Chapter 21

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# SHALES AND OTHER DEGRADABLE MATERIALS

## 1. INTRODUCTION

The designers of earthwork must take precautions when the materials at hand cannot be classified as rock or as soils in terms of their behavior in slopes or in civil engineering works in general. In their in situ form, the geologic formations may have names or appearances that imply rocklike behavior. Once disturbed, however, some of these formations retain the character of rock, but others may degrade to soil-size particles in a time frame that is relevant to the long-term performance of slopes built in, on, or with these materials.

The currently available methods of identifying, classifying, and treating these degradable materials so as to reduce the risk of slope failure are discussed in this chapter. Sedimentary rocks, which constitute the bulk of degradable materials worldwide, are discussed first. Other degradable materials, including weathered igneous and metamorphic rocks, are discussed in less detail. Emphasis is placed on the successful use of these materials in embankments and on their treatment in the formation of cut slopes. In geologic terms, all of the soils and rocks in the earth's crust are "degradable" materials, since all materials modify over geologic time. However, on the human time scale, only comparatively rapid degradation of a strong or hard rocklike material into a weaker soil-like material is of concern to the designer of stable slopes. Certain sedimentary rocks can exhibit a loss of strength that can be of several orders of magnitude within a time frame that may be as short as only a few hours. Such a rapid breakdown is easily identified by straightforward laboratory tests. Other rock materials show no appreciable change in strength over many years. Unfortunately for engineers, the problem materials fall somewhere between these extremes of rock behavior.

Predicting the behavior of degradable materials has been the subject of much research since the early 1960s, and this research thrust continues today. Much of this research can be tied to the construction of large transportation facilities. In the United States, the development of the Interstate highway system required much higher cuts and embankments than had been common in the past. Problem geologic materials that previously could be economically addressed by avoidance, minor mitigation, or maintenance created the need for new engineering solutions. In Central America the construction of the Panama Canal resulted in slope problems that continued decades after construction and are also associated with the properties of degradable materials (Berman 1991).

## 2. GEOLOGICAL CONSIDERATIONS

Degradable materials were not considered in detail in the 1978 landslide report, but the basic principles that govern them were identified:

> Before one can completely comprehend the particular problems of stability, one must under-

> stand the lithology of the physical properties not only of the rock mass itself but of all the materials in the mass.... A sedimentary rock sequence, for example, is markedly different from an igneous series or a metamorphic complex. Each particular type is characterized by a certain texture, fabric, bonding strength, and macro and micro structures. The most important rock properties are the nature of the mineral assemblage and the strength of the constituent minerals; a rock material cannot be strong if its mineral constituents are weak or if the strength of the bonds between the minerals is weak. (Piteau and Peckover 1978, 194)

The strength of the bonds between the minerals is also related to the geologic history of the formation of interest. The resulting hardness is generally due to long-term consolidation under external pressures and not to cementing minerals (McCarthy 1988). Degradable materials can be grouped according to two broad geologic sources, those derived from sedimentary rocks and those derived from igneous and metamorphic rocks.

### 2.1 Degradable Materials from Sedimentary Rocks

Shales constitute about one-half of the volume of sedimentary rocks in the earth's crust. They are exposed or are under a thin veneer of soil over a third of the land area (Franklin 1981). Shales are by far the most pervasive and problematic degradable material. As early as 1948, Taylor stated (53): "Shale itself is sometimes considered a rock but, when it is exposed to the air or has the chance to take on water, it may rapidly decompose."

In the American Geological Institute's *Glossary of Geology*, shale is defined as

> a fine-grained detrital sedimentary rock, formed by the consolidation (esp. by compression) of clay, silt, or mud. It is characterized by finely laminated structure, which imparts a fissility approximately parallel to the bedding.... [It is composed of] an appreciable content of clay minerals and detrital quartz. [Shale includes rocks such as] claystone, siltstone, and mudstone. (Bates and Jackson 1980, 573)

Referring to Huang (1962), Hopkins noted that

> typically, shales are composed of about one-third quartz, one-third clay minerals, and one-third miscellaneous substances. The principal minerals of shales, such as quartz, clay minerals, and hydrated oxides (such as bauxite and limonite), are formed by the weathering of feldspars and mafic igneous rocks. Some associated minerals such as calcite, dolomite, pyrite, illite, and glauconite are formed during and after deposition of the primary minerals. (Hopkins 1988, 8)

Unfortunately, many of the shale particles are less than 1 μm in diameter, and consequently study of their mineralogy is difficult or impossible by simple visual observation. The resulting geologic field classification of shales does not reliably relate to engineering properties.

Terzaghi and Peck described very clearly the geologic processes that lead to the problem properties of shales:

> As the thickness of the overburden increases from a few tens of feet to several thousands, the porosity of a clay or silt deposit decreases; an increasing number of cohesive bonds develops between particles as a result of molecular interaction, but the mineralogical composition of the particles probably remains practically unaltered. Finally, at very great depth, all the particles are connected by virtually permanent, rigid bonds that impart to the material the properties of real rock. Yet, all the materials located between the zones of incipient and complete bonding are called shale. Therefore, the engineering properties of any shale with a given mineralogical composition may range between those of a soil and those of a real rock. (Terzaghi and Peck 1967, 425-426)

These two authors further suggested using an immersion test on intact samples to obtain the relative performance of otherwise "identical sedimentary deposits." As will be discussed in Section 3, this was the direction taken by many researchers of that time.

Just as increasing loads over geologic time play an important role in the interparticle bonding of shale formations, the reverse process, unloading, has significant effects. During the removal of load, "the shale expands at practically constant horizontal dimensions" (Terzaghi and Peck 1967, 426).

During expansion, the interparticle bonds are broken, and joints form at fairly regular spacings. At depths on the order of 30 m, the joints are spaced meters apart and are closed. Closer to the surface, intermediate joints form because of differential movements between the blocks. These joints open, allowing moisture to penetrate. The increase in moisture content may reduce the shear strength, and, if so, new fissures are formed. The final result and slope of any exposed face depend on the interparticle bonding remaining in the shale formation.

## 2.2 Degradable Materials from Igneous and Metamorphic Rocks

Because sedimentary rocks (and shales in particular) are typically formed relatively near the earth's surface and without the extreme heat and pressure that occur at depth, they tend to be mineralogically stable near the surface. Weathering of these materials then involves either a reversal of the consolidation pressure or a dissolution of cement bonds holding the grains or mineral groups together. In contrast, igneous and metamorphic rocks are created under temperature and pressure conditions that are drastically different from conditions at the surface. Macias and Chesworth described the implications of the difference:

> One might therefore expect that they would weather more readily than sedimentary materials.... Generally however, expectations in this regard are not fulfilled. Soils form more readily on sedimentary rocks than on other types and the reason is obviously hydrodynamic. For chemical weathering to take place to any significant degree, water must circulate through the rock, and the open structure of most sedimentary materials is more conducive to this than is the restricted porosity of most igneous and metamorphic rocks...the igneous rocks most susceptible (to weathering) are those with an open structure such as the non-welded pyroclastics. Again, in the metamorphic regime the importance of hydrodynamics is shown in that a vertical disposition of filiation encourages a more facile descent of aqueous solutions and a more rapid weathering, than a horizontal foliation. (Macias and Chesworth 1992, 283)

Weathering of igneous and metamorphic rocks is generally divided into two categories: physical and chemical. Ollier (1969) described in detail several types of physical weathering, including sheeting or spalling (fracturing parallel to a free surface created by erosion, excavation, tunneling, etc.), frost weathering (extension of fractures by expansion of freezing water), salt weathering (extension of fractures by the growth of salt crystals), and isolation (partial disintegration of the rock caused by the volume changes accompanying temperature changes).

Ollier (1969) also described types of chemical weathering, including solution (dissolution of soluble minerals, particularly salts and carbonates), oxidation and reduction (chemical alteration of minerals to form oxides or hydroxides), hydration (incorporation of water to create a new mineral), chelation (leaching of ions such as metals), and hydrolysis (reactions between minerals and the component ions of water).

As noted by Macias and Chesworth (1992), chemical weathering, which brings about mineralogical changes in igneous and metamorphic rocks, is usually more crucial than physical weathering in defining the strength properties of the materials. Physical weathering, however, does provide avenues for water to enter the rock by the creation and extension of fractures and subsequently encourages the more rapid progress of chemical weathering by an increase in surface area exposed to water.

Obviously some minerals, and therefore some rocks, are more susceptible than others to the weathering processes described above. For instance, Bowen's Reaction Series (Goldich 1938), which describes crystallization of magma, may be reversed to model the weathering process: calcium plagioclase weathers more readily than sodium plagioclase, and olivine weathers more readily than biotite, which weathers more readily than muscovite, which weathers more readily than quartz. Thus, rocks containing high percentages of calcium plagioclase or olivine will weather faster than rocks containing high percentages of sodium plagioclase or quartz.

Detailed descriptions of the weathering products of minerals have been provided by Macias and Chesworth (1992), Ollier (1969), and Carroll (1970). In general, most silicates (feldspars and micas in particular) weather into clay minerals. Under more extreme conditions, such as those found in tropical or humid climates, or after long periods of time in a geologic sense, they break down further into oxides and hydroxides of aluminum and iron. The specific types of clay minerals formed depend to a great degree on the parent materials, the pH, and the extent of saturation.

Because the weathering products of rocks are largely a function of the mineralogical composition, certain igneous and metamorphic rock types that share common minerals may share similar weathering characteristics. The following observations may assist in predicting general weathering characteristics of igneous and metamorphic rocks (Ollier 1969; Macias and Chesworth 1992):

1. **Granite and diorite:** Because granites typically exhibit massive structure, they also typically develop unloading fractures when exposed at the surface. Continued physical weathering increases the surface area exposed to chemical weathering. Chemical weathering alters feldspars and micas into clays, whereas quartz persists as a sand. According to Ollier (1969, 81), "weathering often follows the joints, and isolated joint blocks weather spheroidally, leaving 'corestones' of unaltered granite in the center."
2. **Gneiss and amphibolite:** In igneous and metamorphic rocks, feldspars and pyroxenes tend to weather rapidly, amphiboles weather at an intermediate rate, and quartz and accessory minerals are persistent. Ollier (1969, 82) noted that gneiss, in particular, "is rarely as well jointed as granite, so unloading is not common, or at least harder to detect. Minerals are segregated into bands, and bands of the most weatherable mineral affect the total rock strength—a property that often proved troublesome in engineering."
3. **Schist, slate, and phyllite:** Ollier (1969, 82) noted that "these [schists] have marked fissility along the 'schistosity' and this is very important in weathering. They contain some very resistant minerals but weathering is moderately easy. Frost weathering can rapidly break up schist."
4. **Basalt and peridotite:** According to Ollier (1969, 81), "Basalt is attacked first along joint planes, leading eventually to spheroidal weathering. All the minerals are eventually converted to clay and iron oxides, with bases released in solution, and as there is no quartz in the original rock, the ultimate weathering product is often a brown base rich, heavy soil." Peridotite shares mineralogical characteristics with basalt and may be anticipated to weather similarly.

The weathered rock product referred to as *saprolite* is of particular interest in evaluating engineering properties. Saprolites are "rotten rocks," or rocks in which the rock structure is preserved but many of the less durable minerals have altered to clay. Saprolites generally form less stable slopes than their parent rocks because of the increased amount of clay and loss of interlocking structure. They also maintain zones of weakness by preserving the general rock structure, or new zones of weakness may be created by preferential weathering along bands of less stable minerals.

Saprolites that preserve zones of weakness from the original rock structure or contain intact, unweathered blocks may be expected to behave like degradable materials. Saprolites that do not have these characteristics may be expected to behave like deeply weathered soils, whose properties are better described by a system that addresses tropical soils, as discussed in Chapter 19.

This brief geological background clearly establishes that the evaluation of degradable materials is complex and that no single approach to determining the long-term behavior is likely to work for every formation. Thus, many researchers have concentrated on developing identification and classification methods that have regional applications. Local experience and understanding are keys to success when building through, on, or with these materials.

## 3. IDENTIFICATION AND CLASSIFICATION

The investigative procedures for identifying or recognizing potential slope stability problems in shale formations are similar to those described in other chapters of this report. The focus in this section will be on reviewing laboratory and field tests developed for sedimentary rocks, shales in particular.

### 3.1 Shales

#### *3.1.1 Identification*

From a visual reference, the natural topography of regions underlain by shales displays certain characteristics. Terzaghi and Peck (1967, 426) stated: "On shales of any kind, the decrease of the slope angle to its final equilibrium value takes place primarily by intermittent sliding. The scars of the slides give the slopes the hummocky, warped appearance known as '*landslide topography*.'"

In the 1978 landslide report, Rib and Liang (1978) described the typical landforms of shale landscapes and their interpretation from aerial photography. If available, aerial photographs are excellent tools for identifying potential problem sites provided that the user is trained to recognize certain characteristics associated with the diagnostic features.

For thick, uniform shale beds, Rib and Liang described the associated landforms (1978, 57): "Clay shales are noted for their low rounded hills, well-integrated treelike drainage system, medium tones, and gullies of the gentle swale type." Ray (1960, 16) noted that shales "have relatively dark photographic tones, a fine-textured drainage, and relatively closely and regularly spaced joints."

However, it has also been observed that shales are particularly susceptible to landsliding when interbedded with pervious rocks such as sandstones or limestones. In this case, Rib and Liang noted:

> Interbedded sedimentary rocks show a combination of the characteristics of their component beds. When horizontally bedded, they are recognized by their uniformly dissected topography, contourlike stratification lines, and treelike drainage; when tilted, parallel ridge-and-valley topography, inclined but parallel stratification lines, and trellis drainage are evident. (Rib and Liang 1978, 57)

### 3.1.2 Laboratory Tests and Classification Systems

Since the late 1960s, there have been numerous attempts to develop tests to assist design engineers in the difficult task of classifying argillaceous shales and predicting their performance in embankment or cut slopes. These issues are of interest to the mining industry as well as to transportation engineers. The main objective has been to find tests that will reliably differentiate between durable shales that may be treated as rock and those with limited durability that are degradable on a human time scale.

Underwood (1967) discussed in detail the limitations of the various geological, chemical, and mineralogical classification methods of that time. He suggested grouping shales according to significant engineering properties (strength, modulus of elasticity, potential swell) and according to common laboratory tests (moisture content, density, void ratio, permeability). He recognized, however, that developing such a scheme would be a significant effort and he recommended (1967, 116) that "a comprehensive study involving the cooperation of government agencies, private engineering firms, and universities, is needed to produce a satisfactory engineering classification for compaction shales, especially the clay shales."

In the early 1970s, major research efforts were under way at Purdue University sponsored by the Indiana Highway Department and at the U.S. Army Engineer Waterways Experiment Station sponsored by the Federal Highway Administration. Numerous reports resulted from this research, including those by Bragg and Zeigler (1975), Shamburger et al. (1975), Strohm et al. (1978), and Strohm (1978). Because of the widespread occurrence of problem shales and their almost infinite variation of behavior, researchers in a number of state transportation departments, other public agencies, private companies, and universities have continued to refine these earlier studies, to revise the proposed tests, and to apply them to their respective areas.

Although it has been known for decades that certain shales deteriorate rapidly upon immersion in water, Franklin and Chandra (1972), Lutton (1977), and Franklin (1981) have made significant contributions to establishing specific tests for slaking of shales. The tests from these studies are described briefly below.

#### 3.1.2.1 Slake Test

The slake test was originally developed to provide an indication of material behavior during the stresses of alternate wetting and drying, which, to some degree, simulate the effects of weathering. The test procedure and applications have been discussed by numerous authors, including Chapman et al. (1976), Withiam and Andrews (1982), and Hopkins and Deen (1984). The procedure is briefly described below:

1. Choose six pieces of shale *each* weighing about 150 g or the largest pieces available to have a total of 150 g in each group.
2. Identify and photograph each piece beside a millimeter scale.
3. Dry shale to a constant weight at 105°C and record dry weight. (Note: drying the sample is

an important step; the following step of the test must not be started with a field-moist sample.)

4. Place each specimen in a separate jar and cover with distilled water. The condition of the specimens should be checked for the first 10 min, then at 1, 2, 4 or 8, and 24 hr.
5. Remove the specimens from the water and check for any change in pH of the water.
6. Dry to a constant weight and record weight of shale specimens retained on 2-mm (No. 10) sieve. (Note: recording weight of intermediate cycles is desirable so that the results may be compared with those of the slake durability test, described next.)
7. Photograph specimens if significant degradation has occurred or at the end of the last test cycle.
8. Repeat procedure four additional times, or until total degradation, to make five cycles.
9. Calculate the slake index for each of the six samples and take the average:

$$S_I = \frac{(\text{original weight} - \text{final weight})}{\text{original weight}} \times 100$$

This simple test will usually identify the poorly performing shales in a matter of hours. If the specimens are quite resistant, however, this test is time consuming and requires qualitative judgment as to its performance.

#### 3.1.2.2 Slake Durability Test

Franklin (1981) suggested a more severe and less time-consuming test known as the slake durability test, which is summarized below. Obviously, shales that fall apart in the slake test need not be subjected to the slake durability test.

For the slake durability test, a wire-mesh drum made with 2-mm (No. 10) mesh is rotated while partially submerged in a trough of water. The axis of the 140-mm-diameter drum is 20 mm above the water surface.

1. Select 10 pieces of shale (40 to 60 g each) with a total weight of approximately 500 g.
2. Identify and photograph the group beside a millimeter scale.
3. Place the shale fragments in the drum. Weigh drum and shale together. Place drum in an oven and dry the shale to a constant weight at 110°C.
4. Compute natural moisture content; then mount drum in trough.
5. Rotate the drum at 20 revolutions per minute for 10 min.
6. Remove the drum from the water, rinse, dry in oven, and weigh drum and remaining shale.
7. Repeat the cycle four more times to produce five cycles, but calculate the slake durability index ($I_D$) after each cycle. Photograph as necessary.
8. Calculate the durability index as follows:

$$I_D = \frac{\text{weight of shale remaining inside drum}}{\text{original weight of sample}} \times 100$$

Run at least two specimens from each sample of shale and take the average of their durability indexes.

The test proposed by Franklin has been standardized and is described in ASTM D-4644-87 (1992). In this newer test it is recommended that only two cycles be performed before the slake durability index is computed.

From these two tests several agencies have developed classification systems that allow them to determine the method or methods by which they will treat shales in embankment construction. These treatments are discussed in Section 4. In addition, several researchers have proposed classification systems and slope stability evaluations that depend on a number of other laboratory tests, including jar slake, rate of slaking, Atterberg limits, free-swell tests, and point-load strength. The procedures for jar slake, point-load strength, and free-swell tests are described below. Atterberg-limit tests are common in current engineering practice, so no procedural description is given. However, a word of caution should be expressed. Chapman et al. (1976) have noted that Atterberg limits when evaluated for degradable materials are often a function of the energy input and mode of preparation; thus variation in the test can be introduced by the operator.

#### 3.1.2.3 Jar Slake Test

The following procedure for the jar slake test was described by Wood and Deo (1975):

1. A piece of oven-dried shale is immersed in enough water to cover it by 15 mm. [It is important that the shale sample be oven dried. Lutten (1977) reported that damp material is relatively insensitive to degradation in this test when compared with dry material.]
2. After immersion, the piece is observed continuously for the first 10 min and carefully during

the first 30 min. When a reaction occurs, it happens primarily during this time frame. A final observation is made after 24 hr.

3. The condition of the piece is categorized (complete breakdown, partial breakdown, no change), as follows:

| *Jar Slake Index* $I_J$ | *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, develops several fractures, or both |
| 5 | Breaks slowly, develops few fractures, or both |
| 6 | No change |

The reproducibility of the jar slake test was evaluated by Dusseault et al. (1983).

The U.S. Office of Surface Mining Reclamation and Enforcement (1991) has defined "durable rock" as rock that does not slake in water as in the jar slake test. Welsh et al. (1991) proposed a strength-durability classification system that includes the point-load test to "clearly differentiate between strong-durable and weak or nondurable materials." The principal advantages of the test are that the equipment can be taken to the field, irregular samples can be used, and it is inexpensive to perform.

On the basis of work by Olivier (1979), Welsh et al. (1991) selected a dual-index system to categorize rock into three classes (Class I, nondurable and weak; Class II, conditionally stable; and Class III, durable and strong). In addition to the jar slake test, a graph (Figure 21-1) is used that plots the results of the point-load versus the free-swell test. The two tests are described briefly next.

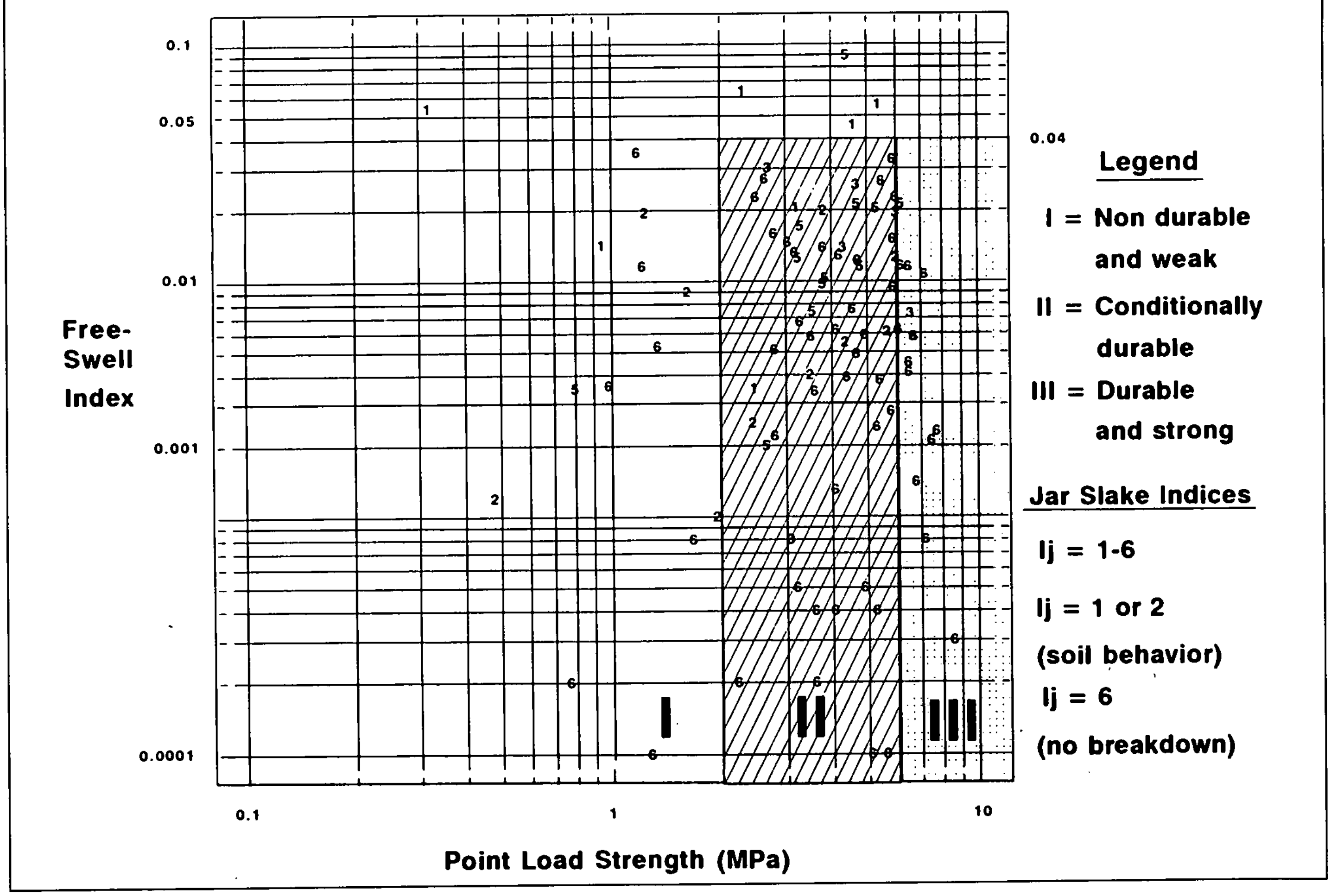



FIGURE 21-1
Strength-durability classification of jar slaking (Welsh et al. 1991).

### 3.1.2.4 Point-Load Test

The history and development of the equipment and the suggested method for determining the point-load index were described more completely by Broch and Franklin (1972). The test was developed principally to be used in the field on rock core or irregular lumps ranging in size from 25 to 100 mm.

The point-load apparatus (Figure 21-2) compresses the rock sampled between the two points of cone-shaped platens. The shape of the cone has been standardized. The radius of curvature of the cone tip (5 mm) is the most critical dimension. The angle of the cone (60 degrees) is of importance only if significant penetration of the cone occurs during testing.

The apparatus must be equipped to measure the distance, $D$, between the platens at failure to within an accuracy of ±0.5 mm. The load is applied hydraulically using a small hand pump and high-pressure ram with low-pressure seals to reduce inaccuracies of load measurement. The load, $P$, is determined from a gauge monitoring the hydraulic pressure in the jack. A maximum-pressure indicator needle on the face of the gauge is necessary to accurately record the maximum pressure or load at failure. The apparatus should have a capacity of 50 kN. After both the distance $D$ and the failure load $P$ have been measured, the point-load index, $I_S$, is determined:

$$I_S = P/D^2$$

where $P$ is the point load at failure and $D$ is the distance between platens at failure of the sample. It should be noted that for hard rock, the initial tested diameter $d$ and the measured distance $D$ are essentially the same.

FIGURE 21-2 Point-load apparatus.

To reduce scatter in the results, the samples should respect certain length-to-diameter ($l/d$) ratios: $l/d \geq 1.4$ for cores and $l/d$ between 1.0 and 1.4 for rock lumps when measured perpendicular to the loaded axis. In addition, when samples indicate to the geologist the potential for significant anisotropic mechanical behavior, samples should be divided and tested in groups to measure the strengths in each direction.

Since strength test results are influenced to some degree by the size of the specimen tested, Broch and Franklin (1972) proposed that the results be adjusted to a reference diameter of 50 mm. In their paper, a number of graphs showed the size effects reported from numerous tests on different rock types. They preferred using graphs to adjust the equation results rather than using factors in the formula because the formula is in stress units and has some theoretical justification. The graphs showed that the point-load strength decreases with increasing diameter. Consequently, when lumps are tested, the authors recommended using or preparing samples so that the initial dimension $d$ be as close as possible to 50 mm.

In order to obtain a statistically valid average value of $I_S$, at least 20 samples from the same formation should be tested (Oakland and Lovell 1982).

### 3.1.2.5 Free-Swell Test

The free-swell test is performed on intact core or from a bulk sample sawed into a rectangular prism. The minimum size should be 10 times the maximum grain size or 15 mm, whichever is greater. For direct testing purposes, NX-diameter core (54 mm) is generally acceptable. To measure the maximum swell, an axis perpendicular to the bedding laminations is chosen. The sample is oven-dried, carefully measured, and placed in water, and the volumetric strain is computed from measurements taken after 12 hr of soaking. Olivier (1979) reported that at least 75 percent of the maximum free swelling occurs within the first 2 to 4 hr of the test. Welsh et al. (1991) reported that for Appalachian shales the proportion of swelling ranged from 80 to 99.1 percent, with an average of 90.8 percent by the end of 4 hr. Consequently, they recommended using the shorter-term test for those shales and approximating the 12-hr results by multiplying the 4-hr results by 1.1.

In the proposed classification shown in Figure 21-1, durable and strong rock (Class III) has a swell of 4 percent or less and a rock strength equal to or greater than 6 MPa and exhibits rocklike behavior during the jar slake test (ranking higher than 2).

In this classification system, any shales with properties less than those of Class III should not be used in drain applications. Therefore, particular care must be taken to avoid placing these materials in or near drainage features used in landslide mitigation works.

Class I material, nondurable and weak rock, has the following characteristics:

- Fails the jar slake test (behaves like a soil),
- Fails during sample preparation for either the free-swell test or point-load test,
- Produces a value less than 2 MPa in the point-load test, or
- Has a free swell greater than 4 percent.

"Hard" shales, as defined by these simple tests, are not all without problems. As discussed by Strohm et al. (1978), the water in the slake durability and jar slake tests should be checked for pH. A pH less than 6.0 indicates an acid condition, and the shale mineralogy should be checked for minerals that can cause chemical deterioration (Shamburger et al. 1975). Chemical deterioration of hard shales in Virginia with $I_d > 90$ percent was studied by Noble (1977), who soaked samples in dilute solutions of concentrated sulfuric acid (18 M) and distilled water as a classification test. He found that a 25 percent solution was more reactive and gave the same ranking in degree of deterioration as the modified sulfate soundness test (ASTM C88).

Noble recommended that hard, dark-colored shales be checked for iron sulfides and chlorite as a clay mineral, since this combination can have great potential for rapid weathering. Upon oxidation and access to water, shales containing iron sulfides (e.g., as pyrite) produce sulfuric acid, which dissolves the chlorite. These chemical soaking tests should be considered in classifying hard shales contemplated for rock fill on important projects where long-term settlement must be kept to a minimum. In contrast to acid reaction, some shales in the western United States have dispersive tendencies (Shamburger et al. 1975) and may react adversely in alkaline (high pH) water.

### 3.2 Igneous and Metamorphic Rock

Little research has been undertaken to quantify how the degree of weathering of igneous and metamorphic rock affects their engineering properties. Cawsey and Mellon (1983) provided an overview of research in experimental weathering of basic igneous rocks. They noted the merits and weaknesses of various tests to reproduce the effects of weathering: Dearman (1976) discussed the use of a weathering classification in the characterization of rock, and Dearman et al. (1978) provided an evaluation of engineering properties based on a visual classification applied to granites (Table 21-1).

To some degree weathered igneous and metamorphic materials may be characterized by the same tests used to characterize shales (Section 3.1.2). This is particularly true for igneous and metamorphic rocks, which have an abundance of clay minerals and few core stones or unweathered blocks. However, a number of differences should be highlighted:

1. Researchers of degradable rocks have investigated shales and designed their classification and testing systems to apply to shales. Correlations with tests applied to weathered igneous and metamorphic rocks have not been established.
2. Weathering rates, and therefore long-term engineering behavior, depend highly on the original mineralogy, all other factors, including climate, being equal. Consequently, fresh rock cuts may weather dramatically near the surface. Similarly, increasing the exposed surface area of the rock fragments by excavation and crushing before their placement as fill may also accelerate subsequent weathering rates.
3. Control of water not only will improve pore-pressure characteristics (as in shales) but will also reduce weathering rates, further improving longer-term behavior.

## 4. ENGINEERING DESIGN CONSIDERATIONS

Using the laboratory tests described above, several researchers have proposed procedures for cut-slope and embankment design and construction using degradable materials. Although these proposals do not have the benefit of a wealth of engineering precedent and experience, they provide general guidelines.

A preliminary evaluation of the characteristics of shales as indicated by a number of standard laboratory tests has been described by Underwood (1965) and is shown in Table 21-2. This evaluation may be applied with care to nonshale degradable rock materials.

## 4.1 Embankment Design

Numerous workers have sought to correlate laboratory tests with construction design parameters for degradable materials. Much of the early work done at the U.S. Army Engineer Waterways

Table 21-1
Rock Mass 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[a] | 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 |
| 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 rock quality designation (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 percent of core stones; as low as 15 if none | 15 as sand | 15 as sand |
| Drilling rates (m/hr) (diamond NX), 2½-in. percussion | 2–4<br>5–7 | 2–4<br>8 | 8–10<br>12–15 | 8–10<br>12–15 | 10–13<br>17 | 10–13<br>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[b] (ohm-m) | 340 | 240–540 | 180–240 | 180–240 | 180 | 180 |

NOTE: Weathering grades (shown here as column headings) are based on Dearman (1976).
[a] 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.
[b] Tends to be determined by joint openness and water-table depth.

Experiment Station for the Federal Highway Administration was summarized into technical guidelines by Strohm et al. (1978), who divided the design of shale embankments into five areas: foundation benching, drainage provisions, material usage, compaction requirements, and slope inclination. The following discussion utilizes a similar grouping.

### *4.1.1 Benching and Drainage*

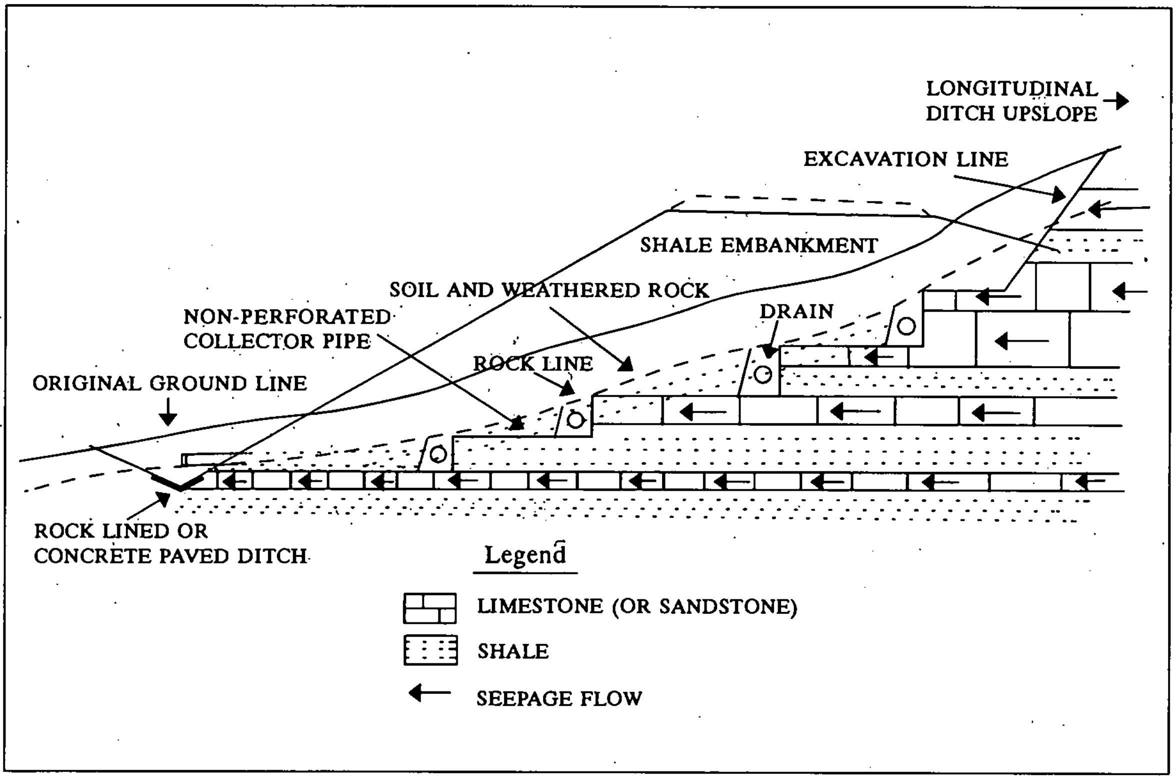
Typical recommended benching and drainage provisions are shown in Figures 21-3 and 21-4, where both longitudinal and transverse (cut-to-fill) situations are illustrated for shales interbedded with sandstone. Obviously the benches must be made into stable ground and the drainage rock must be

Table 21-2
Engineering Evaluation of Shales (modified from Underwood1965 and Wood and Deo 1975)

| PHYSICAL PROPERTIES | | | PROBABLE IN SITU BEHAVIOR [a] | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 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 $2 \times 10^{-6}$ (1400 to 14,000 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/cm³) | 110 to 160 (1.8 to 2.6 g/cm³) | 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 $10^{-10}$ ($3 \times 10^{-7}$ to $3 \times 10^{-12}$ ft/sec) | $>10^{-5}$ ($>3 \times 10^{-7}$ 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 oriented | Favorably oriented | | X | | X | | | X |
| State of stress | > Existing overburden load | = Overburden load | | | X | X | | | X |

[a] Expected problems indicated by X.

FIGURE 21-3
Longitudinal bench drainage tailored to stratification of seepage layers (modified from Strohm et al. 1978).



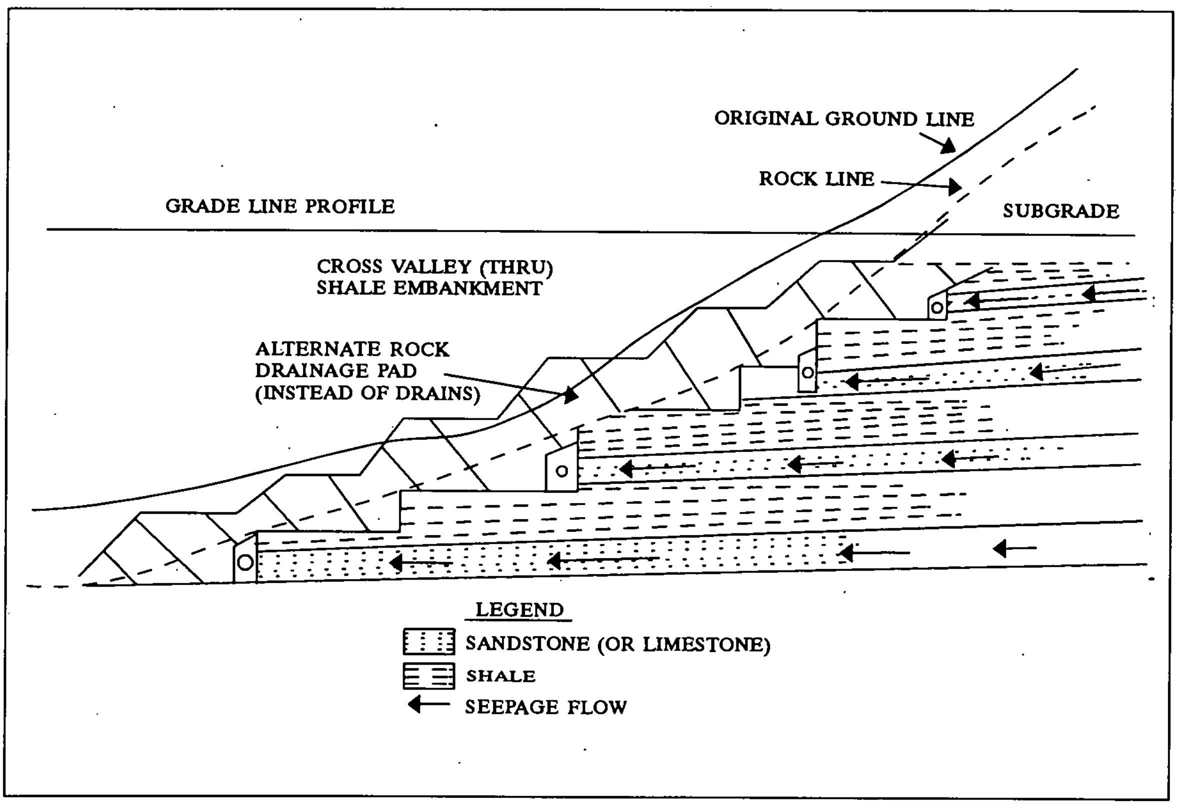




FIGURE 21-4
Transverse bench drainage tailored to stratification of seepage layers, cut-to-fill (modified from Strohm et al. 1978).

durable and not degrade with time. Requirements for this rock might follow the recommendations of Welsh et al. (1991). A typical design is shown in Figure 21-5, reproduced from a report by the Oregon Department of Transportation (Machan et al. 1989). Here the select durable rock embankment is placed to a level above the flood stage of an adjacent river, essentially eliminating the potential for wet and drying cycles in the shale embankment material placed above.

#### *4.1.2 Material Use*

During project development a material use plan should be prepared to cover excavation requirements and ensure that nondurable rocks are placed as soils in thin lifts and that durable rocks are placed as rock fill. The alternative is to place all materials as soils while meeting maximum gradation size and minimum compaction criteria. Wasting of degradable materials, such as shales, can generally be avoided by proper treatment and use. Exceptions might involve extremely wet clay shales, which may not be economically dried by disking or other means.

Strohm et al. (1978) developed design criteria based on the slake durability index, $I_D$, and the jar slake index, $I_J$, as follows:

- $I_D > 90$, $I_J = 6$: These materials can be used as rock fill as long as soil- and gravel-size materials do not exceed 20 to 30 percent of total lift. Too much fine material prevents the rock-to-rock contact necessary for stability and causes long-term settlement.
- $I_D = 60–90$, $I_J = 3–5$: These are hard, nondurable intermediate shales that require special treatment, typically including a high degree of compaction by heavy rollers (see Section 4.1.3).
- $I_D < 60$, $I_J \leq 2$: These materials need to be compacted as soil in thin lifts.

A number of other authors have prepared graphs or tables to help designers prepare the material plans. Lutton (1977) provided an estimate of allowable lift thicknesses as a function of slake durability index (Figure 21-6). The shale rating system proposed by Franklin (1981) (Figure 21-7) groups rock materials according to slake durability index and either plasticity index or point-load strength. Various groupings are assigned a shale rating, $R$, which is used to derive a number of slope design parameters. For instance, Figure 21-8 provides an estimate of allowable lift thicknesses based on Franklin's shale rating.

Santi and Rice (1991) used a modification of the slake index test to provide a preliminary clas-

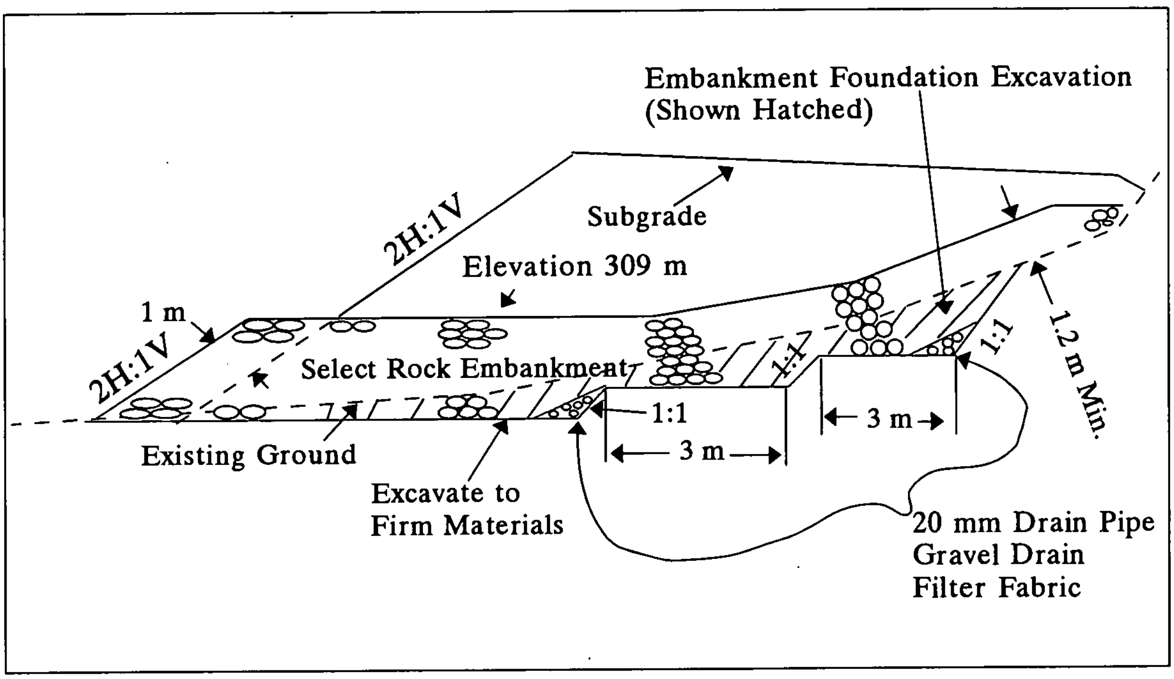



FIGURE 21-5
Oregon benching and drainage detail (modified from Machan et al. 1989).

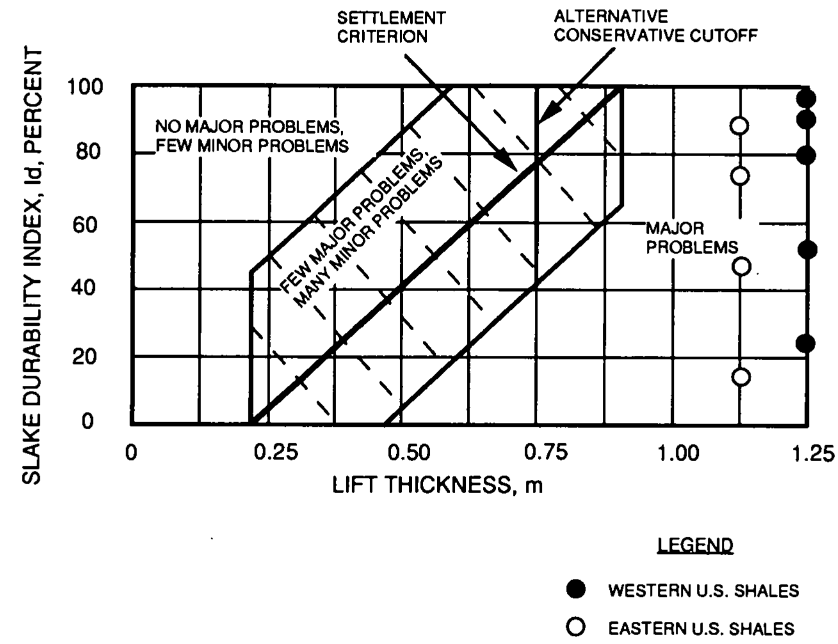



FIGURE 21-6 *(above)* Criteria for evaluating embankment construction on basis of slaking behavior of materials (modified from Lutton 1977).

sification of degradable materials. As is shown in Figure 21-9, they suggested plotting the one-cycle slake value (representing the current state of weathering of a material) against the difference between the five-cycle and one-cycle values (representing the susceptibility to weathering of a material). They term this difference the *slake differential*. The resultant graph is then divided into sections denoting expected material behavior. Such a classification emphasizes the continuum between soil and rock behavior. It also indicates subsequent laboratory tests that are likely to further characterize the material.

Hopkins (1984) tested Kentucky shales extensively in order to correlate slake durability with the California bearing ratio (CBR) used by the state of Kentucky for pavement design. Mathematical expressions were developed to define suitable relationships for design. These expressions convert three different indexes for slake durability to predictions of the Kentucky CBR values.

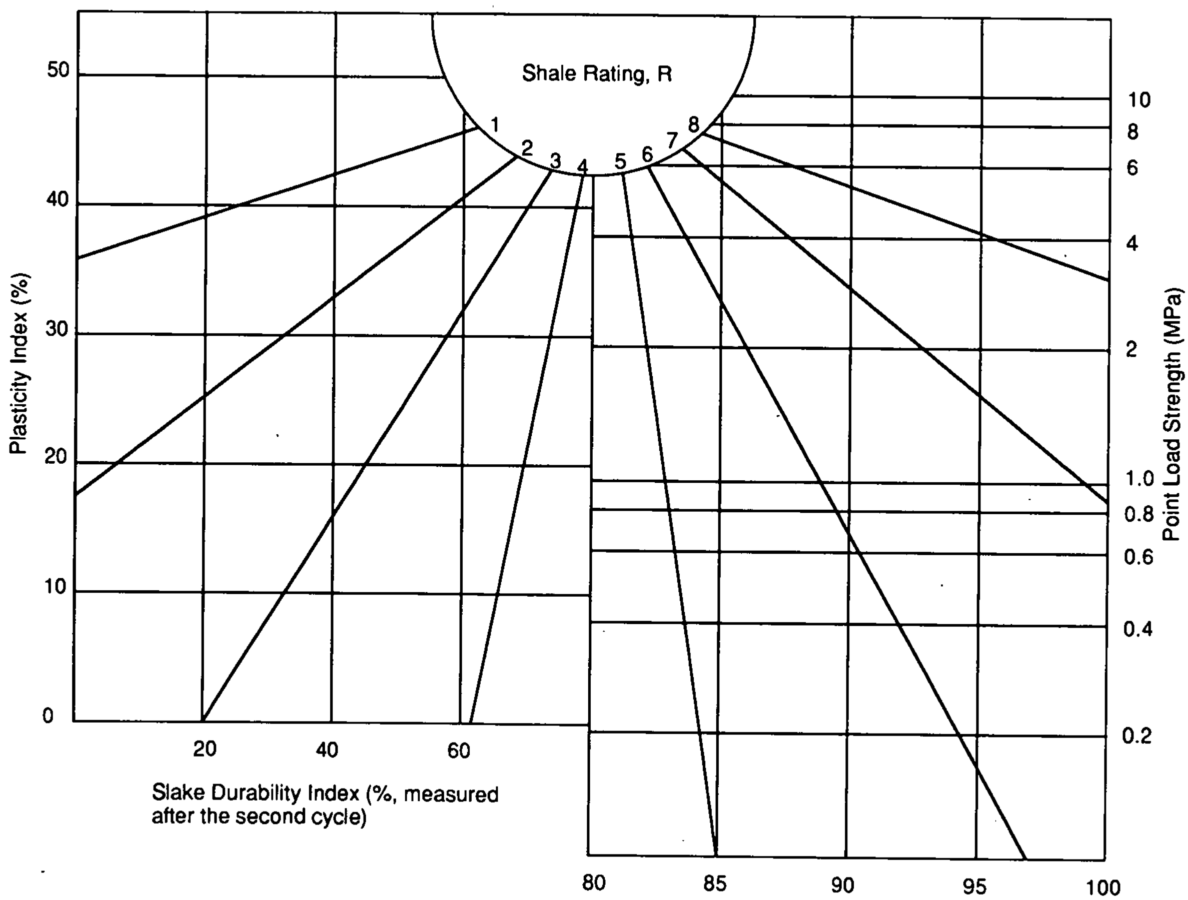


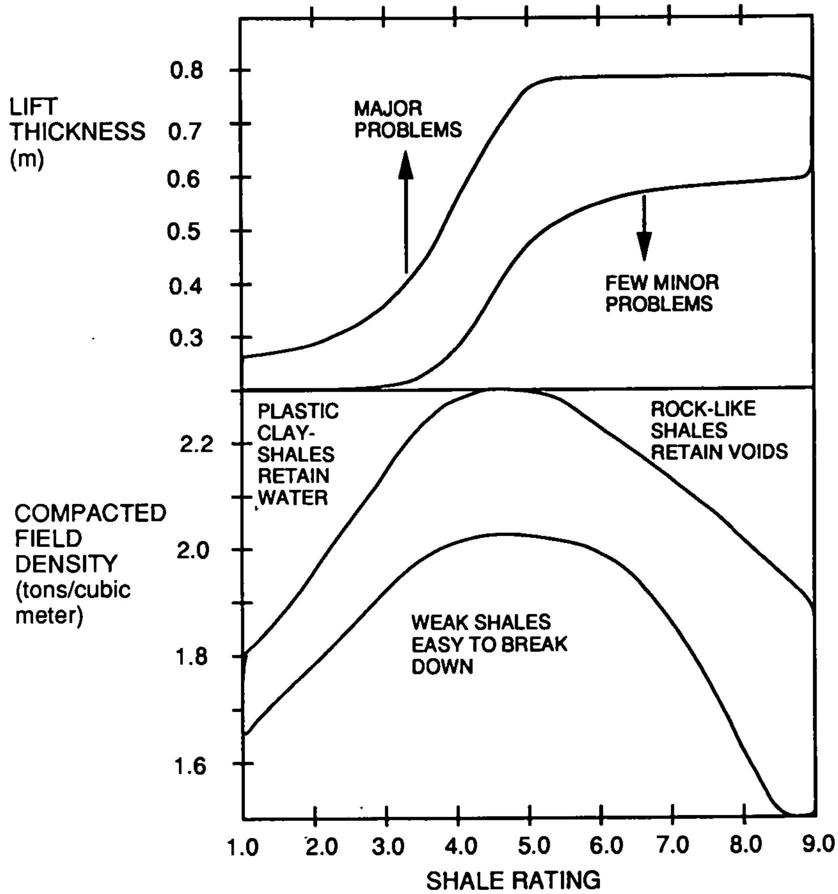

FIGURE 21-7 Shale rating chart. Sample of shale is assigned rating depending on its slake durability and strength if slake durability is > 80 percent, or depending on its slake durability and plasticity if slake durability is < 80 percent (modified from Franklin 1981).

### 4.1.3 Compaction

In the contract documents for a construction project, it is generally good practice to call for test sections or field-scale tests to evaluate construction materials and methods. The test sections are used to develop the required compaction methods and control procedures for nondurable shales before major earthwork is started.

Most specifications require a minimum density of 95 percent of the maximum dry density as determined by AASHTO T-99 and a moisture content within 2 percent of optimum. Some specifications make a point of indicating a value below optimum. However, by their very nature, these materials degrade excessively during laboratory processing and compaction. This degradation can result in unrealistically high dry densities. The tests should be started with the in situ or natural water content, since that is what will be used on the project. It is also necessary to use fresh sample material for each determination of moisture content because of material degradation.

In the field it has been found that generally it is necessary to use two different types of rollers to obtain the specified density. A static or vibratory sheepsfoot roller weighing around 25 000 kg is



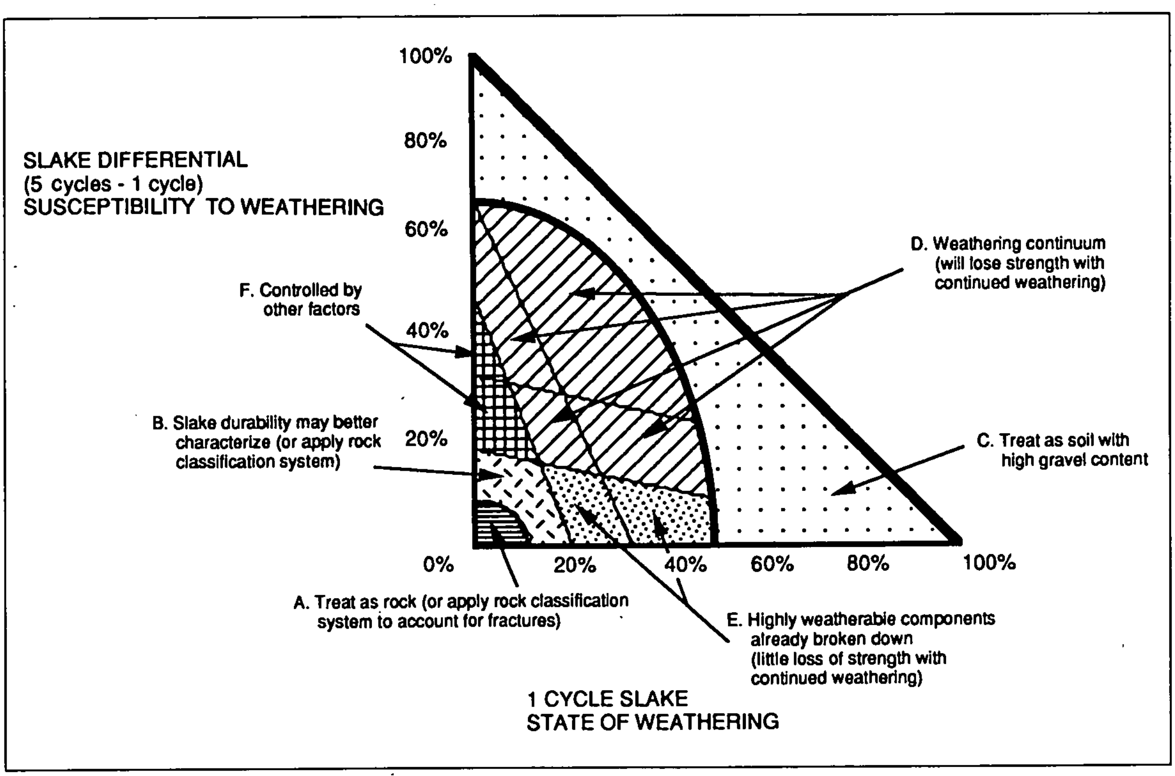




FIGURE 21-8 *(above)* Tentative correlations among shale quality, lift thicknesses, and compacted densities (modified from Franklin 1981).

FIGURE 21-9 Implication of slake differential partitioning. Characteristics of each category and appropriate testing may be estimated as shown (modified from Santi and Rice 1991).

needed to break down large rock fragments. Two to four or more passes may be needed, typically followed by a 46 000-kg pneumatic-tired roller for another two to four passes, which compacts the now-soil-like materials. Loose lift thicknesses are normally specified in the 0.2- to 0.3-m range. The quantity of water to be added or dried off by disking must also be evaluated. All of the above considerations support the test-section approach for determining the proper procedures to use in the field with the equipment provided by the contractor.

### *4.1.4 Slope Design*

In this discussion of embankment slope design the conventional procedures outlined in Chapter 13 of this report have been modified. Two figures from Franklin (1981) have been included. Figure 21-10 uses the shale rating, *R*, obtained from Figure 21-7 to estimate a range of values for cohesion and angle of internal friction. Figure 21-11 provides an estimate of allowable slope angles and embankment heights as a function of *R*.

Perry and Andrews related the mode of slaking to slope stability problems observed in mine spoils ranging in age from 2 to 10 years:

> Little or no stability problems were found where slab or block slaking dominated [degradation to thick, blocky fragments]. Where chip slaking was dominant [degradation to thin, flat segments], the mass appeared to be relatively stable. The chips form an interlocking matrix which is resistant to bulk movement. When slaking to inherent grain size [degradation to fine-grained particles] was found to be the primary mode, stability problems were observed, as evidenced by slips, slides, and similar features. (Perry and Andrews 1982, 27)

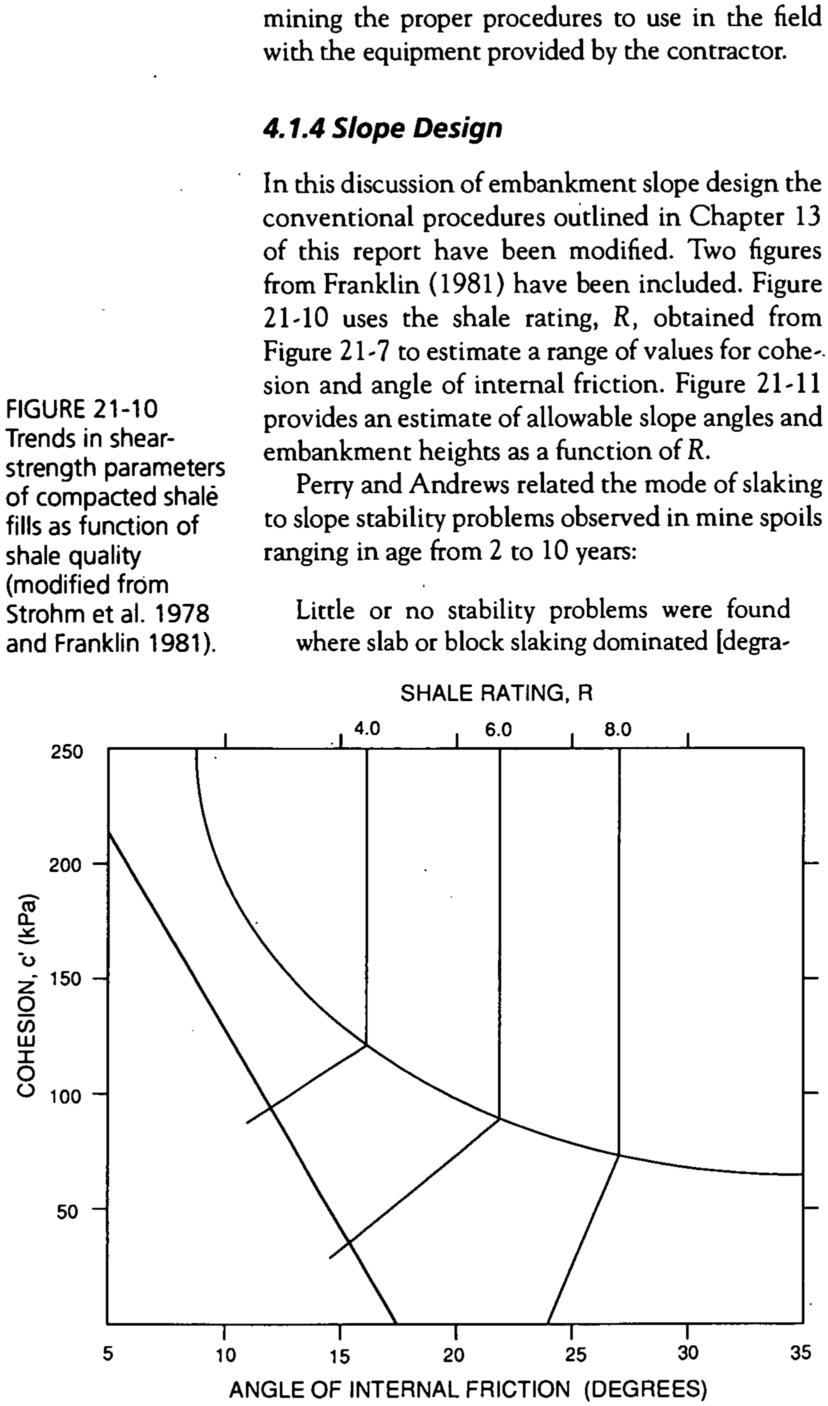



FIGURE 21-10 Trends in shear-strength parameters of compacted shale fills as function of shale quality (modified from Strohm et al. 1978 and Franklin 1981).

In addition to the observations on mass stability, Perry and Andrews (1982) related slaking to erosion problems. Sheet, rill, and gully erosion were observed to occur to varying degrees on all spoils. Where a high proportion of materials that slaked to their inherent grain size was encountered, the most severe erosion was observed, whereas spoils with a high percentage of slake-resistant rocks were least affected. A "pebble pavement" created by an armoring of the surface with resistant small chunks was observed to be effective in controlling sheet erosion.

As a continuation of his earlier study, Hopkins (1988) performed numerous tests on some 40 different shales in an attempt to present predictive equations of engineering parameters from various index tests. For instance, he found that the natural water content of an unweathered shale was a good predictor of important engineering properties.

In addition, Hopkins selected nine types of shales for triaxial testing on remolded specimens compacted to standard-, modified-, and low-energy compaction. The behavior of these complex materials required experience and engineering judgment for interpretation of the results. Hopkins stated:

> Since $\phi'$ and $c'$ values defined by the $(\sigma'_1/\sigma'_3)$ failure criterion are generally higher and lower, respectively, than values obtained from the $(\sigma'_1 - \sigma'_3)$ failure criterion, then it is unclear which set of $\phi'$ and $c'$ to use in a given stability analysis. (Hopkins 1988, 105)

The principal difference lies in the values of $c'$ obtained, which can have a significant influence on the value of the safety factor computed for the slope. Because of this, Hopkins recommended that designers use both sets of parameters in their stability analyses to determine which is the most conservative. In their reports, designers were encouraged to give the method used in the defining $\phi'$ and $c'$ obtained from their laboratory tests.

## 4.2 Cut-Slope Design

The design of cut slopes in degradable materials, particularly in clay shales, is as complex as their geology, which was briefly outlined in Sections 2.1 and 2.2. As described by Terzaghi and Peck (1967, 426), upon unloading, the shales expand horizontally, take on moisture, decrease in strength, and consequently are notorious for delayed and progressive failures.

Duncan and Dunlop (1969) examined the influence of the initial stresses on the stresses near the excavated slope using the finite-element method. In particular, their analyses were conducted to study the behavioral differences of the excavated slope in materials with low and high initial horizontal stresses. Other studies have shown that in heavily overconsolidated clays, the horizontal stresses may be 1.5 to 3 times the current overburden pressures at the site and that in the Bearpaw shale and similar rocks of western North America, horizontal stress has been measured at 1.5 times the overburden pressure at a depth of 20 m.

Duncan and Dunlop concluded:

> Shear stresses around excavated slopes are much larger for conditions representing a heavily overconsolidated clay (high initial horizontal stresses) than for conditions representing a normally consolidated clay (low initial horizontal stresses). The results indicate that shear stresses large enough to cause failure may develop at some points within the slopes, even though the factor of safety according to the usual $\phi_u = 0$ method is considerably greater than unity. (Duncan and Dunlop 1969, 489)

Great care must therefore be used when determining shear strengths of intact shale specimens and when using these values in slope stability calculations. The shear-strength envelopes must be based on residual strength values from tests advanced to large strains.

In view of the difficulties in obtaining and testing samples representative of the materials in a large slope, many engineering firms and transportation departments invoke the local "experience factor" in the design of slopes for minimal maintenance over the long term.

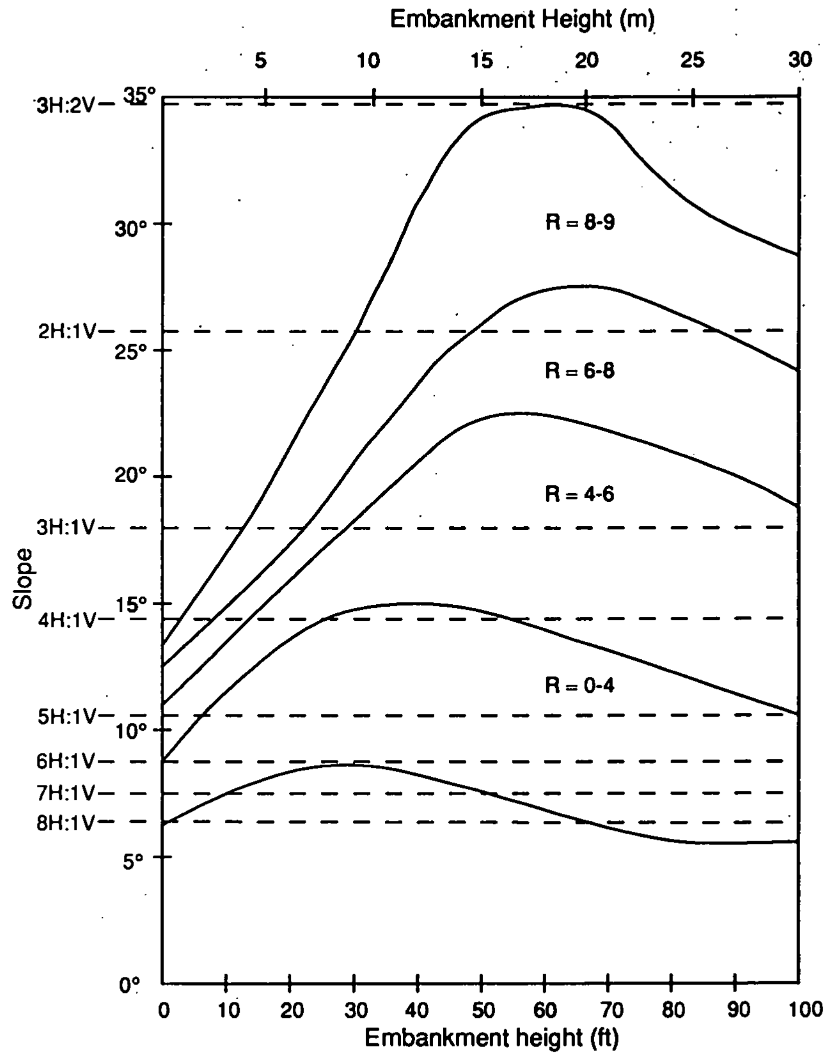



FIGURE 21-11 Trends in stable embankment slope angle as a function of embankment height and quality of shale fill (modified from Franklin 1981).

The number of variables is obviously quite large, and only a few examples are illustrated here. Franklin (1981) addressed this topic. Figure 21-12, reproduced from Franklin's paper, provides an estimate of allowable cut-slope angles as a function of the shale rating *R* and the bedding and joint orientations. State highway departments typically address cut-slope design by using standard plans included in their contracts. These are used to guide the excavation process and locate benches. Typically, the top surface of each bench is located at the top of the degradable materials. Several examples from the Kentucky Department of Highways *Geotechnical Manual* (1993) are shown in Figures 21-13 through 21-19. If rock fall is anticipated, the reader should refer to Chapter 18, Stabilization of Rock Slopes.

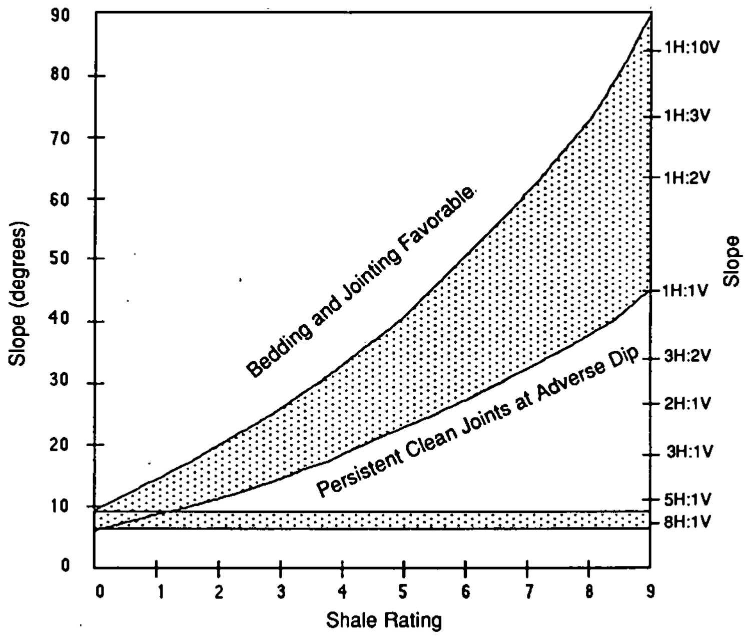



FIGURE 21-12
Trends in stable cut-slope angle as function of character of shale (modified from Franklin 1981).

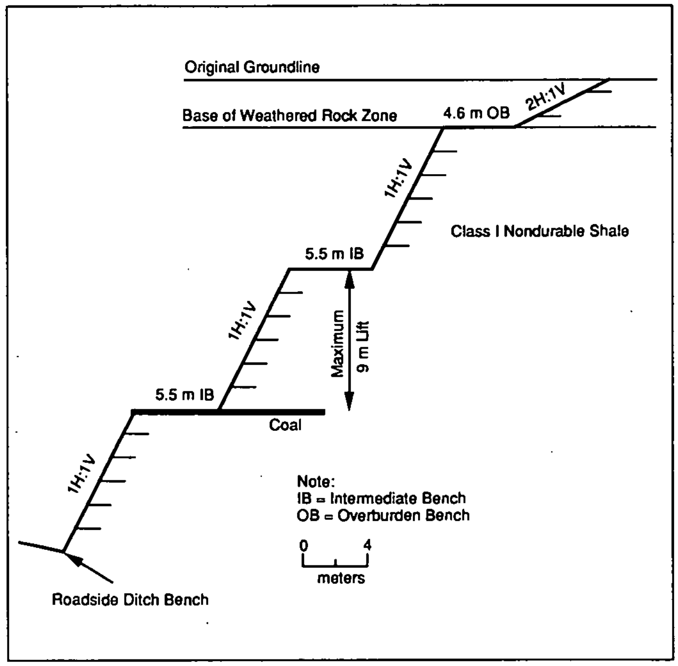



FIGURE 21-14
Typical slope configuration in Class II nondurable shale (modified from Kentucky Department of Highways 1993).

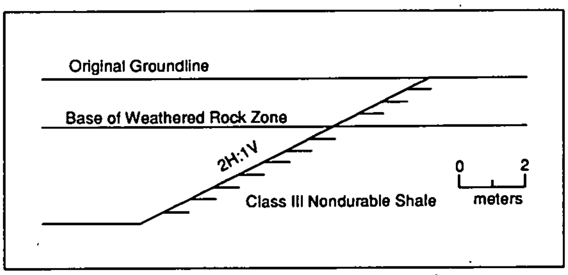



FIGURE 21-13
Typical slope configuration in Class III nondurable shale (modified from Kentucky Department of Highways 1993).

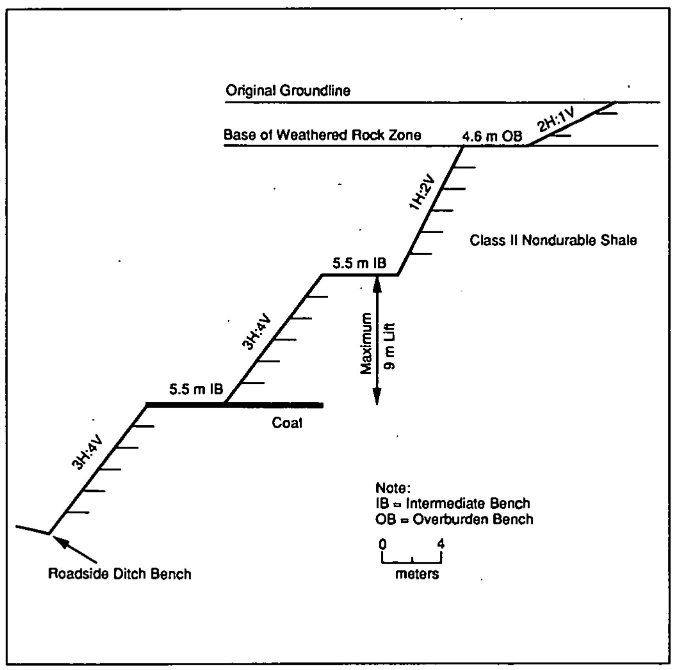



FIGURE 21-15
Typical slope configuration in Class I nondurable shale (modified from Kentucky Department of Highways 1993).

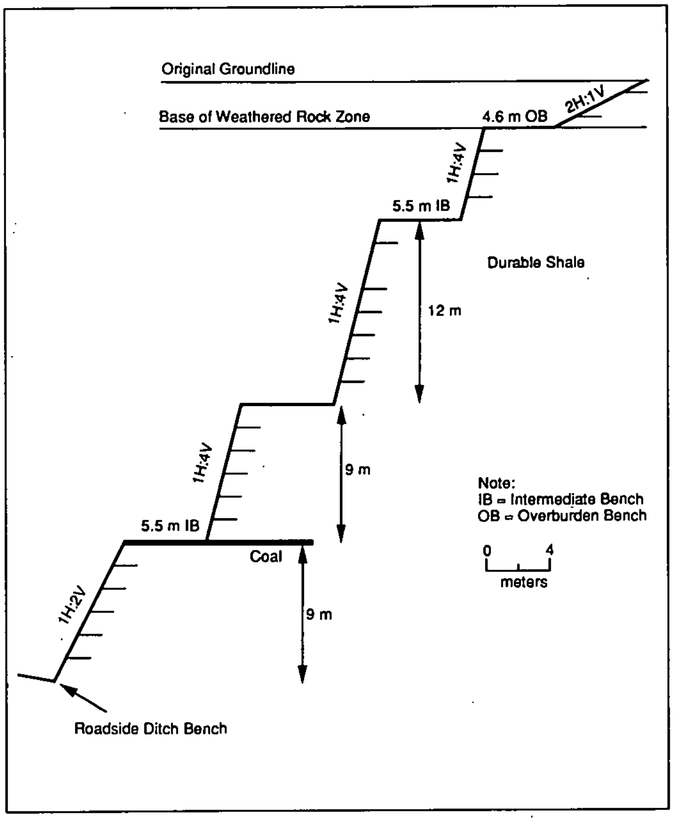
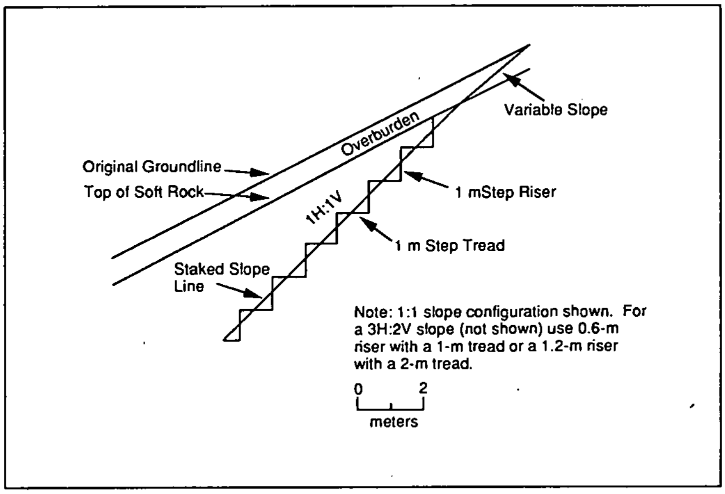
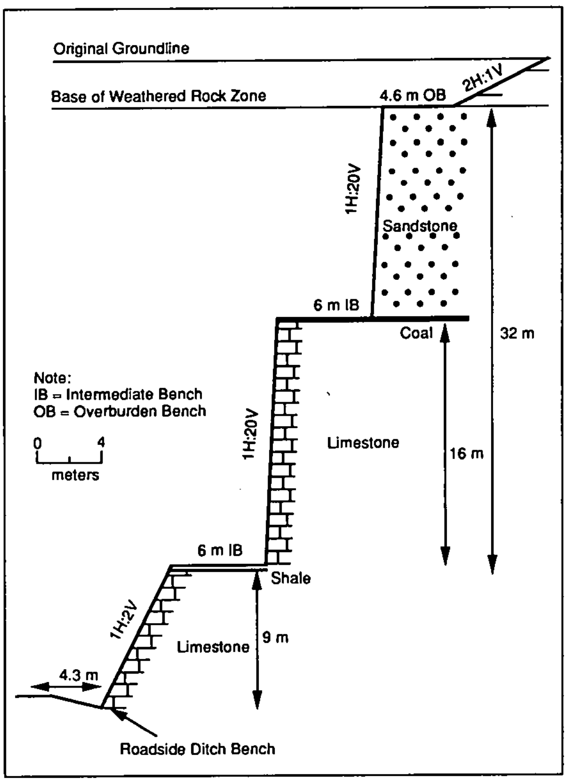



FIGURE 21-16
*(left)*
Typical slope configuration in durable shale (modified from Kentucky Department of Highways 1993).

FIGURE 21-17
*(below left)*
Typical slope configuration in massive limestone or sandstone (modified from Kentucky Department of Highways 1993).

FIGURE 21-18
*(below)*
Typical slope configuration for serrated slopes, which are utilized as means of controlling erosion and establishing vegetation on material that can be excavated by bulldozing or ripping (modified from Kentucky Department of Highways 1993).




## 5. CONCLUSION

It should be clear that the topic of constructing with, in, or through degradable materials is a complex one. There are few absolutes, and one must trust the experience of others. A great deal of reliance is placed on knowing the behavior of slopes in the vicinity and the geologic conditions at the site. It is hoped that the information in this chapter along with that in Chapter 15 on rock slope stability analysis will provide guidance for a successful project.

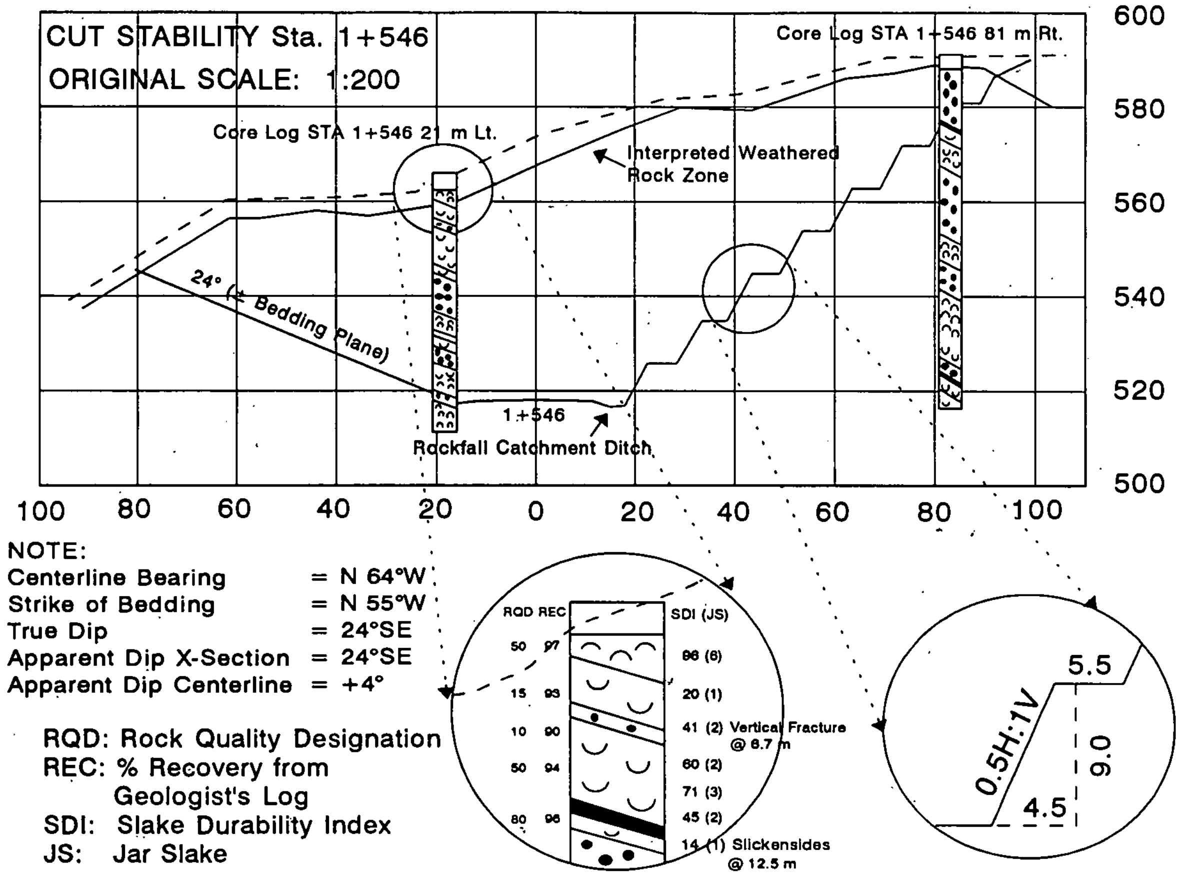



FIGURE 21-19 Through cut with dipping bedding planes (modified from Kentucky Department of Highways 1993).

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