A Shale Rating System and Tentative Applications to Shale Performance

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A "shale rating system" based on three properties—durability, strength, and plasticity—is proposed. A shale sample is assigned a rating value by first measuring its second-cycle slake-durability index. Rocklike shales that have durability values greater than 80 percent for this index are further characterized by measuring their point load strength. Soillike shales that have durability values of less than 80 percent are further characterized by measuring their plasticity index. The shale rating, derived from these test results by using a rating chart, is a continuously variable number in the 0.0-9.0 range. Tentative correlations (trend lines) are proposed that link this rating with aspects of engineering performance such as excavating methods (e.g., whether to dig or to blast), foundation properties (e.g., bearing capacities and foundation moduli), embankment construction (e.g., lift thicknesses and compaction methods), and slope stability (e.g., relations between slope height and angle and failure mechanisms).

Shales constitute about one-third of the rocks in the land surface of the earth and about one-half by volume of all sedimentary rocks (1). Not surprisingly, they are common in engineering projects either in their excavated form as construction materials for shale embankments or in their natural and undisturbed state--for example, in foundations, cut slopes, and underground works.

In spite of its abundance, this important rock type has until recently received little attention. In some ways, it is an unattractive and difficult material to study because it is easily disturbed during drilling, sampling, and specimen preparation. The strength, deformability, and other characteristics of a laboratory test specimen can change by orders of magnitude if the rock is allowed to dry out, shrink, or swell. A further experimental problem is that, whereas the minerals and microtexture of most rocks can be studied easily by using standard optical methods, extremely fine-grained clay minerals require techniques such as electron microscopy or X-ray diffraction.

Shales also vary greatly in their properties and behavior. At some locations, shale slopes stand for many years at near-vertical angles, whereas at others even 10-20° slopes suffer from continual erosion and creep. This has led to a distinction between "clay shales", the softer and more soillike types, and "indurated shales", which, because of their greater cementation and compaction, behave more like harder rocks. The practice of treating shales as either a soillike or rocklike material has been carried into construction specifications, where payment has often been based on a distinction between soil and rock. Problems have occurred with shales of intermediate quality that behave neither as soil nor as hard rock and require special treatment.

There is a clear need for a shale classification system that is capable of distinguishing all grades and qualities of shale and allows a correlation between the type of shale and its performance on engineering projects. In a three-year research program sponsored by the Ontario Ministry of Transportation and Communications (MTC), a shale rating system has been developed for this purpose (2). A rating number R is assigned to a shale according to measurements of the three properties considered fundamental to distinguish one shale from another: durability, strength, and plasticity. Tentative correlations have been developed between the rating number and aspects of engineering performance such as excavating methods (e.g., whether to dig or to blast), foundation properties (e.g., bearing capacities and settlements), embankment construction methods (e.g., life thicknesses and choice of compaction equipment), and slope stability (e.g., relations between slope height and angle and mechanisms of failure in different types of shale). The suggested correlations are based on limited data, and their value and accuracy will improve with use and experience. Nevertheless, it is believed that in their present form they serve to illustrate trends of behavior and will stimulate further research into the performance of this important group of materials.

HISTORIC BACKGROUND

Size-Strength Classification

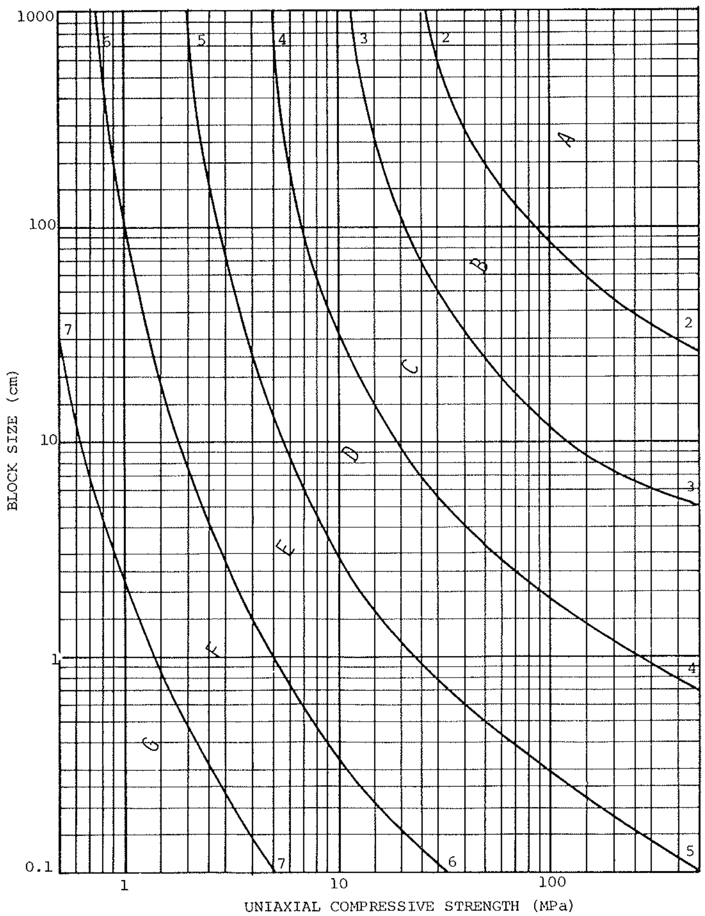
Before considering the subject of shale characterization, it may be helpful to discuss briefly the classification of rocks in general. Of the many characteristics of a rock mass, two in particular appear to be important in determining rock-mass behavior in engineering works: (a) the size of blocks into which the rock mass is divided by intersecting sets of joints and other discontinuities and (b) the intrinsic strength of these blocks. This "size-strength" classification has been applied, for example, to the design of rock tunnels (4,5) as a basis for predicting excavation and support requirements.

The size-strength classification system is shown in Figure 1. Strong, massive rocks plot to the top right of the diagram, whereas weak, broken rocks plot to the lower left. The diagram can be contoured to show classes of rock quality. Evidently, the high classes to the top right represent rock-mass conditions that require minimal support yet are difficult to excavate; i.e., they may require blasting. The lower-quality materials toward the lower left can, conversely, be excavated by rippers, shovels, or front-end loaders, but slopes or tunnels in these materials tend to be less stable.

This simple, two-parameter classification system can be criticized because it ignores a number of properties that have an important influence on rock-mass behavior--for example, the frictional characteristics of rock joints. Some classifications, such as that published by Bieniawski (6) and Barton (7), include a greater number of classification parameters and as a result are somewhat more difficult to apply. The two-parameter approach has been found to be a useful starting point and one that is readily comprehended and used.

The size-strength classification is insufficient, however, when applied to shales or other rocks of limited durability. A sample of shale excavated from the rock mass initially plots at a single location on the diagram; this location depends on the size and strength of rock fragments. When the shale is exposed to weathering, however, it becomes weaker or breaks down to smaller-sized fragments. The effects of short-term weathering processes can be recorded on the diagram in the form of vectors that represent weakening, disintegration, or a

Figure 1. Size-strength classification for rock masses.



combination of the two processes. Different shales vary in their susceptibility to short-term weathering agencies, and a measure of this susceptibility is essential in characterizing shale materials for engineering projects.

Tests for Shale

Some shales can withstand many cycles of wetting, drying, or frost; others soften or break down after only a short period of exposure. Much research has been devoted to methods of assessing the durability of a given shale (8-10). Early tests were qualitative, relying on the immersion of a sample of shale in water and on visual descriptions of the resulting breakdown. Attempted quantitative testing methods were generally more complex, requiring many cycles of freezing or immersion in water or salt solutions. Attempts by Franklin and Chandra (11) to develop a simpler, yet meaningful and reproducible test ultimately led to the development of a 10-min slake test in water, the "slake-durability" test. This test relies on a comparison of dry weights taken before and after slaking in a rotating open-mesh drum. In the slaking process, disintegrated fragments are allowed to pass through the sieve mesh of the drum. In spite of the apparent crudeness of the testing procedure, reproducibility is typically ±2 percent for identical test samples. As a result of extensive research using the slake-durability test, Gamble (12) recommended that the second-cycle slake-durability index be used as a standard for classification purposes. This proposal has been incorporated in a "suggested method" by the International Society of Rock Mechanics (ISRM).

Gamble (12) also proposed a shale classification based on a combination of slake-durability and plasticity indices. This classification can be criticized, however, in that plasticity is only a relevant property for the more soillike shales and is difficult or impossible to measure when the shale has a rocklike consistency. Furthermore, Gamble's classification based on slake durability and plasticity is subdivided into classes of material by way of discrete but arbitrary boundaries, whereas a classification or "rating" in the form of a continuously variable number would seem to be more amenable to correlations with field performance. These limitations led me to develop the alternative shale rating system described below.

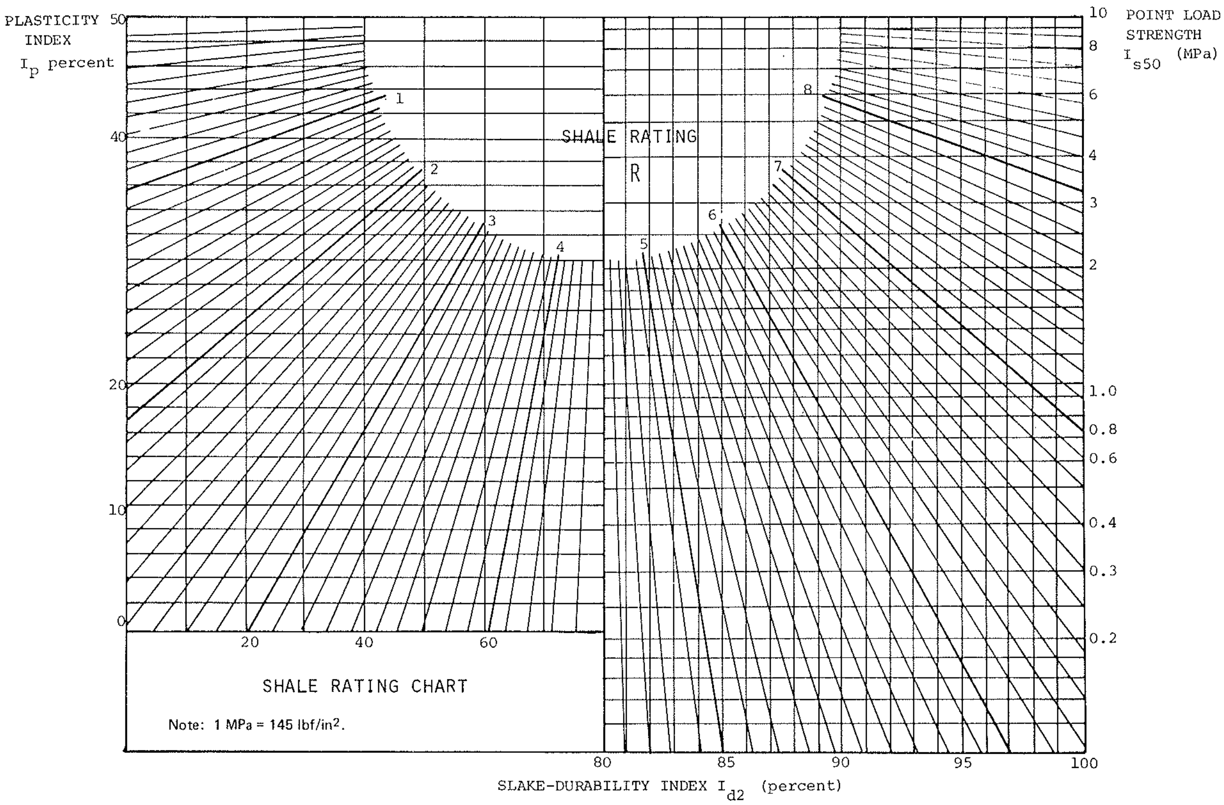
SHALE RATING SYSTEM

The proposed shale rating system is shown in Figure 2, the "shale rating chart". A sample of shale is given a rating number on the basis of (a) its slake durability and strength if the shale is rocklike and has a slake-durability index greater than 80 percent or (b) its slake durability and plasticity if the shale is soillike and has a slake-durability index less than 80 percent.

The rating chart is subdivided by lines that radiate at 2° intervals from the top center of the diagram to give rating values in the 0.0-9.0 range. By interpolating between the lines, a shale can be rated to one (and, if necessary, two) decimal places, which permits a continuous and quantitative classification.

Samples are initially subjected to the slake-dur-

Figure 2. Shale rating chart.



ability test to assess their second-cycle slake-durability index, I_{d2} percent, in accordance with ISRM recommended procedures. If this index is found to exceed 80 percent, the sample is further tested to measure the point-load-strength index (13-14). If the index is less than 80 percent, the fraction passing the slake-durability test drum is subjected to conventional Atterberg-limits determinations to evaluate plasticity index.

The point-load-strength test has been found to be convenient for strength classification of rocks in general and of shales in particular. It requires no specimen preparation or machining and can be conducted in the field before the rock has had a chance to dry or break up. The index used for rating purposes is the strength obtained when the load is applied perpendicular to the bedding planes--i.e., the strongest direction. Supplementary measurements can be made with the load applied parallel to the bedding planes to measure strength anisotropy and "fissility". Samples are tested at their natural moisture content. Point-load-strength values have been found to correlate closely with those obtained in the uniaxial compressive strength test. For classification purposes, uniaxial strengths can be obtained by applying a factor of 24 to the point-load-strength values.

Figure 3 shows the test results obtained for samples of shales collected in Ontario as part of the current research program. The results have been subdivided according to the geologic age of the formation tested. It can be seen that older formations, as expected, are generally stronger and more durable and have higher rating values. Perhaps the most characteristic feature of this diagram, however, is the considerable scatter in durability, strength, and rating values for the majority of formations. The scatter reflects real differences in shale properties as a result of differing degrees of lithification and of in-situ weathering. Evidently the character of these materials differs significantly from place to place throughout the province and even from bed to bed within a single formation. The index test results therefore give important additional information, and the characteristics of these shales cannot be inferred from rock or formation names alone.

It may be noted that Ontario shales are generally more durable and stronger than average shales elsewhere. This is clearly related to geologic age as the data assembled by Patrick and Snethen (15) show (see Figure 4). A review of the percentage of expansive clay present in rocks of various ages clearly shows a marked increase in the expansive clay mineral content of rocks younger than Devonian age. Only the older shales outcrop in Ontario, typically with contents of montmorillonite and other swelling clays in the 0-5 percent range. To find "worse" shales in Canada, one has to go west to the prairie provinces, where Cretaceous or younger shales with swelling mineral contents in the 20-40 percent range are common. Even higher percentages of such swelling minerals are found further south or west--for example, in the Oligocene and Miocene claystones of Texas or the Miocene-Eocene claystones of the Pacific Coast of California.

CORRELATIONS WITH FIELD PERFORMANCE

General Comments

To be of value, a classification system should be readily correlated with the behavior of rock materials observed in construction projects. Unfortunately, if a classification system is new, such a capability for correlation with field performance will be limited by users' lack of experience with the system. This is true in the present case, where

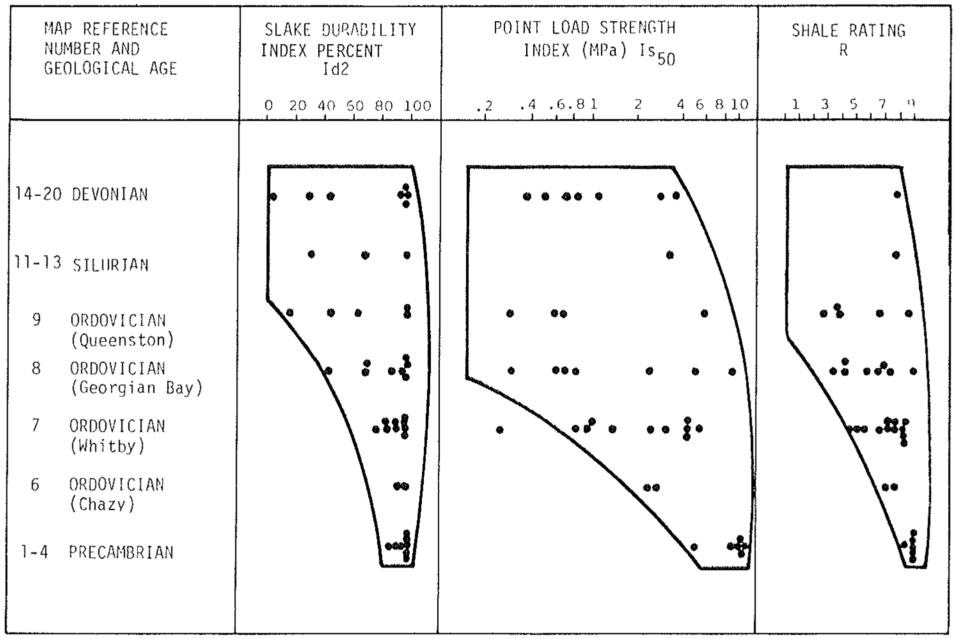


Figure 3. Relation between shale age, rating, and index properties.

Figure 4. Estimates of percentage of expansive clay present in Precambrian through Pliocene-age rocks.

AGE	PERCENT			
Pliocene	65			
Miocene	65			
Oligocene	50			
Eocene	40			
Upper Cretaceous	40			
Lower Cretaceous	20			
Jurassic	20			USA SHALES
Triassic	20			
Permian	40		CANADIAN SHALES	
Pennsylvanian	30			
Upper Mississippian	40			
Lower Mississippian	5			
Devonian	5			
Silurian	5	ONTARIO SHALES		
Ordovician	15			
Cambrian	∿5			
Precambrian	∿5			

the attempted correlations take the form of "trend lines" only and rely on inferred as well as actual data. Seldom were the properties of durability, strength, and plasticity found to have been reported simultaneously for a particular shale. Gaps in the data were filled by a subjective assessment based on published descriptions of shale character and index properties. The proposed trend lines should therefore be taken only as approximate indications of shale behavior and should not be used for design without further cross-checking. Application of the rating system to three areas of rock engineering--embankments, slopes, and foundations--is discussed below.

Shale Embankments

MTC specifications define "earth embankments" as being constructed in layers of loose lift thickness, usually 200 mm (8 in), compacted to 95 percent of ASTM D698 maximum dry density. "Rock embankments", on the other hand, are placed by end dumping in much thicker lifts and with only nominal compaction.

This distinction between "earth" and "rock" can lead to serious construction difficulties and to defects in the completed embankments. The success of attempts to achieve a specified compacted density depends on the character of the shale, the selection of compaction equipment and techniques, and the appropriate matching of equipment and techniques to characteristics. The objective is to achieve the maximum shale breakdown during construction so as to minimize breakdown, or "degradation", during the subsequent service life of the embankment. End-result specifications have generally been found to be inappropriate for the construction of shale embankments, and the trend is to replace these by procedural specifications related to shale character. In Ottawa, for example, the special provisions of a recent contract called for the use of static compactors with tamping- or peg-foot drums to be followed by steel drum units, a combination that was found to be most effective for the harder and more durable shale materials encountered on that project.

Table 1 gives a tentative correlation between the

Table 1. Excavation capabilities of various methods and types of equipment as a function of the character of an interbedded shale and hard-rock sequence.

Method or Equipment	Shale Rating	Limestone (%)	Thickness of Limestone Bed (mm)	
			Average	Maximum
Backhoe or scraper	0.0-5.5	<5	<20	<50
Shovel	0.0-5.5	<10	<50	<100
Medium ripper	3.0-6.0	<20	<75	<125
Heavy ripper	3.0-7.0	<30	<100	<150
Blasting	6.0-9.0	No limitations		

Note: 1 mm = 0.039 in.

character of a shale-limestone formation and the likely excavation requirements for borrow materials. Ease of excavation is governed by a limited number of geologic characteristics. When the borrow is entirely of shale, the key properties are likely to be the strength of the shale (reflected by its rating) and its natural "block size" (governed by the spacing of joints and bedding planes). When, as is often the case, the shale is interbedded with a harder rock such as limestone, the ease of excavation will be greatly affected by the percentages of hard rock in the total rock to be excavated, by the strength of the hard rock, and by the average and maximum thicknesses of the hard-rock bed. Table 1 draws on experience in southern Ontario, where the shales are commonly interbedded with dolomite or limestone that has a uniaxial compressive strength of 150-200 MPa (20 000 to 30 000 lbf/in^2). The table gives a general indication of the performance of various classes of excavating equipment and draws attention to the importance of quantifying the percentages and thicknesses of hard-rock inclusions in a mixed-rock formation. Additional variables should be considered--for example, the depth of excavation and the dip of bedding planes. Ideally, the limitations of each make and model of excavator should be defined in relation to the controlling rock characteristics. Indirect methods of predicting ease of excavation--for example, the use of seismic velocities--are unlikely to be as reliable as direct observation of key properties such as those noted in Table 1.

Figure 5 shows trends in optimum lift thickness and compacted field density as a function of shale rating compiled from data by Lutten (16). Greater lift thicknesses can generally be allowed for shales that have a higher rating. Shales that have rating values in the range of 5.0-8.0 (slake durability greater than 80 percent) can be effectively compacted in lifts of 500-800 mm (20-30 in) if appropriate compaction methods are used. These shales behave substantially as rock fill, retaining a percentage of interfragment void space even after compaction. Shales that have rating values of less than 5.0 require a reduced lift thickness to facilitate complete breakdown of these less durable materials. The degree of breakdown achieved in practice can be assessed from the lower half of Figure 5.

Low values of compacted density, in the range of 1.8-2.0 Mg/m^3 (112-125 lb/ft^3), are typical for plastic clay-shales that retain water between clay mineral grains. The highest densities, in the range of 2.0-2.2 Mg/m^3 (125-137 lb/ft^3), are achieved with intermediate-rating shales that are relatively easy to break down and compact. Field densities again fall to lower values for the more rocklike shales with a rating of 6.0-9.0 because of the retention of significant void space between shale blocks in the fill.

Trends in embankment side slopes, as a function of embankment height and the quality (rating) of shale construction materials, are shown in Figure 6. A general increase in side-slope angle is apparent with increasing shale rating, to a maximum of approximately 35° (1.5:1) for shale rock fills that have ratings in the 8.0-9.0 range. Side-slope angle is also affected by embankment height. Small embankments [typically 5-10 m (17-33 ft) in height] generally have flatter slopes for ease of maintenance. As embankment height increases to 15-20 m (50-65 ft), it becomes uneconomical to design an embankment with flat slopes, and the slopes are generally steepened to the maximum that can be tolerated safely, based on geotechnical considerations. For high embankments [20-30 m (65-100 ft)], the side-slope angle progressively decreases; this reflects the growing importance of embankment stability and the need to maintain acceptable safety factors.

Embankments of significant height are designed by using the standard soil-mechanics method of limiting equilibrium. Calculations require an estimate of shear-strength parameters for the compacted shale material. Figure 7, which is based substantially on data by Strohm and others (17), indicates that as shale rating increases the shale fill becomes progressively more frictional until, at high rating values ($R > 8.0$), the shale behaves essentially as a granular fill with limited cohesion and with an angle of internal friction of >25°.

Embankment permeability is another important parameter to be estimated for design. A review of published field-test data is summarized below:

Type of Material	Rating	Permeability (m/s)
Shale rock fill	8-9	10^{-3}-10^{-5}
Durable shale fill	7-8	10^{-5}-10^{-6}
Moderately durable	5-7	10^{-6}-10^{-7}
shale fill	4-5	10^{-7}-10^{-8}
Well-compacted clay shales	0-4	10^{-8}-10^{-12}

Permeability values of the compacted fill range from 10^{-3} to 10^{-5} m/s (300-3 ft/day) for a shale rock fill to as low as 10^{-8} to 10^{-12} m/s ($3x10^{-2}$ to $3x10^{-6}$ ft/day) for well-compacted shales. Embankment permeability will control the acceptable rate of embankment construction if the development of excess pore-water pressures is to be avoided. It will also govern lateral drainage through the embankment after construction is complete. Permeability will generally decrease during the life of the embankment as a result of shale degradation and the filling of void space.

Cut Slopes in Shales

The long-term stable angle of a slope in shale can vary from about 8° to almost vertical depending on the durability of the shale material. Different slope-failure mechanisms occur in shales that have different rating values.

In shales of low durability ($R = 1.0$-5.0), mechanisms of slaking, erosion, and surface creep predominate as they do in clay embankments. Unprotected steep slopes exposed to continual erosion by surface runoff water develop a pattern of erosion gulleys. The surface layer slakes, and the debris is removed by erosion as fast as it is produced. Although there is usually no safety hazard associated with this mechanism, periodic cleaning of ditches is required, and the appearance of the exposed eroded rock can be unattractive. Slopes that are protected from continuous erosion develop a weathering profile. The thickness of the weathered shale layer may reach 9-13 m (30-43 ft) [for example, in London Clay (England)], although thicknesses of 1-2 m (3.3-6.5 ft) are more common in shales of higher durability. The mantle of weathered shale tends to be unstable and to creep downhill or slide along the contact with the fresh shale. Shallow slab slides typically occur at intervals of 5-10 years when the slopes are steep, exposing fresh shale to further weathering and

Figure 5. Tentative correlations between shale quality, lift thicknesses, and compacted densities.

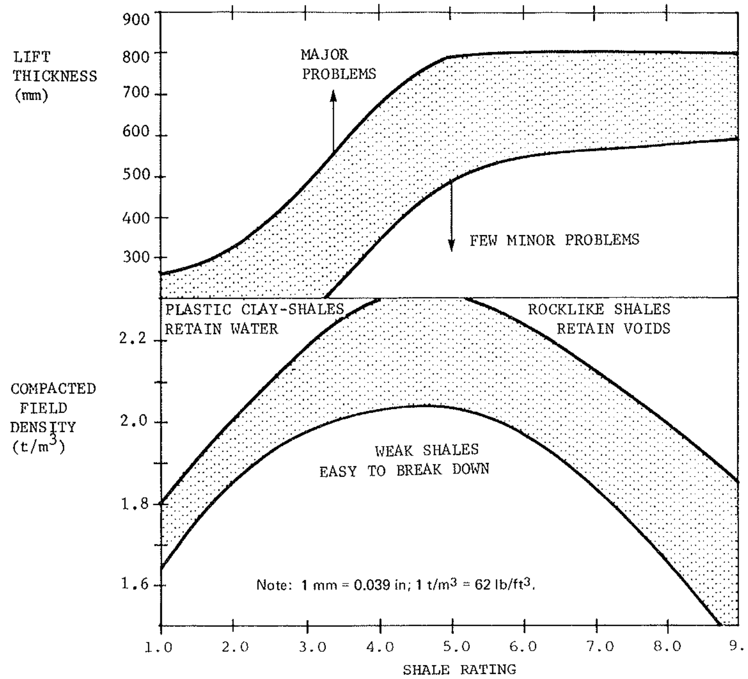


Figure 6. Trends in embankment slope angle as a function of embankment height and quality of shale fill.

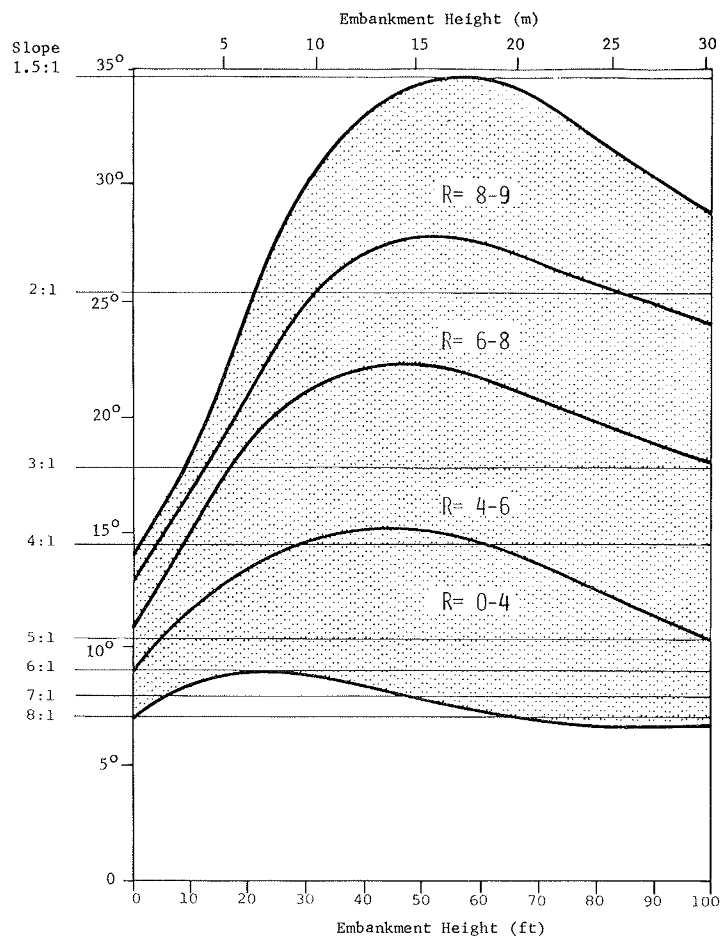
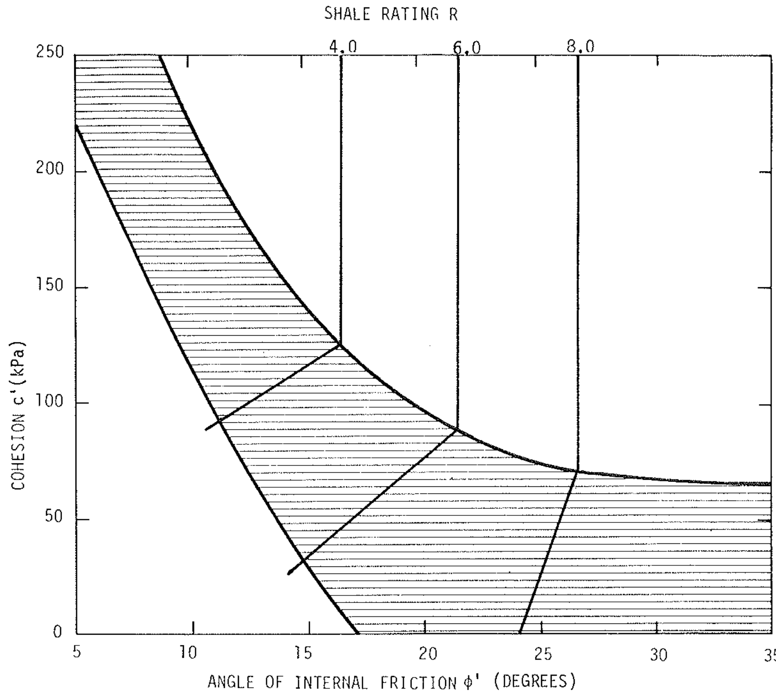


Figure 7. Trends in shear-strength parameters of compacted shale fills as a function of shale quality.



repetition of the cycle. Instability of the surface layer is encouraged if undercutting occurs at the toe of the slope--for example, in river embankments. It is also accentuated by water percolation and frost action along the contact between weathered and unweathered materials. The surface layer is usually more clayey and less permeable than the underlying rocks and thus traps water. Freezing of the layer adds to this damming effect. Eventually, a clay slope will reach a stable angle equal to approximately half the residual angle of shearing resistance of the material (18). Since, in engineering projects, it is seldom practical to design cut slopes this flat (e.g., 10°), one must rely to some extent on cohesion and cementation of the shales to maintain steeper angles over at least decades. In addition, slope stabilization measures are used to improve and maintain stability.

Superficial instability occurs as a result of different mechanisms in shales that have a medium to high rating, such as those of northern Ontario. Wetting, drying, and particularly frost action result in the fragmentation and loosening of cut-slope faces so that large blocks break down into smaller fragments. For example, the Manitoba Department of Highways reports that the Odanah shales of that province are capable of standing vertically to relatively great heights but are cut back to slopes of less than 1.5:1 because, with steeper slopes, blocks of hard shale continually break off. It appears that a recent project that used 1:1 back slopes will require an annual ditch-clearing program.

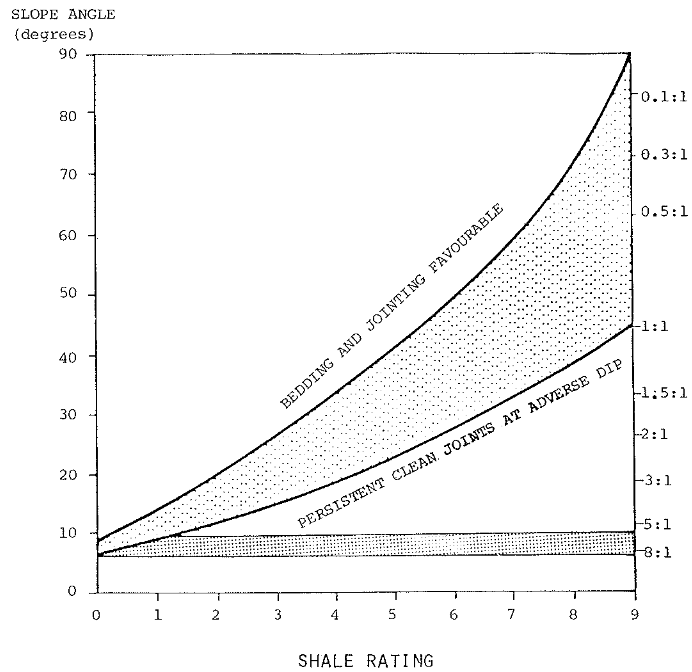
The breaking action of frost results partly from thermal contraction and expansion, accelerated by the wedging action of ice in microfissures and joints. Also contributing are "fossilized" stresses in the rock, which typically reach magnitudes of 6-15 MPa (1000-2000 lbf/in^2) in the near-surface rocks of Ontario, Quebec, and northern New York State.

Deep-seated slope failures are generally more common in shales that have lower ratings. In these shales, the sliding surface may pass through intact shale material and there may be only limited influence from preexisting bedding and jointing. In the harder, more durable shales, slope failures are invariably controlled by the orientations of preexisting discontinuity sets. Wedge or planar slides are bounded by sliding surfaces coincident with preexisting joints and bedding planes.

Figure 8 shows a relation between the stable angle of a slope cut in shale and the quality of the shale material. If the bedding and jointing orientations are favorable (i.e., they dip into the slope face and therefore have little influence on stability), the upper-bound curve of the shaded area in Figure 8 applies. It represents the probable maximum stable angle where failure must occur through intact shale. Near-vertical angles can be reached for a rocklike shale that has a rating in the 8.0-9.0 range. There is likely to be a pronounced increase in the gradient of this curve in the 7.0-9.0 range to accommodate the very steep slopes that are possible in rocklike shales. The lower-bound curve of the shaded area in Figure 8 represents stable slope angles where slope stability is governed by joints "daylighting" in the slope face. It has been assumed that the slope will stand stable at an angle close to the friction angle of the joint or bedding plane. When the joint is tight and clean, its friction angle depends on the strength of the intact shale of the joint walls and so increases as a function of shale rating. The convergence of the upper- and lower-bound curves of the shaded area toward the left side of the figure reflects the comparatively minor effects of jointing in weak and plastic shale materials.

The line at a constant angle of approximately 8-10° in Figure 8 illustrates the potential effect on slope stability of the presence of joints filled with soft and plastic clay. When these joints are present at adverse orientations, they govern the stability of the slope irrespective of how strong and durable the shale elsewhere within the slope may be. It is therefore important to identify the weak "clay mylonite" sheared horizons that are often present in shale formations. These are difficult to observe, since they are often thin and similar in color to the host rock.

Figure 8. Trends in stable cut-slope angle as a function of the character of shale.



Shale Foundations

The Canadian Foundation Engineering Manual (19) defines rock as a material that has uniaxial compressive strength greater than 1 MPa (145 lbf/in²) and cannot be dug by hand with a shovel or pneumatic spade. Shales clearly straddle this arbitrary boundary between soil and rock. A principal feature of foundation shales is their variability in uniaxial compressive strength, which ranges from less than 1 MPa for shales with the consistency of stiff to hard clays to as high as 10-100 MPa (1450-14 500 lbf/in²) for the high-rated shales and argillites. The susceptibility of shales to weathering is usually manifested as an increase of strength with depth. This may be gradual or may occur as an abrupt contrast between the soft, discolored, weathered-shale horizon and the unweathered or "fresh" underlying strata. The softening of shale toward the surface is further aggravated, from the point of view of foundation behavior, by a decrease in block size and bedding-plane spacing. In addition to softening, the shale weathers by splitting and by fragmentation. Modulus variations with depth are clearly illustrated by the results of pressuremeter testing (see Figure 9). In shales, a pronounced anisotropy is also evident, and this leads to deformability normal to bedding being much greater than deformability in the bedding direction.

Allowable bearing pressure is generally controlled by, and can be estimated from, the intact rock strength and the intensity of jointing or bedding (the size-strength parameters discussed earlier). An empirical coefficient that relates the allowable bearing pressure to uniaxial compressive strength is defined in the Canadian Foundation Engineering Manual (19) in terms of ratios of fracture spacing to footing width and of joint aperture to joint spacing. In view of difficulties in making field measurements of joint aperture, the Canadian manual defines three values for the empirical coefficient that depend only on major variations in the spacing of discontinuities: very wide, wide, and moderately close.

These recommendations have been plotted graphically in Figure 10 (19). The contours in the upper right of the diagram apply to the less fractured and stronger shales and illustrate an expected reduction in allowable bearing pressure as the shale becomes weaker, more thinly bedded, or more closely jointed. The Canadian manual is somewhat ambiguous in its treatment of the weaker shales, since recommended bearing pressures for shales with widely spaced joints compute to lower values than those recommended for clays with similar strengths. It might be more realistic if the contours reflected a continuous trend from shale through to clay and there were a gradual decrease in curvature as the material became softer and less influenced by the presence of joints and fissures. The recommendations of the Canadian manual include a safety factor of 3. However, a much greater degree of conservatism is likely. Experimental values of foundation strength often exceed normally used values of foundation bearing pressure by factors from 5 to 50.

Foundation modulus is generally only relevant to the design of heavily loaded structures such as dams and high-rise buildings on shale foundations. Figure 11 (20) shows that the foundation modulus of argillaceous rocks generally increases from 10 to 10 000 MPa (1450-1.4 million lbf/in²) as the character of the material improves from a normally consolidated clay to an indurated, high-durability shale. The ratio of modulus to compressive strength, however, is approximately constant in the 50-200 range, typically 100. Foundation modulus, like bearing capacity, is influenced not only by the strength of the rock material but also by the intensity of jointing in the foundation. A "mass factor" (J) has been defined that relates intensity of jointing to the ratio between field and laboratory deformability values. By using J and a modulus ratio of 100, one can construct Figure 12, which relates field deformability modulus to the size-strength rock classification. As jointing becomes more intense, the field modulus is reduced by joint compressibility until, for very closely spaced joints, the modulus appears to approach a limiting value. The effect of jointing on modulus

Figure 9. Menard pressuremeter test results in shale showing progressive increase in modulus of deformability with increasing depth below surface.

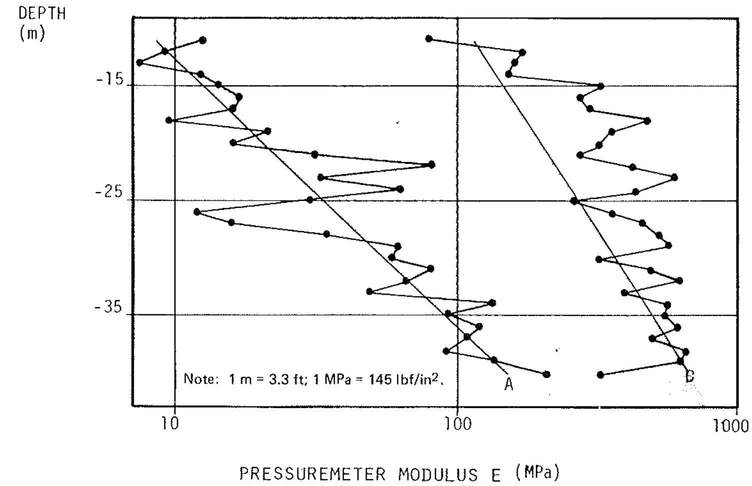
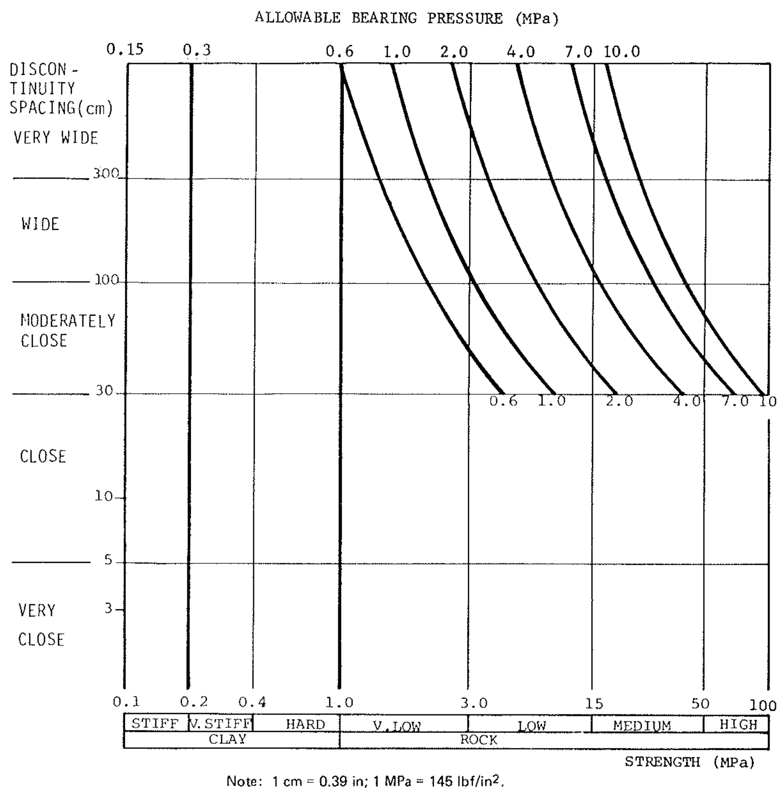


Figure 10. Trends in allowable bearing pressure (shallow foundations) as a function of rock strength and discontinuity spacing.



is most pronounced in the block-size range of 10-100 cm (4-40 in) (close to moderately close joint spacing). The results may be translated into modulus-depth variations by careful borehole or caisson logging to measure joint spacings. For example, in rock that has a laboratory strength of 17 MPa (1465 lbf/in^2), when bedding is spaced at 50 cm (19.5 in) near the surface and 150 cm (58.5 in) at depth, one would expect the field modulus to vary from 500 to 2000 MPa (72 500 to 290 000 lbf/in^2). These values for rock conditions and for modulus are similar to those found in the foundation of the Canadian National Tower in Toronto, where settlements were predicted on the basis of an assumed modulus of 3700 MPa (0.5 million lbf/in^2) to take into account the presence, frequency, and distribution of limestone strata (21).

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Figure 11. Ratio of deformability modulus to compressive strength for clays, shales, and related materials.

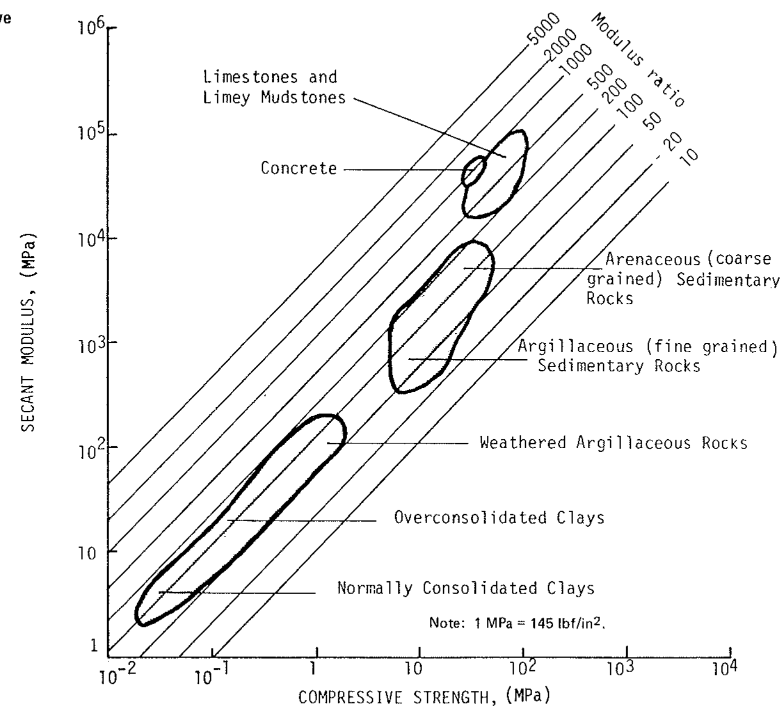
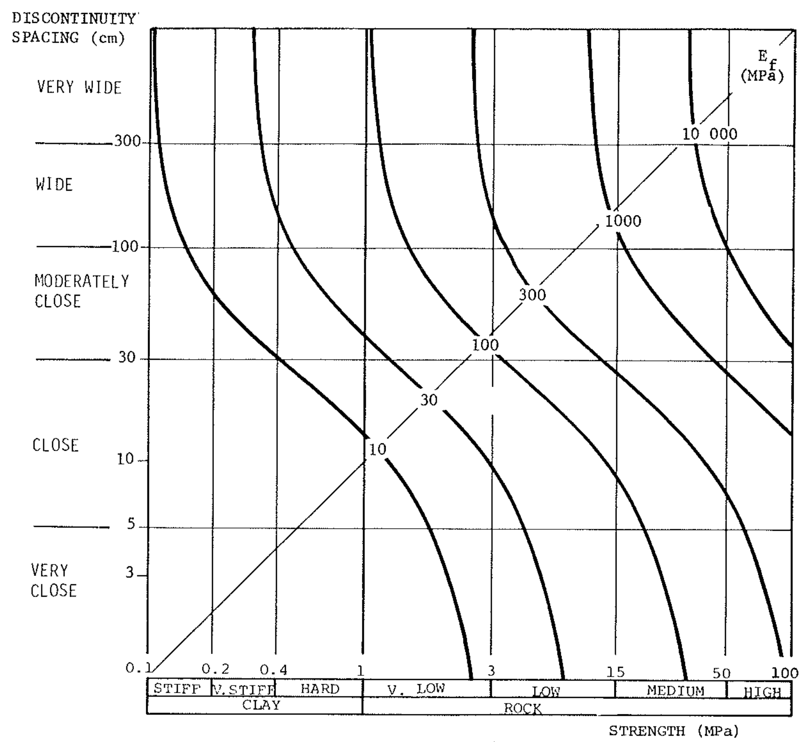


Figure 12. Contours of field modulus of deformability (E_f) of shales as a function of uniaxial compressive strength and discontinuity spacing (assuming modulus ratio = 100).



Note: 1 cm = 0.39 in; 1 MPa = 145 lbf/in2.

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Technical Guidelines for the Design and Construction of Shale Embankments

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In 1974, the Office of Research of the Federal Highway Administration initiated a comprehensive research study to investigate the causes of numerous, large-scale failures of shale embankments on major Interstate routes in several eastern states during the early 1970s and to develop appropriate remedies. The U.S. Army Engineer Waterways Experiment Station was to conduct a three phase, five-year investigation of the shale problem and provide the necessary guidelines to build safe and functional shale embankments at a reasonable cost. Phases 1 and 2 were to be completed in one year and provide interim guidelines for the practicing engineer until the comprehensive guidelines could be developed. Phase 1 involved a state-of-the-art survey of design and construction practices in use at that time as well as a survey of existing problem areas. Phase 2 involved a similar survey of evaluation and remedial treatment techniques for existing distressed shale embankments. Accomplishments from Phases 1 and 2 provided the necessary foundation for the development (under Phase 3) of improved design criteria and construction control techniques for both new construction and existing problem areas. The development of the improved guidelines is described, and the highlights of the major research results are presented.

The Federal Highway Administration (FHWA) recently published a comprehensive engineering manual that provides technical guidelines for the design and construction of shale embankments. These guidelines were developed for FHWA by the U.S. Army Engineer Waterways Experiment Station (WES) at Vicksburg, Mississippi. This paper presents the salient points of the manual and also highlights some of the prominent events that preceded the investigation by the WES researchers. Some of the prominent findings that guided the researchers during the early stages of the investigation are also discussed in order to delineate the basis for some of the guidelines that were developed. Many of these guidelines were taken from other federal agencies and some state highway agencies.

The research study was initiated in 1974 as a three-phase investigation. Phases 1 and 2 were conducted concurrently during the first year of the study to provide preliminary guidance to states that were struggling with inadequate guidelines for correcting existing failures, evaluating potential failures, and constructing new shale embankments. Phase 3 involved the evaluation of existing guidelines and the development of improved guidelines for