium
Monteagle	Limited erosion	Over Geologic Time	Low
Tuscumbia	Limited erosion	Over Geologic Time	Low
Colluvium	Flows/Creep	Continuous - 5 yr	Medium

Mountainside Development Ordinance (1991)

The Mountainside Development Ordinance established a special zoning classification which was superimposed over the existing zoning regulations. A Mountainside Development District was established with a boundary that would encompass those slopes that exceed 15% (run over rise in percent or in degrees 8.5°), although to make a contiguous district it would include some slopes less than 15% (8.5°). Five different levels were established based on the risk of instability and decreasing safety factor. These five levels are as follows:

Level 5 — This level includes all areas of land lying within the boundaries of the Mountainside Development District that have a slope of less than 10% (5.7°).

Level 10 — This level includes all areas of land lying within the boundaries of the Mountainside Development District that have a slope of 10% (5.7°) to 15% (8.5°).

Level 15 — This level includes all areas of land lying within the boundaries of the Mountainside Development District that have a slope of 15% (8.5°) to 25% (14.0°).

Level 25 — This level includes all areas of land lying within the boundaries of the Mountainside Development District that have slopes from 25% (14.0°) to 35% (19.3°). No development is allowed on slopes greater than 35%, with the exception for roads and utility services.

Level K — This level includes all areas of land within the boundaries of the Mountainside

Development District that exhibit any or all of the following geologic formations:

1) Pennington, shaley Upper Bangor, or Pride Mountain geologic formations;
2) Colluvial deposits; or
3) Evidence of mining operations (mine or quarry tailings, or similar geologic conditions).

For all development sites on slopes that are within Level 5 through Level 25, a stability analysis is required with a factor of safety greater than 1.5. In addition, various density controls are required, i.e., only a certain percent of the site can be developed. In general, as the steepness of the site increases then additional open space or undeveloped area would be required. For sites designated as Level K, however, additional geotechnical analysis and testing (in addition to the required zoning regulations for non-district sites) is required both upslope and downslope of where a potential landslide would end. In addition, a professional engineer must certify that the proposed development is designed in accordance with sound engineering standards and practices and that a prudent investigation, geotechnical testing and inspection "on all lands within the proposed development identified as Level K, including geotechnical analysis and testing on all Level K lands and on lands upslope and downslope of the proposed development" had been performed. In other words, a professional engineer must certify that the site is safe from any geologic or man-made hazard, whereas on non Level K sites the engineer must only certify that the site itself is safe from instability. It should be noted that Level K sites do not have any slope requirements with the exception of the 35% slope limit for development.

The practical consequence of this ordinance was essentially to halt development on sites designated as Level K, which included a considerable portion of the land within the Mountainside Development District, since much of the District is covered by some colluvial materials. Two significant problems existed. First, the cost to test and analyze all areas both upslope and downslope of level K sites would be prohibitive for most home development sites. Second, it is not yet possible for geotechnical engineers to meaningfully assess the stability of these geologically complex sites.

In an attempt to remove some land from the Level K designation, developers lobbied for a reclassification of lands. In the ensuing negotiations, the five classifications were redefined into three classifications. The zoning ordinance was then re-issued under the title of "Slope Development District Regulations," which is discussed below.

Slope Development District Ordinance (1996)

As with the previous ordinance, the District's boundary remained the same. However, as noted above, the five levels were changed to three levels, which are defined as follows:

Lower Slope Zone	This zone includes all areas of land within the boundaries of the Slope Development District that are not part of the Upper Slope Zone
Upper Slope Zone	This zone includes all areas of land within the boundaries of the Slope Development District beginning at the elevation contour defining the start of the 20% slope and encompassing all contiguous slopes of 20% or greater.

Hazard Zone — This zone includes all areas of land within the boundaries of the Slope Development District that exhibit any of the following geologic or man-made hazards:

(1) Pennington, shaley Upper Bangor, or Pride Mountain geologic formations;
(2) Colluvial deposits having a depth of five feet or greater or being contiguous to an off-site colluvial deposit that exceeds five feet in-depth; and
(3) Evidence of mining operations (mine or quarry tailings, or similar geologic conditions).

The requirements under the revised ordinance remain basically the same for the new zones. That is, the requirements for the Lower and Upper zones remain the same as for the previous Level 5 through Level 25 zones and the Hazard Zone remains the same as the level K Zone. However, the significant change in the 1996 ordinance is that it eliminates from the hazard zone any site with less than five feet of colluvial deposits on it, whereas previously any site with colluvial deposits would be zoned Level K and would require extensive geotechnical testing and analysis. However, the 1996 ordinance would still require that sites underlain by Pennington, Upper Bangor, or Pride Mountain formations be considered hazardous. While the change in the ordinance allowed some sites to be permitted without a geotechnical investigation, no sites that lie within the Hazard Zone have been permitted for development since enactment of the ordinances. Again, two significant issues affect development: (1) the cost of a geotechnical investigation and (2) the ability of a geotechnical investigation to certify that the site is safe. The latter issue is discussed further below.

ANALYSIS AND DISCUSSION OF THE ZONING ORDINANCE

The main challenges to development within the Hazard Zone are making a meaningful assessment of potential geologic hazards at a site and doing so in a cost effective manner. As discussed above, the primary hazards are landslides, both flow and deep-seated, rock falls, seismic generated slope movement, and sinkholes for site overlain with colluvium. While most geotechnical investigations assess the stability of a given site, it is difficult (and expensive) to assess both upslope and downslope stability. In addition, the potential for seismically generated landslides is only speculative, since too few data have been obtained to either confirm or disprove seismic effects. Also, it is difficult to locate the presence of a developing sinkhole without extensive drilling. Rock falls appear the only recognizable hazard that can be determined and relatively easily discounted if sufficient forest exists between the rock fall areas and the development site.

According to Soeters and van Westen (1996), "slope instability processes are a product of local geomorphic, hydrologic, and geologic conditions; with modification of these conditions by geodynamic processes, vegetation, land use practices, and human activities; and the frequency and intensity of precipitation and seismicity." They further state that "The engineering approach to landslide studies has focused attention on the analysis of individual slope failures and their remedial measures. The techniques used in these studies in accordance with their required large scale and did not allow for zonation of extensive areas according to their susceptibility to slope instability phenomena. The need for this type of zonation has increased with the understanding that proper planning will decrease considerably the costs of construction and maintenance of engineering structures" [p. 129].

The development of the zoning ordinances in Huntsville follows the traditional engineering approach, which focuses on the analysis of recent and/or possibly historic landslides and then sets zoning ordinances based on these analyses and observations. In contrast, Soeters and van Westen (1996) advocate a zoning approach that incorporates knowledge of the processes that affect slope stability. We suggest that this approach may also help resolve the development problems within the Slope Development District. A brief description of the approach by Soeters and van Westen (1996) follows.

Mapping and integrative spatial analysis using GIS methodologies are central to the landslide hazard zonation strategy of Soeter and van Westen. The process begins with rigorous field-based assessment and ends with a probabilistic geographic depiction of landslide susceptibility. Soeters and van Westen divide the analysis into four phases. The initial inventory focuses on documentation of the types, distribution, frequency and spatial density of landslides and identification of homogeneous mapping units. Care must be taken at this stage to distinguish among different modes of slope failure. The next phase, heuristic analysis, involves qualitative geomorphic interpretations of the significance of the spatial patterns of slope failures and preliminary inferences about the principal variables contributing to them. The third phase, statistical analysis, incorporates bi- and multivariate spatial correlation of possible causative factors (geomorphology, soil characteristics, hydrologic properties) with the landslide inventory. Finally, the deterministic analysis phase results in a regional map showing spatial variations in the factor of safety or another index of the likelihood of slope failure. Such an approach will almost certainly result in zoning districts very different from the ones based only on one or two factors, e.g., slope grade and presence of colluvium, in the case of Huntsville, which is how the current zoning ordinance is implemented.

While a complete analysis of applying a zonation approach, as suggested by Soeters and van Westen, to the Huntsville area is beyond the scope of this paper, a number of observations can be made. First, it is clear from the lack of development within the Hazard Zone that developers are unwilling to assume the risk of development or find it uneconomical to conduct the geotechnical investigations required. Moreover, it is questionable whether a geotechnical investigation can actually determine the safety of sites within the Hazard Zone. Because only mudflows have been observed to date, we do not know or understand the mechanism that generated the larger paleolandslides, reoccurrences of which could affect not only sites within the Hazard Zone but also sites all the way to the valley floor outside the boundary of the Slope Development District. Therefore, without an integrated analysis of the slope failures within Huntsville, the ability to *safely* assess the stability of sites within the Hazard Zone remains in doubt. Second, the issue of seismicity is very difficult to assess even with a zonation approach, since the effects of earthquakes in the Eastern United States have not been well understood. Third, given the complex geology of the Huntsville area as discussed above, the current zoning regulations may not be adequate to safe guard development in the upland areas of Huntsville. In light of these considerations, it is suggested that the zonation approach as suggested by Soeters and van Westen (1996) be applied to Huntsville slopes. While the cost of such an undertaking is expensive, it may be the only way to draft zoning regulations that ensure the safety of residents and property while allowing well-judged development of the upland areas of Huntsville and cities in similar geomorphic settings.

CONCLUSIONS

1. Development of steep mountain slopes in Huntsville. Alabama is accelerating in spite of incomplete understanding of the geologic hazards associated with urbanization of these previously uninhabited areas
2. Current zoning regulations are based on a traditional engineering approach. which focuses attention on the analysis of recent landslides. However. additional geologic hazards including large deep-seated landslides and the possibility of seismically triggered slope failures may not be adequately accounted for using such an approach.
3. Implementation of the regulations have caused development to cease in areas designated as lying within the Hazard Zone. An integrated, process-orientated zonation strategy may provide a better approach to understanding instability of the area's slopes.

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Assessing Weak Rock Excavatability: Site Characterization and Predictive Techniques

HARDY J. SMITH
Research Civil Engineer, Rtd., Geotechnical Laboratory,
USAE Waterways Experiment Station, Vicksburg, MS

ABSTRACT

Weak rock presents special problems in exploration and site description as well as in engineering applications of site information as in assessment of rock excavatability. There is a general trend toward the use of mechanical excavation equipment in increasingly difficult materials, where detailed excavatability assessment is needed. Such an assessment requires a description of the rock and rock structure as well as methods of relating such site information to the difficulty of excavation. Recent years have seen improvements in exploration and site description. Developments in drilling exploration and field determination of a strength index for weak rock are presented.

Although more can now be readily learned about subsurface conditions, a similar trend in predictive methods for excavatability assessments has not been observed. The need for a proven predictive system for mechanical rock excavation is explored. Some common approaches for excavatability assessment are reviewed. Mechanical comminution of rock is considered as a function of rock material parameters such as intact strength, bedding, joint spacing and structural orientation.

INTRODUCTION

The term "excavatability" (or "dredgeability" as applied to rock underwater) is defined as the ability to excavate rock with respect to known or assumed equipment (excavation geometry), methods, and in situ material characteristics. The excavation of rock by mechanical means involves several considerations: (1) breaking up or cutting the rock; (2) removal of comminuted material from the cutter or pick area as work progresses; and (3) the overall excavation process including disposal, environmental and political concerns. An estimate of overall production in a rock dredging operation must take all these considerations into account, and any estimate of excavatability for rock with respect to particular material-machine characteristics must involve the first two of these considerations. However, the primary concern in this paper is the first consideration only, and it involves the direct application of rock mechanics. The assumption is made that once the rock is broken up it will be removed from the cutter or pick area as necessary. Certainly, other considerations may also be of critical importance. However, the ability to break up or cut the rock is of primary importance; and any assessment of this ability requires both good site characterization involving geologic and engineering properties and a predictive method for excavatability. Historically, the ability to predict excavatability has not been sufficient to avoid major economic impacts in most excavation projects involving rock.

BACKGROUND

The general trend toward the use of mechanical (non-blasting) excavation equipment in increasingly difficult materials is seen in mining, tunneling, dredging, and open excavations (Smith, 1994a; Bennett et al. 1985; Hignett, 1984; Caterpillar, 1988a). As larger and more powerful machines have become available, their use as an alternative to drilling and blasting of weaker rock has been pushed by both economic considerations and a desire to avoid blasting in developed or populated areas. The rock excavation projects of the US Army Corps of Engineers (Corps) have followed this trend. Corps dredging projects are now typically excavated by contract, and in recent years the Corps has experienced numerous differing site condition claims on rock projects. These claims have typically been large, often into millions of dollars and sometimes far exceeding the original bid cost. These differing site condition claims are commonly based on the contention that the rock encountered is harder to dredge with available equipment than the contractor had inferred from bidding information. Several example rock dredging claim situations were outlined by Smith (1986b). Similar claims frequently occur in surface excavations such as in the construction of the Beaver Dam cut-off trench in which a claim settlement in excess of $3M was recently made. Such claims necessarily hinge upon either the description of the rock material or the predicted performance of particular equipment in excavating such material, the two being interrelated. Assessment of excavatability depends on (1) an accurate description of the rock and rock mass from which engineering behavior can be inferred and (2) the use of a predictive method to determine the ease (or difficulty) of excavation. Even in those cases where these two critical elements are known, and an assessment of relative excavatability can be easily made, equipment capabilities of prospective contractors may vary widely. Equipment capability is not within the scope of this paper; however, it is often a complex issue which involves much more than equipment type and horsepower. Other factors (typically known only to the owner, if at all) include wear, state of maintenance, strength of superstructure, etc. which influence efficiency in production and the ability to absorb reactive forces produced by the excavator.

SITE DESCRIPTION AND EXPLORATION

In Invitations for Bid (IFB's) for dredging work, common practice has been to provide boring logs at selected locations. Unconfined compressive strength and/or geologic description are frequently given, although unconfined compressive strengths are usually not routinely obtained at all borings. However, in some cases where unconfined compressive strengths were given, contractors have encountered unexpected problems. On other projects, contractors have assumed that rock of a given geologic description would be easily excavated with the same or similar equipment previously used on rock of the same type and general description, only to find that excavator performance was vastly different at a new site. Intact rock strength and a general geologic description are important but often not the controlling factors for excavatability. Obvious to most involved in dredging and other excavation is the fact that several engineering parameters influence rock excavator performance. Unconfined compressive strength is almost always considered, but other concerns, particularly joint spacing and laminations, are recognized as having critical influence. However, the use of such information is necessarily subjective and/or based on individual experiences. With the possible exception of ripping, no systematic methods are known which provide an index of difficulty as a function of material properties. Whether a judgement of excavatability is made based on experience with the same equipment in similar rock or on a more systematic determination, a geotechnical characterization of the site to be excavated is first necessary.

These problems are more often encountered in weak or weathered rock which have a greater range of engineering properties and which are more variable in areal and vertical extent.

Recent research performed at the U S Army Waterways Experiment Station (WES) as a part of the Corps Dredging Research Program has developed applications of two new technologies which have capabilities beyond traditional exploration. Although developed to better characterize weak rock to be mechanically dredged, both these technologies have applicability to, and have been used for, site characterization for other rock excavations.

DRILLING PARAMETER RECORDER

The first technology for application to site exploration is the drilling parameter recorder (DPR) which significantly enhances what can be learned from traditional subsurface rock borings (Smith, 1994a,b). DPR is a generic name for systems used to record the operating characteristics of a drill rig. Devices ranging from paper chart recorders to computerized systems for monitoring drilling production rates and efficiencies are commercially available, but virtually all of them record data relative to elapsed time. For site characterization work, the data record must be in direct correspondence to position in the bore hole. This is the primary reason for selection of an Enpasol recorder and related software for this DPR system. For the purpose of this article, "DPR" refers to the WES-modified DPR system using the Enpasol recorder and software by Solentanche. The DPR system described here is the first of its kind to be used in the United States. The DPR monitors operational parameters of an exploration drill rig such as bit pressure, advance rate, rotation speed, relative torque, and position in the bore hole. The associated software can be used to compute other parameters for which relationships are known such as the specific energy of drilling, or to compute estimates of rock parameters for which correlation with drilling parameters have been demonstrated, such as unconfined compressive strength (UCS). The DPR provides a continuous record of drilling parameters which can be related to material properties of the rock or rock mass, using both directly-measured or computed drilling parameters. With the DPR, sites can be better characterized and exploration costs reduced. Methods of use include roller bit drilling for most holes, using drilling parameters to correlate with a small number of more costly cored holes, saving field production and laboratory costs for a given number of holes, or extending needed coverage for limited exploration funds. In conventional coring operations, drilling parameters can be used to estimate material properties and to determine geologic contact elevations with certainty even where core recovery is poor or nonexistent.

Description of the Drilling Parameter Recorder System

To improve the characterization of in situ rock properties, a hydraulic drill rig was instrumented by the WES. The DPR monitors, measures, and records drilling parameters that reflect the operation of the drill rig thereby producing a record of the characteristics of the formation being drilled. In WES operations the following parameters were recorded on an analog graphical plotter and digitally stored on tape by a microcomputer integrated into the equipment:

(1) relative torque indicated by pressure to hydraulic motor for the drill string

(2) downthrust on the drill bit
(3) rate of advance or penetration speed
(4) rotation rate
(5) holdback pressure on drill string
(6) time to drill one digitized increment of depth
(7) drilling fluid pressure

Although the drilling fluid pressure was not used for the dredging applications, it has been used by the WES in grouting work. The software program allows the user to select numerical parameters, plotted parameters, and other pertinent information related to the bore hole. A more comprehensive description of the DPR, dredge site application, and development of capabilities can be found in the WES report on improving site characterization (Smith 1994a).

DPR Field Use

Initially designed for construction exploration, the Enpasol DPR was previously used in Europe on underground construction and grouting projects. The most critical task of the research was to prove the use of the DPR systems in rock exploration for dredging applications. The DPR was installed at WES on a Longyear HC 150 palletized hydraulic drill rig and was first used for subaqueous drilling in New York Harbor in December 1988. The DPR was subsequently used at Grays Harbor, WA, Wilmington Harbor, NC and at Kings Bay, GA. At these field sites the jack-up barge proved best suited to DPR exploration as it provided a very stable, yet mobile drilling platform. However, successful records of drilling parameters were obtained using both anchor and spud barges. In addition to routine site characterization, DPR records were used directly by the New York District to provide a top-of-rock assessment in a complex geology. Field records indicated good correlation with actual site conditions. Data from these field explorations were also used as a basis for developing computed parameters such as specific energy of drilling and in situ strength.

DPR Results and Graphic Displays

The DPR software produces graphic displays of any parameter in several alternative formats. However, much can be inferred from the basic or directly-measured drilling parameters which are observed in real time on an analog plotter and can also be plotted from stored digitized information. For example, since downthrust pressure is relatively constant due to the design of the drill rig, the holdback pressure can be observed to infer relative bit pressure. However, the BITFORCE parameter must be calculated, involving these two pressures, piston size, and the weights of the drill head and rod. Relative torque can be observed by directly monitoring the pressure to the hydraulic drive motor. ROTATION is observed in the field in pulses per second. TIME is recorded for the drill bit to advance one depth increment of 5 mm; it is the inverse of the SPEED parameter and can be used as an advance rate measure in very hard rock. These "raw" data have certain obvious correlation with material character. For example, if other parameters remained unchanged and relative TORQUE increased, a tougher and stronger material would be indicated. Similarly, an increase in rock strength would normally produce an increase in the parameter TIME and a decrease in ROTATION.

Figures 1 and 2 demonstrate example DPR outputs. Data were obtained and interpreted from a single interval of a boring made at Wilmington Harbor, NC. The DPR can be used for all the sizes of core bits and roller bits for which the drill rig has capability. In this case, a 4 x 5.5-in core barrel was used. The recovered core was badly fragmented and eroded, with the largest fragment about 0.8 ft long; approximately 70 percent of the drive was recovered.

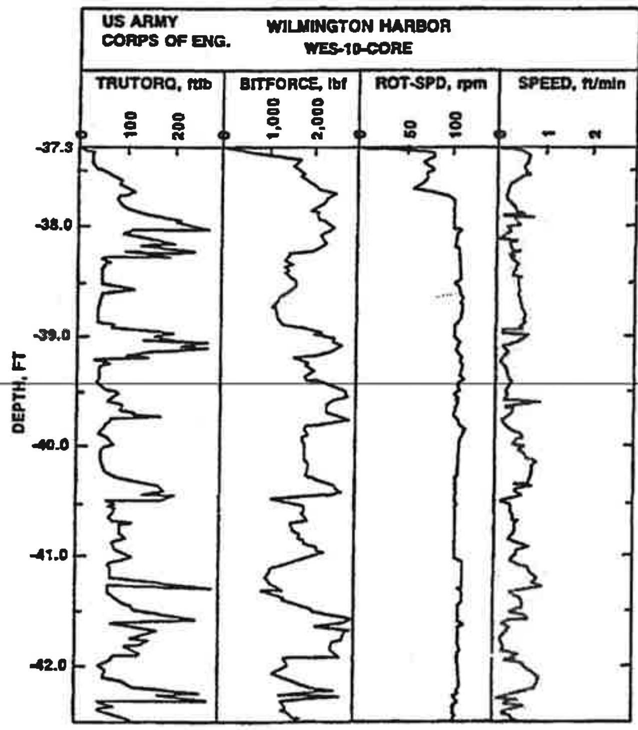

Figure 1. Calculated drill rig mechanical parameters from the directly observed parameters.

Figure 1 shows calculated mechanical parameters describing the rig behavior during drilling the above-noted boring. The TRUTORQ parameter resulted from applying a correlation equation derived from torque versus pressure calibration data. The BITFORCE parameter is the cumulative sum of directed forces and weights bearing on the bit. ROT SPD is rotational speed of the drill string and bit recomputed from pulses per second. SPEED is the rate of advance computed from the TIME parameter.

Specific Energy of Drilling

The specific energy of drilling (E_s) is the energy expended per unit volume of material removed by the drill bit or core barrel. E_s has been incorporated in the DPR as a computed drilling parameter. Because the DPR directly measures only a "relative" torque (pressure to the hydraulic drive motor) and torque was necessary to determine E_s, a torque versus pressure calibration equation was determined using a reaction disk and an in-line torque cell. This calibration is specific to the drill rig so that use of the DPR system on other equipment would require a similar determination and modification of the torque equation. The specific energy expended by rock dredging equipment is expected to correlate with E_s for at least the more massive rock materials but this correlation awaits actual dredging at sites where the DPR system has been used. E_s is computed directly from known physical relationships with no empirical correlation involved. The software determines this parameter from known torque, speed of rotation, advance rate, bit force, and cross-sectional area.

Drilling Parameter Correlation with Unconfined Compressive Strength

Review of DPR records and rock core strengths from field sites, revealed some correlation with UCS. Also, available data from drilling rates in the mining and tunneling industries (Howarth and Rowlands, 1987; and Somerton, 1959) indicate a correlation of drilling parameters (bit force, rpm, and advance rate) with UCS or drilling preformance. Potential for field application of such correlation is good since UCS is an accepted measure of strength, and since UCS correlations can be used immediately, unlike the correlation of specific energy of drilling with energy of excavation, which must await actual rock dredging in order to be established.

Figure 2 displays a combined-parameter estimate of UCS based on Somerton's drilling index using a site-specific fit. A correspondence of this estimated UCS with actual strengths of rock core taken from the same positions in bore holes was observed; however, rock strength at this site is highly variable so that a very large body of data would be required to establish a correlation with confidence.

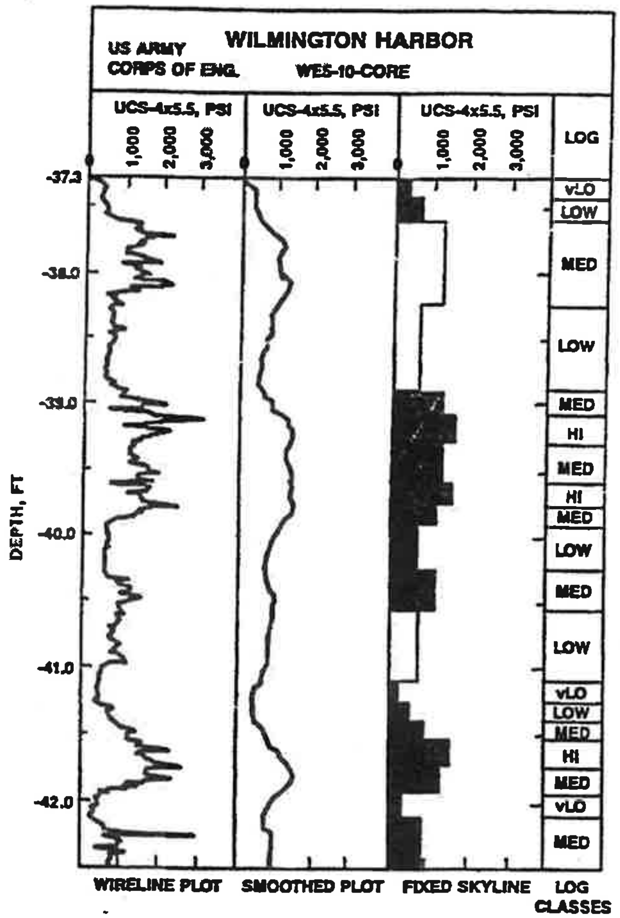

Figure 2. Four graphical forms of combined-parameters of unconfined compressive strength from data shown in Figure 1.

Variability in rock strengths is typical of coastal deposits and most other natural sites. Because field data were so highly variable, the correlation of DPR data with UCS was further established under controlled conditions. Selected uniform natural rock materials and several rock simulants were used to obtain drilling parameter records for materials of known strengths. Figure 3 shows diagrammatically the placement of two natural materials and rock simulants of several strengths. (Although not used for strength correlation, building brick were placed as shown to obtain DPR records in known jointed material. Each layer of brick was indicated on DPR records, which also showed clearly the joint between the two Berea sandstone blocks.) DPR drilling tests were completed in these materials and summary results are shown in Figure 4. For the data sets obtained in each material one averaged data point was determined as indicated. A best fit is shown of UCS plotted against bit force times the square root of rotation divided by advance rate, which is Somerton's index. This correlation, with correlation coefficient of 0.84, is based on DPR records and UCS tests on materials having a wide range in strength — from 300 psi to 10,400 psi. Better correlations (using the general realtionship of these parameters rather than Somerton's index) have been obtained in cases based on fewer materials covering a smaller strength range (Smith 1994a).

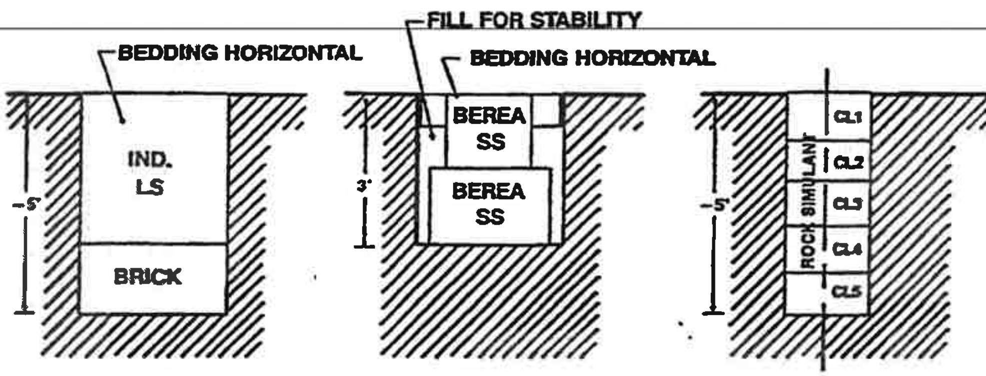

Figure 3. Rock and rock simulant set-up for DPR laboratory drilling tests.

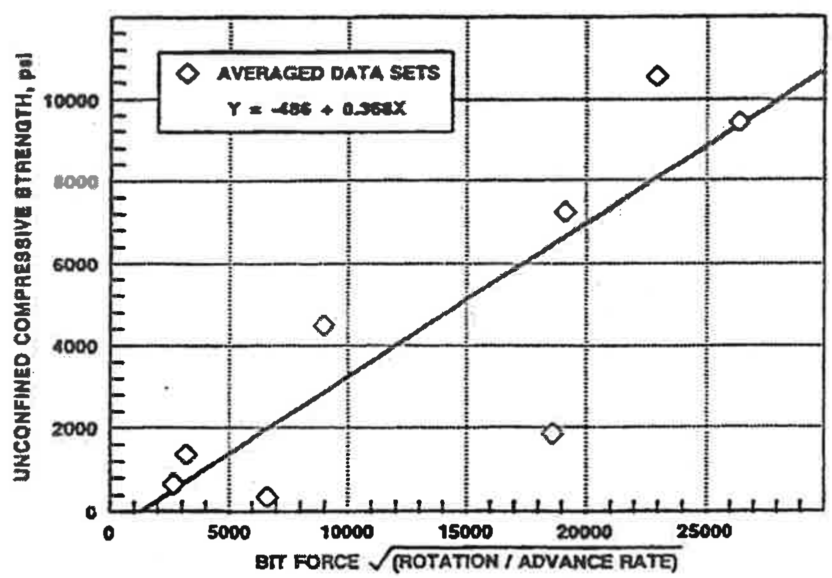

Figure 4. UCS correlation from DPR drilling tests.

POINT LOAD TESTING

Parallel with the DPR work in improving site characterization, research was conducted to develop the point load test as a field strength index for the weak rock typical of many mechanically-dredged coastal deposits (Smith, 1994a). Because point load tests are low in cost and the test apparatus is hand portable with ease, some Corps Districts had begum to use the point load test for monitoring strengths of dredged material prior to the research reported here. However, its use was limited since available published information on the point load test was related to hard rock and no correlations with Unconfined Compressive Strength (UCS) or other common strength parameters were available for weak, saturated rock similar to that at dredging sites. In order to establish modified testing procedures and correlations with UCS for weak saturated rock, point load tests and UCS tests were conducted on rock from the DPR exploration sites and other harbors as well as on other rock selected for its uniformity.

Point Load Test Standards

A proposed standard for the point load test was published by USAE Waterways Experiment Station (WES) in the Corps' *Rock Testing Handbook* (USAEWES 1982). This standard was based largely on Broch and Franklin (1972), supplemented with other works (Bieniawski 1975) and the WES experience. Subsequently the International Society for Rock Mechanics (ISRM) published a suggested method for determining point load strength (ISRM, 1985). This ISRM standard was incorporated in the new *Rock Testing Handbook* (USAEWES 1989) as RTH Std 325-89, replacing the original *Rock Testing Handbook* standard. There were few significant changes in this new standard. One change recommended a reference or standard international size of 50 mm where data from size-dependent point load tests on various-sized specimens were to be converted to one size, as is necessary when point load strengths are used for strength classification purposes. Subesquently, the American Society for Testing and Materials (ASTM) published ASTM D5731-95 "Standard Test Method for Determination of Point Load Strength Index of Rock". This standard is based on the earlier publications given above, and is closely parallel with the ISRM's recommendations. In the following abridged description of the point load tester and the point load index, the specific restrictions and definitions of terms given are consistent with the ISRM's "Suggested Method for Determining Point Load Strength", unless otherwise indicated.

The Point Load Tester

Point load tests are performed by loading the sample between two platens having 60-degree conical points and a 5-mm point radius. Thus, a sufficient point load can be provided to fail even hard igneous samples using a small portable test apparatus. The apparatus consists of an adjustable passive platen and an active platen providing the load through a hydraulic ram; pressure is provided by a second piston manually advanced by a mechanical screw with handle or by a manually operated reciprocating piston with check valve. A hydraulic pressure gauge records pressure at failure, and the gauge reading is multiplied by the area of the piston to give total point load, P, on the specimen. Different gauges can be used to produce accurate readings for both very high and very low point loads to accommodate a wide range of rock materials. More detailed requirements for test apparatus geometry, measuring provisions, and calibration are given in the references cited above. A point load tester may be constructed using the criteria given in these publications, but several manufacturers of testing equipment now market point load testers. Both small hand-portable testers intended for field use and larger, more convenient to use, laboratory testing machines are available.

Point Load Index

Results of point load tests are usually expressed in terms of the point load strength index I_s which is, in accordance with the standards cited above, determined by dividing the total load P by D_e^2 where D_e is the equivalent diameter. The index for a given size core is directly related to the material's tensile strength and can be correlated with UCS. Point load tests may be performed on core specimens without standard preparation or on a series of irregular rock fragments. Tests can be carried out using three different sample geometries, summarized below.

When first introduced, point load strength was mainly used to predict UCS (Broch and Franklin 1972), which was the established test for general rock strength classification. UCS is certainly the only widely accepted strength criteria for dredging applications today. However, even when making correlations to obtain UCS, the I_s should be given. I_s is size dependent and should be correlated to a standard size when published. As given above, the international standard diameter is 50-mm. This index, written $I_{s(50)}$, is often used directly for hard rock classification. The NX core size (54-mm), which is often used in U.S. practice, is close to this size and correction to NX size is common especially where the site exploration used NX-sized core. This strength index would then be designated as $I_{s(NX)}$. Procedures for correcting as-taken I_s to a standard size are given in ISRM's suggested method and in ASTM D5731-95, but the testing of samples close to a standard size is recommended to minimize error.

Unconfined Compressive Strengths

Correlation of I_s with UCS is both material-specific and size-dependent. Therefore, for best accuracy this correlation should be established for each site-specific material. In this case, a number of UCS tests would be necessary; but even so, the time and cost saving for large numbers of strength tests would be significant using the point load tester. On the average, UCS is 20-25 times the point load strength ($I_{s(50)}$), but can vary over a much wider range (ISRM 1985). In reconnaissance exploration where site-specific correlations or other material-specific information is not available, the UCS can be estimated using a size correlation graph or table (Bieniawski 1975, ASTM D5731-95) to obtain the point load index to UCS conversion factors. For example, a conversion factor of approximately 24 is found if using the common NX (54-mm) core size.

Point load tests on igneous and the harder sedimentary rocks could be expected to have a reasonable correlation with UCS using the factors indicated. However, since these results were developed based on hard rock data (Bieniawski 1975, Broch and Franklin 1972), such size correlation graphs provide no basis for use of point load test results for the weaker rock materials.

The Comparative Testing Program

The primary purposes of this testing program were to demonstrate the applicability of the point load test method for weak, saturated dredged rock and to determine any correlation with UCS. In accordance with these major purposes most testing was done on saturated samples. Although outside the scope of this article, some testing of oven dried sandstones and limestones was done to show wet versus dry strength behavior. Point load tests and UCS tests were conducted on rock from several sources: Core of weak rock from several harbors were used; also, Indiana limestone and Berea sandstone were selected for their uniformity, and a rock simulant was used to provide a uniform rock material of low strength.

Data Base System

A data base system was developed to store, retrieve and compare rock test data: the Point Load Index and Unconfined Compressive Strength Data Base System (PLUCS). The PLUCS is an open-ended system, which contained data from over 400 rock tests from 10 different material

Table 1. Summary of number and types of tests contained in the PLUCS data base.

Rock Material	W-Wet D-Dry	UCS	$I_{s(NX)}$
Wilmington Harbor Lime rock	W	21	22
Kings Bay Lime rock	W	21	12
Port Everglades Lime rock	W D	15 3	13 3
Brunswick Harbor Lime rock	W	1	1
Grays Harbor Silty sandstone	D	1	12
Indiana limestone	W D	30 29	35 36
Dardanelle sandstone	W D	11 11	– –
Ozark sandstone	W D	11 10	– –
Berea sandstone	W	16	29
Rock simulant	W	32	31

sources. About three-fourths of these tests were performed on wet samples (see summary of data contents, Table 1). In addition to displaying summary data from individual tests such as type test, sample dimensions, and breaking strength, the PLUCS system will, for a specified material and/or source location, scan the data base and compute average strengths, wet/dry strength ratios, and unconfined compressive strength versus point load index correlation factors. Most point load index tests in this data base were performed on NX-sized (54-mm) samples, a size commonly used by the Corps. Since the point load index I_s is influenced by sample size, and correction to standard size must be made for strength comparison or rock classification purposes, the PLUCS software automatically corrects index values to NX size when data are entered so that all index values recorded in and displayed by the system are $I_{s(NX)}$, although actual sample dimensions are stored.

The PLUCS Data Base System is a self-contained system that can be executed without additional software and can be executed on any IBM-compatible personal computer (PC). The executable version of PLUCS has been published and updated through the Corps Dredging Research Program (Smith 1992, 1994a). All data in PLUCS are given in English (Non-SI) units. A copy of the PLUCS Data Base System is available through the Geotechnical Laboratory, WES, or from the author.

Test Results and Observations

Comparison of averaged strength parameter values of UCS and $I_{s(NX)}$ for the biohermal lime rock from Port Everglades, Wilmington Harbor, and Kings Bay resulted in similar UCS to $I_{s(NX)}$ correlation factors for each of these sites, as shown in Table 2. Although these materials were highly variable as is indicated by the high standard deviations for each of the test sequences, correlation factors are consistently within a small range. The average correlation factor is 14.3, with a corresponding standard deviation for the three sites of less than 7 percent. The apparent inconsistency may be explained by the way in which the samples were taken. Since the primary purpose of this test was to obtain a correlation factor for this variable material, and sufficient material was not available to obtain a large number of samples in each strength range at the sites, point load and unconfined compressive test samples were taken in sets from discrete small volumes of material. In the case of Point Everglades, core was taken for both types of tests from large rock fragments taken from the harbor bottom. Each intact fragment could reasonably be assumed to be much more uniform in strength than the material over the site. Cores for both point load and unconfined compressive samples were taken from each fragment and although each sampling set was too small to infer a reliable correlation factor, all such data were lumped together in the data base from which could be computed a site-specific UCS to I_s correlation factor. The use of a correlation factor so derived makes the assumption that the correlation factor, being material-specific, would change little over the site within the same material, even though strength changes may occur within the range for that material at the site.

A similar bias relative to random sampling was accomplished for the rock from Kings Bay and Wilmington Harbor. Rock was taken in the field using a 4-in. core barrel. Although core recovery was often poor, many core pieces were long enough so that one 45-mm core for point load testing and two 35-mm cores sufficient for unconfined compressive testing could be taken parallel to the original core centerline. Core samples for all these tests were taken to maximize the number of samples from a limited amount of material. Accordingly, a perfect pairing of point load tests with

Table 2. Comparison of results from unconfined compressive and point load testing for biohermal lime rock.

Location	UCS		$I_{s(NX)}$		K
	No. of Samples	**mean* psi**	**No. of Samples**	**mean* psi**	**Correlation Factor**
Wilmington Harbor	21	4,347 (3,191)	22	329 (210)	13.2
Kings Bay	21	3,436 (4,446)	12	232 (260)	14.8
Port Everglades	15	2,141 (1,080)	13	143 (108)	15.0

* Standard deviation shown in parenthesis.

unconfined compressive tests in adjacent material was not possible for all samples, as is evident from Table 2. However, the sampling and testing plan used, coupled with the consistent nature of the rock material, produced $I_{s(NX)}$ to UCS correlation factors consistent over the three sites.

Special Testing Procedures

Testing procedures used were consistent with the ISRM's "Suggested Method for Determining Point Load Strength" and the ASTM D2938-86 "Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens". However, some special additional precautions were needed in point load testing of some rocks.

Most of the biohermal lime rock, even though essentially isotropic, was either vuggy or had local inclusions of weaker material. In this case point load platens must bear on the harder portions of the nonuniform sample to produce the desired tensile loading of the overall cross section. Some coastal materials are weak but sufficiently brittle such that the point load platens can produce a local crushing failure and embed without failing the entire sample. A sample failed in this way is shown in Figure 5. Little of this material was encountered in the comparative testing program; however, in such a case, a valid point load test is not possible. Strength of such materials could, of course, be determined in laboratory UCS tests. However, a field strength test may be desired so that core can be tested in as-taken condition, to save laboratory costs, or for other reasons.

A field strength test on such friable rock is possible using the point load tester. Material encountered in the comparative testing program on which the platens produced local crushing was weak enough so that the entire cross section of core could be easily loaded in compression with a point load tester. Direct UCS tests have been successfully made on such material using flat platens configured to pivot on the point load platens. For material having UCS values under 2,500 psi or more, UCS can be determined in the NX size with a point load tester with alternative flat platens if

Figure 5. Core sample showing local crushing failure at point load platens.

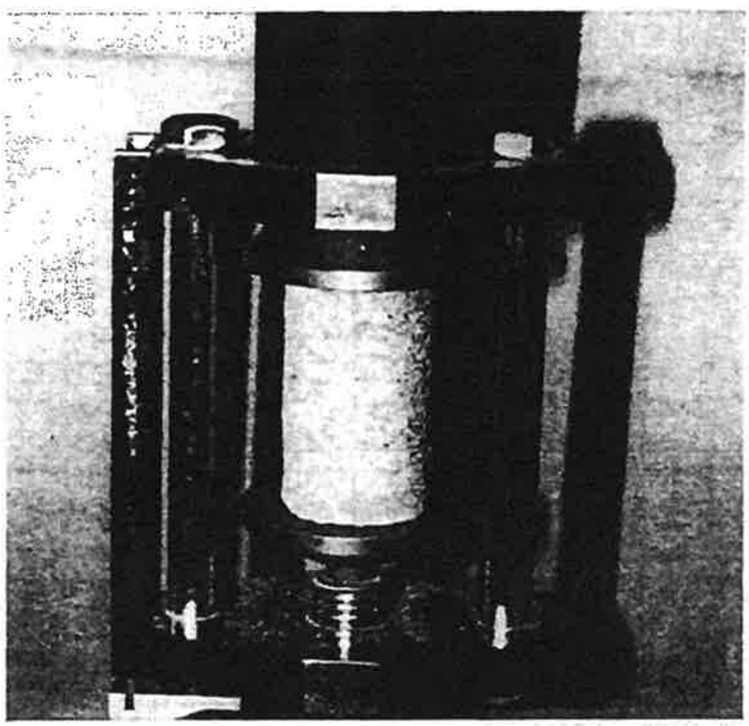

Figure 6. Flat platens pivoted on point load platens to load sample in axial compression.

it is capable of testing the harder igneous rocks in its normal point loading mode. The use of the point load tester for direct UCS tests in the field requires a small rock cutoff saw for sample preparation and does not meet all of the requirements for standard laboratory tests; however, results should reasonably be as consistent as point load strengths and are quite suitable for a field-determined strength index. To demonstrate this technique a test was performed as shown in Figure 6. Test results were entered in the PLUCS data base as an unconfined compressive test, identified with "FL" after the sample number. This sample was from Port Everglades.

Point Load Index Strength Correlation to Unconfined Compressive Strength

The average correlation factor for the three lime rock sites discussed above was 14.3, which is low compared with an expected value of 24 based on hard rock testing experience. Because weak rock materials are by nature nonuniform in strength, the rock simulant was used to further show that consistent point load test results could be obtained for very weak saturated materials and to obtain a correlation factor for a material in this strength range. A total of 32 unconfined compressive tests on this material resulted in an average UCS of 626 psi, with a standard deviation of only 9.3 percent. A total of 31 point load tests were performed resulting in an average $I_{s(NX)}$ of 73.5 with a standard deviation of 16 percent. The corresponding I_s to UCS correlation factor is 8.5. The lowest correlation factor found for a natural rock site was 13.2; however, that was for material of much higher strength. These results show that site-specific correlation factors for weak, saturated materials can easily be one-half or less of published values for hard rock.

Comparative tests on Berea sandstone and Indiana limestone, both selected for their uniformity, resulted in $I_{s(NX)}$ to UCS correlation factors consistent with hard rock testing experience as shown in Table 3. These tests were performed using the same procedures and test equipment as those for the weaker materials, except as noted above under "Special Testing Procedures".

Table 3. Comparison of results from unconfined compressive and point load testing for selected sandstone and limestone.

Material	UCS		$I_{s(NX)}$		K
	No. of Samples	mean* psi	No. of Samples	mean* psi	Correlation Factor
Berea sandstone, W	16	7,284 (703)	29	313 (26.9)	23.3
Indiana limestone, W	30	9,375 (498)	35	408 (98.4)	23.0
Indiana limestone, D	29	11,780 (987)	36	433 (94.6)	27.2

* Standard deviation shown in parenthesis.

Review of the data discussed and displayed above and other data from the PLUCS showed a consistent trend toward lower correlation factors for materials of lower strength. The PLUCS was used to compute a correlation factor for materials for which both $I_{s(NX)}$ and UCS were available. Average UCS for each material type was plotted against those correlation factors as shown in Figure 7. Although shown in the PLUCS content summary (Table 1.), data from Brunswick Harbor were not used because only one test of each type was performed. However, data from Grays Harbor were used since several point load tests were performed and the material showed good uniformity for a weak natural material, with a standard deviation in strength of 31.9 percent. Standard deviations for the other averaged data sets are given above. The author opines that additional data is needed to further establish the relationship between rock strength and the correlation factor, probably resulting in a concave down curve. If UCS is solved for as a function of $I_{s(NX)}$, using the relationship shown between UCS and K, the resulting equation would have a singularity well within the range for hard rock applications, and thus such a relationship would be flawed. However, the linear fit shown is sufficient to demonstrate clearly that site-specific or material-specific correlation factors are lower for weaker rock, and that correlation factors in the neighborhood of 10 could be encountered in very weak rocks.

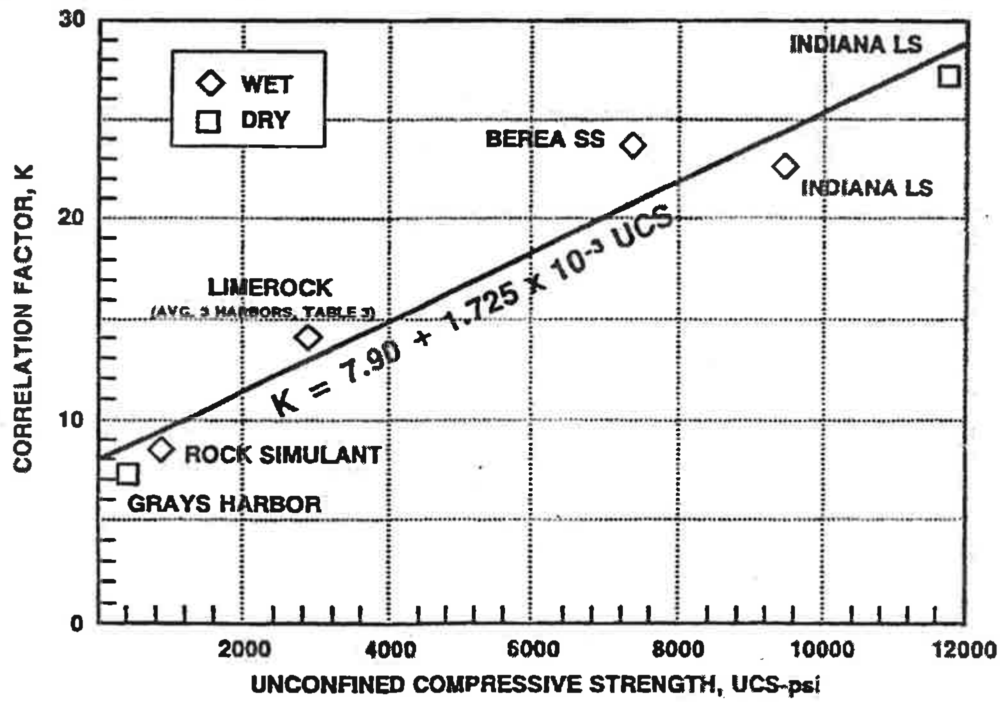

Figure 7: Variations of UCS to $I_{s(NX)}$ correlation factor for low strength rock.

ASSESSING EXCAVATABILITY

Many rock and rock mass parameters can influence excavatability. Perhaps the most often used and widely accepted parameters for excavatability assessment are (UCS) and refraction seismic velocity. The UCS, which is often given in bidding documents for dredging work, has most frequent and suitable application in the more massive and/or weaker materials, while seismic velocity can give a good indication of excavatability in highly fractured rock masses with high intact strengths. Even so, a much wider range of rock mass parameters have influence. Consider two sites having different excavation characteristics: a comparative assessment of the two sites based on only one rock mass parameter must either make the assumption that other influencing parameters change in an orderly way as a function of the parameter under consideration, or make the assumption that other parameters are constant; neither of these assumptions is always valid. Excavatability can be influenced by rock type, strength, degree of weathering, rock structure (including fractures, joints, schistosity, laminations, and orientation), rock fabric, and seismic wave velocity, some of which are interdependent.

Several authors have developed rock classification schemes which take many of these parameters into account to produce a description of rock mass quality. These classifications are useful in various engineering applications, which include or can be adapted for estimating excavatability (Bieniawski, 1974; Weaver, 1975; Kirsten, 1982; Smith, 1986a, 1987). Some of these classification schemes cover a wide range of materials from loose soil to rock requiring blasting; some classifications lack the desired resolution for rock, while others are primarily for other engineering applications or for only one specific excavation geometry. In the case of underwater excavation the same rock parameters may be expected to govern; however, a given rock mass excavated underwater will usually be weaker than similar rock encountered in the more usual surface excavations, because both UCS and joint strength are less for saturated conditions (Vutukuri, Lama and Saluja, 1974; Smith, 1992; Bieniawski, 1974). Also in dredging rock, excavator geometry and available reactive forces are significantly different. Development of systematic method(s) is needed to estimate dredgeability of rock as a function of material properties. In order to develop a predictive system for dredgeability, a determination must be made as to which rock and rock mass parameters are important in particular dredging operations and the relative influence of these parameters based on excavation geometry and site conditions. Such a system is necessarily empirically based and must depend on sufficient case history information or on a large-scale parametric study based on tests under controlled conditions to determine the relative influence of rock mass parameters such as intact strength, joints and fractures, bedding and orientation. Adequate case history information is not available and the large-scale parametric study has not been conducted in large part due to the high costs involved. However, contributing to the resolution of these needs, the Permanent International Association of Navigation Congresses (PIANC) has produced a report on the "Classification of Soils and Rocks to be Dredged" (PIANC, 1984) in which recommended means of describing the engineering characteristics of the rock are given, and several of the rock mass parameters given above are listed. Although helpful, this classification is descriptive in nature rather than deterministic, and provides no means of rating the relative difficulty in dredging various rock materials.

Estimating Mechanical Excavatability

In the absence of a well defined system for assessing mechanical excavatability, the methods mentioned above, as well as equipment-specific knowledge of individuals, can be used to estimate relative difficulty. However, all methods of direct mechanical excavation commonly used for rock excavation involve some elements of ripper action, and ripping is the primary means of mechanical pretreatment for rock dredging. On-the-surface ripping is perhaps the most basic and well defined means for mechanical excavation of rock. Accordingly, rippability is recognized as an important index property for dredgeability of weak rock.

The refraction seismograph method for rippability assessments was developed during 1958 by Caterpillar, Inc. A detailed description of this widely accepted method can be found in Caterpillar's "Handbook of Ripping" (1988a), and in the US Army Corps of Engineer Manual EM 1110-1-1802, "Geophysical Exploration" (1979). The relationship of this method to other rock mass parameters was explored by Smith (1986b) who gave an approximate correlation between seismic velocity under average conditions and a rippability rating (RR) based on the weighted influence of several rock mass parameters. The refraction seismograph method has rarely been used for dredging or marine engineering applications because of the difficulties in handling the hydrophones and energy sources at sea, the need for accurate positioning on the bottom, and the cost compared with seismic reflection methods. However, seismic reflection, which has been more widely used for these applications, cannot provide a quantitative measure of rock quality.

Relative Descriptive Classification	Very hard ripping or blasting	Hard Ripping	Average ripping	Easy Ripping	Very easy ripping
Rock Hardness*	Very hard > 70 MPa	Hard 25 – 70 MPa	Medium hard 10 – 25 MPa	Soft 3 – 10 MPa	Very soft < 3 MPa
Rating	$\geq$ 10	5	2	1	0
Rock weathering	Unweathered	Slightly weathered	Weathered	Highly weathered	Completely weathered
Rating	10	7	5	3	1
Orientation	Very unfavorable	Unfavorable	Slightly unfavorable	Favorable	Very favorable
Rating	15	13	10	5	$\leq$3
Joint spacing (expressed in ripping depth , D)	> 3D	D to 3D	D/3 to D	D/20 to D/3	< D/20
Rating	30	25	20	10	5

*Corresponding to unconfined compressive strength.

Figure 8. Underwater rippability rating chart.

The underwater rippability rating (RW) was proposed by Smith (1987) which was based on a modification of the RR, cited above. Because the RW is based on an extrapolation of dry excavation technology and has no direct empirical basis, no attempt was made to relate it to production rates as is possible with the RR. The RW was proposed as providing a measure of relative difficulty with higher ratings corresponding to more difficult ripping. The RW reflects several changes over the RR. Two parameters, joint continuity and joint gouge, which are readily observed in open excavation and tunneling applications but are rarely observed in dredging exploration, were not used, resulting in a four parameter system. As an example of these rating schemes, Figure 8 shows influence values for the RW rating, which retains similar parameter values but uses descriptions tailored to dredging applications. Rock hardness relates directly to UCS and is in agreement with the Core Logging Committee, Association of Engineering Geologists (1978). Orientation for rippability is most adverse for vertical or horizontal bedding and favorable near 45 degrees. Accordingly, that rating number may be expressed by:

$$\text{Orientation Rating} = \frac{|\alpha - 45|}{3}$$

where α is the angle of the bedding with the horizontal taken in the direction of ripping. The joint spacing is expressed in multiples of ripping depth, D, in order to account for a wide range of ripper sizes, including the large single-tooth rippers, rock picks on cutter heads, and bucket teeth. Using the table a total underwater rippability rating can be determined by adding the rating for each of the rock mass parameters shown, resulting in a quantitative estimate of relative ripping difficulty.

CONCLUSIONS

The DPR has provided information on subsurface rock material properties based on drilling records for a wide range of field conditions. It can be used to lower exploration costs by roller bit drilling most holes, using the drilling parameters to provide site-specific correlations with a number of cored holes. This method allows for more bore holes at a site for the same cost since roller bit drilling is faster and also it requires no casing over water. This approach is particularly valuable when rock materials are highly variable with depth and over the site area. When core is taken in coastal deposits, core recovery is typically poor. In this case, the DPR records show where in the core run material was recovered; and, geological contact elevations can be determined with certainty even when no core is recovered.

The point load test has been shown to be useful for weak rocks. Consistent, repeatable test results as well as correlation with unconfined compressive strengths have been shown. Point load index to unconfined compressive strength correlation factors are low for such materials. Applicable established testing procedures should be followed (ASTM D5731-95, ISRM 1985) with special additional precautions/procedures in the case of vuggy or weak and friable rock. If a site-specific correlation factor is not possible for correlations of I_s to UCS, a material-specific correlation factor should be used, such as is available in the PLUCS for several rock materials. The use of published average correlation factors based on hard rock testing should be employed only for very rough approximations or to assess relative strengths in the field, since results could be in error by a factor of two or more as has been shown for some weak, saturated rock.

Recent years have seen improvements in exploration and site description, with developments in drilling exploration and rock testing as well as the means to interpret and display field data. Although more can now be learned about subsurface conditions, a similar trend in predictive methods for rock excavatability has not been observed. An empirically based systematic predictive method is needed. While sufficient case history information does not now exist, the development of such a system could come from observations of future rock dredging and surface excavation operations, documented for the purpose of developing a predictive system. Certainly this process requires many case histories and could only come about during the next few decades as major constructions are excavated. In order to assess the relative influence of key rock mass parameters such as joint spacing, structural orientation, and intact strength within a time frame to benefit the future deepening projects, large-scale parametric studies under controlled conditions are necessary.

Underwater and surface excavations present a wide spectrum of site conditions, excavation geometry, and equipment capability. Because the ripper is the primary means of mechanical pretreatment for weak rock, and ripper geometry on a smaller scale is incorporated in most rock dredging equipment, rippability can provide a reasonable index of dredging difficulty. The RW provides a systematic means of quantifying relative rippability while taking into account a wider spectrum of rock mass properties than those usually considered for dredging exploration. Although the RW is limited to assessment of relative ripping difficulty, the RR can be related to production. In applying these rippability ratings, the use of more than one method of assessment is specifically recommended and would be of particular value where unusual conditions exist or are suspected.

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Summary of Pre-Symposium Questionnaire

PAUL M. SANTI
Department of Geological and Petroleum Engineering, University of Missouri-Rolla, Rolla MO 65409

ABDUL SHAKOOR
Department of Geology, Kent State University, Kent, OH 44242

ABSTRACT

The final presentation at the Symposium on the Characterization of Weak and Weathered Rock Masses consisted of a panel discussion among the symposium speakers. In order to provide a basis for discussion, a questionnaire was distributed and completed prior to the symposium. This paper summarizes the responses to the questionnaire.

In general, the most significant weak and weathered rock problems identified by the respondents correspond with the common blunders they also identified: collecting and preserving representative samples for testing, measuring or estimating strength values that will apply over the life of the project, and detecting and accounting for the rock's reactivity to water. There was some agreement among the respondents regarding the important field characteristics to record, namely, bedding thickness and orientation, state of weathering, joint spacing and friction, rock strength, and for weathered rock, the nature of the matrix materials. Many more field characteristics were suggested, though with less agreement among the respondents.

Respondents noted that laboratory tests were highly project specific, but the preferred tests include slake durability, unconfined compressive strength, and swelling tests. Less popular, but important laboratory tests include shear strength, mineralogy (x-ray diffraction), and Proctor density and moisture content. There was little agreement over the preferred classification scheme for weak or weathered rock, with ten respondents suggesting nine different schemes.

INTRODUCTION

The final presentation at the Symposium on the Characterization of Weak and Weathered Rock Masses consisted of a panel discussion among the symposium speakers. In order to provide a basis for discussion, a questionnaire was distributed and completed prior to the symposium. This paper summarizes the responses to the questionnaire, and interprets the significance of the range and type of responses.

Questionnaires were mailed to the 18 authors and co-authors participating in the Symposium and concurrent Special Publication. Ten authors completed and returned questionnaires (56%), three individuals did not consider themselves qualified to complete the questionnaire, and five were unaccounted for.

The questionnaire consists of 14 questions, some of which contain a suggested list of possible answers and some of which are completely open-ended. The questions may be grouped into four categories: General Significance, Field Characterization, Laboratory Characterization, and Common Blunders.

The background of the respondents varies considerably. Although most of the respondents are engineering geologists, their respective work with weak and weathered rock ranges from field classification to laboratory test methods to design input for a variety of engineering projects. The majority of authors and co-authors currently hold academic jobs. However, the reader should note that more than half of the respondents are either currently employed by consulting or AE-firms or have been employed by them in the past. Consequently, the overall level of practical experience with weak and weathered rock represented by the questionnaire is very high.

GENERAL SIGNIFICANCE

This series of questions is intended to provide an overview of the engineering difficulties associated with weak and weathered rock and preferred solutions available to the practitioner. Responses are categorized as "Suggested" and "Volunteered." Suggested responses were listed as example answers in the questionnaire. In every case, there were several more suggested responses in the questionnaire than were selected by the respondents. Volunteered responses were write-in answers which were not specifically suggested in the questionnaire. The "# of Responses" column lists the number of respondents who selected a particular answer. In some cases, a respondent's vote was split between answers, resulting in a decimal fraction for "# of Responses."

1. In your experience, what has been the single most significant problem in dealing with weak and weathered rocks?

Suggested Possible Responses	*# of Responses*
collecting samples appropriate for lab testing	3
measuring / estimating strength	3
lateral inhomogeneity	1
reactivity to water (slaking, swelling)	1
Volunteered Responses	
detailed field mapping for 3-D characterization	1
good site characterization where core recovery is poor	1
Suggested Responses Which Were Not Selected	
selecting representative samples	0
vertical inhomogeneity	0
loss of strength during the project time frame	0
correlating block and mass properties	0
lab results out of range for standard soil or rock tests	0

2. In your experience, what has been the second most significant problem in dealing with weak and weathered rocks?

Suggested Possible Responses	*# of Responses*
reactivity to water (slaking, swelling)	2
correlating block and mass properties	1.5
lab test results out of range for standard soil or rock tests	1.5
selecting representative samples	1
collecting samples appropriate for lab testing	1
measuring / estimating strength	1
Volunteered Responses	
applying petrographic analysis	1
change in strength from in situ to lab	0.5
field estimation of strength	0.5
Suggested Responses Which Were Not Selected	
lateral inhomogeneity	0
vertical inhomogeneity	0
loss of strength during the project time frame	0

3. Please indicate how successful you have been applying standard rock and soil mechanics methods to deal with weak material.

Suggested Possible Responses	*# of Responses*
highly successful, no problems noted	0
moderately successful, experience and caution are sufficient	4
slightly successful, works in some cases but not in others	2
unsuccessful, standard weak rock methods must be developed	4

4. Recognizing that there are different levels of work in sample preparation and reliability of test results, which strength test do you think is the most appropriate for a typical weak rock characterization project?

Suggested Possible Responses	*# of Responses*
unconfined compressive strength	2.3
use of a rock mass rating system	2.3
backcalculation based on standing slopes, tunnels	2
point load strength	1.3
plate load strength	1
Volunteered Responses	
depends on material and situation	2
Suggested Responses Which Were Not Selected	
triaxial compressive strength	0
Schmidt hammer rebound	0
direct shear strength	0

5. Which durability test do you think is the most appropriate for a typical weak rock characterization project?

Suggested Possible Responses	*# of Responses*
slake durability	6.3
slake index	1.3
jar slake	0.3
Suggested Responses Which Were Not Selected	
rate of slaking	0
hardness tests	0
Los Angeles abrasion test	0
modified soundness test	0

6. Which system do you prefer to classify and characterize weak rock (references are given at the end of this summary)?

- Palicki (1995) (suggested by two respondents),
- Geological Society Engineering Group Working Party (1995),
- Blatt (1982),
- Deere and Miller (1966),
- RMR (Bieniawski, 1973),
- Q (Barton, et al, 1974),
- Santi (1995),
- Franklin (1981),
- Dick and others (1994), and
- System should be a function of need, application, and site specifics.

FIELD CHARACTERIZATION

The two questions below are aimed at revealing not only the most important items to be noted in the field, but also the range of minor details which the respondents have found useful in their analyses. No list of suggested responses was included, so all responses were volunteered by the respondents. The responses were ranked by awarding five points for each time an answer was the first one listed by a respondent, four points for each time an answer was the second one listed, and so on. In some cases two answers were written on the same line, so the point value was split.

7. What are the five most important details to be recorded in the field to best characterize a weak rock mass?

Volunteered Responses	*Score*
bedding thickness and orientation	19
state of weathering	19
joint friction and joint surface conditions	15
estimate of rock strength	13
joint spacing	13.5
rock name and description	10.5
induration of fresh sample	8
presence of water	8
presence of strong interbeds in weak rock mass	8
evidence of slope instability and failure modes	6
reaction to water / slaking	5
sustainable slope angle	5
accurate topography	5
homogeneity	5
lateral and vertical variation	4
site conceptual geologic model	3
conditions under which data was collected	1

8. What are the five most important details to be recorded in the field to best characterize a weathered rock mass?

Volunteered Responses	*Score*
state of weathering	39.5
nature of matrix/clay mineralogy	12
ease of manual excavation / induration	11
estimate of rock strength	10
corestone size / joint spacing	8
nature of contact between fresh and weathered rock	7
presence of water	7
rock name and description	5
natural slope angle	5
evidence of slope instability and failure modes	5
accurate topography	5
type of weathering (erosion, chemical, etc.)	4.5
% corestones	4
sonic velocity	4
thickness of weathered material	4
site conceptual geologic model	3
reaction to water / slaking	2
remnant planes of weakness	1

LABORATORY CHARACTERIZATION

The goal of these questions is to reveal the preferred laboratory tests for various types of projects encountering weak rock. As with the "Field Characterization" questions, no suggested responses were included in the questionnaire, and points were awarded based on the place in each respondent's list.

9. If a client called you for help in designing a geotechnical investigation for a <u>cut slope</u> in shale, what five laboratory tests would you say are the most important?

Volunteered Responses	*Score*
slake durability	30
unconfined compressive strength	15
direct shear strength	12
x-ray diffraction / clay mineralogy	11
plasticity indices	8
rate of slaking	4
triaxial compressive strength (saturated)	4
specific gravity / unit weight	4
jar slake	3
slake index	3
freeze-thaw	3
absorption	3
grain size distribution	2
swell potential	2
point load	1

10. If a client called you for help in designing a geotechnical investigation for a an <u>embankment</u> built of excavated shale pieces, what five laboratory tests would you say are the most important?

Volunteered Responses	*Score*
slake durability	27
Proctor density and moisture content	13
swell potential	8
plasticity indices	7
slake index	7
bulk density	5
modified soundness	5
direct shear strength	5
freeze-thaw	4
porosity	3
jar slake	3
rate of slaking	3
triaxial compressive strength	3
point load strength	3
grain size distribution	2
modified soundness	2
x-ray diffraction / clay mineralogy	1
soundness as recompacted	0.5

11. If a client called you for help in designing a geotechnical investigation for a tunnel through shale, what five laboratory tests would you say are the most important?

Volunteered Responses	*Score*
unconfined compression strength	15
slake durability	12
swell potential	8
RQD	5
elastic modulus	5
sonic velocity	5
jar slake	4
x-ray diffraction / clay mineralogy	3
unit weight	3
triaxial compressive strength	3
shear strength of discontinuities	2

12. If a client called you for help in designing a geotechnical investigation for a building foundation on shale, what five laboratory tests would you say are the most important?

Volunteered Responses	*Score*
free swell / swelling pressure	18
unconfined compressive strength	11.5
direct shear strength	8
x-ray diffraction / clay mineralogy	6
plate load strength	5.5
slake durability	7
sonic velocity	5
plasticity indices	5
Jar Slake	5
pyrite / gypsum content for swelling potential	4
unit weight	3
triaxial compressive strength	3
slake index	3
grain size distribution	2

COMMON BLUNDERS

One of the most problematic aspects of weak and weathered rock is that many practitioners do not recognize that it is a material which will behave differently than typical soil and rock. The following two questions are intended to point out some of the more common mistakes encountered by the respondents. No responses were suggested for these questions, and all answers were volunteered by the respondents. Answers listed more than once show the total number of times listed in parentheses.

13. Please name one or two blunders that a person unfamiliar with weak rock might make.

- Ignore reactivity to water (3),
- Assume fresh rock behavior is maintained over time (3),
- Fail to preserve in situ moisture content prior to testing (2),

- Take no special precautions to collect good samples and get valid test results,
- Collect weathered specimen for testing when unweathered is needed,
- Incomplete field work to characterize the rock mass,
- Neglect to integrate design requirements with geologic model,
- Assume weak rock totally unacceptable for any use,
- Assume geologic continuity across site,
- Neglect strength variations across mass of rock,
- Fail to acquire representative samples,
- Ignore effects of ground water and excavation on rock properties,
- Neglect rock mass properties,
- Assume intact strength is the only or primary influence on engineering behavior, and
- Assume that a small number of borings is sufficient to characterize the site.

14. Please name one or two blunders that a person unfamiliar with weathered rock might make.

- Assume that a corestone is bedrock,
- Ignore remnant planes of weakness,
- Fail to identify the weathering processes acting at the site,
- Neglect to relate lithology/petrography to observed physical characteristics,
- Assume weak rock totally unacceptable for any use,
- Fail to recognize the degree of weathering,
- Use an incomplete geologic interpretation,
- Rely on strength values which do not represent the range of site conditions,
- Fail to preserve in situ moisture content prior to testing,
- Assume fresh rock behavior is maintained over time,
- Neglect rock mass properties,
- Ignore reactivity to water,
- Assume that rating for weathering and geologic description are all that are required, and
- Overestimate the field performance of the rock mass.

CONCLUSIONS

In general, the most significant weak and weathered rock problems identified by the respondents correspond with the common blunders they also identified: collecting and preserving representative samples for testing, measuring or estimating strength values that will apply over the life of the project, and detecting and accounting for the rock's reactivity to water. There was some agreement among the respondents regarding the important field characteristics to record, namely, bedding thickness and orientation, state of weathering, joint spacing and friction, rock strength, and for weathered rock, the nature of the matrix materials. Many more field characteristics were suggested, though with less agreement among the respondents. Respondents noted that laboratory tests were highly project specific, but the preferred tests include slake durability, unconfined compressive strength, and swelling tests. Less popular, but important laboratory tests include shear strength, mineralogy (x-ray diffraction), and Proctor density and moisture content. There was little agreement over the preferred classification scheme for weak or weathered rock, with ten respondents suggesting nine different schemes.

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