The main consideration involves determining which shales can be placed as rock fill in thick lifts and which shales must be placed as soil and compacted in thin lifts. Test pads should be constructed to determine the required procedures (lift thickness, watering, disking, type of compactor, and number of compactor coverages) for each different shale material.

The common persistence and eventual magnification of shale-embankment distress suggest the need for early evaluation and treatment of embankment problems. Existing distressed embankments should be evaluated by performing a systematic review of design, construction, and maintenance records plus a comprehensive field and laboratory investigation to define the cause of distress or failure. The primary consideration in the remedial treatment of shale embankments should be surface and subsurface drainage measures. When other remedial techniques are applied, drainage measures are usually a necessary supplement.

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Stability of Waste-Shale Embankments

BRUCE C. VANDRE AND LOREN R. ANDERSON

Research conducted by the U.S. Forest Service and Utah State University on the stability of waste-shale embankments is described. Mine-waste embankments can be distinguished from other engineered fills by their variable and loose nature, by the lack of control of gradation and density during construction, and by their deformation tolerance. Stability requirements dictated by government regulations generally focus on the protection of adjacent surface resources rather than on the utility of the embankment. Laboratory and field investigations indicate that waste shales in southeast Idaho have high void ratios, moderate permeability, and low-plasticity fines and are susceptible to collapse settlement on saturation. Commonly occurring slope movements can be classified as slumps, shallow flow slides, and foundation spreading. Fully developed rotational slides are not common in southeast Idaho. The deep slope movements generally result from a reduction in toe support caused by groundwater, excavation, or weak foundation soils. Shear-strength testing of shales at different gradations, durabilities, and moisture conditions indicates that ultimate shearing resistance can be differentiated at two levels that can be related to material conditions. The design of mine-waste embankments should be based on limiting conditions that may include maximum probable precipitation, maximum credible earthquake, saturation or nonsaturation, and index shear-strength parameters. The use of stability charts in design analysis is frequently justified by the simplified nature of the limiting conditions. The emphasis in the design of mine-waste embankments is to control the location of different material types and drainage more than to control slope inclinations.

Surface mining involves the removal and disposal of large quantities of overburden material, much of which is shale. This material is often disposed of in large waste embankments several hundred meters high that vary in volume from several million to several hundred million cubic meters. Until recently, most of these embankments were not engineered. These embankments can be distinguished from highway or earth-dam embankments by the variability of the material properties and the large void space within the embankment material. Because of uncontrolled placement and the variable nature of the waste-embankment materials, conventional procedures for evaluating stability and shear strength are of limited usefulness.

The purpose of this paper is threefold:

1. To present a discussion of the construction methods and resulting characteristics of waste embankments,
2. To present laboratory and field test data that characterize the properties of the waste-shale embankments constructed in the mountains of southeastern Idaho, and
3. To propose the use of index shear-strength determinations for the design of waste-shale embankments.

GENERATION AND DISPOSAL OF WASTE SHALES

Since shale is the most abundant rock type occurring at the earth's surface, it is not surprising that large quantities of mineable minerals are associated with shales. Coal frequently occurs in a rock unit known as a cyclothem, which consists of successive beds of sandstone, shale, clay, coal, and limestone. This succession of beds is repeated many times in the strata of the coal fields. Frequently, the shale and sandstone members form the greater part of a cyclothem and coal, clay, and limestone are subordinate. Commonly, 30-70 percent of the waste rock associated with the surface mining of coal is shale (1). In southeast Idaho, 50-70 percent of the waste rock associated with the surface mining of phosphate ore is shale (2). The remainder of the waste rock is predominantly chert and limestone.

The disposal of waste rock can be a major aspect of the mining operation. Waste-to-ore ratios can typically vary from 1:1 for underground mining to 20:1 for the strip mining of coal. The waste-to-ore ratio for phosphate is approximately 6:1. The allowable economic waste ratio depends on commodity price, refining cost, ore grade, and mining method.

The optional locations for the waste fills are valleys, sidehills, and the mining excavation. Generally, stability is not a concern in the case of waste material placed as backfill. Backfilling is limited by the cost of the double handling of materials or the presence of future potential ore in the excavation. Backfilling is done in some states to satisfy reclamation requirements. Even when backfilling is done, some surface waste placement cannot be avoided because the volume of excavated material after fragmentation will increase and exceed the size of the excavation.

METHODS OF WASTE-EMBANKMENT CONSTRUCTION

Methods of constructing waste embankments at the phosphate mines of southeastern Idaho vary depending on the steepness of the terrain at the mine and the type of earthmoving equipment used. The difference in construction methods causes differences in the engineering properties of the embankment material.

At one mine, where the topography is steep, the embankments are built by dumping waste material over a slope at the end of the embankment (end dumping). The material flows down the slope at an angle equal to the angle of repose of the material. The vertical heights of such embankments often exceed 30 m (100 ft) and have been as high as 90 m (300 ft). The area of the dump is increased as material is continually placed over the edge of the embankment. End dumping results in high void ratios because the bulk of the material is not compacted during placement. Segregation of particle size also results. The large materials roll to the bottom of the slope, and the finer materials remain near the top. If the coarse material near the base of the dump is durable, it behaves as a rock fill. There is grain-to-grain contact of the coarse particles, and the voids are generally not filled with fines. Near the top of a dump, the fines frequently control the permeability and shear strength of the material.

Waste dumps at a second mine, where the topography is comparatively flat, are placed in horizontal layers approximately 0.5 m (1.65 ft) thick. Wheel tractor scrapers place the material. Some compaction is achieved by the scraper wheels as they pass directly over the material. The densities vary because portions of the fill do not receive direct wheel contact. Final slopes are generally graded to 3 horizontal to 1 vertical at both mines for reclamation purposes.

Other common means of waste-embankment placement, not used in southeast Idaho, include aerial tramways and conveyor belts. These free-fall methods also result in particle segregation and high void ratios.

EMBANKMENT PERFORMANCE REQUIREMENTS

The disposal of mine waste can be a liability from the perspective of both the mine operator and the surface resource manager. In mining, overburden disposal is a nonproductive cost. In addition, waste embankments can change or create potential hazards for future land use or result in adverse environmental impacts. Some federal regulations (e.g., 36 C.F.R. § 252.1) require waste disposal operations to be "conducted so as, where feasible, to minimize adverse environmental impacts" and to take into consideration (a) air and water quality standards, (b) protection of fish and wildlife habitats, (c) reclamation, and (d) harmony with scenic values. Embankment instability may adversely affect any or all of these factors. The prime function of waste embankments is disposal. Performance requirements generally consider the protection of other resources more than the utility of the embankment.

From an engineering perspective, waste embankments can be distinguished from highway embankments and dams with respect to design life, maintenance, and settlement tolerance. The design life or term of service for highway embankments or dams is frequently limited by the durability of the appurtenant structures. The appurtenances can also control the allowable deformation limits of the embankment. Asphalt, concrete, and steel have finite and somewhat predictable service lives. Waste embankments are landforms and, geologically, landforms are inherently unstable. It's just a matter of time, which is uncertain.

Stability depends on future physical conditions such as rainfall, earthquakes, groundwater development, or changes in the properties of embankment materials. Theoretically, "worst" conditions such as the probable maximum precipitation and the maximum credible earthquake can be estimated. These conditions are the upper physical limits extrapolated from existing knowledge, which is limited. The uncertainty of future conditions is the primary basis for the risk of future instabilities. Whereas worst conditions are considered in the design of dams because of adverse consequences, such as loss of life, worst conditions are considered in the design of waste embankments because of the ever-present possibility of their occurrence. Designing for lesser conditions may be more acceptable when probability, rather than consequences, is the basis for a worst-conditions design. Acceptance of lesser design conditions must consider optimization (mineral development versus environmental protection), cost-effectiveness, and society's reluctance to consciously impose risks on future generations. To enable government agencies to approve "optimum" designs, regulations must permit discretionary acceptance rather than promulgate mandatory design standards. However, discretionary authority may result in inefficient, time-consuming, or inequitable approval practices.

Magnitudes of settlement that would generally be unacceptable for roadway or dam embankments are acceptable for waste embankments. The limiting deformation for waste embankments is the amount that would disrupt surface drainage, create excess pore pressure, or create other hazards for stability.

SLIDE CLASSIFICATION

Slope movements common to non-water-impounding mine-waste embankments may be classified as slumps, shallow flow slides, or foundation slides (3). In southeast Idaho, slumping has generally been limited to shallow depths. Edge slumps result from the oversteeping of the upper portion of the slope. This can be caused by an accumulation of fines or by temporary cohesion associated with moisture (4). Fully developed rotational slides are not common in shales that have a low clay content. However, deep-embankment slides can result from a reduction in toe support caused by groundwater, excavation, or weak foundation soils. If the foundation shear strength exceeds the embankment shear strength and is frictional in nature, and if excess pore pressures do not occur in the foundation, sliding surfaces will be confined within the embankment.

Shallow flow slides are frequently initiated by rain or snowmelt. Infiltrating water can saturate surface soils, provided an adequate supply of water is available to fill the air voids and the runoff or rainfall intensity exceeds the infiltration rate. The depth of saturation depends on soil permeability, porosity, degree of saturation, and the available supply of water. Flow slides occur because of the shear failure of the soil or the collapse of the soil structure. These slides can disrupt reclamation activities and can affect locations at considerable distances below the waste embankment.

Foundation slides involve shearing at the embankment-foundation interface or below the embankment. End dumping, a rapid-loading condition, can result in the development of excess pore pressures in the foundation soils and cause a slope wedge to translate laterally. The foundation soils can be shoved or pushed ahead of the advancing toe. This type of sliding is classified as foundation spreading. The development of excess pore pressure depends on the degree of saturation, the permeability of the foundation material, and the rate of advancement of the dump slope. If the loading exceeds the shear strength because of pore-pressure buildup, foundation spreading will occur. Generally, the slope will stabilize after a period of time if the fill placement is discontinued. To resume dumping and avoid future movements, the dump height or advancement rate would need to be decreased. If the drained residual foundation strength is unusually low, buttressing and confinement or translation to flatter terrain may be needed to stop the slide. For saturated clay foundations, generally it is not feasible to limit rates of fill placement to avoid undrained loading conditions. In addition, it is not common practice to construct dump slopes rapidly enough to develop excess pore pressures in sand foundations. The rate of advancement of dumps supported by silty material can be critical to foundation stability.

Embankments dumped on steep slopes may translate along the contact between the embankment and the foundation. These slides may occur during embankment construction because of the steepness of the foundation or can be initiated later by the decay of organic matter, earthquake forces, the melting of buried snow, or other occurrences of groundwater. The slope of the natural ground determines both the potential for and the consequences of sliding. As the slope of the foundation increases, so do the sliding potential and the potential area of impact of the sliding.

DESCRIPTION OF THE INVESTIGATION

In 1978 (phase 1 of the investigation), laboratory and field testing were undertaken to classify and evaluate the general engineering properties of overburden materials wasted in the mining of phosphate ore in southeastern Idaho (5). In the spring of 1980, waste-shale materials were sheared at low normal pressures to evaluate strength parameters for the analysis of shallow flow slides (6). In the fall of 1980 (phase 2), waste-shale materials were tested to develop index shear strengths for use in the stability analysis of mine-waste embankments.

Phase 1

Classification

Seven shale samples, weighing approximately 110 kg (50 lb) each and representative of the finer particle sizes, were obtained from waste embankments at two different phosphate mines in southeastern Idaho. Particles larger than 51 mm (2 in) were discarded. The average and range of grain-size distributions are shown in Figure 1. The liquid limit and plastic limit, averaged for the classification samples, were 23 and 19 percent, respectively. Most of these samples were classified as silty gravel (Unified Soil Classification GM).

Compaction

To provide some perspective as to the range of compaction occurring in layer-placed embankments, 15 density tests were taken at locations beneath or between the wheels of the hauling equipment. Laboratory compaction curves (AASHTO T99-74) were determined for the samples at each density location. The average degree of laboratory compaction for locations subjected to wheel traffic was 90 percent; the average compaction for locations not subjected to wheel traffic was less than 80 percent. The compaction range was highly variable (standard deviation of 6). The average moisture content of the density tests was approximately 2 percent below optimum. Changes in the placement moisture content of waste shales can vary with the time of year as well as other climatic conditions. The field density tests were taken during the summer.

In the construction of end-dumped embankments, only the finished areas and haul roads receive wheel compaction. These areas represent a small fraction of the entire waste embankment. Density testing on an angle-of-repose slope is difficult; one test, however, had a relative compaction of 79 percent.

Permeability

Constant-head permeability tests were performed on samples compacted to densities that varied from 86

Figure 1. Particle-size gradation of representative samples of waste shale.

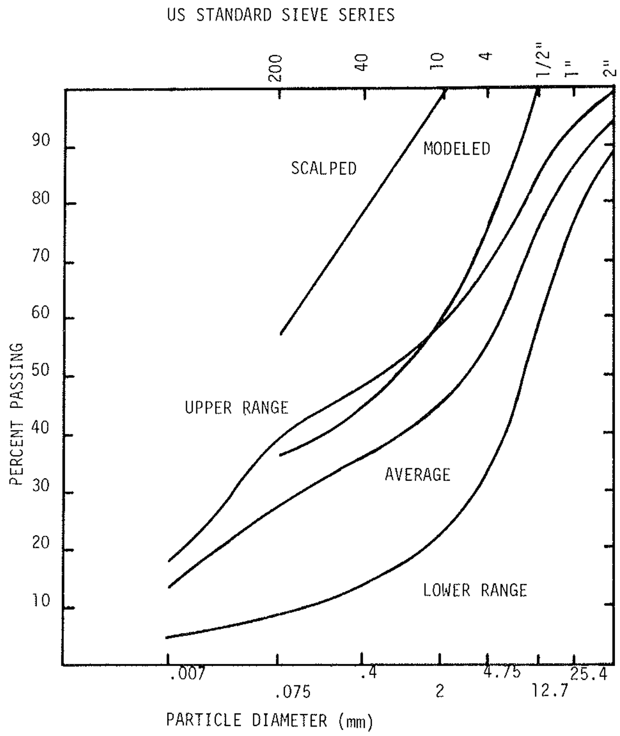
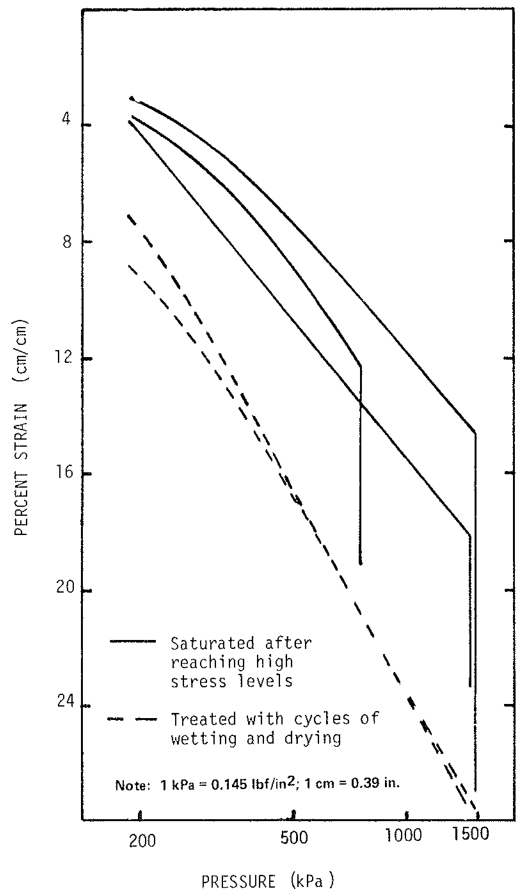


Figure 2. Strain versus log pressure for coarse waste-shale material showing the effects of moisture.



to 95 percent of the laboratory maximum at a moisture content 2 percent dry of optimum (AASHTO T-99). The average permeability for four tests was 3×10^{-5} cm/s (0.9×10^{-5} in/s). This permeability is believed to be more representative of layer-placed embankments than end-dumped embankments and interior fill rather than exterior fill. The effects of weathering and siltation from surface erosion could substantially reduce infiltration at the dump surface.

Compression Tests

Confined compression tests were run on the classification samples to evaluate the settlement characteristics. Samples representing two different grain-size distributions were used for testing. One set of samples was prepared to represent the coarse shale material that occurs near the bottom of end-dumped embankments where there is substantial grain-to-grain contact of the coarse particles. A second set of samples represented the waste-shale material that has substantial fines. The coarse samples were prepared by passing crushed cherty shale material through sieves and retaining the fraction passing the 4.75-mm (no. 4) sieve but retained on the 0.8-mm (no. 20) sieve. A 6.4-cm (2.5-in) diameter consolidometer was used to perform the compression tests. Different moisture treatments were used in performing the tests. Two tests were performed on samples that were subjected to wetting and drying cycles during the tests. Three tests were performed by increasing the normal load on dry samples and then saturating the samples after they had reached a given stress level. The results of these tests on the coarse material are shown in Figure 2. The slope of the strain versus log of stress curves was nearly twice as steep for samples subjected to wetting and drying cycles as for samples that were compressed dry. Complete saturation of relatively dry samples caused an immediate increase in compression (collapse settlement). However, the magnitude of vertical strain is about equal to the final normal pressure for both the cyclic moisture condition and the collapse condition.

Similar tests were performed on compacted samples of waste shale that contained significant fine material. The saturation collapse settlement was not as great for the fine material, and increased compaction decreased the magnitude of compression.

Total and Effective Shear-Strength Parameters Versus Compaction

The effect of compaction on the strength of partially and fully saturated waste-shale samples was evaluated from the results of 23 consolidated-undrained triaxial shear tests with pore-pressure measurements. The triaxial test specimens had a diameter of 3.6 cm (1.4 in) and were prepared by passing one of the classification samples through a 2.0-mm (no. 10) sieve and discarding the plus-2-mm material. The scalped test gradation is shown in Figure 1.

The triaxial shear tests were run on samples compacted to 80, 90, and 100 percent of the laboratory maximum (AASHTO T-99) at a moisture content approximately 2 percent dry of optimum. The tests were run at confining pressures of 69, 207, and 345 kPa (10, 30, and 50 lbf/in^2). The strength envelope was based on the maximum deviator stress reached before a strain of 8 percent. The Mohr-Coulomb strength envelopes had a shear-strength intercept that curved upward with increasing confining pressure. Both intercept and curvature

Table 1. Friction angle versus compaction and moisture content for waste-shale material from triaxial shear test results.

Relative Compaction[a] (%)	Void Ratio	Water Content (%)	Friction Angle ϕ (°)	
			Total	Effective
80	0.90	14	31.0	
90	0.69	14	37.0	
100	0.52	14	47.5	
80	0.90	-[b]	15.0	29.5
90	0.69	-[b]	15.0	33.0
100	0.52	-[b]	37.5	39.0

[a]AASHTO T99. [b]Saturated.

increased with compaction. To indicate the effects of compaction on the partially saturated samples, the friction angles, as defined by a straight-line envelope through the origin of the axes and tangent to Mohr's circle at the 345-kPa confining pressure, are given in Table 1. The complete results, including the stress-strain curves, have been presented elsewhere (5).

Saturation after placement may be a possible future condition, as a result of extreme flooding or the melting of large snow masses that were buried in the embankment during construction. Therefore, consolidated-undrained triaxial shear tests with pore-pressure measurements were also run on saturated samples. Saturation was achieved by using back pressure. Both total and effective strength parameters were determined. The results are summarized in Table 1, along with the partially saturated test results. These results are based on best-fit linear strength envelopes through the origin of the axes for shear stress and normal stress.

Direct Shear Tests at Low Normal Pressure

Thirty direct shear tests were performed at normal pressures less than 27.6 kPa (4 lbf/in^2) on laboratory-molded samples of waste shale. The gradation of these samples modeled the average gradation of the classification samples presented in Figure 1.

The grain-size distribution of the modeled material was obtained by shifting the grain-size-distribution curve for the average, approximately parallel to itself, to the desired maximum particle size for the laboratory specimen. The liquid limit and the plastic limit of the tested material were 27.4 and 24.5 percent, respectively. The samples were loosely placed in the test equipment and compressed to void ratios that averaged 0.87 with a standard deviation of 0.03.

The samples were saturated by upward seepage of water under a hydraulic gradient of one and were maintained in a saturated condition for approximately 16 h before testing. The tests were conducted in accordance with the method given by AASHTO T236-72. All samples were sheared under consolidated-drained conditions with a constant rate of shear displacement of 0.7 mm/min (0.03 in/min) and a gap spacing of 6.4 mm (0.25 in). The samples were 10.2 cm (4 in) in diameter and approximately 5.1 cm (2 in) in height. The slow rate of shear displacement allowed sufficient time to ensure total dissipation of pore-water pressure within the sample, and effective stress parameters were obtained.

Failure was defined as the maximum shearing stress or the shearing stress at 10 percent lateral strain (shear displacement divided by the sample diameter), whichever occurred first. The friction angle and the strength intercept obtained from the best-fit strength envelope were 29° and 1.3 kPa (0.18 lbf/in^2), respectively. The average coefficient of variation (ratio of the standard deviation to the mean) for the test shear-strength values obtained at three different normal pressures was 20 percent.

Phase 2

Testing samples with scalped gradations will yield conservative estimates of shear strength for many field conditions. However, the use of this approach has the following limitations for the design of mine-waste embankments: Testing of representative samples is not possible, and fine-grained material strengths may be overly conservative for some mine-waste-embankment conditions. In the laboratory testing performed to examine the relationship among shear strength, gradation, and durability, it was expected that shear strength could be indexed to particle-size classification. As testing progressed, the importance of the identification of shear-strength components became apparent. The following discussion of the phase 2 investigation consists of a review of the mechanics of shear resistance of granular materials, test results, conclusions, and recommendations for indexing waste-embankment shear strength.

Mechanics of Shearing Resistance

The drained shearing resistance of granular material consists of the following components: sliding friction, particle rearrangement, dilatancy, and particle crushing (7,8). The slope of the Mohr-Coulomb strength envelope can be explained in terms of the different shear-resistance components. At lower normal pressures, the slope of the envelope is controlled by resistance to particle rearrangement and by dilatancy and sliding friction. For example, medium-grained uniform quartz sand experiences very little crushing below 37 kPa (5 lbf/in^2) (9). As the normal pressure increases, the slope of the envelope decreases when crushing has a net effect of reducing dilatancy effects. At very high pressures, particle crushing requires considerable energy and, together with a possible small increase in friction, causes the strength envelope to steepen.

When the strength envelope is curved, the friction angle can be determined at various normal pressures ($\tan^{-1}$ shear stress/normal stress) rather than measuring the inclination of the strength envelope over a range of normal pressures. This "point" friction angle has been referred to as the "equivalent" friction angle in a British study of coarse colliery waste materials (10).

The friction angle for a best-fit, straight-line strength envelope will not reflect all of the dilatancy and crushing contributions to shear resistance, which vary with normal pressure. This approach may be overly conservative if it eliminates crushing contributions. In addition, a straight-line envelope is conservative if the shear-strength intercept is eliminated from contribution because it is mistakenly attributed to negative pore-pressure effects. The variation of shear strength with normal pressure may also be examined by plotting the point friction angle as a function of the logarithm of the normal pressure. A linear relation commonly exists (11).

At a constant-placement relative density, gradation appears to have pronounced influence on the shear strength of rock-fill materials whereas, at a constant-placement void ratio, the influence of gradation seems to be negligible (12). A resulting similarity in void ratios appears to be the explanation for the acceptable use of modeled gradations in shear-strength testing. In the shearing of a loose granular material, deformation tends to increase the shear strength by decreasing the void ratio (particle rearrangement). It is apparent that, when shear strength is determined by strain criteria, loose materials in which there is a large difference between minimum and maximum void ratios will have low shear strengths compared with materials that have a small range of possible void ratios. The energy required for compression particle rearrangement is less than that required for dilatancy or crushing.

It is common knowledge that the fines content [minus 0.075-mm (no. 200) sieve] affects the shear strength of granular materials. The American Association of State Highway and Transportation Officials (AASHTO) and Unified Soil Classification systems suggest that approximately 15, 35, and 50 percent fines contents have a significant effect on the engineering properties of soils (13). The mechanics of how fines affect shear strength have not been explained. Intuitively, the presence of fines is expected to decrease resistance to particle rearrangement and dilatancy. Studies that have examined the relation between shear strength and fines content in mine-waste materials have not considered the combined effect of relative density and grading in their examinations (11).

Laboratory Testing Program

The U.S. Forest Service performed 97 direct shear tests and 8 triaxial tests on 6.4-cm (2.5-in) diameter samples for the phase 2 investigation. Utah State University performed 21 direct shear tests on 10.2-cm (4 in) diameter samples. The direct shear tests were performed at normal pressures that varied from 12 to 372 kPa (1.8-53.9 lbf/in^2). The triaxial tests were performed at confining pressures of 138, 345, or 690 kPa (20, 50, or 100 lbf/in^2). The 10.2-cm direct shear apparatus was used primarily to test the coarser gradations. Triaxial testing was performed for comparative purposes and to obtain higher normal pressures than would be practical with the direct shear apparatus.

Five gradations (A through E) of durable, low-plasticity shale were tested (see Figure 3). These gradations included the significant fines content recognized by the engineering soils classification systems previously mentioned. The minus-2-mm (no. 10) particle-size content was based on the ratio of percentage finer than 2 mm to percentage finer than 0.075 mm (no. 200) of the average gradation for the classification material (Figure 1). The 2-mm particle is the boundary size between sand and gravel in the AASHTO Soil Classification system. It is also a reference size in slake-durability testing. The linear particle-size distributions between the control sizes--maximum test particle size 2 mm and 0.075 mm--are reasonable, considering that placement methods result in poorly graded particle distributions.

For comparative purposes, two other materials--quartzite and nondurable shale--were tested at gradation E. The nondurable shales degraded into a pile of flakes after immersion in water, whereas the durable shale formed only a few fractures.

The direct shear samples were tested by using a rate of shear displacement that varied from 0.5 to 1.3 mm/min (0.02-0.05 in/min). The gap spacing equaled 1 cm (0.4 in) for the 1.25-cm (0.5-in) maximum particle gradations and was equal to or greater than the maximum particle size for the other gradations. Moisture conditions for the tests were either air-dried or saturated.

Test Results and Discussion

The shear-strength parameters and void ratios for the different gradings and materials are summarized in Table 2. The void-ratio ranges were estimated by considering the range of densities measured for 10 loose placements at each grading and random compression measurements at the various normal or confining pressures. The terms point friction angle and envelope friction angle are defined in Figure 4.

In the air-dry condition, the gradations of the durable and nondurable shales compressed from 3 to 8 percent at normal pressures that varied from 99 to 371 kPa (14-54 lbf/in^2) and confining pressures as high as 690 kPa (100 lbf/in^2). In the saturated condition, the durable shale compressed up to 12 percent and the nondurable shale compressed rapidly up to 24 percent at the 371-kPa loading. The quartzites compressed less than a fraction of 1 percent at the 371-kPa normal pressure loading for both the dry and saturated conditions.

All shale gradations except gradation E exhibited peak shear strengths (curve type 1 in Figure 5) at normal pressures from 99 to 371 kPa. Shale at gradation E developed peak shear strengths (curve type 3 in Figure 5) at normal pressures of less than 37 kPa (5 lbf/in^2), at which level dilatancy could not be prevented. Type 3 curves were also developed by material tested at gradation A and at the higher triaxial confining pressures (significant compression).

The effect of saturation on the durable shales was to change the shape of the stress-strain curve from a flattening curve (type 2) to a continuously rising curve (type 4). The shear strength of the durable saturated shale generally reached the shear strength of the air-dry shale within 15 percent

Figure 3. Gradations for shear-strength testing (phase 2).

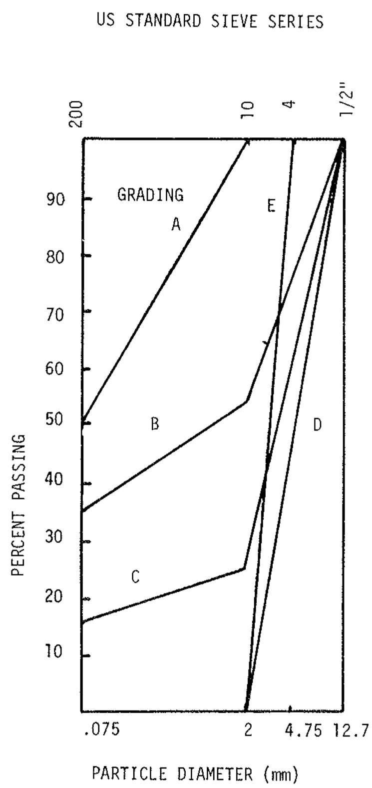


Table 2. Summary of shear-strength parameters.

Sample	Grading	Initial Void Ratio	Normal Pressure (kPa)	Average Point ϕ (°)		Envelope ϕ (°)/Intercept (kPa)	
				Peak	Ultimate	Peak	Ultimate
Durable shale	A	0.8-0.95	12.4-37.1	42	40	35/5	35/0
			99.4-371.9	39	36	37/30	37/4
			99.4-371.9		31[a]		28[a]/18
	B	0.5-0.65	119.5-352.0	37	35	37/0	29/33
	C	0.65-0.70	119.5-352.0	43	39	36/48	25/75
	D	1.15-1.25	119.5-352.0	47	40[a]	39/42	35[a]/24
	E	1.05-1.2	12.4-37.1	50		42/57	
			99.4-371.9		39		38/3
			99.4-371.9		39[a]		39[a]/0
Nondurable shale	E	1.05-1.2	99.4-371.9		39		37/10
		0.9-1.1[a]	99.4-371.9		32[a]		29[a]/13
Quartzite	E	1.05-1.2	12.4-37.1	49		39/70	
			99.4-371.9		40		40/0

Note: 1 kPa = 0.145 lbf/in^2.

[a]Saturated sample.

Figure 4. Definitions of envelope and point friction angle.

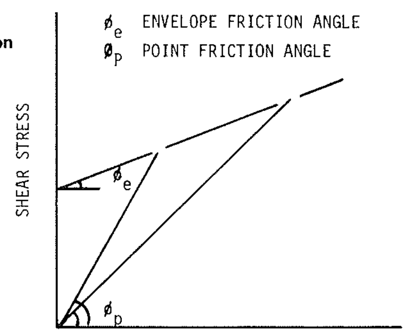
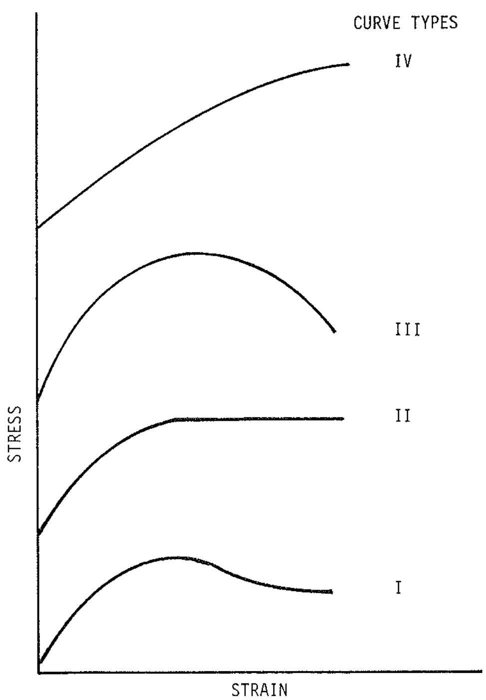
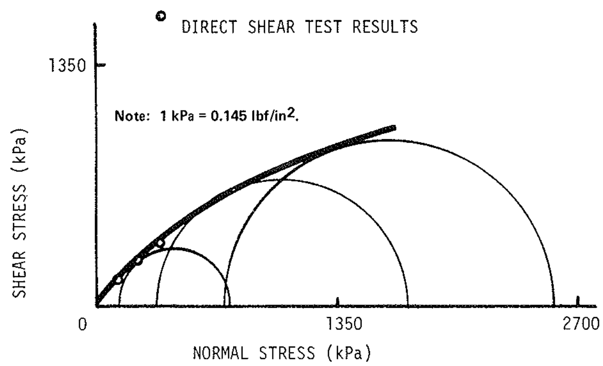


Figure 5. Classification of laboratory stress-strain curves.



strain deformation. The stress-strain curve for the nondurable shales was the flattening type for both the saturated and dry conditions; however, the saturated shear strength was significantly lower.

Figure 6. Comparison of triaxial and direct shear test results (gradation D).

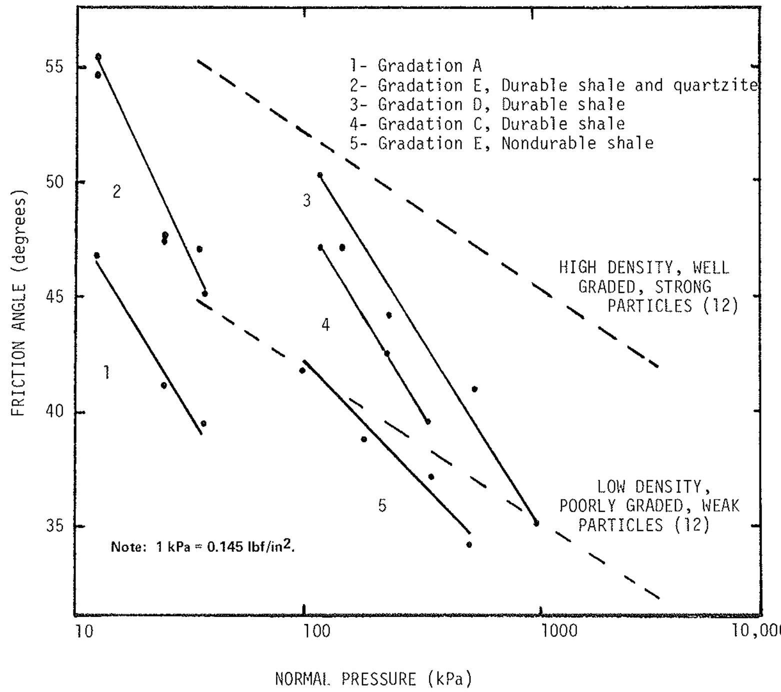


The friction angle for the saturated nondurable shales (32°), averaged for the different normal pressures, nearly equaled the similarly averaged saturated ultimate friction angle for the fine-grained gradation A (31°). After saturation, the collapsed void ratio of gradation E nearly equaled that of gradation A. The nondurable shales collapsed to such an extent on exposure to water in the triaxial apparatus that saturation of the sample under confining pressure became extremely difficult because of the decreased permeability. Saturation and shear testing degraded the nondurable shale samples from zero percent passing the 2-mm (no. 10) sieve (gradation E) to 59 percent passing the 2-mm sieve and 11 percent of the total sample finer than the 0.075-mm (no. 200) sieve. The durable shale samples also degraded during saturated shearing to 36 percent passing the 2-mm sieve and 4 percent of the total sample finer than the 0.075-mm sieve.

The Mohr-Coulomb strength envelope for gradation D of the durable shales determined by triaxial testing is shown in Figure 6. The direct shear test results, also included in this figure, fit on the strength envelope for the triaxial tests. The variation in shear strength between triaxial and direct shear tests reported for mine-waste materials by others (14) could be explained by differences in normal pressure ranges. The other triaxial test results also closely agreed with the averaged direct shear results. However, the variation among replicated direct shear tests suggests that repetitive testing is advisable.

Averaging the test friction angles for the shales in each column in Table 2 has produced the following results: For point friction angle, peak = 43° and

Figure 7. Friction angle versus log pressure for different particle gradations and durability.



ultimate = 37°; for envelope friction angle, peak = 38° and ultimate = 35°. If one considers the previously described shear-resistance theory for granular materials, the difference between the point peak and point ultimate friction angles (6°) can be attributed to dilatancy. The envelope friction angle for the peak strengths (38°) also appears to remove dilatancy effects in that its value nearly equals that of the point ultimate friction angle (37°). The parameters for the ultimate friction angle envelope cannot be explained in terms of shearing-resistance components.

Point friction angles are plotted in Figure 7 as a function of the logarithm of the failure normal pressure. Steeply sloping plots 1 through 4 for peak friction angles indicate the severe effect of normal pressure on dilatancy. The flatter slope for ultimate friction angles for the nondurable shale (plot 5) suggests that normal pressure has a less detracting effect on crushing contributions to shear resistance. The ultimate shear resistance for the durable shales at gradations A and E and the quartzite were not significantly affected by normal pressure within the testing load range.

At low normal pressures [<37 kPa (<5 lbf/in^2)], dilatancy appears to be controlled more by grain size than by material type. Dilatancy for plot 1 (the fine-grained gradation) is lower than that for plot 2, which fits the friction-angle/normal-pressure points for both the quartzite and the durable shale.

The laboratory test results given in Table 2 exhibit two levels of ultimate friction angles, approximately 31° and 38°. These friction angles reflect moisture conditions, grain sizes, and stress levels. Saturated nondurable or fine-grained shales can be assigned an index shear-strength value of 31°. It appears reasonable to assign an index shear-strength value of 38° to nonsaturated waste shales irrespective of grading and durability. The rationales for using ultimate strength for index purposes are as follows:

1. If the strain at every point along a sliding surface in an embankment could be measured, it would vary from near zero at the most recently mobilized point to a relatively high value at the initially mobilized point. The peak shearing resistance occurs at only one value of strain. Eventually, the ultimate resistance can be available along the entire failure path.
2. Ultimate strength values do not include dilatancy effects, which vary with placement conditions.

STABILITY ANALYSIS AND DESIGN

When in situ embankment conditions cannot be reliably predicted, design decisions should be based on limiting conditions. Limiting conditions for mine-waste embankments include maximum probable precipitation, maximum credible earthquake, index shear-strength parameters, and saturated or unsaturated material.

A sliding wedge is frequently used to analyze the stability of granular soils. The wedge is recognized to be a fundamental configuration for self-weight-type analysis of clastic materials (15). A theoretical limiting or worst condition for a sliding wedge would be the full arching condition (16). Unequal compression of loose embankment material can initiate arching. For shales, the shearing resistance available along the sliding surface will depend on the material conditions. Unless positive drainage is ensured by the embankment design and construction control, saturated conditions should be assumed. However, the design for saturated conditions is often not practical.

The infinite slope model can be used to analyze the potential for shallow flow sliding. For this type of sliding to occur, a saturated wetting front must develop. For material that has a shear-strength intercept, the stable slope angle will be affected by the depth of saturation. In most temperate climates, maximum probable precipitation events will supply enough water to saturate to such a depth that the stabilizing effect of the strength intercept is offset. If the strength intercept is ignored (limiting condition), design slopes become quite flat and stability is controlled by the occurrence of saturation. However, highly permeable material can be used to prevent the development of a wetting front at the final construction-slope location.

Stability charts can be developed for use in the analysis of the stability of mine-waste embankments. The use of stability charts is often justified by the simplified nature of limiting conditions. Charts have been developed for analyzing deep-seated wedge slides, foundation spreading, and shallow flow slides (17). The average normal pressure on a wedge sliding surface has also been charted as a function of embankment height and foundation inclination.

In deciding on design safety requirements, it is important to make a distinction between construction slopes and abandoned and/or reclaimed slopes. Frequently, the safety factor can be lower for construction slopes than for the final slopes. This reduction can be justified by a comparatively short-term risk and the opportunity for remedial or mitigating measures. In addition, assuming that the upper level of shearing resistance will be available may be reasonable for construction evaluations but not reasonable for long-term conditions.

The most controversial stability issue associated with the regulation of mine-waste embankments is the acceptability of end-dumping construction methods and the abandonment of angle-of-repose slopes. If one uses the index shear-strength parameters and chart analysis, angle-of-repose slopes appear reasonably safe in relation to deep-seated sliding, provided the slope of the foundation is less than 14° and saturation can be prevented beneath the slope. The potential for shallow flow slides can be eliminated by using durable, highly permeable materials in the construction of the final slopes. However, the use of these materials would limit revegetation opportunities. The potential for raveling cannot be eliminated but can be mitigated by designing benches. It appears, then, that the construction of waste embankments by end dumping may be acceptable in situations that involve (a) restricted site locations, (b) select waste materials, and (c) limited revegetation requirements.

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