THE DESIGN OF SPOIL BANKS AND HOLLOW FILLS

FOR SURFACE MINING

by

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Chapter 1

INTRODUCTION

The surface mining reclamation and enforcement provisions published in December 1977 [27] and subsequent revisions [10] place a heavy burden on the surface coal mine industry with respect to the maintainance of spoil banks and hollow fills. An economic problem exists as legislation demands that construction techniques be based on the "worst operating conditions," while no variance is allowed for different site conditions.

The predominant concern is the stability of the compacted fill material, which for engineering purposes, is termed slope stability. This report develops the following concepts, as they relate to the design of spoil banks and valley fills: (1) factors affecting spoil slope stability; (2) methods of stability analysis; (3) legal requirements for fill construction under public law 95-87; (4) a check-list for typical valley fill construction; (5) environmental considerations; and (6) practical impacts of recent federal regulations on the mining industry.

Before one can hope to intelligently evaluate, or design a structure, a complete understanding of the engineering fundamentals is essential. The principles of soil mechanics, as they relate to slope stability are developed in Chapter two.

In addition, the concepts of soil shear strength, seepage, dilatancy and compaction are developed as they relate to slope stability.

Chapter three introduces the two possible modes of slope failure: plane and cylindrical. The methods used to evaluate the possibility of failure, as proposed by recognized experts, are presented, as well as their theoretical development.

Chapter four deals with the legal requirements of fill construction. These are grouped under the following headings: (1) site investigation; (2) site preparation; (3) placement of fill material; (4) configuration of the fill; and (5) surface drainage. Certainly, this would represent the order in which the steps necessary for fill construction would be performed.

The next chapter develops the basic phases of surface mine operation. It is believed that this is necessary to properly plan and consider all aspects of fill construction. These steps are: (1) site feasibility and planning; (2) mining; and (3) reclamation. The last area, reclamation, has the greatest impact on the environment. This subject is expanded in Chapter six which deals with water quality.

Certainly, surface mining results in substantial generation of sediment. There is evidence that sediment ponds that are used to remove sediment from runoff, are not always efficient. It appears that chemical addition may be necessary, in some cases, to meet effluent standards under Public Law 95-87. This process is developed.

Chapter seven presents some conclusions concerning the necessity of concurrent compaction in fill construction. This requirement has met with considerable opposition from the mining industry [9] and their comments appear justifiable.

Further impacts of Public Law 95-87 are addressed in Chapter eight. Finally, the report ends with conclusions and recommendations for further work.

Chapter 2

FACTORS AFFECTING SPOIL SLOPE STABILITY

2.1 Shear Strength

An analysis of the stability of a slope can be divided into two parts: (1) determining the internal surface along whose line the greatest stress exists; (2) determination of the magnitude of this stress, and the soil's ability to resist it. The stress that affects slopes is shear, and the soils resistance to failure is dependent on its ability to withstand shear. The slope will remain stable if the shearing strength of the soil, of which it consists, is at all times greater than the stress on its most stressed internal surface. Those factors determining a soil's shear strength are cohesion and the internal angle of friction. Since the values of these factors must be determined before any stability calculations may be performed, it would seem appropriate to define and show the relationship of these terms to a soil's shear strength before developing more advanced concepts. This is done in the following section.

2.2 Cohesion and Friction

The shear strength of soil is largely a function of the internal angle of friction ϕ. For those soils considered in this report, the cohesion component of shear strength is of minor importance, since mine spoils have relatively little cohesion [14,3]. Figures 2.1A and 2.1B illustrate the determination of these two factors by the direct shear

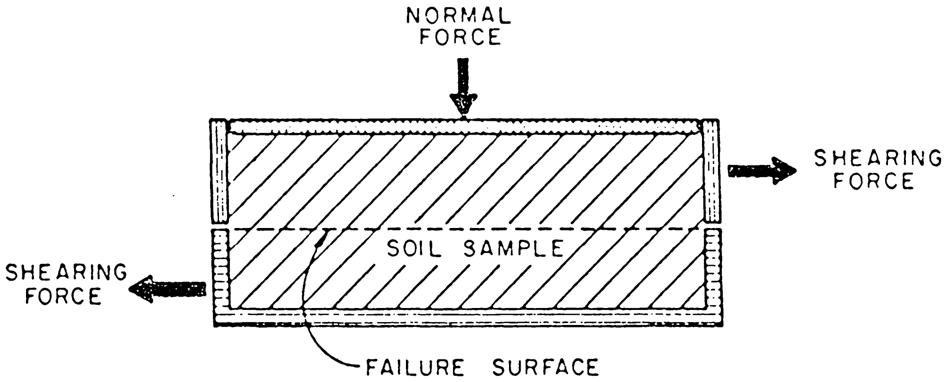


Figure 2.1A. Shear box (from E. D'Appolonia, 1976).

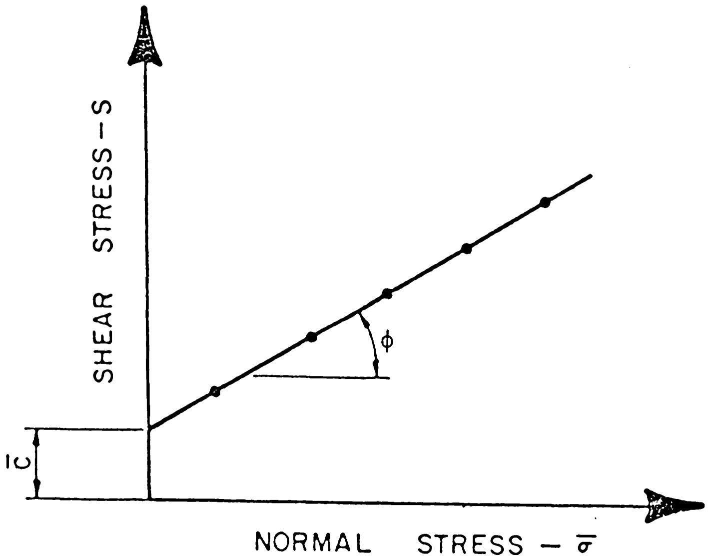


Figure 2.1B. Test results (from E. D'Appolonia, 1976).

test. In equations used to determine the amount of shear stress that a soil can resist, $\tan \phi$ is always multiplied by the force acting normal to the failure surface being considered. The reason for this is made obvious by Figure 2.1B, since it shows that

$$\tan \phi = \frac{\text{shear stress}}{\text{normal stress}} .$$

The basic relationship between normal stress σ on a section through a mass of soil and the shearing stress s per unit area resisting shear can be represented as:

$$s = \sigma \tan \phi + c$$

This equation is known as Coulomb's equation.

It must be realized that the values of c and ϕ used in this equation are merely two empirical coefficients that are determined for a particular soil based on laboratory experiments [17]. In Coulomb's equation, cohesion is an abbreviation of apparent cohesion, not true cohesion, which would actually include the coulomb c and an appreciable part of $\sigma \tan \phi$ [38,18]. In those soils classified as cohesive, their shear strength is not increased by normal forces. The shearing strength which is independent of normal pressure, is called cohesion or no-load shearing strength [37]. It can therefore be concluded that in this situation, the term cohesion does not denote any inherent strength from interparticle attraction.

The friction angle ϕ of a soil is primarily influenced by the interlocking of grains, which must be lifted over one another for lateral movement to occur [24]. Since the lifting must be done against the normal stress σ, the lift force and sliding friction is proportional to σ. In soils, ϕ is therefore designated the angle of internal friction, because it represents the sum of sliding friction plus interlock [37]. This concept of particle interlocking will be developed later in the report, when considering dilatancy. Because of the mechanics of this interlocking of granular particles, moisture does not directly influence these mechanisms in most cohesionless soils because the intense stresses at the contact points between grains force the water molecules aside [41]. In recent work, Huang has verified this to be true in mine spoils [14]. In saturated conditions, however, water can affect shear strength indirectly and must be considered. The effects of pore water pressure will be dealt with in the following section.

2.3 Seepage

Although Section 816.72(A), (B) of Public Law 95-87 provides for the prevention of saturation or seepage of water through valley fills, it would seem appropriate to examine the effects that seepage can have on the fill's stability. As discussed in the previous section, when a small percentage of the voids in the soil are occupied by water, it is easily displaced and has no significant impact on stability. However, if water is allowed to flow through the fill and cause saturation to occur, it can have a major impact on stability [40,44]. In these saturated areas, the water moves between and around individual particles and a level of equilibrium is established. The upper boundary of this area is called the phreatic surface. The determination of the position of the phreatic surface is critical in the analysis of the stability of existing soil structures due to the resulting effects, the most significant being buoyancy.

2.3.1 Buoyancy

All material particles below the phreatic surface are acted upon by the natural buoying effect or force that water exerts on all submerged bodies [40]. This force is 62.4 pounds of lift for each cubic foot of water displaced. Even in the finest grained soils, all voids are connected to neighboring voids [35]. This, then, would require stability analysis calculations to consider saturated soils to be treated as if submerged. This is accomplished in stability analysis by reducing the normal component of the soil's weight by some percentage of unit water weight. This will be shown in a later development.

2.3.2 Pore Pressure

The effect of pore pressure can be understood by examination of Figure 2.2 in conjunction with the following development of effective stress taken from Perloff and Baron [27]. Several definitions of terms are necessary to understand this development.

A_{th} = The total horizontal projection of the cutting surface, for the soil mass considered

A_{ch} = The horizontal projection of the contact area between the solids lying in the cutting surface.

A_{wh} = The horizontal projection of the portion of the cutting surface which passes through water.

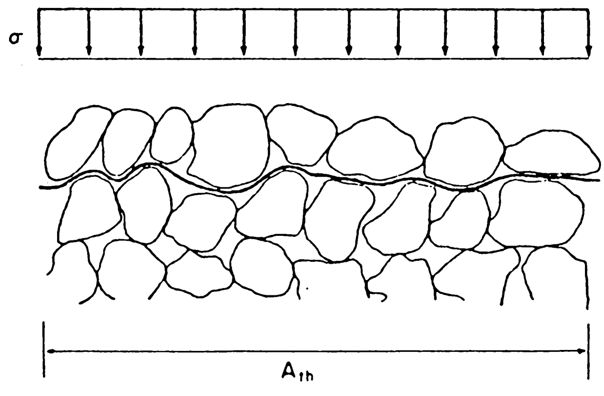


Figure 2.2. Soil mass subjected to average stress (from Perloff and Baron, 1976).

From Figure 2.2 [27] it can be seen that

$$\sigma A_{th} = \sigma^{*} A_{ch} + U A_{wh} \tag{2.1}$$

where σ^{*} is the actual intergranular stress at points of contact, and U is the pressure of water.

Dividing Equation (2.1) by A_{th} gives:

$$\sigma = \sigma^{*} \frac{A_{ch}}{A_{th}} + U \frac{A_{wh}}{A_{th}} \tag{2.2}$$

For coarse material the area of water A_{wh} will be close to the cross sectional area A_{th}, therefore $\frac{A_{wh}}{A_{th}}$ will be unity for practical purposes. Therefore, Equation (2.2) becomes

$$\sigma = \sigma^{*} \frac{A_{ch}}{A_{th}} + U \tag{2.3}$$

The $\sigma^{*} \frac{A_{ch}}{A_{th}}$ term is generally called effective stress σ' and Equation (2.3) becomes

$$\sigma = \sigma' + U \tag{2.4}$$

which is the principle of effective stress first proposed by Terzaghi (1923)[38].

It can be reasoned from Figure 2.2 that if water carries some of the normal stress σ, then the soil particles will carry less load. This will result in a loss in shear strength, since although water can accept normal loading due to its incompressibility, it cannot sustain a static shear stress [27].

The effect of pore pressure U can be seen in Figure 2.3. The Mohr circle for effective stress is shifted to the left of the circle for total stress by an amount equal to the pore water pressure [27]. Therefore, a positive pore pressure causes a reduction in shear strength, but it does not affect existing shear stress directly, nor does it change ϕ'. Its only effect is on normal stress, which then modifies shear strength.

2.3.3 Dilatancy

To further appreciate the effect of pore pressure the concept of dilatancy must be considered. Dilatancy is the tendency of a particle mass to change volume upon application of a shear strain [27]. From Figure 2.4, it would follow that for loose material, the volume change would be negative, due to compaction. For dense compacted material, the volume change would be positive, since the particles would need to "unlock" themselves, or expand for movement to occur. Obviously, compacted soil would have greater effective shear strength due to interlocking of particles. Also, due to the dilatancy principle, any tendency of dense material to move laterally due to shear force would be countered by a reduction of pore pressure due to expansion. For less dense material, shearing forces would tend to cause compaction, which would cause higher pore pressure, with resulting decrease in the fill's ability to resist shear forces. If the fill design did not allow for soil saturation, and it did occur, the reduction in the safety factor might be sufficient to allow failure to occur, as it did in Aberfan, Wales [27].

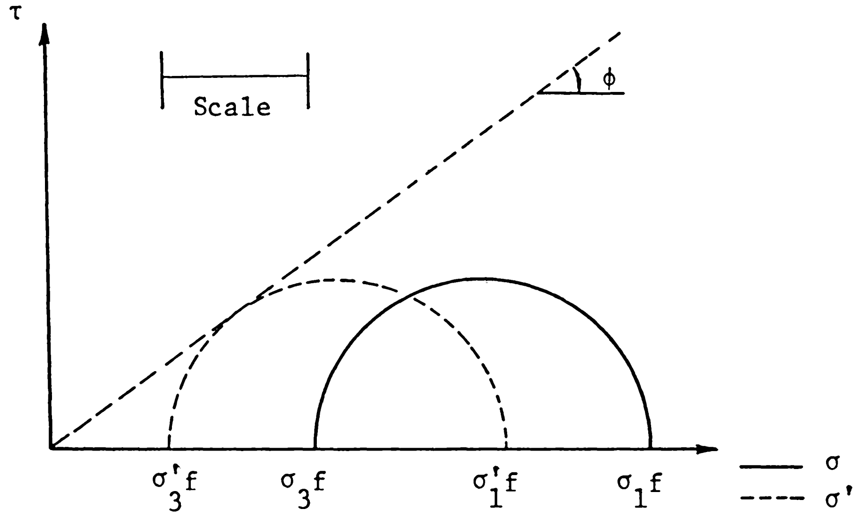


Figure 2.3. Mohr circles.

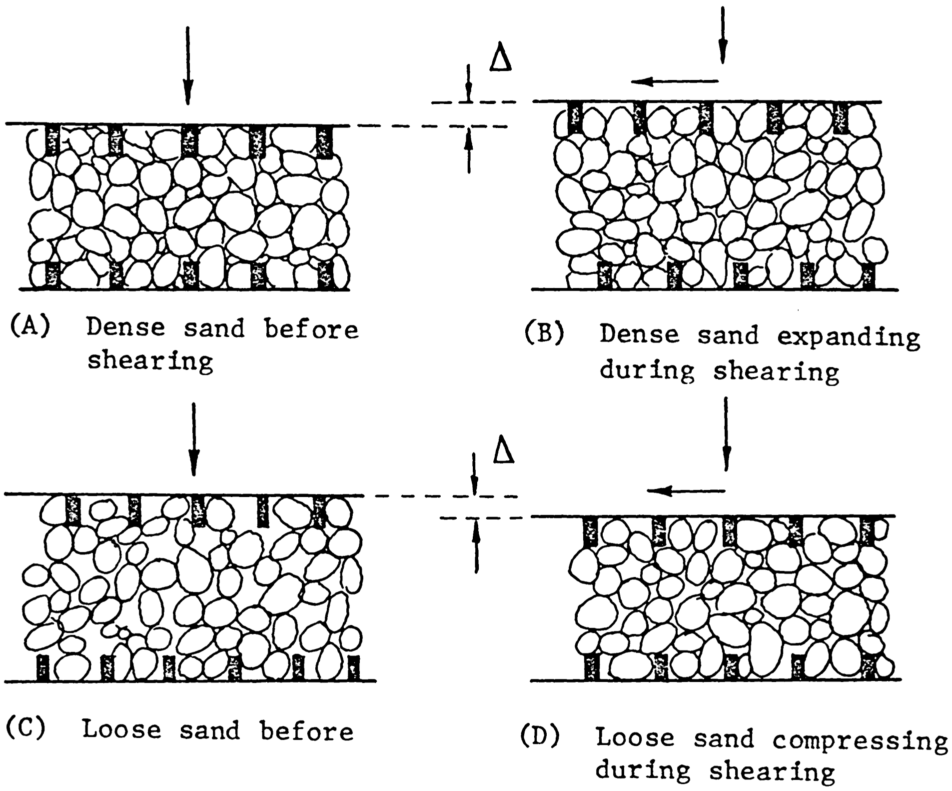


Figure 2.4. Dilatancy of sand (from Perloff and Baron, 1976).

It is only after an understanding of all of these factors, direct and indirect, that determine the soil's ability to resist shear can the Coulomb equation be fully appreciated.

$$s = \bar{c} + (\sigma - u) \tan \bar{\phi}$$

$$s = \bar{c} + \bar{\sigma} \tan \bar{\phi}$$

Where: s = shear stress on the surface at failure

$\bar{c}$ = effective cohesion

$\bar{\phi}$ = angle of effective internal friction

σ = total stress acting normal to the failure surface

u = pore water pressure acting on the failure surface

$\bar{\sigma}$ = effective stress acting normal to the failure surface.

This relationship is the key to evaluating those forces resisting shear failure. The following section will deal with the evaluation of the shear s and cohesion $\bar{c}$ using laboratory tests.

2.4 Contemporary Methods of Determining Spoil Strength

Before being concerned with various determinations of soil strength there are several important facts that should be kept in mind. There is not a complete understanding of the shear strength of soils. Data applications and shear strength behavior theories are continually being refined. Also, conditions in the field cannot be exactly modeled in the laboratory [42]. These limitations do not mean, however, that safe embankments cannot be constructed. Soil mechanics has long been sufficiently understood to allow design of embankments with full confidence in their safety, provided the designer recognizes the accuracy and applicability of this data.

There are two basic modes of testing soils for stability analysis: effective stress analysis and total stress analysis. The stress between individual soil grains is called "effective stress," and the pressure of the water surrounding the soil grains is called pore pressure. The combination of these two stresses is the total stress.

In the effective stress analysis of shear strength tests, water is allowed to drain from the sample during the test. Alternatively, the pore pressures under loading may be monitored in an attempt to evaluate actual field conditions. This method of analysis is generally considered most appropriate for evaluating long term conditions where seepage is expected [42].

When performing a total stress analysis, water is allowed to drain from the samples during testing. This method of testing is considered most appropriate for evaluating relatively short term conditions [42].

The laboratory procedures used in shear strength testing are beyond the scope of this report. It would be hard, however, to overemphasize the importance of these tests, and care should be taken to insure that they are performed by skilled technicians in a professional manner, and the results carefully evaluated.

The three standard shear strength test methods are: direct shear, triaxial compression, and unconfined compression. Applicability of these tests are given in Table 2.1 which was prepared by D'Appolonia Consulting Engineers [42].

2.4.1 The Direct Shear Test

The Direct Shear Test, shown in Figure 2.1A is the simplest and easiest method for measuring shear strength. This method is used most often for testing cohesionless soils, while cohesive soils are usually tested by the triaxial method or the unconfined method. To construct the graph shown in Figure 2.1B, a series of tests is conducted under various normal loads. The results are plotted as shown, with failure shear stress on the vertical axis and the normal stress on the horizontal axis. The line of best fit through the points is called the failure envelope. The angle of effective internal friction ϕ and the apparent cohesion $\bar{c}$ may be read from the graph. Either drained or undrained direct shear tests may be conducted.

2.4.2 The Triaxial Compression Test

In the triaxial compression test, a cylindrical soil sample inside a thin rubber membrane is placed inside a chamber filled with a liquid. The chamber is pressurized to apply three-dimensional pressure to the sample, while an axial load is applied to the top and bottom of the specimen through plates. A series of tests at various load applications are performed to establish a failure envelope. The variety of load applications that may be used to simulate field conditions is generally considered to be the triaxial compression method's strongest point [42].

TABLE 2.1

SUMMARY OF LABORATORY TESTS FOR DETERMINING SOIL SHEAR STRENGTH

Type of Test	Relative Frequency of Application[(1)]				Preparation Prior to Applying Load	Drainage Conditions During Test	Parameters Determined	Remarks
	Coarse-grained Soils[(2)]	Fine-grained Soils	Coarse Refuse	Fine Refuse				
1. <u>Direct Shear</u>:								
Drained[(3)]	1	3	1	1	Consolidated Under Normal Load	Drained	"Effective" Stress	Difficult to control rate of test to assure drained condition
Undrained[(4)]	NA	1	NA	NA	Consolidated Under Normal Load	Undrained	Approximate "Total" Stress	Difficult to conduct quickly enough to assure no drainage
2. <u>Triaxial Test</u>:								
Unconsolidated Undrained (UU)	NA	2	NA	3	Unconsolidated	Undrained	Approximate "Total"	Also called quick (Q) test
Consolidated Undrained (CU)	1	1	1	1	Consolidated Under Isotropic Pressure	Undrained	"Total" Stress and "Effective" Stress	Pore pressures are measured to give "effective" stress condition
Consolidated Drained (CD)	2	1	2	1	Consolidated Under Isotropic Pressure	Drained	"Effective" Stress	Also called slow (S) test
3. <u>Unconfined Test</u>:	NA	2	NA	NA	Unconsolidated	Undrained	Approximate "Total" Stress	Sample must have sufficient cohesion to maintain shape without support

<u>Source</u>: E. D'Appolonia Consulting Engineers, 1976

<u>Notes</u>: (1)1 = frequently used; 3 = applicable, but seldom used; NA = not applicable

(2)When coarse-grained soils are used only for the filter and drainage layers within an embankment, strength parameters are generally higher than those of the protected soil; and laboratory strength tests are not required.

(3)The maximum particle size in a direct shear test should not exceed 5% of the minimum dimension of the shear surface without distorting the results (Lewis, 1952).

(4)Maximum diameter of gravel should not exceed 15 to 20% of the specimen diameter (Holtz and Gibbs, 1956).

2.4.3 The Unconfined Compression Test

The unconfined compression test generally has little application for shear analysis. It provides an approximate measure of the undrained, short-term strength of soil for a total stress analysis.

After a review of test results obtained using the direct shear and triaxial compression method of testing soil shear strength, Huang concluded that for mine spoil: direct shear tests yield data comparable to that obtained from triaxial tests, and that the speed and ease with which the direct shear test can be carried out outweighs its disadvantages [14]. Similar conclusions were reached by D'Appolonia Consulting Engineers for mine waste [42].

2.5 Typical Spoil Characteristics

The purpose of this section is to present typical characteristics of mine spoils that are required for slope design. Those spoils presented were taken from actual embankments in eastern Kentucky, and were evaluated in a previous Master's thesis [33].

Table 2.2 shows the properties of 16 different mine spoils. Figure 2.5 shows the determination of an average shear strength for the spoils tested. This graph was developed using methods previously developed in this report. From the failure envelope $\bar{c}$ = 0.1 TSF and $\bar{\phi}$ = 30°. Examination of these data indicates that for uncompacted eastern Kentucky mine spoil these values for effective friction and effective cohesion would represent those most likely encountered in design of embankments.

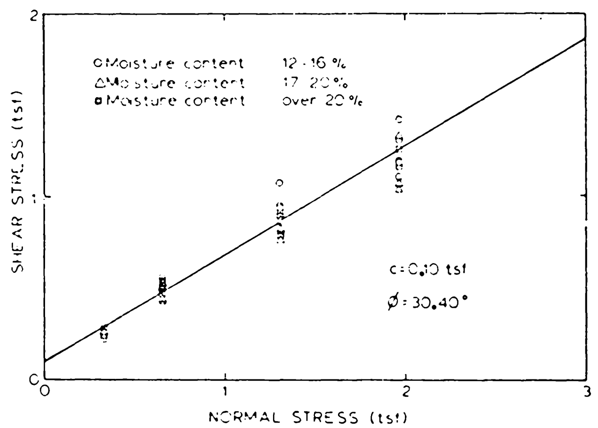


Figure 2.5. Shear strength of Eastern Kentucky mine spoil samples (from Huang, 1978).

TABLE 2.2

PROPERTIES OF MINE SPOILS FROM EASTERN KENTUCKY

Site Number	County	Liquid Limit (%)	Plasticity Index (%)	Natural Moisture Content (%)	Peak Shear Stress, tsf, at Normal Stresses of			
					0.33 (tsf)	0.65 (tsf)	1.31 (tsf)	1.96 (tsf)
1	Johnson	26	7	19.5	--	0.452	0.810	1.39
2	Floyd	31	7	15.9	--	0.408	0.898	1.282
3	Floyd	28	8	16.9	--	0.546	0.875	1.182
4	Floyd	34	9	30.9	--	0.545	0.755	0.915
5	Pike	25	5	--	--	0.570	1.020	1.480
6	Pike	35	13	14.7	--	0.470	0.795	1.045
7	Pike	31	8	16.7	--	0.500	0.890	1.330
8	Pike	--	NP	17.5	--	0.475	0.890	1.330
9	Knox	35	10	16.5	0.260	0.540	0.920	1.300
10	Knox	26	8	14.8	0.280	0.500	0.900	1.190
11	Knox	26	2	17.3	0.265	0.500	0.800	1.105
12	Knox	39	13	19.6	0.265	0.510	0.830	1.065
13	Knox	32	5	16.6	0.240	0.560	1.070	1.420
14	Bell	36	11	18.5	0.230	0.435	0.765	1.030
15	Bell	28	6	21.9	0.240	0.450	0.800	1.160
16	Bell	32	9	23.5	0.225	0.440	0.855	1.255
				Average	0.251	0.498	0.867	1.202
				Standard Deviation	0.019	0.044	0.087	0.152
				Coefficient of Variation	0.078	0.089	0.100	0.126

Source: Huang, 1978.
Note: 1 tsf = 95.8 kN/m^2

Table 2.3 shows the factor of safety that Huang calculated for the same 16 spoil banks shown in Table 2.2. He states that because the spoil banks had not failed, although their safety factor is near unity at zero pore pressure, they are stable because of their good drainage conditions [14]. One might also conclude that the method of calculating the safety factor is somewhat conservative. A later section of this report will use a statistically determined "worst condition" of these spoil characteristics to evaluate the requirements for slope stability under Public Law 95-87.

TABLE 2.3

SAFETY FACTOR OF STABLE SPOIL BANKS IN EASTERN KENTUCKY

Number	$\bar{c}$ (psf)	$\bar{\phi}$ (degree)	a (degree)	β (degree	H (feet)	Factor of Safety at Pore Pressure Ratios of 0.0	0.25	0.5
1	200	28.2	25	37	37.3	1.35	1.16	0.94
2	120	32.2	21	31.5	31.4	1.65	1.36	1.05
3	440	26.0	24	36.5	48.8	1.73	1.50	1.27
4	760	15.6	21	37	24.1	2.97	2.84	2.70
5	240	34.7	27	37	18.1	2.55	2.29	1.99
6	320	25.0	26	38.5	53.5	1.28	1.40	0.93
7	200	31.8	25	37	22.3	1.89	1.63	1.36
8	0	34.2	26.5	37	36.1	0.93	0.68	0.45
9	120	33.3	14	34	53.1	1.26	1.00	0.73
10	200	29.9	18	36.5	39.2	1.40	1.16	0.92
11	240	27.0	24	35.5	48.8	1.30	1.12	0.91
12	280	26.4	21	41	75.4	0.94	0.78	0.59
13	80	36.6	20	35	54.4	1.28	1.01	0.73
14	160	27.0	12	32	53.0	1.11	0.91	0.69
15	200	28.3	28	39	40.2	1.28	1.10	0.90
16	40	32.3	27	39	27.7	1.01	0.81	0.60

Source: From Huang, 1978. Note: 1 foot = 0.305 meter and 1 psf = 47.9 N/m^2.

Chapter 3

METHODS OF STABILITY ANALYSIS

Slopes created as a result of surface mining must be considered to have two possible modes of failure [14]. First, those failures are occurring along a plane surface. This could be the ground surface at the bottom of the fill, or some other line created by the placement of one dissimilar material above another. In the second mode, cylindrical failure, the line of slippage can pass through layers of homogenous or inhomogenous material, rather than along them. As early as the middle of the nineteenth century, the French engineer Collin (1846) recognized from observation of slope stability failures in the field that the failure surface was curved rather than plane [27]. Generally, most methods for stability analysis used involve the application of the theory of limit equilibrium, and the use of statics.

3.1 Failure Criteria

Usually, for plane failure there will be only one calculation necessary since this type of failure will occur along some known line. However, for cylindrical failure, the line of failure is not known, and must be determined by a method of trial and error.

The literature reviewed indicates that the simplified Bishop method is currently the most used method of cylindrical failure analysis [4,14,27]. Although computer programs are available for determining slope stability, the use of these programs requires considerable experience and engineering judgment and may become quite expensive if applied improperly [19]. Based on this conclusion, Huang has proposed a method of analysis for cylindrical and plane failure based largely on the use of charts and tables that have been developed [14]. The most practical procedure for stability analysis is to assume a critical circular surface, unless definite knowledge exists of critically oriented weak bands [4]. An introduction to circular failure is given in the following section.

3.1.1 Cylindrical Failure

The method of slices assumes that the failure surface may be defined as a cylinder whose axis is oriented parallel to the strike of the slope [27].

The effect of slope steepness and height on stability are shown in Figure 3.1 [40]. Slopes A and C are the same height and the failure arcs contain approximately the same weight of material. The two important factors illustrated are: for the greater slope, the length of the failure surface is shorter; also the steeper angle of the failure surface reduces the normal component of W, and therefore reduces the resisting force caused by friction. While the first factor reduces the resisting force, the second increases the disturbing force, which has the result of reducing the safety factor. In Figure 3.1 slopes B and D also show the disadvantages of steep slopes. The addition of weight (W_2) is located farthest from the center of rotation and therefore adds greatly to the disturbing forces, while the additional length of failure plane

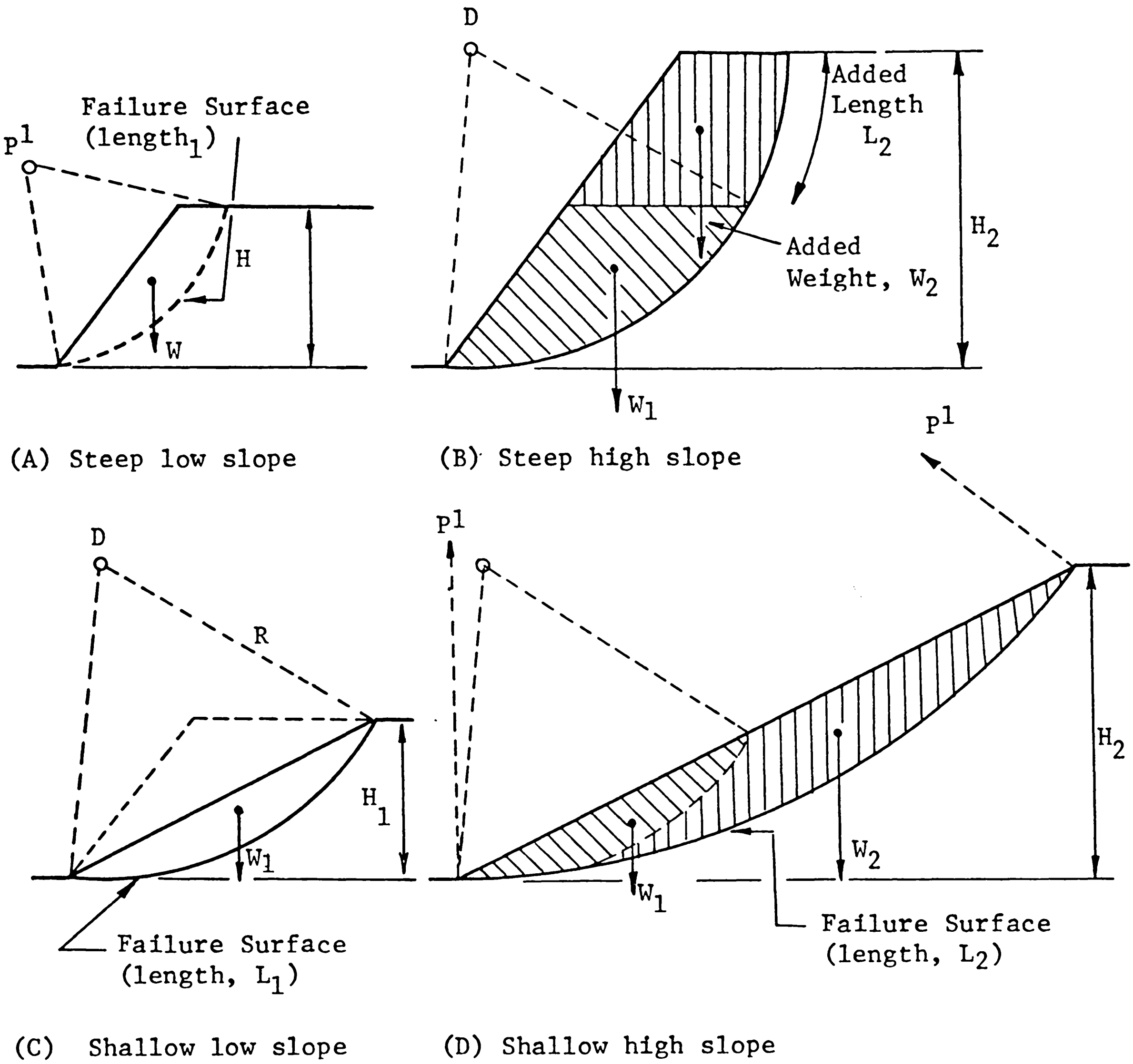


Figure 3.1. Effect of slope steepness and height on stability (from E. D'Appolonia, 1976).

adds little to the resisting forces. Here again the steep slope results in a smaller factor of safety, as compared to the lesser slope.

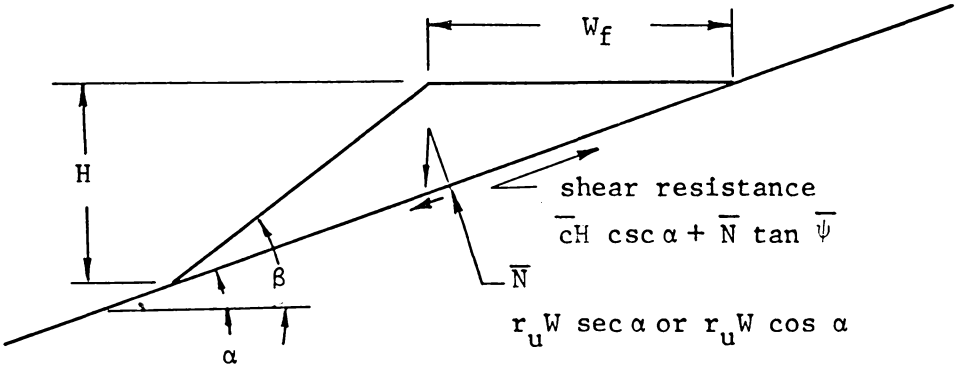
3.1.2 Plane Failure

A typical section of a spoil bank is shown in Figure 3.2. This represents the most simple soil embankment that would be encountered in mining operations. The calculation of its factor of safety for shear failure is relatively simple. Examination of Figure 3.2 reveals that the situation is hardly more than a simple physics problem. A mass is situated on an inclined plane, and is subject to the common forces of gravity and friction. Development of the mathematical formulas necessary for calculation of the factor of safety are given in the following discussion.

This analysis of plane failure is based on recent work by Huang [14], utilizing Figure 3.2 for the resolution of those forces acting on a spoil bank.

The force resisting failure is composed of two parts: one due to cohesion, $\bar{c}H \csc \alpha$, and one due to friction, $\bar{N} \tan \bar{\Psi}$. If there is no seepage, $N = W \cos \alpha$. If seepage exists, $\bar{N} = W \cos \alpha - r_u \sec \alpha$, where r_u is the ratio between the pore pressure along the failure plane and the overburden pressure. Since the unit weight of mine spoil is roughly twice that of water Huang lets r_u vary between 0.5 for complete saturation to 0 for no seepage. The following equation is used to calculate the effective normal force:

$$\bar{N} = (1 - r_u) \cos \alpha.$$



W = total weight of the fill

$\bar{N}$ = effective force normal to the failure plane

$\bar{c}$ = effective cohesion of soil

H = height

α = degree of natural slope

β = degree of outslope

Figure 3.2. Forces acting on a spoil bank (from Huang, 1978).

Based on the above, the following equation for the factor of safety can be written:

$$F = \frac{\bar{c}H \csc \alpha + (1 - r_u) W \cos \alpha \tan \bar{\psi}}{W \sin \alpha}$$

If the full width, W_f is known the following equation can be written:

$$F = \frac{\bar{c}W_f \sin \beta/\sin(\beta - \alpha) + (1 - r_u) W \cos \alpha \tan \bar{\psi}}{W \sin \alpha}$$

in which

$$W = 1/2\ W_f^2 \sin \alpha \sin \beta/\sin (\beta - \alpha).$$

Therefore

$$F = 2 \csc^2 \alpha \left(\frac{\bar{c}}{\gamma W_f}\right) + (1 - r_u) \cot \alpha \tan \bar{\psi} \qquad (3.1)$$

Sample calculations using Equation (3.1) will be given in a following chapter.

3.2 Legal Requirements for Slope Stability

The key factors in determining the legal requirements under Public Law 95-87 [10] for slope stability are given below.

816.71. Disposal of Excess Spoil: General Requirements.

(i) where the slope in the disposal area exceeds 1V:2.8H (36 percent), or such lesser slope as may be designated by the regulatory authority based on local conditions, keyway cuts (excavated to stable bedrock), or rock toe buttresses shall be constructed to stabilize the fill. The slope of original ground at the toe of the fill shall not exceed 1V:5H (20 percent).

816.72. Disposal of Excess Spoil: Valley Fills.

(a) The fill shall be designed to attain a long-term static factor of safety of 1.5 based upon data obtained from subsurface exploration, geotechnical testing, foundation design, and accepted engineering analyses. (g) The outslope of the fill shall not exceed 1V:2H (50 percent). The regulatory authority may require a flatter slope.

Apparently in an attempt to define what is meant in Section 816.72(a) by "accepted engineering analyses," the Federal Register [10] gives a list of references for each section of the law. Under Section 816.71-1816.73 Disposal of excess spoil, a total of 54 literature sources are given. Without the resources of the federal government the acquisition and assimilation of such a volume of information is beyond the bounds of this report. However, this does not preclude a review of a number of those references listed so as to obtain a representative cross section of the available information on this subject. This report presents the results of such an effort.

After making a review of the literature, there are some basic conclusions that may be reached so far as determination of slope stability. (1) Although there are many procedures for determining slope stability, all are based on the same fundamental principles. (2) Examination of past slide failures indicates that failure to apply basic design principles, and to realistically model field conditions has been the cause of failure, rather than utilization of one design method over another which might change the safety factor by 5 or 10 percent [27].

With a required safety factor of 1.5 there could be, barring acts of nature, little chance of slope failure, if the true factor of safety was calculated using any accepted method that is based on sound engineering principles. There would seem to be little justification for concerning oneself with differential equations and computer programs, if more mundane methodologies would allow sufficient accuracy for a safe design. In practice, certain things must be accomplished in given periods of time. It should follow then, that the easiest and quickest method of acceptable slope analysis should be employed. This enables the engineer to have more time to evaluate pertinent aspects of design other than through calculations. With this in mind, the following sections develop and present what acknowledged experts in the field of slope stability believe to be a state of the art for the practical design of stable slope embankments.

3.3 Bishop Method of Slices

The following development of the Bishop simplified method of slices is based on work by Perloff and Baron [27]. Using Figure 3.3, the relationship between the forces causing and resisting failure is as follows:

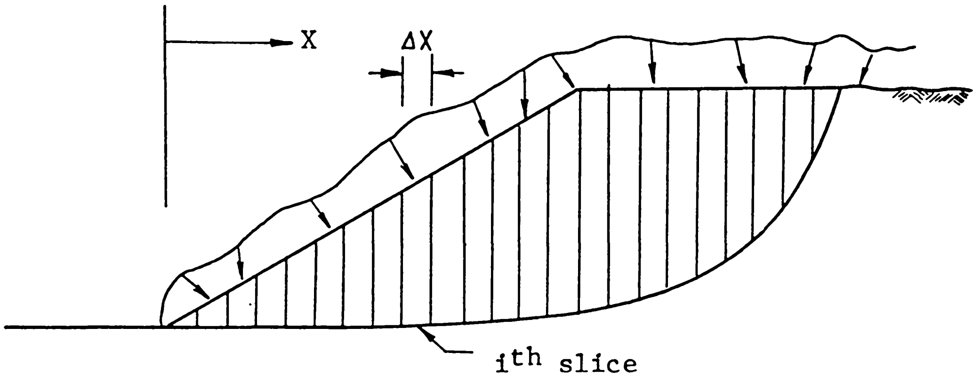
$$S_i = \frac{1}{FS} (c_i \; \Delta X_i \sec \theta + N_i \tan \phi_i) \qquad (3.2)$$

where,

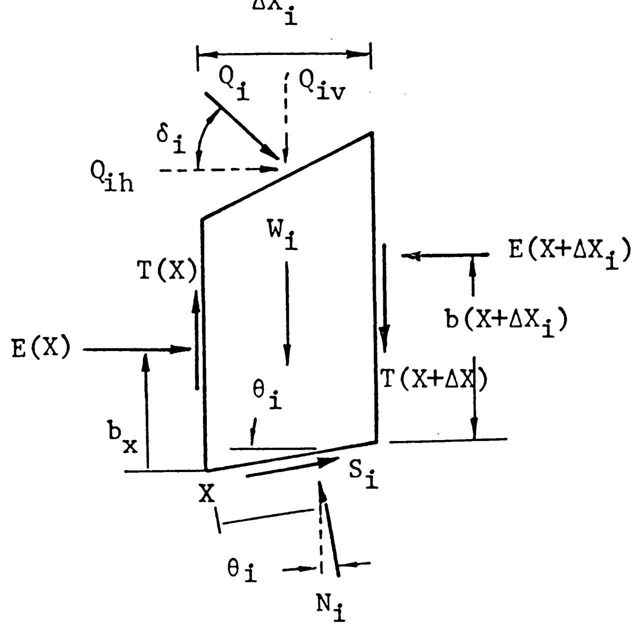
FS = factor of safety

c_i = cohesive component of strength

$\Delta X_i \sec \theta_i$ = length of surface



3.3 (A) Sliding mass.



3.3 (B) Thin slice of sliding mass (from Perloff and Baron, 1976).

$N_i \tan \phi_i$ = normal force multiplied by internal angle of fric- friction.

If the failure surface is assumed to be an arc, N_i can be eliminated by taking moments about the center of the assumed circle:

$$W_i R \sin \theta_i - S_i R + Q_{iv} R \sin \theta_i - Q_{ih} d_i + E_{x+\Delta x} (R \cos\theta_i - b_{x+\Delta X}) - E_x (R \cos \theta_i - b_x) + (T_{x+\Delta x} - T_x) R \sin \theta_i = \theta \qquad (3.3)$$

where

d_i = moment arm of Q_{ih}.

Substituting Equation (3.2) into Equation (3.3) and dividing by R, the forces on the element can be expressed in terms of the factor of safety:

$$W \sin \theta_i - \frac{1}{FS} (c_i \Delta x_i \sec \theta_i + N_i \tan \phi_i) + Q_{iv} \sin \theta_i - Q_{ih} \frac{d_i}{R} + E_{x+\Delta x} (\cos \theta_i - \frac{b_x + \Delta_x}{R}) - E_x (\cos \theta_i - \frac{b_x}{R}) + (T_{x+\Delta x} - T_x) \sin \theta_i = 0 \qquad (3.4)$$

Equation (3.4) can now be summed over n slices. For a sufficiently small slice, the forces on the sides of the slice, E and T, will be cancelled since they oppose each other. This will give the factor of safety as follows:

$$FS = \frac{\sum_{i=1}^{n} (c_i \Delta x_i \sec \theta_i + N_i \tan \phi_i)}{\sum_{i=1}^{n} [(W_i + Q_{iv}) \sin \theta_i - Q_{ih} \frac{di}{R}]} \qquad (3.5)$$

To determine N_i, the forces acting in the vertical direction are summed, giving:

$$\Sigma F_v = 0$$

$$W_i + Q_{iv} - N_i \cos\theta_i - S_i \sin\theta_i + T_{x\Delta x_i} - T_x = 0 \tag{3.6}$$

Substituting Equation (3.2) into Equation (3.6) and rearranging terms will give the normal force:

$$N_i = \frac{W_i + Q_{iv} - \dfrac{c_i \Delta x_i \tan\theta_i}{FS} + T_x - T_{x+\Delta x_i}}{\cos\theta + \dfrac{\sin\theta_i \tan\phi_i}{FS}} \tag{3.7}$$

By substituting Equation (3.7) into Equation (3.5), N_i can be eliminated. After simplification this gives:

$$FS = \frac{\Sigma\left[\dfrac{c_i\Delta x_i + [(W_i + Q_{iv}) + (T_x - T_{x+\Delta x_i})] \tan\phi}{\cos\theta_i + \dfrac{\sin\theta_i \tan\phi_i}{FS}}\right]}{\Sigma[(W_i + Q_{iv}) \sin\theta_i - Q_{ih}\dfrac{di}{R}]} \tag{3.8}$$

For equilibrium the following assumptions can be made:

$$\Sigma(T_x - T_{x+\Delta x}) = 0 \quad \text{and} \quad \Sigma\left[\frac{(T_x - T_{x+\Delta x}) \tan\phi_i}{\cos\theta_i + \dfrac{\sin\theta_i \tan\phi_i}{FS}}\right] = 0$$

This gives the following:

$$FS = \frac{\Sigma\left[\dfrac{c_i\Delta x_i + (W_i + Q_{iv}) \tan\phi_i}{\cos\theta_i + \dfrac{\sin\theta_i \tan\phi_i}{FS}}\right]}{\Sigma[(W_i + Q_{iv}) \sin\theta_i - Q_{ih}\dfrac{di}{R}]} \tag{3.9}$$

In addition to those forces shown in Figure 3.3, the effect of pore water pressure must be considered. Incorporating this factor into Equation (3.2) gives:

$$S_i = \frac{1}{FS}\left[c_i' \Delta X_i \sec\theta_i + (N_i - u_i \Delta X_i \sec\theta_i)\tan\phi_i'\right] \tag{3.10}$$

where,

c_i' = cohesive component of strength in terms of effective stress

ϕ_i' = apparent angle of shearing resistance in terms of effective stresses

u_i = pore water pressure at the base of the slice.

Following a procedure analogous to the development of Equation (3.9), the factor of safety can be expressed in terms of effective stresses as follows:

$$FS = \frac{\Sigma\left[\dfrac{c_i'\Delta X_i + (W_i + Q_{iv} - u_i\Delta X_i)\tan\phi_i'}{\cos\theta_i + \dfrac{\sin\theta_i\tan\phi_i'}{FS}}\right]}{\Sigma\left[(W_i + Q_{iv})\sin\theta_i - Q_{ih}\dfrac{di}{R}\right]} \tag{3.11}$$

Several authors have presented a graph (Fig. 3.4) for determining the value of

$$\cos\theta_i + \frac{\sin\theta_i \tan\phi_i}{FS},$$

Its use can result in fewer laborious calculations when using the method of slices for stability analysis.

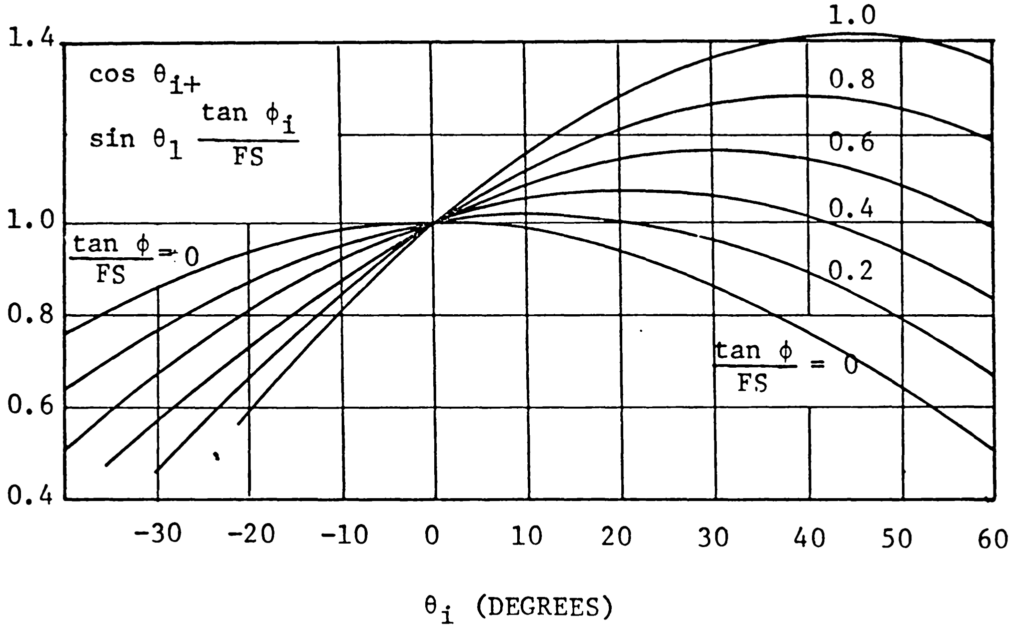


Figure 3.4. (From Perloff and Baron, 1976.)

Correction factors for the Bishop simplified method of slices.

3.3.1 Application

The following is a sample calculation for the simplified Bishop method of slices. Figure 3.5 shows a typical section to be analyzed. The necessary calculations are shown in Table 3.1. This analysis is taken from Perloff and Baron [27].

3.4 Huang's Method of Stability Analysis

The most recent and comprehensive investigation of stability of spoil banks and valley fills has been done by Dr. Huang [14]. By the use of computer programs he has developed a series of design charts that may be used for determination of factors of safety for the various modes of failure associated with mine spoil. This report will show the development of equations used in these programs as well as those for the required additional calculations. It is felt that the purpose of this report will be best served by this development without the generation of voluminous data employing these programs. These programs are available in reference 14.

3.4.1 Plane Failure for Hollow Fills

The determination of the factor of safety for a hollow fill is considerably more complicated than for the simple spoil bank. Examination of Figure 3.6 shows that the line along which shear failure can occur must be considered as two independent surfaces. Although an accurate section of field conditions would show more than two lines, it has been concluded that sufficiently accurate analysis can be made using two lines [14]. Figure 3.6 also shows that the mass above each

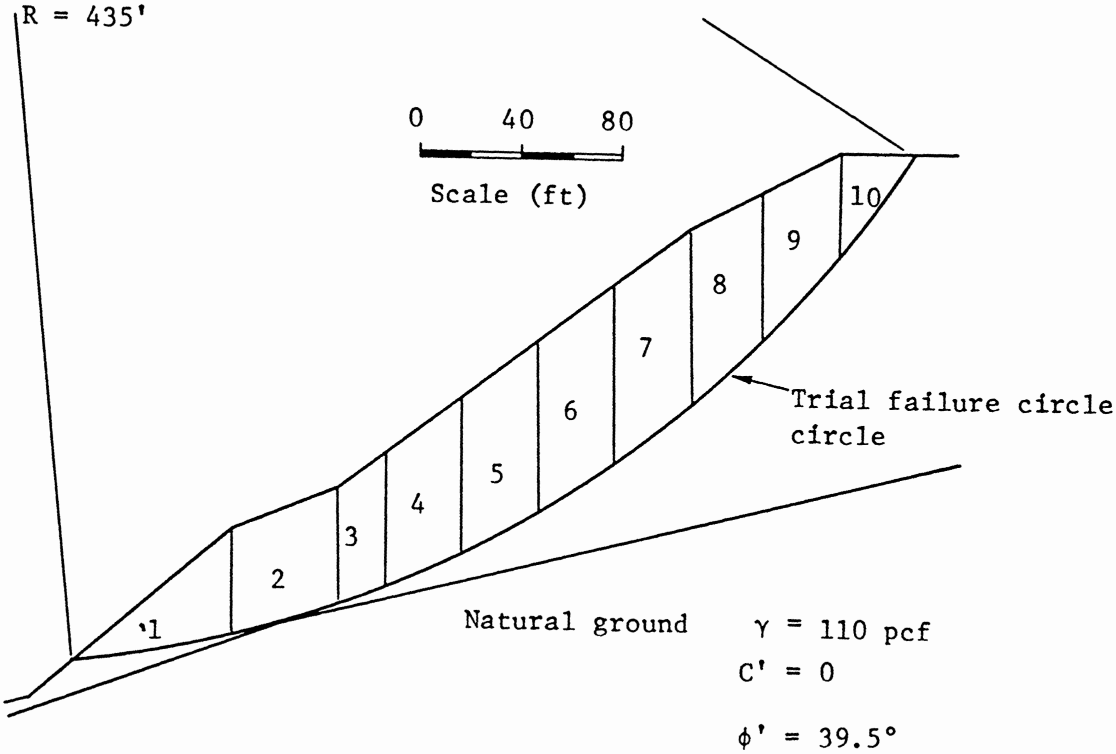


Figure 3.5. (From Perloff and Baron, 1976.)

Figure for calculations shown for trial failure circle in table 3.1.

TABLE 3.1

CALCULATIONS FOR TRIAL FAILURE CIRCLE

Slice No. (A)	Δx (ft) (B)	Δz (ft) (C)	W (kips) (D)	W tan φ (kips) (E)	θ (deg) (F)	sin θ (G)	W sin (kips) (H)	Trial FS = 1.30		Trial FS = 1.40		Trial FS = 1.50	
								$(I_1)^a$	E/I_1 (kips) J_1	$(I_2)^a$	E/I_2 (kips) J_2	$(I_3)^a$	E/I_3 (kips) J_3
1	62	21	143	118	9	0.156	22	1.09	108	1.08	109	1.07	110
2	42	44	203	167	16	0.276	56	1.14	147	1.12	149	1.11	150
3	18	50	99	82	21	0.358	35	1.16	70	1.14	72	1.13	73
4	30	59	195	161	24	0.406	79	1.17	137	1.15	140	1.14	142
5	30	66	218	180	28	0.469	102	1.18	152	1.16	155	1.14	158
6	30	71	234	193	33	0.545	128	1.18	163	1.16	166	1.14	170
7	30	72	238	196	38	0.616	147	1.18	166	1.15	170	1.13	174
8	28	66	203	167	43	0.682	138	1.16	144	1.13	147	1.11	151
9	30	52	172	141	48	0.743	128	1.14	124	1.11	127	1.08	131
10	30	21	69	57	55	0.819	57	1.09	52	1.06	54	1.02	56
							Σ = 892		Σ = 1263		Σ = 1289		Σ = 1315
									$FS_1 = \frac{\Sigma(J_1)}{\Sigma(H)}$		$FS_2 = \frac{\Sigma(J_2)}{\Sigma(H)}$		$FS_3 = \frac{\Sigma(J_3)}{\Sigma(H)}$
									= 1.42		= 1.44		= 1.47

Source: Perloff and Baron, 1976.

[a] $\cos\theta + \sin\theta \frac{\tan\phi'}{FS}$ obtained from Figure 3.4.

γ = 110 pcf; c' = 0: ϕ' = 39.5°.

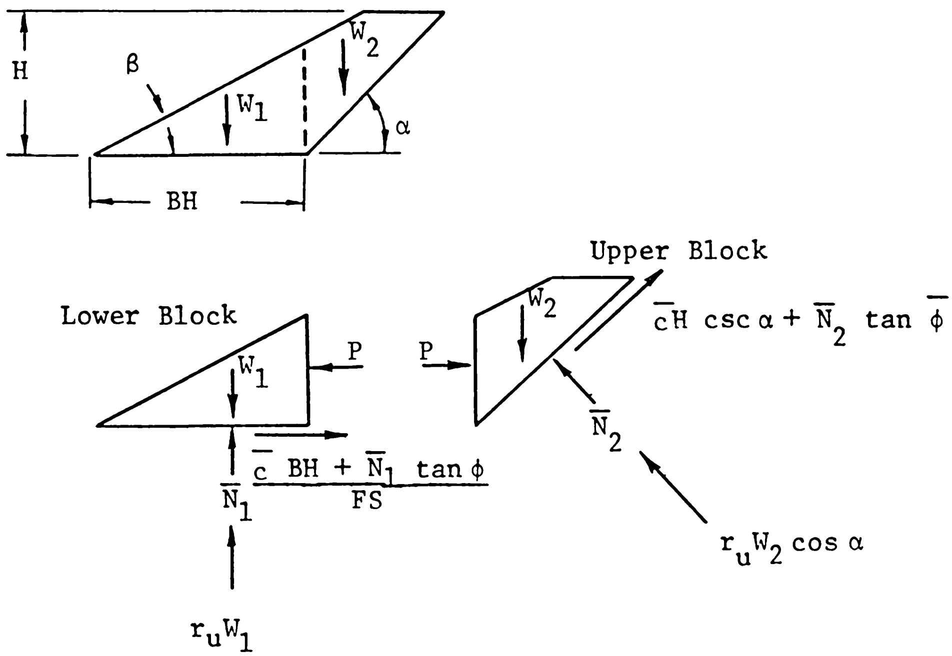


Figure 3.6. Forces acting on a hollow fill (from Huang, 1976).

potential failure plane must be considered as an independent body. This results in considerable calculation, since the problem requires a simultaneous solution. The required mathematical formulas for determining the factor of safety for hollow fills have been presented by Huang and are given in the following pages.

Figure 3.6 shows the forces acting on a hollow fill. For the lower block the following equations may be obtained:

$$\bar{N}_1 = (1 - r_u)W_1$$

$$P = (\bar{c}\ BH + \bar{N}_1 \tan \bar{\psi})/F$$

where

W_1 = weight of lower block

B = ratio between the base width and height

From the two equations the following equation can be written:

$$P = [\bar{c}\,BH + (1 - r_u)W_1 \tan \phi]/F$$

For the upper block the following equations may be written:

$$P = [\bar{c}\ BH + (1 - r_u)W_1 \tan \bar{\psi}]/F \qquad (3.12)$$

$$\bar{N}_2 = P \sin \alpha + (1 - r_u)W_2 \cos \alpha \qquad (3.13)$$

$$W_2 \sin \alpha = P \cos \alpha + (\bar{c}\,H \csc \alpha + \bar{N}_2 \tan \bar{\psi})F \qquad (3.14)$$

A solution of Equations (3.12), (3.13), and (3.14) as a quadratic equation obtains the factor of safety.

$$aF^2 + bF + c = 0$$

where,

$$a = d \sin \alpha$$

$$b = -\left[\frac{\bar{c}}{\gamma H}(B \cos \alpha + \csc \alpha) + (1 - r_u)(d + e) \cos \alpha \tan \bar{\psi}\right]$$

$$c = -B \sin \alpha \tan \bar{\psi} \left[\frac{\bar{c}}{\gamma H} + (1 - r_u) \frac{e}{B} \tan \bar{\psi}\right]$$

$$d = \frac{W_2}{\gamma H^2} = 1/2 \left[\cot \alpha - (1 - B \tan \beta)^2 \cot \beta\right]$$

$$e = \frac{W_1}{\gamma H^2} = 1/2\ B^2 \tan \beta$$

3.4.2 Cylindrical Failure

Figure 3.7 can be used to show the basic factors involved in determining the factor of safety for slope stability.

Ignoring seepage,

$$F = \frac{\sum_{i=1}^{n}(\bar{c}\, b_i \sec \theta_i + \gamma b_1 h_1 \cos \theta_i \tan \bar{\psi})}{\sum_{i=1}^{n} \gamma b_1 h_1 \sin \theta.}$$

A more complex situation involving two soil types is developed as follows: Figure 3.8 can be used in the determination of those factors considered in the analysis of cylindrical failure. The resisting moment, ignoring pore pressure and assuming $\gamma_1 = \gamma_2 = \gamma$, can be expressed as

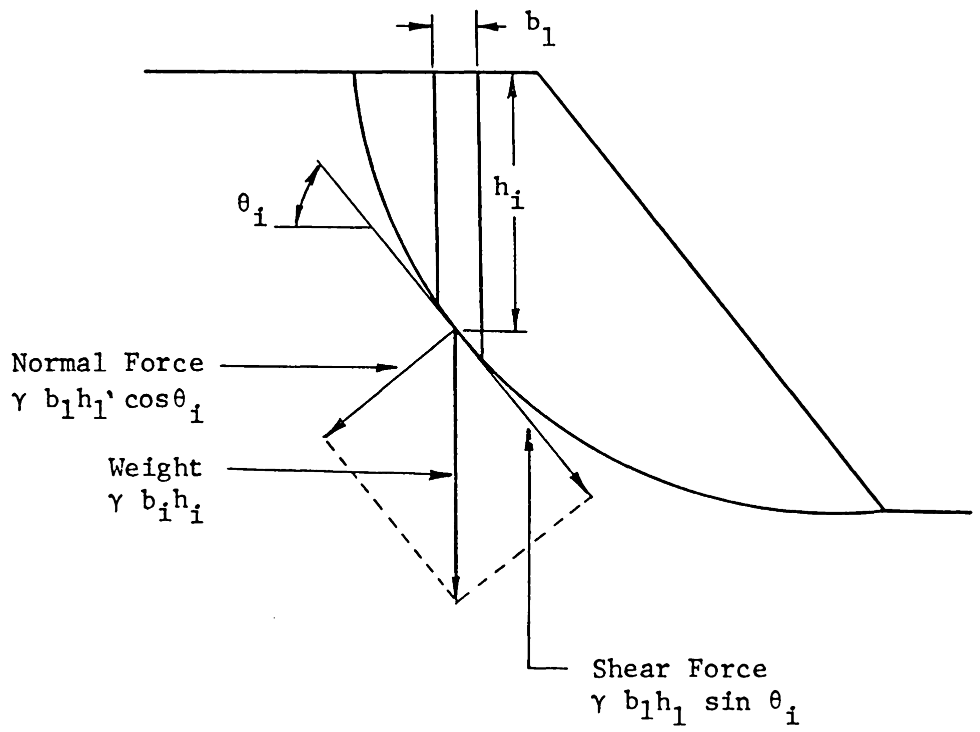


Figure 3.7. Stability analysis by the method of slices (from Huang, 1978).

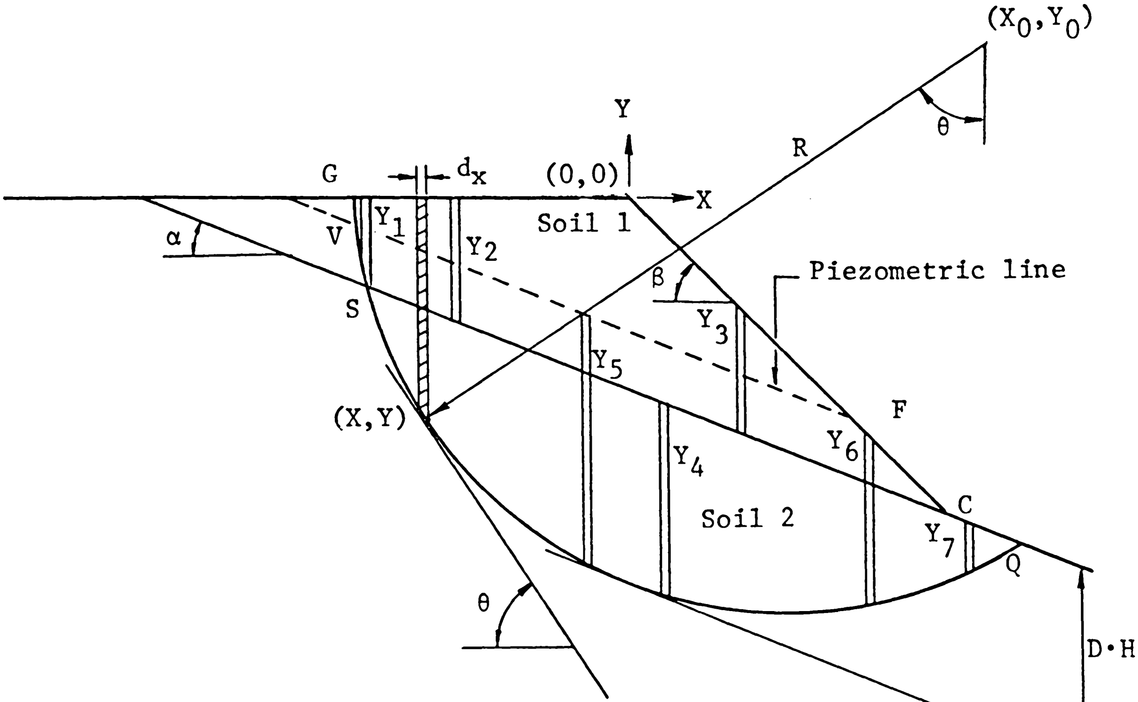


Figure 3.8. Determining the factor of safety by the direct integration method (from Huang, 1978).

$$M_r = R[\bar{c}_1 GS + \bar{c}_2 SQ + (\tan\bar{\psi})\gamma \int_{x_G}^{x_S} Y_1 \cos\theta\, dx +$$

$$(\tan\bar{\psi}_2)\ \gamma\ (\int_{x_S}^{o} Y_2 \cos\theta\, dx + \int_{o}^{x_c} Y_3 \cos\theta\, dx +$$

$$\int_{x_S}^{x_Q} Y_4 \cos\theta\, dx)].$$

The overturning moment can be expressed as follows:

$$M_o = R \cdot GQ \cdot (\gamma H/N_s),$$

where,

$$N_s = R \cdot GQ \cdot \gamma H/M_o, \text{ and}$$

$$c_d = \gamma H/N_s = \text{developed cohesion}$$

The factor of safety can now be expressed as,

$$F = \frac{M_r}{M_o} = N_s\ [(\frac{\bar{c}_1}{\gamma H})L_1 + (\frac{\bar{c}_2}{\gamma H})(1 - L_1) +$$

$$(\tan\bar{\psi}_1)F_1 + (\tan\ \bar{\psi})F_2].$$

where

L_1 = length factor = GS/GQ and

F_1, F_2 = friction factor for soils, or

$$F_1 = \frac{1}{H \cdot GQ} \int_{x_G}^{x_S} Y_1 \cos\theta\, dx$$

$$F_2 = \frac{1}{H \cdot CQ} \left(\int_{x_s}^{o} Y_2 \cos\theta \, dx + \int_{o}^{x_c} Y_3 \cos\Theta \, dx + \int_{x_s}^{x_Q} Y_4 \cos\theta \, dx \right).$$

With the consideration of pore pressure the equation becomes

$$F = N_s \left[\left(\frac{\bar{c}_1}{\gamma H}\right) L_1 + \left(\frac{\bar{c}_2}{\gamma H}\right)(1 - L_1) + (1 - r_{u1}) (\tan\bar{\psi}) F_1 + (1 - r_{u2})(\tan\bar{\psi}) F_2 \right]. \quad (3.15)$$

If only case 1, where ψ is considered, Equation (3.15) can be simplified to

$$F = N_s \left[\frac{\bar{c}}{\gamma H} + \frac{(1 - r_u)\tan\bar{\psi}}{N_f} \right], \quad (3.16)$$

where N_f = friction number, which is the reciprocal of the friction factor, F_1. Because of this simplification, a correction factor must be applied to factors of safety calculated using Equation (3.16). For this Huang provides a table that apparently has an empirical basis. The use of this table (Figure 3.9) requires the calculation of the percent of cohesion resistance, Pc, and a subsequent correction (Fig. 3.10).

$$P_c = \frac{\frac{\bar{c}}{\gamma H}}{\frac{\bar{c}}{\gamma H} + (1 - r_u)\frac{\tan\bar{\psi}}{N_f}}$$

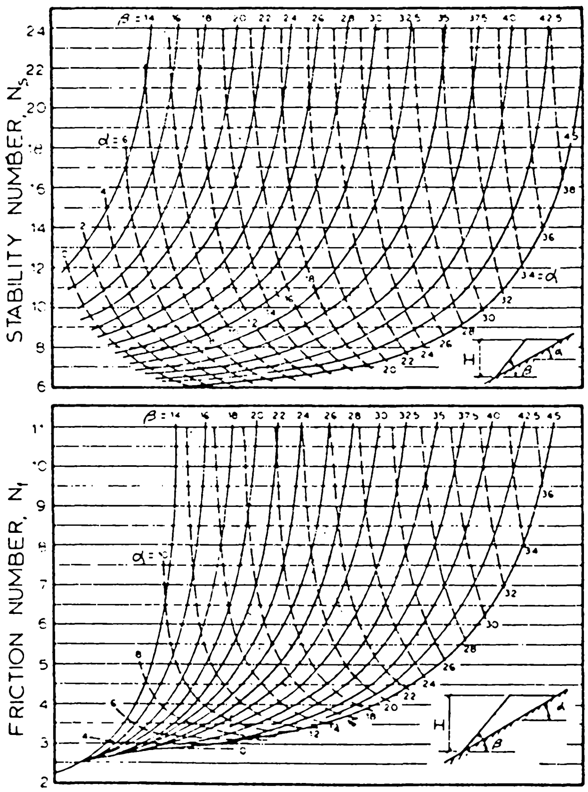


Figure 3.9. Stability chart for various combinations of α and β in a spoil bank (from Huang, 1978).

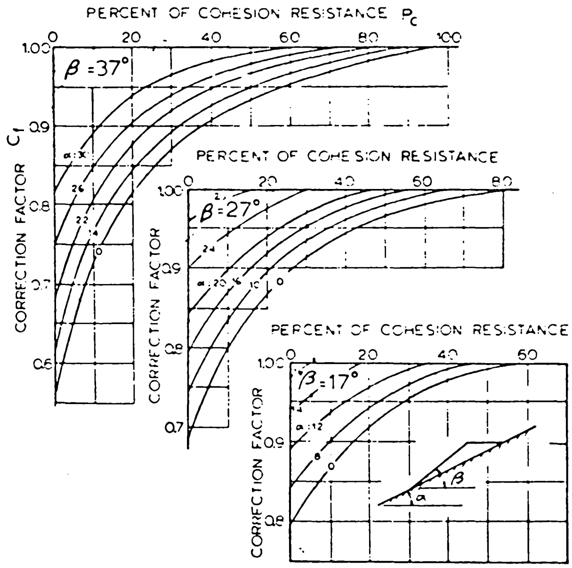


Figure 3.10. Correction factor for the factor of safety computed by Equation 3.17 (after Huang, 1978).

3.4.2.1 Application

The following example has been presented by Huang to illustrate the determination of safety factors for cylindrical failure:

$$F = N_s \left[\frac{\bar{c}}{\gamma H} + \frac{(1 - r_u)\tan\bar{\psi}}{N_f}\right] \tag{3.17}$$

$$P_c = \frac{\dfrac{\bar{c}}{\gamma H}}{\dfrac{\bar{c}}{\gamma H}(1 - r_u)\dfrac{\tan\bar{\psi}}{N_f}} \tag{3.18}$$

Given $\beta = 20°$, H = 200 feet (61.0 m), $\bar{c}$ = 200 psf (9.6 kN/m^2), $\bar{\psi} = 30°$, γ = 125 pcf (19.6 kN/m^3), and $r_u = 0.1$, determine the factor of safety for a = 0°, 4°, 8°, and 12°, respectively.

Solution: When $\beta = 20°$ and $\alpha = 0°$, from Fig. 3.9, $N_s = 9.5$ and $N_f = 2.6$. From Equation (3.17), F = 9.5[200/(125 x 200) + (1 - 0.1) tan $\bar{\psi}$/2.61] = 9.5(0.008 + 0.200) = 1.976. From Equation (3.18), P_c = 0.008/(0.008 + 0.200) = 3.8%. From Fig. 3.10, C_f = 0.80, so F = 0.80 x 1.976 = 1.581. By the same procedure, it will be found that the factors of safety for $\alpha = 4°$, 8°, and 12° are 1.562, 1.554, and 1.486, respectively.

The stability charts shown in Figures 3.11 and 3.12 were developed by Huang using the ICES-LEASE computer program. They are used for the determination of N_s and N_f. These, in turn, are used for the calculation of the factor of safety for cylindrical failure for hollow fills. This procedure is the same as in the previous example for spoil banks.

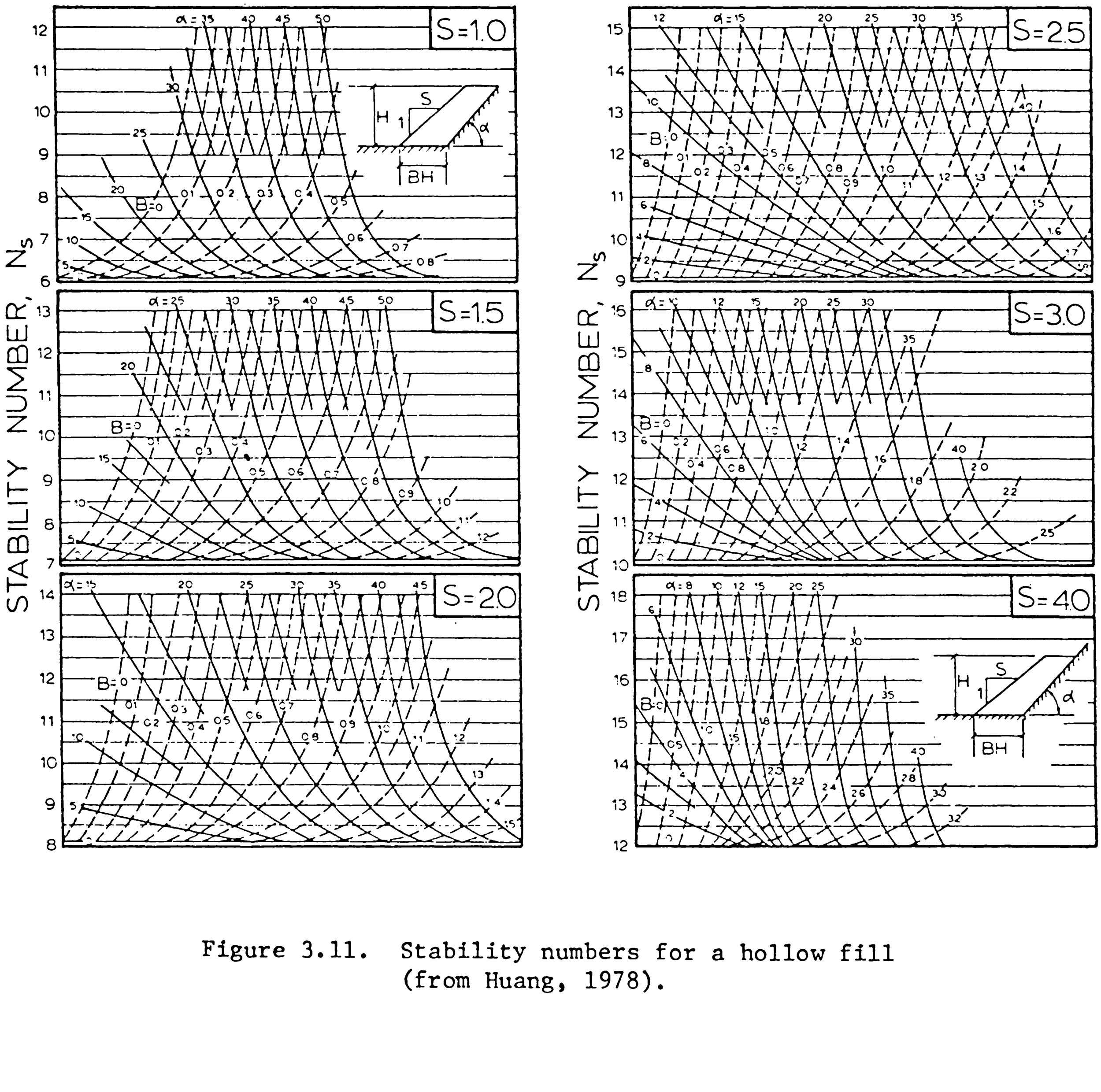


Figure 3.11. Stability numbers for a hollow fill
(from Huang, 1978).

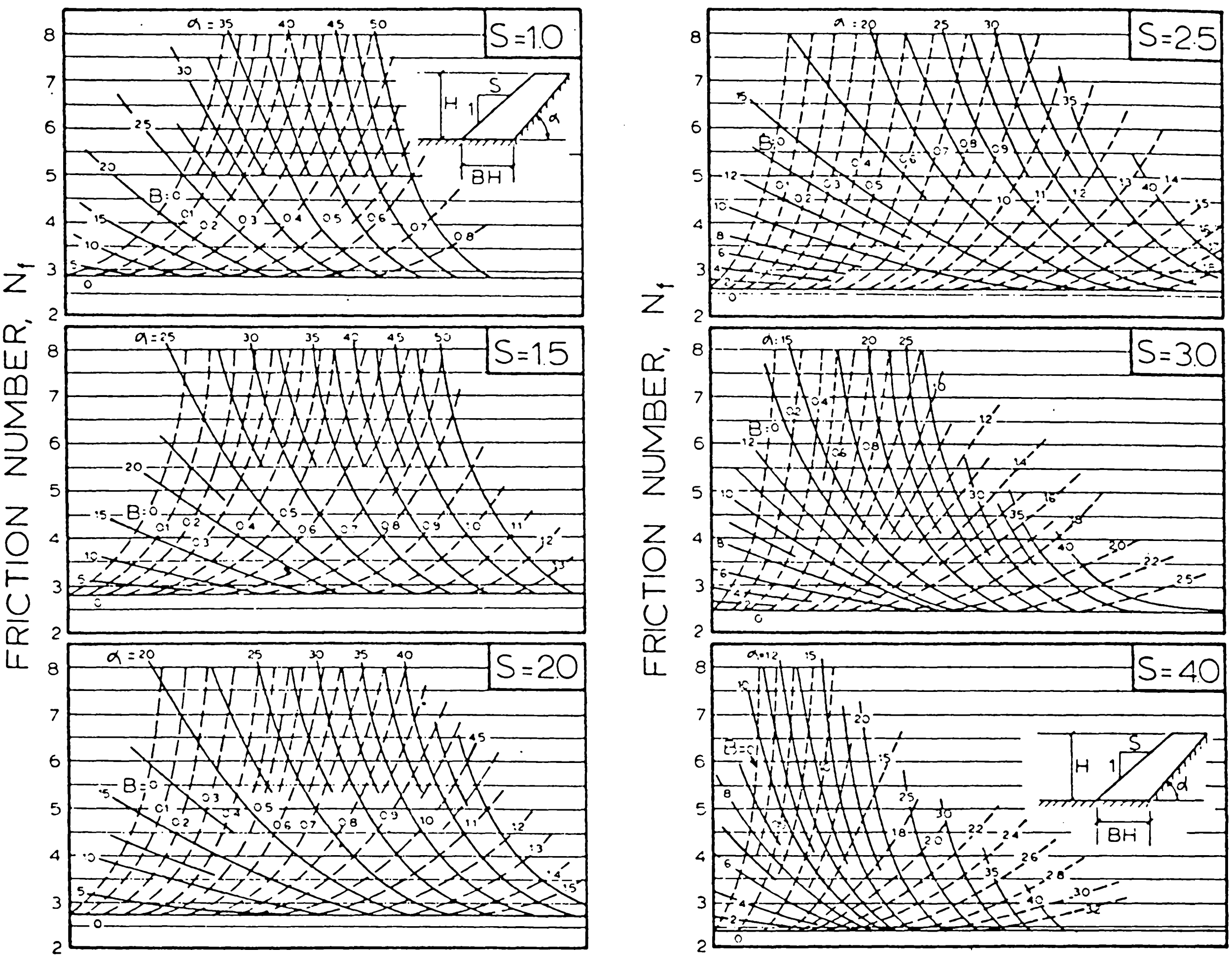


Figure 3.12. Friction number for a hollow fill
(after Huang, 1978).

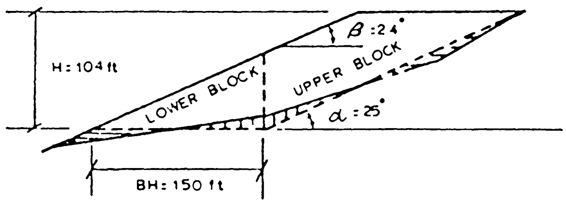


Figure 3.13. Approximation of a natural slope by two straight lines (after Huang, 1978).

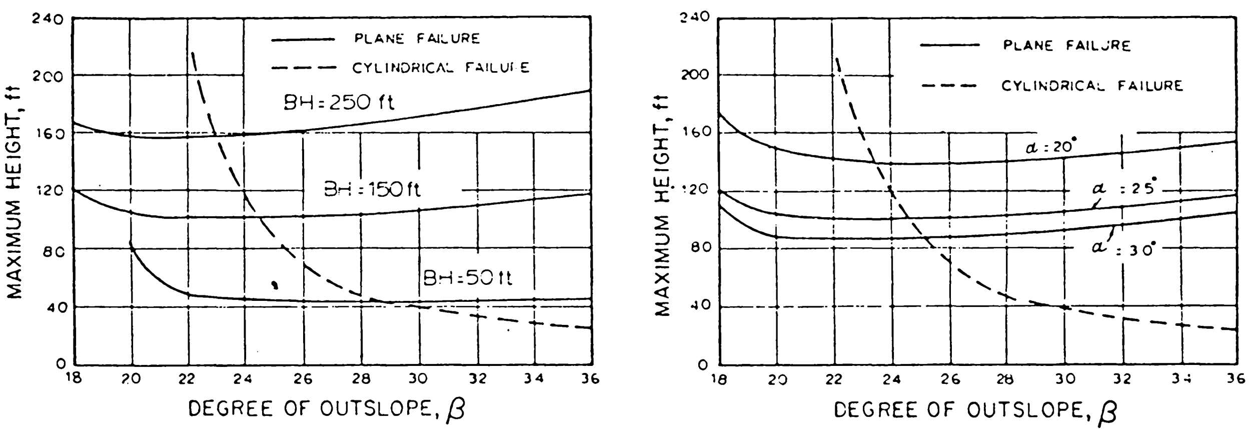


Figure 3.14. The relationship of base width, angle of outside, and slope on allowable height of a hollow fill (from Huang, 1978).

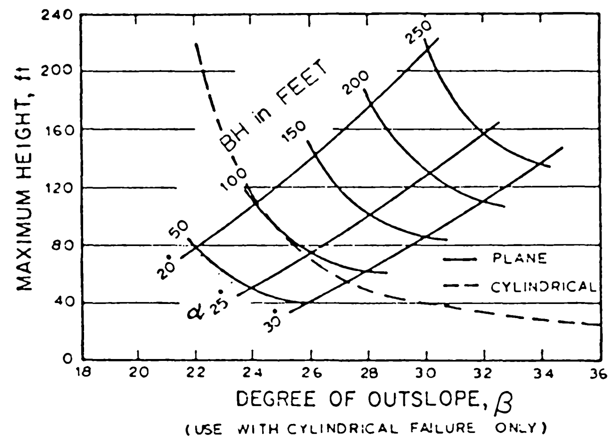


Figure 3.15. Design chart for a hollow fill (after Huang, 1978).

Figure 3.13 may be considered to be representative of a typical section of a hollow fill. Figures 3.14 and 3.15 were developed by Huang for a safety factor of 1.5 using his method developed in this report. The following fill material characteristics were assumed: $\bar{c}$ = 200 psf, $\bar{\psi}$ = 30°, r_u = 0.05, γ = 125 pcf; for interface material (used for plane failure calculations) $\bar{c}$ = 160 psf, $\bar{\psi}$ = 24°, and r_u = 0.1. It is obvious from these figures that the degree of outslope has relatively little effect on plane failure. However BH and degree outslope do affect cylindrical failure. These aspects with relation to fill configurations required by Public Law 95-87 will be developed in the following section.

3.5 Cylindrical and Plane Failure Analysis of Spoil Banks

In Chapter 2 the methods of calculating the factor of safety for cylindrical and plane failure of slopes was developed. Although it is recommended that a slope be analyzed for both types of failure, clearly each will generally govern under different specific conditions.

Figure 3.16 represents those conditions under which shear or cylindrical failure will govern. The material properties used for this solution are the statistical worst developed from Huang (Table 7.2), $\bar{c}$ = 126.5 psf, ϕ = 28.30, γ = 125, factor of safety = 1.5. The degree of natural slope and height of the fill are shown; the resulting coordinates give the maximum degree of outslope that would be acceptable under given conditions. There are several important generalizations that can be made, based on Figure 3.16. Over the range of

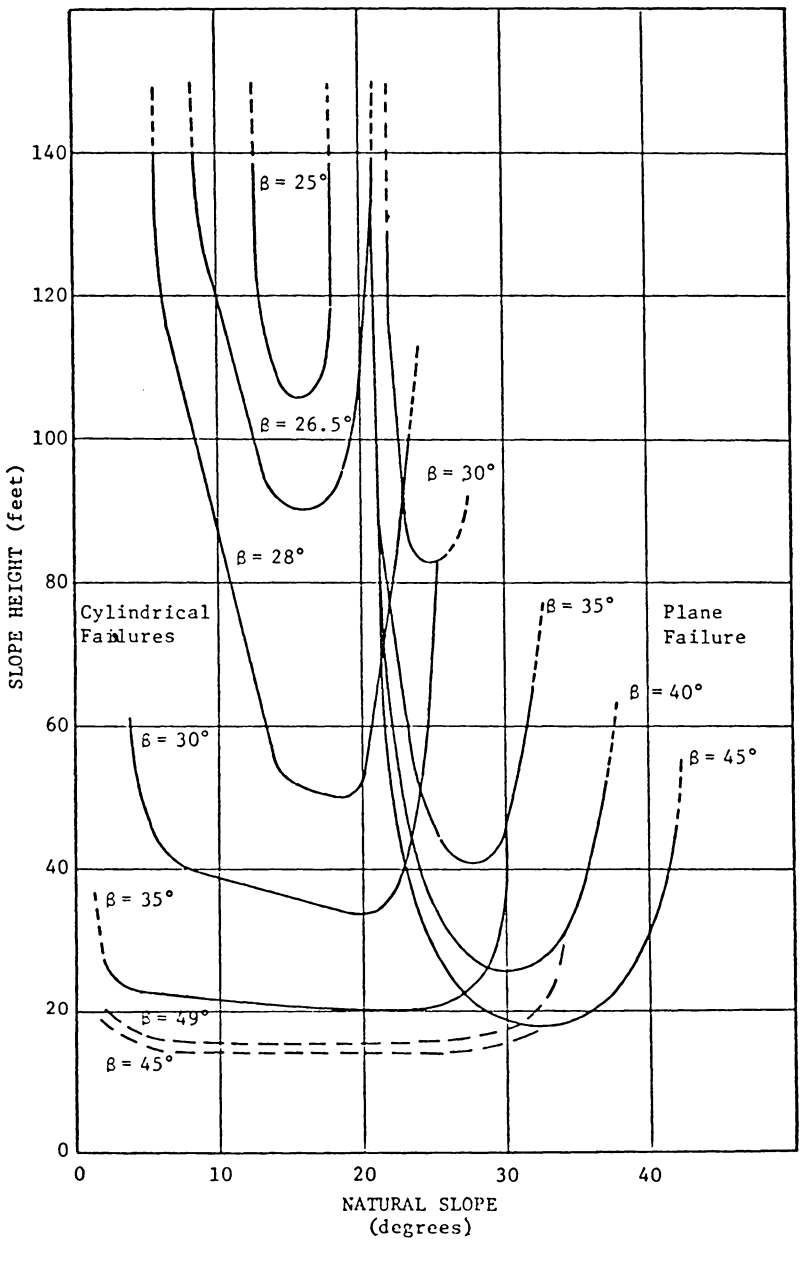


Figure 3.16. Factors determining cylindrical and plane failure.

conditions calculated for, shear failure governs in only a relatively small area. This region could be defined roughly as when the degree of outslope is within five degrees of the natural slope. Obviously, as the degree of outslope approaches the degree of natural slope, for a soil exhibiting a residual shear expressed as cohesion, the height of the slope will theoretically approach great heights. Recalling that at the natural angle of repose, an embankment may theoretically be of infinite height; one must conclude that the angle of repose for this material must be something slightly less than 25°. There would therefore, seem to be good reason for the common use of a 1v:2H slope, which equals 26.5°. Since this is considered to be representative of poor material, the typical, or average material encountered would have a higher natural angle of repose, and thus a greater margin of safety would exist. The increase in embankment height for cylindrical failure, just prior to encountering shear failure is apparently the result of the cohesion term in the basic factor of safety equation, becoming dominant. As the outslope approaches the natural slope, the soil mass contained within the critical arc would decrease to the point where the ratio of failure surface area to the failure component of weight would increase.

One must keep in mind that this graph is not unlike most others of its nature, in that its accuracy decreases at extreme values. It is believed that such a graph as shown in Figure [3.16] would be of considerable value in making estimates of the stability of spoil banks in an area having material of consistent characteristics.

Chapter 4

LEGAL REQUIREMENTS

Table 4.1 lists the sections of the Federal Register [10] where the legal requirements, under Public Law 95-87, may be found for the following subjects: head of hollow fills, valley fills, and steep slope mining. It is believed that it would serve no purpose to reproduce all of these requirements here. However, it would be useful to list the key regulations as they pertain to the design of spoil banks and valley fills for surface mining. These key parts of Public Law 95-87 are listed in the general order in which they would concern an engineer in the design of these structures. They are: (1) site investigation, (2) site preparation, (3) placement of fill material, (4) configuration, and (5) surface drainage. Also included in this chapter are figures to help the reader visualize some basic concepts involved with this subject.

4.1 Site Investigation

816.71 Disposal of Excess Spoil--General Requirements

(1) The disposal area shall not contain springs, natural water-courses, or wet-weather seeps unless lateral drains are constructed from the wet areas to the underdrains in a manner that prevents infiltration of the water into the spoil pile.

TABLE 4.1

REGULATIONS PERTAINING TO FILL DESIGN

Item	Regulation			
Head-of-Hollow Fills	816.43	816.71	817.43	817.71
	816.51	816.73	817.51	817.73
Valley Fills	816.43	816.72	817.51	817.72
	816.51	816.73	817.71	817.73
	816.71	817.43		
Steep Slope Mining	816.102(c)	817.102(c)	Part 826	

Regulation	Description
816.43	Hydrologic Balance: Diversions and Conveyance of Overland Flow and Shallow Ground Water Flow.
816.51	Hydrologic Balance: Protection of Groundwater Recharge Capacity.
816.71	Disposal of Excess Spoil: General Requirements.
816.72	Disposal of Excess Spoil: Valley Fills.
816.73	Disposal of Excess Spoil: Head-of-Hollow Fills.
817.43	Hydrologic Balance: Diversions and Conveyance of Overland Flow and Shallow Ground Water Flow.
817.71	Disposal of Underground Development Waste and Excess Spoil: General Requirements.
817.72	Disposal of Underground Development Waste and Excess Spoil: Valley Fills
817.73	Disposal of Underground Development Waste and Excess Spoil: Head-of-Hollow Fills.

Table 4.1--Continued

Regulation	Description
817.102	Backfilling and Grading: General Grading Requirements.
826	Special Permanent Program Performance Standards--Operations on Steep Slopes.
816.102(c)	Backfilling and Grading: Grading Requirements. Small Depressions

(n) The foundation and abutments of the fill shall be stable under all conditions of construction and operation. Sufficient foundation investigation and laboratory testing of foundation materials shall be performed in order to determine the design requirements for stability of the foundation. Analyses of foundation conditions shall include the effect of underground mine workings, if any, upon the stability of the structure.

4.2 Site Preparation

816.71 Disposal of Excess Spoil: General Requirements.

(c) All vegetative and organic materials shall be removed from the disposal area and the topsoil shall be removed, segregated, and replaced under sections 816.21-816.23 before spoil is placed in the disposal area.

816.72 Disposal of Excess Spoil: Valley Fills.

(b) A subdrainage system for the fill shall be constructed in accordance with the following:

(1) A system of underdrains constructed of durable rock shall--

(i) be installed along the natural drainage system;
(ii) extend from the toe to the head of the fill; and
(iii) contain lateral drains to each area of potential drainage or seepage.

(2) A filter system to insure the proper functioning of the rock underdrain system shall be designed and constructed using standard geotechnical engineering.

(3) In constructing the underdrains, no more than 10 percent of the rock may be less than 12 inches in size and no single rock may be larger than 25 percent of the width of the drain.

Rock used in underdrains shall meet the requirements of paragraph (a)(5) of this section. The minimum size of the main underdrain shall be:

Total amount of fill material	Predominant type of fill material	Minimum size of drain, in feet	
		Width	Height
Less than			
1,000,000 Yd^3	Sandstone	10	4
1,000,000 Yd^3	Shale	16	8
More than			
1,000,000 Yd^3	Sandstone	16	8
1,000,000 Yd^3	Shale	16	8

(5) Rock used shall not have less than 50 percent wear in 500 revolutions in the Los Angeles Rattler Test (AASHTO T-96-70), shall not have less than 15-percent weight loss in 5 cycles of the sodium sulfate test (ASTM, C088, AASHTO T-1-4), and shall not contain less than 30 percent by volume of clay or clay minerals as determined by standard petrologic analytical tests, and shall not be acid forming or toxic-forming.

4.3 Placement of Fill Material

816.71 Disposal of Excess Spoil: General Requirements

(a) Spoil not required to achieve the approximate original contour shall be hauled or conveyed to and placed in designated disposal areas within a permit area other than mine working or excavations, only if the disposal areas are authorized for such purposes in the approved mining and reclamation permit and only in accordance with sections 816.71-816.73. The spoil shall be placed in a controlled manner to ensure--

(2) stability of the fill; and

(3) that the land mass is suitable for reclamation and revegetation compatible with the natural surroundings.

(e) The disposal areas shall be located on the most moderately sloping and naturally stable areas available as approved by the regulatory authority. If such placement provides additional stability and prevents mass movement, fill materials suitable for disposal shall be placed upon or above a natural terrace, bench, or berm.

(f) The spoil shall be hauled or conveyed and placed in a controlled manner, concurrently compacted as necessary to ensure mass stability and prevent mass movement, covered, and graded to allow surface and subsurface drainage to be compatible with the natural surroundings, and to ensure long term stability.

816.72 Disposal of Excess Spoil: Valley Fills.

(c) (1) Spoil shall be hauled or conveyed and placed in a controlled manner and concurrently compacted as specified by the regulatory authority in lifts no thicker than 18 inches in order to--

(i) achieve the densities designed to ensure mass stability;
(ii) prevent mass movement;
(iii) avoid contamination of the rock underdrain or rock core; and
(iv) prevent formation of voids.

(c) (2) The person who conducts the surface mining activities may use lifts of greater thickness than required under paragraph (c)(1) of this section if he has demonstrated to the regulatory authority by density monitoring tests that the density throughout the thickness of the lift is equal to or greater than the density specified in the design referred to in paragraph (2) of this section, except that in no event shall lift thickness exceed 4 feet.

826.12 Steep Slopes: Performance Standards.

(a) Spoil, waste materials, or debris, including that from clearing and grubbing or haul road construction, and abandoned or disabled equipment, shall not be placed or allowed to remain on the downslope.

(b) The highwall shall be completely covered with compacted soil and the disturbed area graded to comply with the provisions of 30 CFR 816.101-816.106 including, but not limited to, the return of the site to approximate original contour. The person who conducts the surface coal mining and reclamation operation must demonstrate to the regulatory authority, using standard geotechnical analysis, that the minimum static factor of safety for the stability of all portions of the reclaimed land is at least 1.3.

816.103 Backfilling and Grading: Covering Coal and Acid--and Toxic-Forming Materials.

(a) Cover

(1) A person who conducts surface mining activities shall cover, with a minimum of 4 feet of the best available nontoxic and noncombustible material, all exposed coal seams remaining after mining and any acid forming or toxic forming combustible materials, or any other materials identified by the regulatory authority, that are exposed, used, or produced during mining.

4.4 Configuration of the Fill

816.71 Disposal of Excess Spoil: General Requirements.

The spoil shall be placed in a controlled manner to ensure--

(3) That the land mass is suitable for reclamation and revegetation compatible with the natural surroundings.

(g) The final configuration of the fill must be suitable for postmining land uses approved in accordance with section 816.124 except that no depressions or impoundments shall be allowed on the completed fill.

(h) Terraces shall not be constructed unless approved by the regulatory authority.

(m) If any portion of the fill interrupts, obstructs, or encroaches upon any natural drainage channel, the entire fill is classified as a valley or head-of-hollow fill and must be designed and constructed in accordance with the requirements of sections 816.72 and 816.73, respectively.

816.72 Disposal of Excess Spoil: Valley Fills.

(e) The tops of the fill and any terrace constructed to stabilize the face shall be graded no steeper than 1v:20h (5 percent). The vertical distance between terraces shall not exceed 50 feet.

(g) The outslope of the fill shall not exceed 1v:2h (50 percent). The regulatory authority may require a flatter slope.

816.73 Disposal of Excess Spoil: Head-of-Hollow Fills.

(a) The fill shall be designed to completely fill the disposal site approved by the regulatory authority to the approximate elevation of the ridgeline. A rock-core chimney drain may be utilized instead of the subdrain and surface diversion system required for valley fills. If the crest of the fill is not approximately at the same elevation as the low point of the adjacent ridgeline, the fill must be designed as specified in section 816.72, with diversion of runoff around the fill.

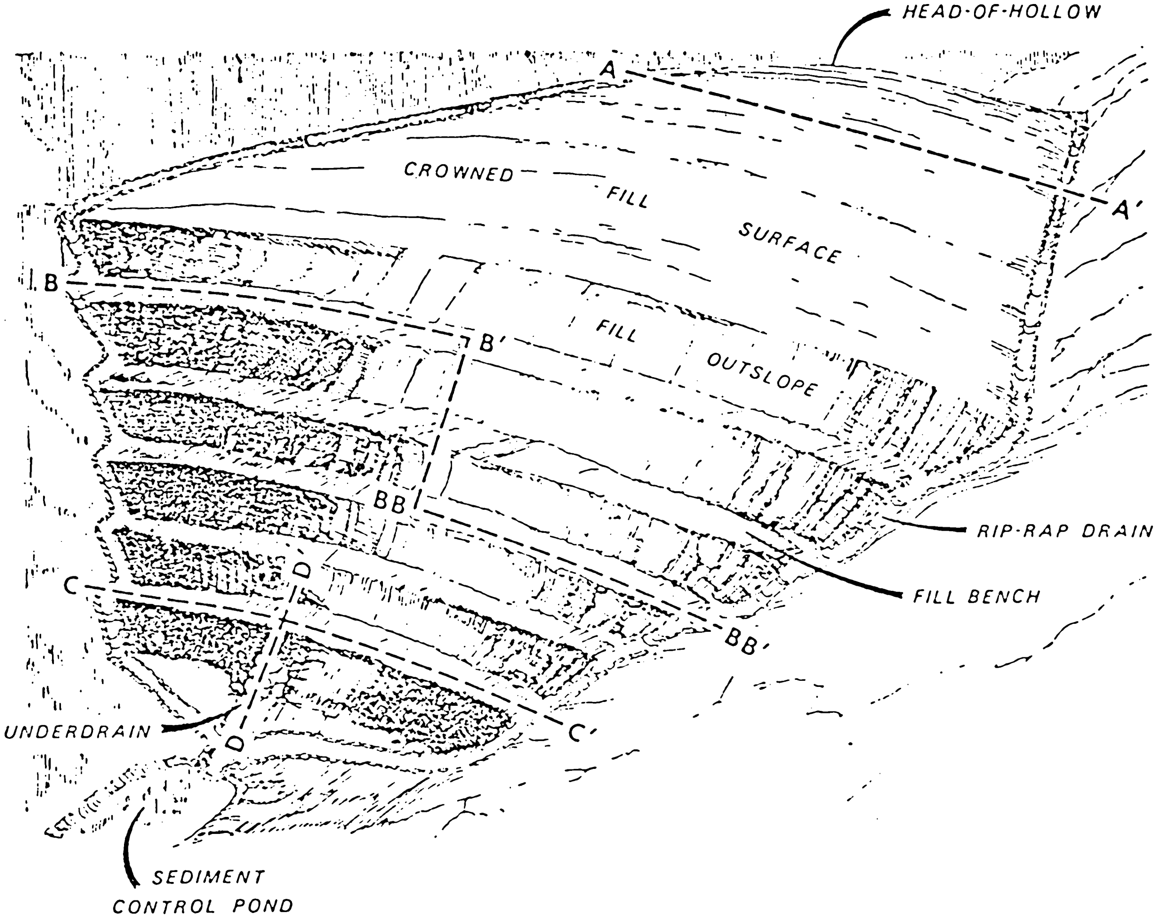


Figure 4.1. Valley fill (after Skelly and Loy, 1978).

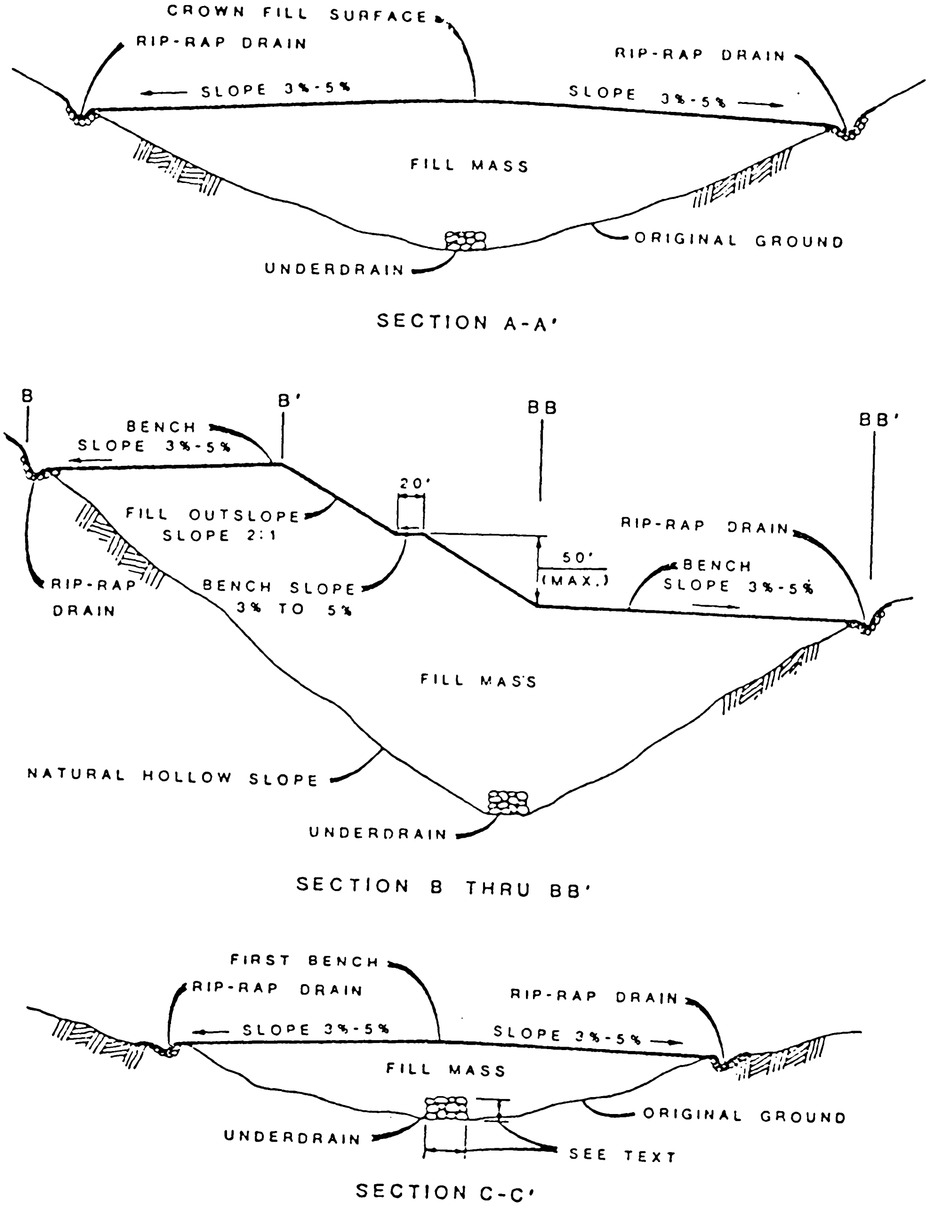
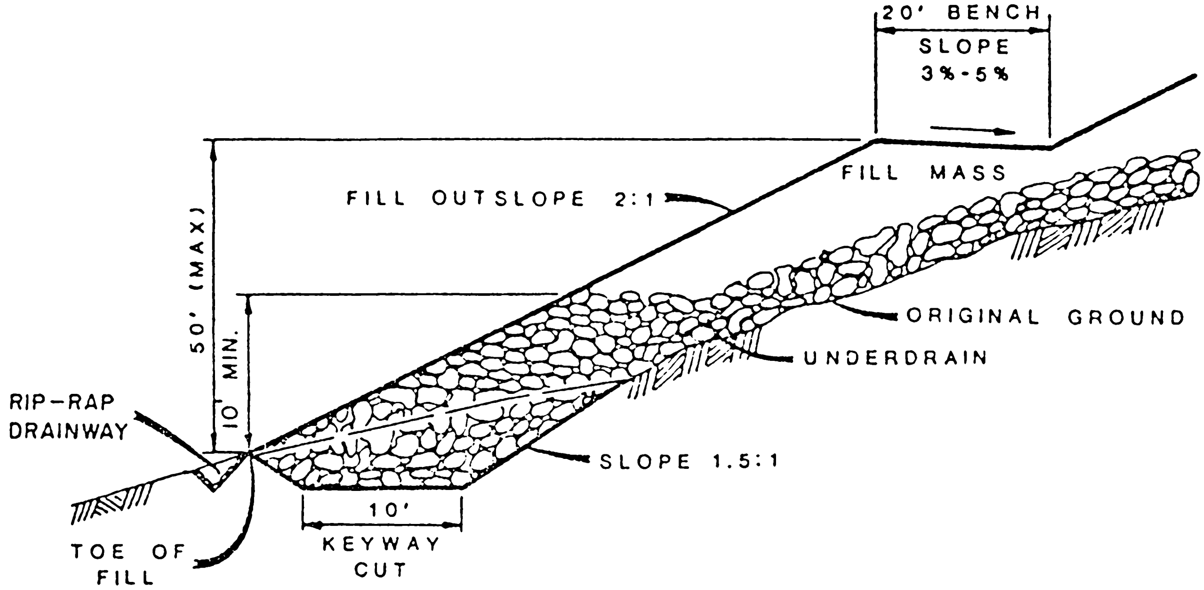
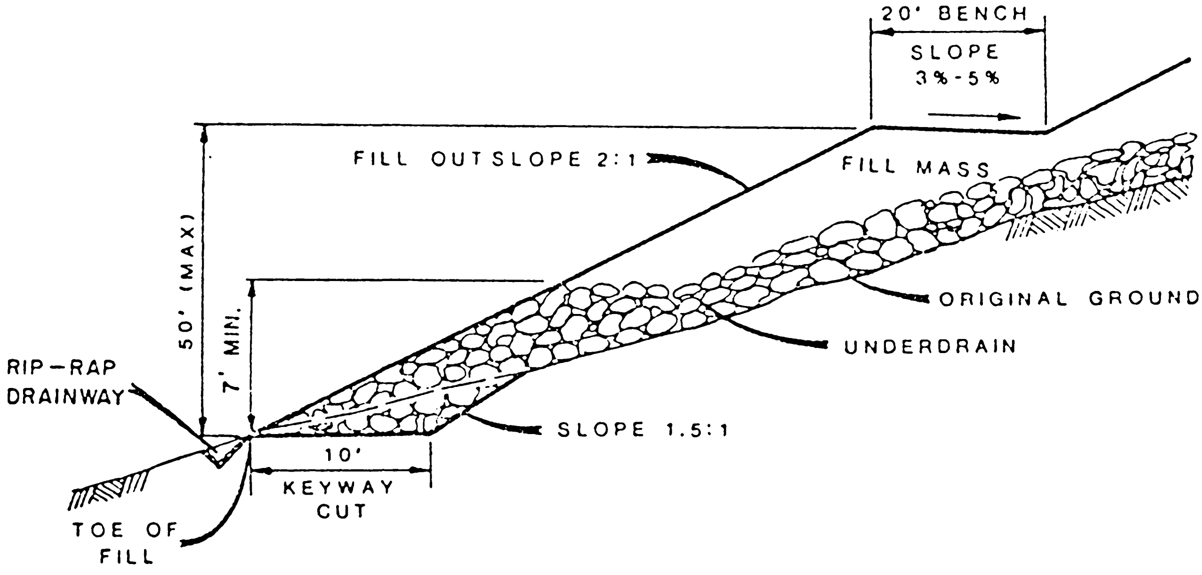


Figure 4.2. Typical sections for a valley fill (after Skelly & Loy, 1978).



A. TRAPEZOIDAL KEYWAY CUT



B. BENCHED KEYWAY CUT

Figure 4.3. Typical sections for a valley fill (after Skelly & Loy, 1978).

825.12 Performance Standards: Special Bituminous Coal Mines Developed after August 3, 1977.

(d) (2) (i) The final mine area shall be backfilled, graded and contoured to the extent necessary to return the land to the use approved by the regulatory authority.

4.5 Surface Drainage

816.71 Disposal of Excess Spoil: General Requirements.

The spoil shall be placed in a controlled manner to ensure--

(1) That leachate and surface runoff will not degrade surface or ground waters or exceed the effluent limitations of section 816.42;

(d) (3) Slope protection shall be provided to minimize surface erosion at the site. All disturbed areas including diversion ditches that are not riprapped shall be vegetated upon completion of construction.

816.72 Disposal of Excess Spoil: Valley Fills.

(d) Surface water runoff from the area above the fill shall be diverted away from the fill and into stabilized diversion channels designed to pass safely the runoff from the 24-hour duration 100-year frequency storm or larger event specified by the regulatory authority. Sediment control structures shall be provided at the discharge of the diversion ditch before entry into the natural watercourse in accordance with section 816.46. Surface runoff from the fill surface shall be diverted to stabilized channels off the fill which will safely pass runoff from a 24-hour duration, 100-year frequency storm diversion design shall comply with requirements of section 816.43(f).

(f) Drainage shall not be directed over the outslope of the fill.

Chapter 5

A CHECK-LIST FOR CONTEMPORARY VALLEY FILL CONSTRUCTION

The primary purpose of this chapter is to develop and present the engineering concepts necessary for design of valley fills. However, as was indicated in the introduction, these structures are relatively new to the mining industry, and were developed primarily as a result of the great volumes of spoil produced in mountaintop removal methods. It should therefore follow that some basic knowledge of the process involved in the production of the material of which the valley fill is constructed would be relevant. Unless otherwise noted, this chapter is taken largely from a recent state of the art review of mountaintop removal techniques that has been presented by Skelly and Loy [34].

The mountaintop removal mining method shares the same three phases with other surface mining methods. These are site feasibility determination, actual mining, and post mining reclamation.

Before any mining operation may be initiated, it must first be determined if there will be sufficient economic impetus for mining, after all factors are considered. This is essentially reduced to a matter of determining if the cost of mining (the removal of overburden, extraction of coal and subsequent reclamation) will be offset by the value of the resulting product, coal. If the resulting margin of profit is determined to be sufficient, by management, then the operation is deemed feasible. The mechanics of this decision making process are beyond the scope of this report. However, the following would represent key factors to be considered in the site feasibility phase.

- Site Feasibility

  --pit exploration

  --core borings

  --historical geological and mining records

5.1 Site Feasibility

The site requirements for valley fills may be found in the previous chapter, which deals with the legal requirements. The basic requirement for the fill area is that: (1) it will not be subject to flooding; (2) it will provide a foundation of sufficient strength to support the mass of the fill. Examination of topographical maps and brief field reconnaissance is generally sufficient to determine the likelihood of flooding, or existence of streams in an area. The structural integrity of the foundation however, will not be so obvious. Study of geologic maps as well as core drilling is required to determine the nature of the underlying strata. The necessity of locating any weak material or sources of water should be obvious from the previous discussion of factors affecting mass stability. As indicated previously, if the mining operation itself is not expected to be worthwhile, there is little need to concern oneself with valley fills. The following pages deal with the basic considerations involved with making this determination.

Generally, anyone involved in mining will have some general idea as to where coal beds exist. In coal bearing regions, it is not usually a question of whether or not seams exist, but how thick they are, and of what quality their coal is. For steam coal, the BTU value per ton and percentage of sulfur is usually the prime concern. For metalurgical coal, the fixed carbon and impurities in the coke produced are most important.

Usually any large company contemplating sizable operations has a special department which deals specifically with exploration. This exploration involves "drill programs," which consist of drilling holes at some distance apart in the areas of interest. If these holes produce information indicating that further investment is appropriate, more holes are drilled in order to provide a more complete picture of the seam thickness, overburden characteristics, etc. Often, in heavily mined regions, the operators will be familiar with the coal seams being investigated, due to previous experience, and therefore will be concerned primarily with thickness.

Any coal seam which could be mined by mountaintop removal, will have an outcrop which is readily accessible. The coal in the outcrop will not be representative of the overall seam quality, due to oxidation. However, examination of the coal seam in situ will allow evaluation of the difficulties likely to be encountered in the important overburden removal process. If, after exploration, and subsequent reserve evaluation, the project still appears feasible, it will be turned over to the operations department for final evaluation.

5.2 Mining

The actual mining process may be divided into the following general categories for consideration:

- Mining

  --preplanning
  --site preparation
    silt pond construction and erosion control
    clearing and grubbing
  --drilling and blasting
  --overburden removal
  --coal removal

Preplanning immediately follows the exploration process. At this point all information accumulated during exploration will be turned over to the "operations" department. This is where the final decisions are made as to mine feasibility, operational methodology, and application for mining permits from the numerous regulatory agencies [26].

This permit process, in recent years, has become increasingly difficult due to the increased burdens placed on the surface mining industry by the Government. With the regulations contained within Public Law 95-87, the operator must face the grim prospect of not being able to mine his coal unless he can "show" regulatory authorities that reclamation standards can be met. Part of this process involves demonstration "through accepted professional standards" that earthwork resulting from mining will be of sufficient stability. After the mine layout and sequence of operations has been determined and the necessary permits obtained, operations may begin.

The first thing that must be done, before any mine site or fill area preparation may start, is to construct a sediment pond. Before discussing those factors involved with the design of sediment control structures, it would be appropriate to review: (1) the problems caused by sediment; and (2) methods used to limit its generation. This subject has been covered by Hittman Associates in a recent publication, and the following has been taken from their work [13].

Detrimental Effects of Sediment

- occupies water storage in reservoirs
- fills lakes and ponds
- clogs stream channels
- settles on productive land
- destroys aquatic habitat
- creates turbidity that detracts from recreational use of water and reduces photosynthetic activity
- degrades water for consumptive uses
- increases water treatment costs
- damages water distribution systems
- acts as a carrier of other pollutants (plant nutrients, insecticides, herbicides, heavy metals)
- acts as a carrier of bacteria and viruses

Sediment is widely regarded as the greatest source of water pollution in the United States [13].

Hittman Associates have also presented the following list of sources of sediment from surface mining:

- failure to install perimeter control measures prior to the start of clearing and grubbing
- exposure of soils on steep slopes

- overclearing--clearing too far above the highwall or below the outcrop line
- clearing and grubbing too far ahead of the pit, exposing the soil for an excessive length of time
- improper placement and/or protection of salvaged and stockpiled topsoiling material
- creation during clearing and grubbing operations of a soil surface that impedes infiltration and/or concentrates surface runoff (for example, leaving ripper marks or dozer cleat marks that run up and down the slope rather than along the contour)
- poor design and construction of roads

The rationale involved in the control of sediment is basically: (1) expose for the shortest period of time, the smallest area possible; (2) reduce runoff by stabilization; (3) intercept and detain runoff; and (4) divert concentrated flows into ponds [11]. This last factor, sediment ponds, is the most critical. Although much can be done to limit the amount of sediment generated during mining, the complete elimination of sediment laden runoff is impossible. For this reason, construction of sediment ponds are absolutely essential, as they represent the last line of defense against the pollution of streams.

The subject of sediment pond design will be covered more extensively in the following chapter. However, the following list will provide some idea of the numerous factors that must be considered.

- Design
  --drainage area
  --precipitation
  --topography
  --area to be disturbed by mining operations
  --water quality standards
  --soil characteristics

- Types of Sediment Control Structures
  --primary
    windrowed vegetation
    rock check dam
  --final
    gabion structure
    excavated pond
    earthen or rock dam

- Construction
  --material
  --access
  --configuration and size
  --location
  --topography

- Maintenance
  --silt cleanout
  --bank stability

All runoff from the area disturbed by mining operations must flow through a sediment pond [15]. Although such water may require acid neutralization, this process will not be covered here, since acid water is not a product of fill construction, as is sediment. The pond must obviously be situated at some point between the land disturbed and the stream to be protected. The most common location is at a short distance below the toe of the valley fill [6].

After the sediment pond is constructed, clearing and grubbing of the fill area and mine site may begin. Generally, the initial cut on a new mine site will be parallel to the outcrop, and upslope a sufficient distance to provide a 15 foot strip of material around the perimeter of the operation. This barrier results in little loss of coal since most of it will have been weathered [2]. It does, however, support backfill material, and helps the control of water drainage [45].

The actual method of excavation can vary a great deal, after the initial cut. The factors bearing on this are:

- number of coal seams and overburden characteristics
- spoil volume and haul distance
- equipment type, size, and quantity
- post mining land use

It is obvious that the first factor, combined with site topography will have the greatest effect on subsequent earthwork operations. The most important aspect of this phase, the removal of the overburden, is blasting. The major considerations in this operation are: (1) drill hole size and spacing along with drill bench level; (2) type of explosive, load, and detonation pattern. Those physical features that would be site specific are:

- overburden characteristics
- fracture sizing
- noise and vibration restrictions
- displacement of material due to blasting

After the overburden has been broken up, its removal begins. A major advantage of mountaintop removal is that it allows the use of very large earthmoving equipment that had previously been restricted to area mining. These machines reduce unit operating costs, and permit a greater ratio of overburden of coal to be removed, than would otherwise be possible [1]. After the removal of overburden begins, the necessity of a fill site becomes obvious.

The fill area, prior to spoil placement, will be cleared of all organic material. By definition, a valley fill will have a "U" shape; and in its bottom will be placed a rock underdrain. The first material deposited will be at the lowest elevation of the site area. The spoil will be placed in lifts of the thickness required, and will be concurrently compacted; all subsequent lifts will be placed in the same manner. As the fill operation continues, grading for drainage and configuration will be in keeping with requirements under Public Law 95-87.

5.3 Reclamation

The reclamation process for the valley fills begins after the completion of each lift. This concurrent reclamation results in: (1) reduced erosion; (2) reduced saturation of fill material; (3) surface stabilization; and (4) prolonged sediment pond cleanout intervals [28]. This stabilization process generally involves revegetation only. However, chemical binders in some cases are used to provide temporary binding of soil, which will allow grasses time to begin growth [23]. A listing of commonly used grasses and chemical binders for mine reclamation has been prepared by Hittman Associates [13], and may be found in Appendix I. This firm's publications also give recommended methods for their use. A table listing environmental impacts associated with head of hollow fills has been prepared by Skelly and Loy Consulting Engineers, and may also be found in Appendix I.

In conventional contour surface mining, reclamation involves returning the mined land to its original contour. This is obviously not done in mountaintop removal. The degree of effort expended upon the relatively level mine site after the coal is removed will depend on the intended post mining land usage [39]. If the site will be used for a structure, such as a school or hospital, there would be little need for revegetation. If, however, green areas, or housing developments were planned, some spoil material would probably be used to create a rolling terrain prior to revegetation. Regardless of ultimate use, the creation of large flat areas in mountainous regions results in a most valuable land form.

Chapter 6

WATER QUALITY

This chapter will deal with the prevention of siltation of streams through the use of sediment ponds. As indicated previously, their design will not be covered in this report, as this has been adequately covered by Hittman Associates [11]. There is some evidence however, that traditional design considerations can result in ponds that cannot always meet effluent standards [12,32]. The cause of this problem and its solution will be given in the following pages.

Before progressing further into this subject, the key requirements under Public Law 95-87 that deal with sediment control should be presented [10].

6.1 Legal Requirements

816.46 Hydrologic Balance: Sedimentation Ponds.

(a) General requirements. Sedimentation ponds shall be used individually or in series and shall--

  (1) be constructed before any disturbance of the disturbed area to be drained into the pond

  (2) be located as near as possible to the disturbed area and out of perennial streams.

(c) Detention Time. Sedimentation ponds shall provide a 24-hour theoretical detention time for the water inflow or runoff entering the pond from a 10-year 24-hour precipitation event.

(c)(1) The regulatory authority may approve a theoretical detention time of not less than 10 hours, when the person who conducts the surface mining activity demonstrates that--

(i) the improvement in sediment removal efficiency is equivalent to the reduction in detention time as a result of pond design.

(c)(3) The regulatory authority may approve a theoretical detention time of less than 24 hours to any level of detention time when the person who conducts the surface mining activities demonstrates to the regulatory authority that the chemical treatment process to be used--

(i) will achieve and maintain the effluent limitations;

(ii) is harmless to fish, wildlife and related environmental values.

6.2 Sediment Pond Performance

Table 6.2 shows a summary of effluent parameters from ten sediment ponds that were studied by Skelly and Loy [34]. There are several things that should be kept in mind when studying the data presented in Table 6.2. Permission had to be obtained from the companies that owned the facilities studied. Although they remained anonymous, one would expect these operations to be the best the company had. All of the

TABLE 6.1

816.42 HYDROLOGIC BALANCE: WATER QUALITY STANDARDS AND EFFLUENT LIMITATIONS

EFFLUENT CHARACTERISTICS[1]	MAXIMUM ALLOWABLE[2]	AVERAGE OF DAILY VALUES FOR 30 CONSECUTIVE DISCHARGE DAYS
Iron, Total	7.0	3.5
Manganese, Total[3]	4.0	2.0
Total Suspended Solids[4]	70.0	35.0
pH	Within Range of 6.0 to 9.0[5]	

[1]To be determined according to collection and analytical procedures adopted by the Environmental Protection Agency's regulations for wastewater analyses (40 CFR 136).

[2]Based on representative sampling.

[3]The manganese limitations shall not apply to untreated discharges which are alkaline as defined by the Environmental Protection Agency (40 CFR 434).

[4]In Arizona, Colorado, Montana, New Mexico, North Dakota, South Dakota, Utah, and Wyoming, total suspended solids limitations will be determined on a case-by-case basis, but they must not be greater than 45 mg/l (maximum allowable) and 30 mg/l (average of daily value for 30 consecutive discharge days) based on a representative sampling.

[5]Where the application of neutralization and sedimentation treatment technology results in inability to comply with the manganese limitations set forth, the regulatory authority may allow the pH level in the discharge to exceed to a small extend the upper limit of 9.0 in order that the manganese limitations will be achieved.

ponds studied were receiving runoff from hollow fill areas. It is obvious from the table that those sediment ponds in West Virginia have effluents of a quality much higher than those of Kentucky. It should be noted that in West Virginia, runoff from a valley fill must be directed into the rock-core chimney drain that is required by that state. For that reason, it is likely that some sediment is removed as the water passes through the rock core, which makes comparison of pond efficiencies difficult. West Virginia is acknowledged to have the most stringent surface mining laws in the eastern United States. This is certainly reflected in the effluent data shown.

Certainly, a 24-hour detention time would result in a pond as large as would be realistically feasible. This has been a standard for a number of years. However, Hill, in a review of sediment pond efficiency reported that in a study of nine of the "better constructed ponds in Kentucky, Pennsylvania, and West Virginia, that half did not meet proposed EPA effluent guidelines" [12]. The reason for this is obvious after studying Tables 6.3 and 6.4. One can expect at least one percent or more of the sediment entering a pond, having a 24-hour detention time, of not settling out. Since this problem is the result of fine particles not having sufficient detention time, a coagulation process is obviously required. Before discussing the application of this process, it would be appropriate to present a brief discussion of the theory and principles involved in coagulation.

TABLE 6.2

SUMMARY OF EFFLUENT FROM SEDIMENT PONDS

MINE SITE	TURBIDITY			TOTAL SOLIDS			SUSPENDED SOLIDS		
	Max.	Min.	Mean	Max.	Min.	Mean	Max.	Min.	Mean
West Virginia									
BA	10	<5	-	550	401	-	26	3	-
HA	45	<5	14	1170	349	495	32	3	15
MA	75	<5	40	372	290	304	34	22	30
OA	100	5	38	180	151	164	50	8	26
PA	40	5	19	200	114	161	28	<1	14
State	100	<5	22	1170	114	352	50	<1	19
Kentucky									
EA	90	5	48	230	204	217	60	1	30
FA	15	<5	8	3700	450	2841	19	3	11
GA	220	<5	56	680	54	488	150	2	40
IA	250	10	88	1260	466	694	275	6	82
KA	150	<5	52	688	240	509	150	4	49
State	250	<5	51	3700	54	105	275	1	44
Overall	250	<5	37	3700	54	223	275	1	32

All units mg/1 except where noted.
(After Skelly and Loy, 1978).

TABLE 6.3

TIME AT WHICH PARTICLES WILL SETTLE IN STILL WATER AT 10°C
(SPECIFIC GRAVITY = 2.65)

DIAMETER OF PARTICLE mm.	ORDER OF MAGNITUDE	TIME REQUIRED TO SETTLE 1 FOOT
10.0	Gravel	0.3 seconds
1.0	Coarse sand	3 seconds
0.1	Fine sand	38 seconds
0.01	Silt	33 minutes
0.001	Bacteria	35 hours
0.0001	Clay particles	230 days
0.00001	Colloidal particles	63 years

(From Hittman Associates, 1976).

TABLE 6.4

PARTICLE SIZE DISTRIBUTION INCOMING SUSPENDED SOLIDS

PERCENT FINER BY WT.	SMALLEST PARTICLE SIZE[a] mm.	LARGEST PARTICLE SIZE[b] mm.
100	0.5	4
90	0.1	3.5
80	0.7	2.3
70	0.045	1.7
60	0.035	1.0
50	0.028	0.6
40	0.017	0.4
30	0.008	0.27
20	0.0045	0.2
10	0.002	0.1
5	0.001	0.07
1	0.005	0.01

[a]Obtained from summation of all distributions and represents the smallest particle sizes.

[b]Obtained from summation of all distributions and represents the largest particle sizes.

(From Hittman Associates, 1976).

6.3 Theory of the Coagulation Process

There seems to be two reasons why particles are not removed by sedimentation ponds. Firstly, the particles are too small to settle out in the detention time provided. Secondly, the particles carry an electric charge which results in repulsive interactions that cause the particles to remain suspended in the water. It should then follow that if some means could be used to reduce these electrical charges and bring the particles together to form larger particles, efficient sedimentation would occur. This process can be accomplished by coagulation. Based on the above, a review of the factors involved in this process is appropriate. A review of the coagulation process has been presented by O'Melia [25]. The following is taken from this source.

There appears to be three reasons why colloidal particles have electrical charges. Firstly, the substitution of a different atom in the crystal lattice of a particle with a resulting change in charge. This is apparently quite common in the layered structures of clays. Secondly, colloids can have groups on their surface that are subject to ionization, with a subsequent charge modification (the pH of the solution would therefore be important in determining the extent to which these surface groups ionize). Lastly, the adsorption of ions onto the colloid, with subsequent charge modification. "The specific adsorption of ions arises from hydrogen bonding, covalent bonding, or van der Waals forces, and can be augmented by electrostatic attraction" [25].

Now that some reasons as to why particles have electrical charges are known, one can look at the resulting problems and their solutions.

There are two generally accepted theories dealing with the relative stability of the colloidal systems of interest here. Firstly, the double-layer theory. This deals with a concentration of counter ions near the colloids surface and a diffuse layer extending outward dependent upon a concentration gradient (this simple concept considers only electrostatic attraction and diffusion interactions). Van der Waals forces are considered to be important in this system. "Increasing the electrolyte concentration compresses the diffuse layer, decreases the magnitude of the repulsive energy of interaction at intermediate separating distances, eliminates the potential energy barrier, and thereby destabilizes the colloid" [25]. Presumably then, van der Waals forces are then able to cause the particles to come together.

The second method by which coagulation can be achieved involves chemical bridging between a polymer molecule and colloids. This polymer would have many sites onto which colloids could attach themselves. Depending on chemical and physical conditions, this adsorption can be accomplished in many ways. "Postulated interactions include ion exchange, hydrogen bonding, the formation of coordinative bonds and linkages, van der Waals forces between the coagulant and the colloid, and repulsion of the coagulant by the aqueous phase (e.g. surface active agents)" [25]. The floc resulting from this process will obviously be desirable for subsequent sedimentation. The next subject of interest are the chemicals that are used in these two theories of colloidal destabilization.

Aluminum (III) and iron (III) are the most commonly employed chemicals to implement the mechanisms propesed by the double layer theory (O'Melia points out that (III) in this case denotes the ability to chemically combine with three valence bonds). All metal cations that are added to water are hydrated to form metal hydroxides. If these metals are added to water in quantities sufficient to exceed the solubility of the resulting metal hydroxides, hydroxo metal complexes are formed. These are readily adsorbed onto the collodial particles with resulting destabilization. The pH is important in this system (O'Melia's definition of isoelectric point--"Many colloids (e.g. clays, protein colecules) have both positive and negative charge sites. The extent to which these sites are charged depends upon the pH of the colloidal system. Sites which can assume a negative charge (e.g. carboxyl groups) tend to do so as the pH of the system is increased. Sites which can assume a positive charge (e.g. amino groups) tend to do so as the pH of the system is lowered. There exists a hydrogen ion concentration (pH) at which the net primary charge on such a colloidal particle is zero. This pH value is called the isolectric point") [25]. Anionic polymers will dominate above isoelectric point and positive charged inorganic polymers will dominate below.

The other type of chemicals of interest in coagulation are synthetic organic polymers. A polymer may generally be described as a chain of ionizable groups, although some have no ionizable components and are called nonionic polymers. The polymers effectiveness will obviously depend on its ability to attach colloids to its many surfaces. The molecular weight, and degree of branching of the polymer are important, as are pH and concentrations of ions in the water, in determining its effectiveness.

After particles have been destabilized or have become attached to polymer chains, they must be brought together to form flocs. This is generally called flocculation. In systems having small particles and low turbulence, this is accomplished by Brownian movement. Orthokinetic flocculation, or mechanical stirring is associated with larger particles and higher turbulence. The power input of these systems can be critical in balancing flocculation with forces that can shear the floc that has already been formed. The importance of initial dispersion (rapid mix) of the appropriate coagulant and subsequent contact (flocculation) with the colloids is critical in determining the removal efficiency that will be achieved.

6.4 Application of Coagulation

Apparently there has been only one significant published paper on the use of coagulation to remove sediment from surface mine runoff. Considerable effort was required to locate and obtain a copy of it. This might explain why this process has seen little use by the mining industry. A more important reason could be that electricity for the necessary equipment is not always available at sediment pond sites. Also, protection of equipment from the weather and the necessary operation and maintenance would represent considerable effort and expense. However, in view of recent enactment and enforcement of Public Law 95-87, it appears that operators might have no choice but to upgrade their water treatment facilities [19]. For this reason, a review will be made of a paper presented by Richard E. McCarthy to the Research and Applied Technology Symposium on Mined-land Reclamation in 1973 [2].

McCarthy's paper deals with the use of chemical coagulants for removal of sediment from water emanating from a surface mine in the Pacific Northwest. He indicates that lab tests were performed to determine the degree of difficulty that might be experienced in attempting to remove the sediment from the runoff at the site of interest. These tests revealed that "a significant fraction of the soil (clay) that would be subjected to runoff would become near colloidal and remain suspended in the water for more than a week" [2]. Knowing that the existing sediment ponds would provide a maximum of 23 hours of detention time, the author began investigating the possibility of using a "high molecular weight organic polelectrolyte with a high charge density" to flocculate the fine particles.

Further lab studies, using jar test apparatus, gave the required dosages of the chemical for removing various concentrations of solids. The final optimum dosage was determined to be 10 ppm for the water studied. This would certainly vary, depending on the chemical used and sediment characteristics. Laboratory studies would, therefore, be required for each specific situation.

After determining the chemical to be used and the required dosage, McCarthy's next problem was in the design and construction of the facilities for application of the coagulation process. After considerable difficulty a chemical metering system was devised using the following key components:

1. Liquid level capacitive probe
2. Liquid level monitoring and control instrument with flow proportional output circuit
3. Automatic pump control
4. Chemical metering pump

This system was located at the effluent end of the first pond in a two-pond system. The first pond removed the larger particles, while the second pond acted as a clarifier after the coagulation process had been implemented. The metering system works as follows:

1. The liquid level capacitive probe is used in conjunction with a rectangular weir to measure outflow of the pond.
2. The liquid level monitoring and control instrument receives the signal from the probe and converts it to 10-50 MA DC output for use by the automatic pump control.
3. The pump control unit then amplifies the signal it receives and regulates the chemical feed pump, which supplies the chemical.

The above system is shown in Figure 6.1.

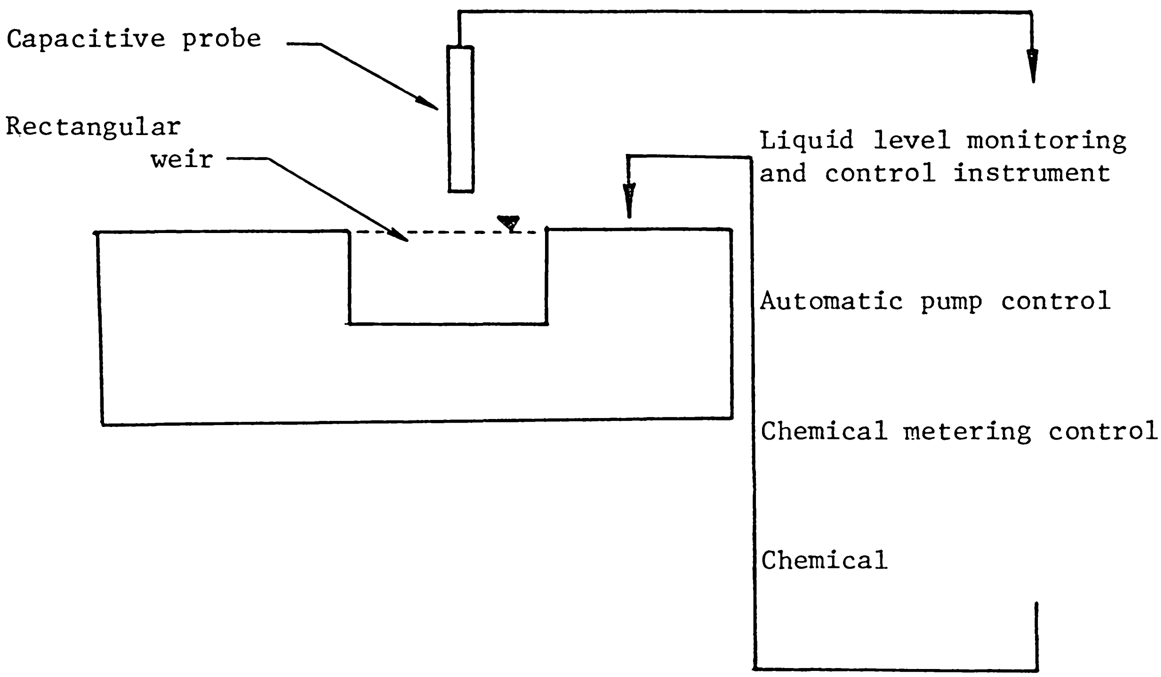


Figure 6.1. Chemical metering system (after McCarthy, 1973).

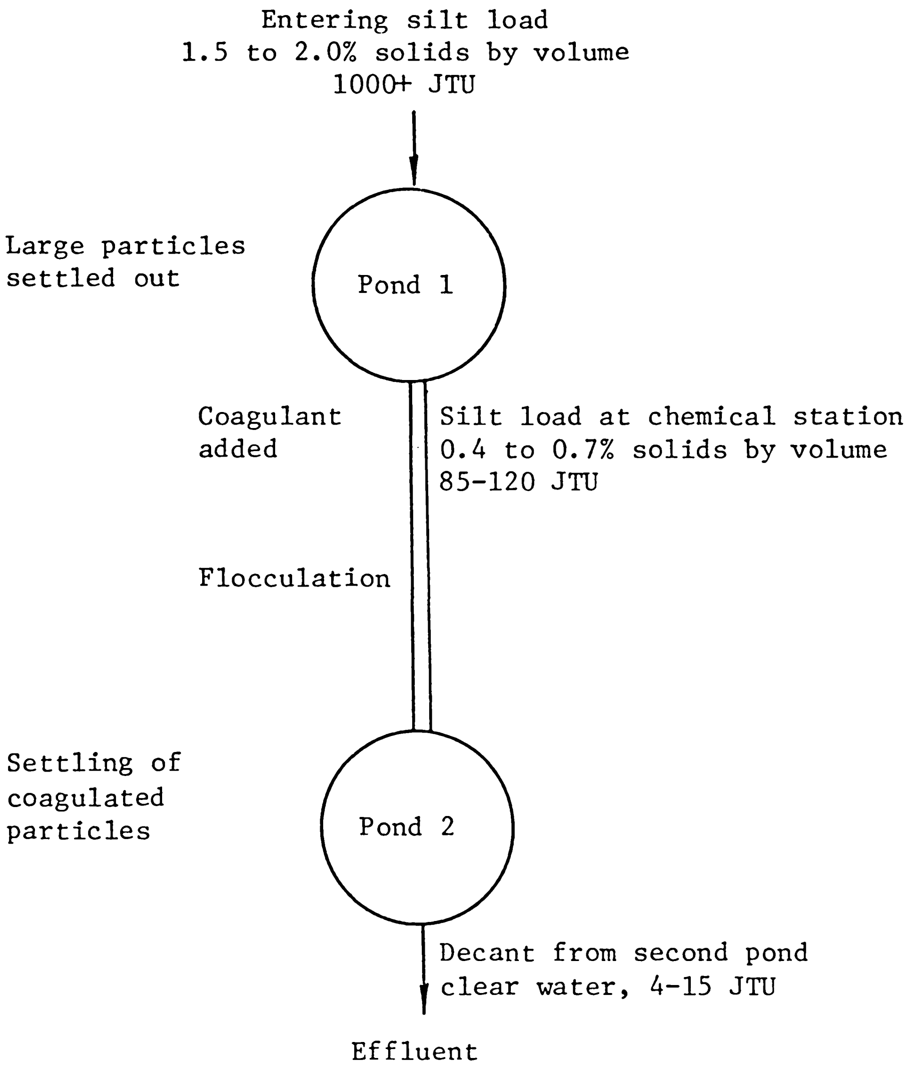


Figure 6.2. Chemical treatment system.

After chemical is introduced into the stream, mixing is required for effective flocculation to occur. This was accomplished by "designing into the spillway, fins and angle iron obstructions to cause turbulence and mixing." Subsequent flocculation was accomplished by "a 50 foot half round culvert with baffles every 5 feet." Certainly, this would appear to be a rather crude device for implementing the coagulation process, but one must keep in mind the primitive site conditions normally associated with surface mining. The success of any treatment process is represented by its effluent. The results of this successful system are shown in Figure 6.2.

Chapter 7

EVALUATION OF COMPACTION REQUIREMENTS UNDER PUBLIC LAW 95-87

7.1 The Effect of Compaction

Based on concepts previously developed in this report, there can be no doubt that compaction increases the shear strength of mine spoil. What is not clear is how much of an increase in strength is obtained. Basically Public Law 95-87 requires concurrent compaction "to ensure mass stability and prevent mass movement." One must conclude that those responsible for this legislation are implying that without concurrent compaction, mass stability cannot be insured.

Based on the literature review for this report, there does not appear to be any studies of existing spoil slopes, compacted and uncompacted, that allow one to evaluate the difference in their stability. However, the results of shear tests for compacted and uncompacted mine spoil have been presented by Huang [14]. Table 7.1 shows the results under conditions indicated. These results have been plotted, and are shown in Figure 7.1.

It is obvious from the failure envelope that compaction has very little impact on the material's angle of internal friction. Clearly the apparent cohesion is increased by a maximum value of 300 psf for the test conditions. This increase in cohesion appears significant, but in large fills with very high normal forces acting at the failure plane, this parameter decreases in importance. It is therefore postulated

TABLE 7.1

PEAK SHEAR STRESSES FOR MINE SPOIL
UNDER VARIOUS TESTING CONDITIONS

SPECIMEN STRAIN RATE	UNCOMPACTED 0.005 in/min		UNCOMPACTED 0.03 in/min		COMPACTED 0.005 in/min		COMPACTED 0.03 in/min	
Normal Stress (tsf)	Dry Density (pcf)	Peak Shear Stress (tsf)	Dry Density (pcf)	Peak Shear Stress (tsf)	Dry Density (pcf)	Peak Shear Stress (tsf)	Dry Density (pcf)	Peak Shear Stress (tsf)
0.65	100.6	1030	103.2	1002	116.7	1162	116.6	1462
	100.8	872	102.7	956	126.3	1190	113.7	1462
	101.1	880	101.7	966	121.9	1200	119.3	1462
	Average	927	Average	975	Average	1184	Average	1462
1.31	109.8	1640	107.9	1686	119.2	1874	119.7	1996
	109.1	1574	106.8	1640	121.4	1856	118.0	2006
	111.1	1724	106.3	1650	119.7	1836	120.7	2118
	Average	1646	Average	1659	Average	1855	Average	2040
40' Embankment	-	2960	103.7	2932	120.8	3158	119.0	3392
2.61	114.1	2970	-	3046	118.4	3168	123.7	3242
	112.0	2952	112.9	3074	123.4	3168	122.9	3252
	Average	2961	Average	3017	Average	3165	Average	3295

Note: 1 inch = 2.54 centimeter, 1 pcf = 157 N/m^3,
and 1 tsf = 95.8 kN/m^2.

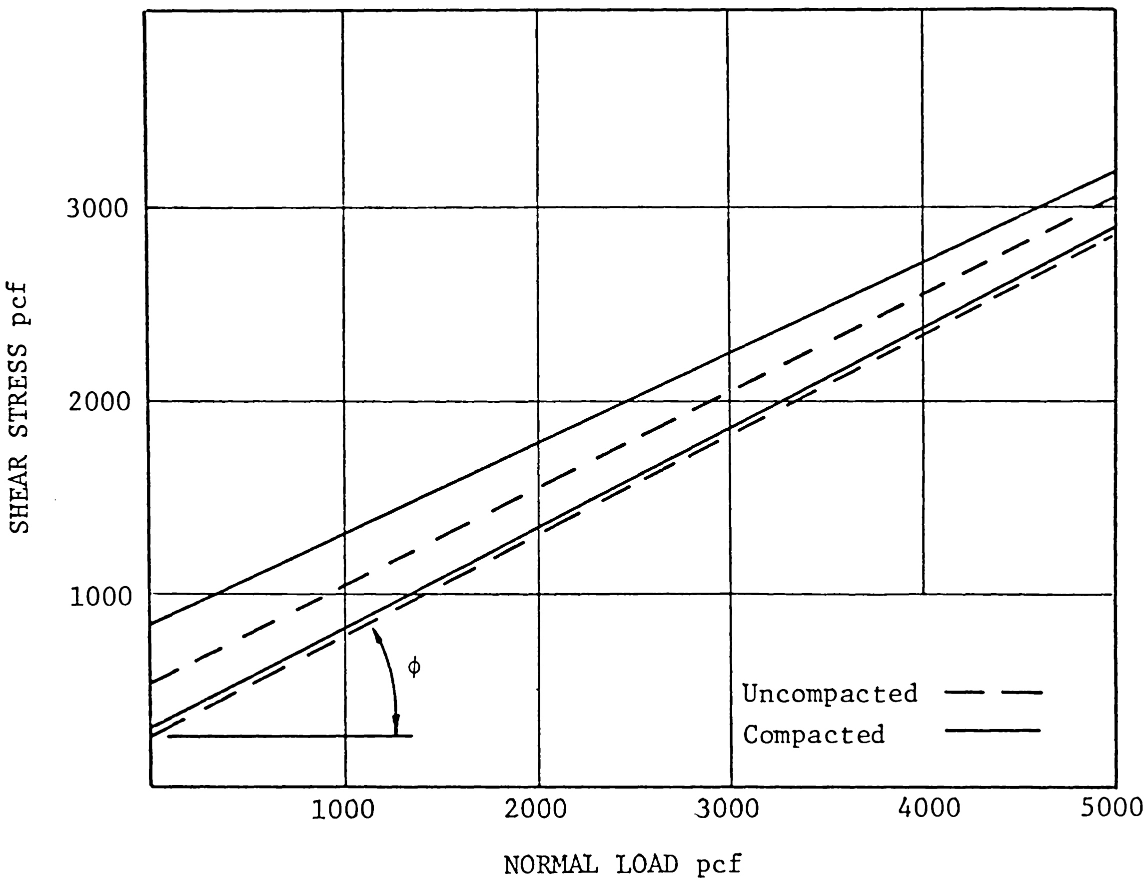


Figure 7.1. Determination of effect of compaction on soil shear strength.

that a stable slope, using typical surface mined spoil, may be achieved without compaction. This concept is pursued further in the following pages.

The legal requirements for compaction of spoil placed in valley fills is covered in section 816.72 (c)(1) of Public Law 95-87. The exact wording may be found in Chapter 4 of this report. Concurrent compaction is stated to be necessary to "achieve the densities designed to ensure mass stability, prevent mass movement, and prevent formation of voids." There should be no question that those compaction requirements stated in section 816.72 (c)(1) would result in achievement of these goals. The question would seem to be, however, is concurrent compaction necessary to accomplish this?

In Chapter 2 of this report, the important concepts of dilatancy and saturation were developed, as they relate to mass stability. With the requirement of a rock underdrain, combined with adequate surface drainage, the possibility of a fill becoming saturated is very remote. Furthermore, any fill is subject to a varying degree of settling, depending upon the amount of compaction performed during placement. This settlement will result in greater density and increased interlocking of particles, which will subsequently increase the internal angle of friction, resulting in greater stability. Settlement would obviously have an opposite effect under saturated conditions, as the water trapped within the pores is forced to accept a greater load, less normal force is exerted on the solid material. Since water can withstand no lateral stress and as the N Tan ϕ value decreases, the likelihood of failure will increase. Failure under saturated conditions, based on this situation, will tend to be instantaneous. This is just the opposite situation that is present under unsaturated conditions. Based on the above, in situations where settling of the fill mass would not have deleterious effects on the environment, or result in a hazard to life, there would seem to be little need for concurrent compaction. A realistic compromise might be to limit concurrent compaction to the critical outslope area and limit the outslope to 26.5°.

7.2 Analyzing the Stability of Uncompacted Fill

The objective of this section of the report is to demonstrate, while using established field data, that slope stability may be maintained without compaction. The argument centers on an analysis utilizing the statistically worst spoil material.

Table 7.2 shows the characteristics of 16 spoil banks in eastern Kentucky [14]. A statistical analysis of the cohesion and friction angle values shown, indicates that. $\bar{C}$ = 126.5 psf and $\phi = 28.3^{\circ}$ (for uncompacted material) are representative of the worst material likely to be encountered. In calculating safety factors, the geometric constraints imposed by Public Law 95-87 (section 816.71 (i) and section 816.72 (g), Chapter 4) are used. Thus, the stability analysis is for the worst (most unstable) configuration permitted by law. The sections of the fill, and the stability calculations, are shown in Figures 7.2, 7.3, 7.4, 7.5 and Table 7.3.

TABLE 7.2

SAFETY FACTOR OF STABLE SPOIL BANKS
IN EASTERN KENTUCKY

SITE NUMBER	$\bar{c}$ (psf)	$\bar{\delta}$ (degree)	α (degree)	β (degree)	H (feet)	FACTOR OF SAFETY AT PORE PRESSURE RATIOS OF		
						0.0	0.25	0.5
1	200	28.2	25	37	37.3	1.35	1.16	0.94
2	120	32.2	21	31.5	31.4	1.65	1.36	1.05
3	440	26.0	24	36.5	48.8	1.73	1.50	1.27
4	760	15.6	21	37	24.1	2.97	2.84	2.70
5	240	34.7	27	37	18.1	2.55	2.29	1.99
6	320	25.0	26	38.5	53.5	1.28	1.10	0.93
7	200	31.8	25	37	22.3	1.89	1.63	1.36
8	0	34.2	26.5	37	36.1	0.93	0.68	0.45
9	120	33.3	14	34	53.1	1.26	1.00	0.73
10	200	29.9	18	36.5	39.2	1.40	1.16	0.92
11	240	27.0	24	35.5	48.8	1.30	1.12	0.91
12	280	26.4	21	41	75.4	0.94	0.78	0.59
13	80	36.6	20	35	54.4	1.28	1.01	0.73
14	160	27.0	12	32	53.0	1.11	0.91	0.69
15	200	28.3	28	39	40.2	1.28	1.10	0.90
16	40	32.3	27	39	27.7	1.01	0.81	0.60

Note: 1 foot = 0.305 meter and 1 psf = 47.9 N/m^2.

(After Huang, 1978).

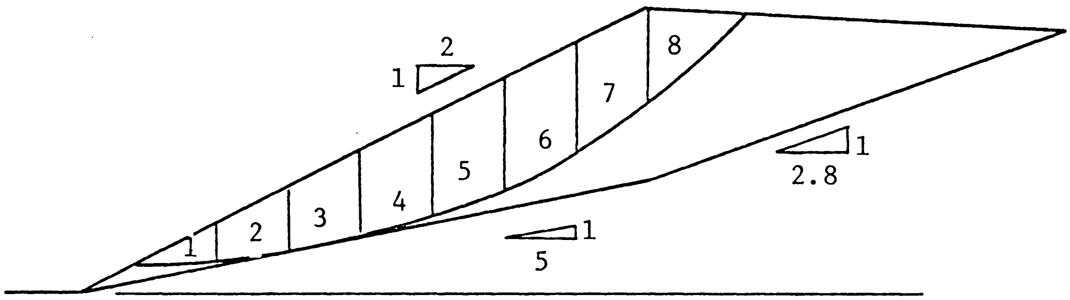


Figure 7.2. Cylindrical section, Trial 1.

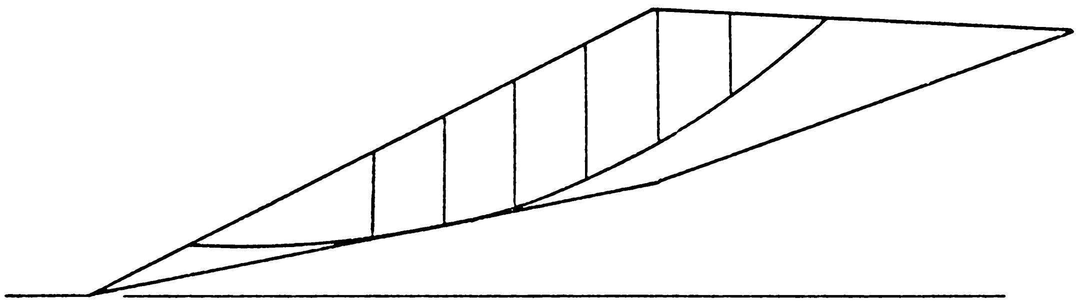


Figure 7.3. Cylindrical section, Trial 2.

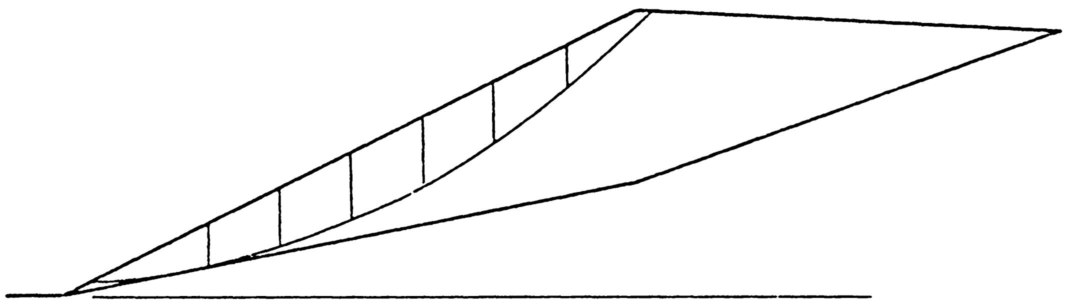


Figure 7.4. Cylindrical section, Trial 3.

TABLE 7.3

STABILITY CALCULATIONS FOR
CYLINDRICAL FAILURE

SLICE AREA	γ = 125 UNIT SLICE WEIGHT -LBS	φ = 28.3 N TAN φ E	Θ	N SIN Θ	I	E/I
1 6	18,750	10,096	0	0	1	10,096
2 18	56,250	30,287	5	4,903	1.03	29,404
3 26	81,250	43,749	11	15,503	1.04	42,066
4 33	103,125	55,527	17	30,151	1.04	53,391
5 37	115,625	62,258	22	43,314	1.05	59,293
6 37	115,625	62,258	28	54,283	1.04	59,863
7 35	109,375	58,887	35	62,735	1.01	58,304
8 24	75,000	40,383	41.5	49,696	0.99	40,790
				∑ 257,585		∑ 353,207 + 31,625
	$\frac{\text{Tan } \phi}{1.5}$ = 0.36		384,832 ÷ 257,585 = 1.49			384,832

250 x 126.5 = 31,625 lbs. cohesion

Safety Factor = 1.49

TRIAL 1 (Figure 7.3)

SLICE AREA	UNIT SLICE WEIGHT LBS	N TAN φ E	Θ	N SIN Θ	I	E/I
1 39	121,875	65,617	0	0	1	10,096
2 34	106,250	57,205	11	20,273	1.03	55,539
3 42	131,250	70,665	14	31,752	1.04	67,947
4 45	140,625	75,712	22	52,679	1.04	72,800
5 47	146,875	79,077	26	64,386	1.04	76,036
6 39	121,875	65,617	31	62,770	1.03	63,706
7 21	65,635	35,332	39	41,299	0.99	35,689
				∑ 273,159		∑ 437,333 + 30,360
			467,693 ÷ 273,159 = 1.7			467,693

C = 126.5 x 240 = 30,360 lbs. cohesion

Safety Factor = 1.7

TRIAL 2 (Figure 7.4)

TABLE 7.3 (Continued)

STABILITY CALCULATIONS FOR
CYLINDRICAL FAILURE

SLICE AREA	UNIT SLICE WEIGHT -LBS	N TAN ϕ E	Θ	N SIN Θ	I	E/I
1 12	37,500	20,192	0	5,866	1.03	19,607
2 17	53,125	28,605	17	15,532	1.04	27,505
3 22	68,750	37,018	20	23,514	1.04	35,594
4 23.5	73,437	39,542	24	29,869	1.04	38,021
5 22	68,750	37,018	30	25,754	1.03	35,940
6 18	56,250	30,287	37	33,852	0.99	30,593
7 10	31,250	16,826	44	21,708	0.95	17,711
				$\sum$ 156,095		$\sum$ 204,971 + 25,300
			230,271 $\div$ 156,095 = 1.47			230,271

C = 200 x 126.5 = 25,300 lbs. cohesion

Safety Factor = 1.47

TRIAL 3 (Figure 7.5)

CONDITIONS

$\alpha = 21.5$
$\beta = 26.5$
$BH = 125$
$H = 200$
$B\ (200) = 125$
$B = .625$

$$d = \frac{1}{2}\ [\cot\alpha - (1 - B\tan\beta)^2 \cot\beta]$$
$$.5\ [2.53 - [(1 - (.625)(.4986)]^2\ 2.006]$$
$$d = .794$$

$$e = \frac{1}{2}\ (.625)^2 \tan\beta$$
$$\frac{1}{2}\ (.625)^2\ (.4486)$$

$$\alpha = d \sin 21.5$$
$$= (.794)(.3665)$$
$$\alpha = .291$$

$$b = -\ [\frac{\bar{c}}{\gamma H}\ (\ \csc\alpha + \csc\alpha) + (1 - r_u)(d + e)\csc\alpha\tan\psi]$$
$$-\ [.0101[(.625)(.9304) + (2.7285)] + (.8914)(.9304)(.5384)]$$

$$\underline{b = .781}$$

$$c = -\quad \sin\alpha\tan\psi\ [\frac{\bar{c}}{\gamma H} + (1 - r_u)\ \frac{e}{\beta}\tan\psi\ ]$$
$$-\ [(.625)(.5384)[.0191 + .1558(.5384)]$$

$$\underline{c = .0316}$$

$$\alpha F^2 + bF + c$$

$$\frac{.781 + \sqrt{(.781)^2 - 4\ (.291)(.0316)}}{2(.291)} = 2.7$$

Factor of Safety for Plane Failure = 2.7

FIGURE 7.5

CALCULATIONS FOR PLANE FAILURE
FOR FIGURE 7.3 (Using Huang's Method).

With a minimum factor of safety for cylindrical failure of 1.47 it is obvious that a slight reduction in the degree of outslope would make the slope legally stable. With a factor of safety of 2.7 for shear failure, one must question the necessity of the restrictive requirements for natural slopes. It may be reasoned that: if only that portion of the fill contained within the circular arc is subject to failure, then there would be no need to compact all of the material in the fill in order to prevent failure. To restate a previous conclusion: if stability through compaction is insisted on by any regulatory authority, surely this compaction need only be carried out in the area of the final slip circle. That is to say at the extremities of the final head of hollow or valley fill.

Chapter 8

THE IMPACT OF PUBLIC LAW 95-87 ON THE MINING INDUSTRY

In an attempt to evaluate some of the ramifications of recent legislation on surface mining, some statements have been taken from current literature addressing this subject. The following statements are from an analysis published by the U.S. Department of the Interior on the Surface Mining Control and Reclamation Act of 1977 [15].

> In the post-implementation case for the Appalachian Supply District, the primary effect of the regulations is to slow down the expansion of output in the surface mining sector and encourage more underground production. (page 125)
>
> The cost of regulation implementation for surface mined coal ranges from a high of $2.16 per ton in the steep contour mining regions of Appalachia to a low of $.02-.05 per ton in the areal surface mines of the north Rocky Mountains and the west. (page 135)

Many coal producers disagree with these government figures and insist that they are quite conservative. Some of their comments follow. The following statements are taken from a recent published article by an Engineer, with Consolidation Coal Company on the impact of Public Law 95-87 [32]:

> Economic analyses have indicated that a $3 to $5 per ton of coal difference exists between a totally compacted fill, . . . as compared to a controlled side-dumped valley fill . . . This is a savings of $120 to $200 million at one mine which would not be passed to the consumer due to the unjustified regulation.

> . . . compaction of the entire embankment would result in overall strength in the entire embankment being relatively high. However, the front portion of the embankment is still the only critical section which governs mass stability.

It is believed that there is sufficient evidence to support this last statement, concerning the portion of the fill that governs stability, based on concepts developed in this report. The economic impact, therefore, is uncertain, but as with other regulations, it is likely to be severe. As this report has so far shown, part of this cost increase is questionable.

Further considerations involve the operation of the regulatory program. It is, therefore, useful to examine its early interpretation and implementation. The department created by Public Law 95-87 to enforce its regulation is the Office of Surface Mining (OSM). The following is a summary of OSM's activities from May 29, 1978 to December 1, 1978 [19].

After nine months of enforcement of the interim regulations of Public Law 95-87, 2,189 coal operations have been inspected throughout the country. OSM weekly reports indicate 458 coal operators have received notices of violation and 161 have been closed due to violations. The state of Kentucky, the leading state in coal production, had 416 operations inspected and 46 have been shut down. The fact that West Virginia has had only 6 shutdowns, is not surprising, since this state is generally acknowledged to have the most stringent surface mining laws. In fact, some West Virginia operators favored the passage of Public Law 95-87, since it would result in more uniform regulations nationwide and thus enable their coal to be more competitively priced.

A summary of OSM inspection results is shown in Table 8.1. The small operators have been hit hardest by these regulations. Certainly they do not account for a majority of the coal produced, but one must wonder at the incongruity of such laws. While the Federal Government continues to call for increased coal production, it closes mines.

In conclusion, it will take time to fully understand the impacts of this new law. Furthermore, recent court action by Virginia will slow the process of the compliance schedule (Table 8.2), a process already slated to be phased up to February 3, 1981.

TABLE 8.1

OSM INSPECTION
RESULTS:
MAY 29, 1978 TO
DECEMBER 1, 1979

INSPECTION RESULTS BY STATE

STATE	*NOV	*CO	STATE	*NOV	*CO
Kentucky	109	46	Texas	0	2
Virginia	55	27	Oregon	2	0
Ohio	60	10	Georgia	0	1
Tennessee	42	25	Kansas	1	0
Alabama	33	21	North Dakota	1	0
West Virginia	37	6			
Indiana	34	4			
Illinois	24	8			
Pennsylvania	26	3			
Arkansas	8	5			
Oklahoma	7	1			
Missouri	7	1			
Maryland	7	0			
Iowa	5	0			
Utah	3	2			

INSPECTION RESULTS BY REGION

REGION	*NOV	*CO
II	184	93
I	125	36
III	118	22
IV	28	8
V	3	2

*Notice of Violation

*Cessation Orders

(After Landmarc, 1979).

Table 8.2

OPERATOR COMPLIANCE SCHEDULE
FOR PUBLIC LAW 95-87

AUGUST 3, 1977

Public Law 95-87 signed into law. Part 716.7 (Prime Farmlands) effective on permits issued on or after Aug. 3, 1977 section 510 (d)(1) of the act.

OCTOBER 1, 1977

Begin collection of reclamation fee on all coal production.

DECEMBER 13, 1977

Interim regulations published in Federal Register (42 days late). In section 710.11(d)(1) states, "The requirements of this chapter apply to operations conducted after effective date of the regulations (Dec. 13, 1977) on lands from which the coal has not yet been removed and to any other lands used, distrubed . . ."

DECEMBER 31, 1977

Special reclamation fee due on last quarter of production.

JANUARY 15, 1978

Last day for advertisement in local newspaper for notice of request for small operator exemption.

JANUARY 30, 1978

The abandoned mine reclamation program regulation is to be promulgated and published (Sec. 405(a)).

Last day for payment of special reclamation fee from last quarter.

FEBRUARY 3, 1978

Section 710.11(a) states that all state permits issued by a state on or after Feb. 3, 1978 must contain provisions to comply with the relevant performance standards as required by Section 502(b) of the act of the initial regulatory program. Last date for small operators to submit an application requesting an exemption from compliance from the performance standards.

Table 8.2 (Continued)

It is also the last date for the submittal of plans for reconstruction of pre-existing, non-conforming structures or facilities that are physically impossible to bring into compliance by effective date of regulations (Section 710.11(d)(2)).

Plan must provide starting date of May 4, 1978 and completion date of Nov. 4, 1978. A definitive list of pre-existing facilities is not available but definitely includes sedimentation ponds, haul roads, off-site spoil storings and head of hollow fills. Facilities and structures not serving active mining areas after May 4, 1978 need to be reconstructed.

<u>MARCH 31, 1978</u>

First quarter of reclamation fee payable is due.

<u>APRIL 30, 1978</u>

Final day for payment of first quarter reclamation fee.

<u>MAY 3, 1978</u>

On and after May 3, 1978, any person conducting coal mining operations shall comply with the initial regulatory program, except:

1. Certain provisions of operations on Indian Lands (Part 710.11(b));

2. Pre-existing, non-conforming structure or facility granted a variance;

3. Special exemption for small operators on state lands covering all general and special performance standards except for Part 716.2(a)(1)-Handling of Spoil.

<u>AUGUST 3, 1978</u>

The permanent reclamation program regulations promulgated (Section 501(b)).

<u>JANUARY 1, 1979</u>

Small operators receiving an exemption must start complying with the interim standards (Section 502(c)).

Table 8.2 (Continued)

<u>FEBRUARY 3, 1979</u>

State programs must be submitted to the secretary for approval (18 months after passage of the act).

<u>AUGUST 3, 1979</u>

Secretary approves or disapproves state program (6 months after receipt of the application from the state).

<u>OCTOBER 3, 1979</u>

Operators must submit permit application if state program is approved.

<u>JUNE 3, 1980</u>

The final date for any state failing to submit a program or resubmit an acceptable program before the federal government takes over the permanent administration of the act in that state.

<u>FEBRUARY 3, 1981</u>

All operations must have approved permit under this act.

(From Mining Congress Journal, 1978).

Chapter 9

SUMMARY

In the past, ignorance or disregard of the fundamental principles of engineering have resulted in irreparable damage to the environment and loss of life. The time has long passed when this can be tolerated.

In recent years, stringent laws have been enacted to insure that surface mining will result in as little harm to the environment as is possible. The impact of Public Law 95-87 on the mining industry cannot, at this time, be fully evaluated. There can be little doubt, however, that its implementation will result in some coal reserves not being mined, and the cost of mining those that can be will be increased.

In order to intelligently design a valley fill, a basic understanding of the surface mining process should be acquired. A logical progression in designing a valley fill is as follows: site investigation, site preparation, placement of fill material, configuration of the fill, and surface drainage. With these factors forming a basic framework, a safe, workable, environmentally acceptable fill may be designed and constructed.

During this report, particular attention has been paid to fill properties, and their subsequent ability to resist failure. It has been suggested that adequate stability can be obtained without utilizing concurrent compaction, contrary to the expectations of the federal law. Further investigation in this area is warranted, particularly in areas which have as yet not been investigated; i.e., Virginia and West Virginia. This aspect is given further consideration in the following section "Recommendations for Further Research."

In conclusion, it is believed that this report develops all significant aspects dealing with the design of spoil banks and valley fills for surface mining, in accordance with Public Law 95-87. As stated previously, there are many unknowns in this area, but certainly all that is known should be available to the engineer in order for him to achieve his objectives.

Chapter 10

RECOMMENDATIONS FOR FURTHER WORK

As stated previously, there are many unknowns in the field of soil mechanics. It seems that this is particularly true, with respect to specific applications such as spoil banks and valley fills.

One fundamental aspect of fill design, the strength properties of the fill material, is ignored within the law. That is, no leeway is allowed when a spoil can be shown to have high shear strength characteristics. A general lack of field data may be responsible for this oversight. In addition, those spoil characteristics presented in Chapter 2 were only taken from one foot beneath the surface [14]. However, the study from which this data was taken probably represents one of the most valuable contributions to this subject to date. Nevertheless, it is obvious that material just beneath the surface is hardly representative of the entire fill. A project that involved obtaining cores of the entire depth of existing fills would give more representative results. The samples obtained would allow study of the effect of natural compaction, due to overlying material, on the soils shear strength.

A drilling program would also permit a determination of the height of the phreatic surface, and subsequent degree of saturation; this would give an indication of the effectiveness of the underdrain system used. Without such studies, many questions regarding actual results of regulations concerning underdrains placement and concurrent compaction will continue to be unknown.

Indications are that more research needs to be done in the use of the coagulation-flocculation process, as applied to the removal of sediment from surface mine runoff. This process has been used successfully for many years in municipal water treatment plants, so the problem is in transferring the available technology. Special equipment is needed for efficient and reliable operation under the adverse and primitive conditions that exist at surface mine sites. With this development, the problem of environmental damage resulting from the earthwork involved in surface mining could be objectively solved.

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APPENDIX

AREAS OF ENVIRONMENTAL IMPACT ASSOCIATED WITH HEAD-OF-HOLLOW FILLS

CONSTRUCTION ELEMENTS	ENVIRONMENTAL PROBLEM AREAS	MAGNITUDE OF PROBLEM	SOLUTION TO THE PROBLEM	RELATIVE COST TO ESTIMATE PROBLEM[1]
SITE SELECTION	1. Springs or wet areas within fill zone	Moderate	1. Pick a different hollow. 2. Provide secondary drains to main underdrain	Moderate
			3. Provide rock blanket	High
	2. Steep slopes at proposed fill	Severe	1. Pick a better location 2. Use a keyway cut at toe 3. Install rock buttress at toe 4. Utilize thin lifts and additional compaction	Moderate
	3. Long distances between fill toe and sediment pond	Moderate	Move pond closer to fill toe	Moderate to High
SITE PREPARATION	1. Trees not cut and removed from within fill zone 2. Surface organic material not removed	Severe	Cut and windrow or burn trees.	Low
SEDIMENT POND CONSTRUCTION	1. Construction area around sediment pond not revegetated	Moderate	Grade and seed immediately after construction of pond.	Low
	2. Poor quality control in construction of rock structure	Severe	Closer control of rock size and placement during construction.	Moderate
FILL CONSTRUCTION — WEST VIRGINIA	1. Easily weathered rock used in core drain.	Severe	Closer supervision and rock selection	Low to Moderate
	2. Core clogs at fill bench intercept	Minor	Closer construction supervision	Low
	3. Erosion between sides of fill & original ground	Minor	Blend fill side slope into original surface	Low
	4. Wet areas not drained into core	Severe	Construction of lateral drains to core	High
	5. No upper size limit on core rock	Minor to Severe	Establish upper limit for rock size	Low
FILL CONSTRUCTION — KENTUCKY	1. Easily weathered rock used in underdrain	Severe	Placement of selected rock underdrain	High
	2. Fine grained spoil mixed with underdrain	Severe	Modify current construction practices	High
	3. Insufficient compaction of fill spoil	Moderate to Severe[2]	Modify current construction practices	High
	4. Severe surface erosion	Moderate	1. Divert water away from fill surface 2. Provide a non-erodable surface for water to flow across	Moderate to High
	5. Rill erosion of fill face	Minor	Reduce length of fill face by adding diversion benches	Low

[1]Relative additional cost in relation to current practices [2]Depending on spoil composition and intended land use.

Source: (after Skelly and Loy, 1978).

CHARACTERISTICS OF COMMONLY USED GRASSES[a] FOR REVEGETATION PURPOSES

Common name	Botanical name	Season: Cool	Season: Warm	Season: Dry (not droughty)	Site suitability: Well drained	Site suitability: Moderately well drained	Site suitability: Somewhat poorly drained	Site suitability: Poorly drained	Growth habit[b]	pH range[c]	Use suitability: Erodible areas	Use suitability: Waterways and channels	Use suitability: Agriculture[d]
1. Bahiagrass	*Paspalum notatum*		x	x	x	x			P	4.5-7.5	x	x	x
2. Barley	*Hordeum vulgare*	x			x	x			A	5.5-7.8	x		x
3. Bermuda grass	*Cynodon dactylon*		x	x	x	x	x		P	4.5-7.5	x	x	x
4. Bluegrass, Canada	*Poa compressa*	x		x	x	x			P	4.5-715	x		x
5. Bluegrass, Kentucky	*Poa pratensis*	x			x	x	x		P	5.5-7.0	x	x	x
6. Bluestem, big	*Andropogan gerardi*		x		x	x	x		P	5.0-7.5	x		x
7. Bluestem, little	*Andropogan scoparius*		x		x	x			P	6.0-8.0	x		x
8. Bromegrass, field	*Bromus arvensis*	x			x	x	x		A	6.0-7.0	x		x
9. Bromegrass, smooth	*Bromus inermis*	x	x	x	x	x			P	5.5-8.0	x	x	x
10. Buffalograss	*Buchloe dactyloides*		x			x	x		P	6.5-8.0	x		x
11. Canarygrass reed	*Phalaris arundinacea*	x		x	x	x	x	x	P	5.0-7.5	x	x	x
12. Deertongue	*Panicum clandestinum*		x	x	x	x	x	x	P	3.8-5.0	x	x	
13. Fescue, creeping red	*Festuca rubra*	x		x	x	x	x		P	5.0-7.5	x	x	x
14. Fescue, tall	*Festuca arundinacea*	x			x	x	x		P	5.0-8.0	x	x	x
15. Grama, blue	*Bouteloua gracilis*		x	x	x	x	x		P	6.0-8.5	x		

CHARACTERISTICS OF COMMONLY USED GRASSES[a] FOR REVEGETATION PURPOSES

	Common name	Botanical name	Season			Site suitability				Growth habit[b]	pH range[c]	Use suitability		
			Cool	Warm	Dry (not droughty)	Well drained	Moderately Well drained	Somewhat poorly drained	poorly drained			Erodible areas	Waterways and channels	Agriculture[d]
16.	Grama, sideoats	*Bouteloua curtipendula*		x		x	x			P	6.0-7.5	x		x
17.	Indian grass	*Sorgastrum nutans*		x			x	x		P	5.5-7.5	x		x
18.	Lovegrass, sand	*Eragrostis trichodes*		x		x				P	6.0-7.5	x		x
19.	Lovegrass weeping	*Eragrostis curvula*		x	x	x	x	x		P	4.5-8.0	x		x
20.	Millet, foxtail	*Setaria italica*		x	x	x	x			A	4.5-7.0	x		x
21.	Oats	*Avena sativa*	x		x	x				A	5.5-7.0	x		x
22.	Oatgrass, tall	*Arrhenatherum elatius*	x		x	x				P	5.0-7.5	x		x
23.	Orchardgrass	*Dactylis glomerata*	x		x	x	x	x		P	5.0-7.5	x		x
24.	Redtop	*Agrostis alba*	x		x	x	x	x	x	P	4.0-7.5	x	x	x
25.	Rye, winter	*Secale cereale*	x		x	x	x			A	5.5-7.5	x		x
26.	Ryegrass, annual	*Lolium multiflorum*	x			x	x	x		A	5.5-7.5	x		x
27.	Ryegrass, perennial	*Lolium perenne*	x			x	x	x		P	5.5-7.5	x		x
28.	Sandreed, prairie	*Calamouilfa longifolia*		x	x	x				P	6.0-8.0	x		
29.	Sudangrass	*Sorghum sudanense*		x	x	x	x	x		A	5.5-7.5	x		x
30.	Switchgrass	*Panicum vergatum*		x		x	x	x		P	5.0-7.5	x	x	x
31.	Timothy	*Phleum pratense*	x			x	x	x	x	P	4.5-8.0	x		x
32.	Wheat, winter	*Triticum aestivum*	x		x	x	x	x		A	5.0-7.0	x		x

CHARACTERISTICS OF COMMONLY USED GRASSES[a] FOR REVEGETATION PURPOSES

Common name	Botanical name	Season: Cool	Season: Warm	Season: Dry (Not droughty)	Site suitability: Well drained	Site suitability: Moderately well drained	Site suitability: Somewhat poorly drained	Site suitability: Poorly drained	Growth habit[b]	pH range[c]	Use suitability: Erodible areas	Use suitability: Waterways and channels	Use suitability: Agriculture[d]
33. Wheatgrass, tall	<u>Agriopyron elongatum</u>	x		x	x	x	x	x	P	6.0-8.0	x	x	x
34. Wheatgrass, western	<u>Agropyron smithii</u>	x		x	x	x	x	x	P	4.5-7.0	x	x	x

[a]Grasses should be planted in combination with legumes. Seeding rates, time, and varities should be based on local recommendations.

[b]P = Perennial; A = annual.

[c]Many species survive and grow at lower pH; however, optimum growth occurs within these ranges.

[d]Hay, pasture, green manure, winter cover, and nurse crops are primary agricultural uses.

Note.--Prepared in cooperation with Soil Conservation Service plant material specialists and State conservationists.

Source: After Hittman and Associates.

REMARKS: (APPENDIX I):

1. Tall, extensive root system. Maintained at low cost once established. Able to withstand a large range of soil conditions. Scarify seed.
2. Cool season annual. Provides winter cover.
3. Does best at a pH of 5.5 and above. Grows best on well drained soils, but not on waterlogged or tight soils. Propagated vegetatively by planting runners or crowns.
4. Does well on acid, droughty, or soils too low in nutrients to support good stands of Kentucky bluegrass.
5. Shallow rooted; best adapted to well-drained soils of limestone origin.
6. Strong, deep rooted, and short underground stems. Effective in controlling erosion.
7. Dense root system; grows in a clump to 3 feet tall. More drought tolerant than big bluestem. Good surface protection.
8. Good winter cover plant. Extensive fibrous root system. Rapid growth and easy to establish.
9. Tall, sod forming, drought and heat tolerant. Cover seed lightly.
10. Drought tolerant. Withstands alkaline soils but not sandy ones. Will regenerate if overgrazed.
11. Excellent for wet areas, ditches, waterways, gullies. Can emerge through 6 to 8 inches of sediment.

12. Very acid tolerant; drought resistant. Adapted to low fertility soils. Volunteers in many areas. Seed not available.

13. Grows in cold weather. Remains green during summer. Good seeder. Wide adaptation. Slow to establish.

14. Does well on acid and wet soils of sandstone and shale origin. Drought resistant. Ideal for lining channels. Good fall and winter pasture plant.

15. More drought resistant than sideoats grama. Sod forming. Extensive root system. Poor seed availability.

16. Bunch forming; rarely forms a sod. May be replaced by blue grama in dry areas. Feed value about the same as bit bluestem. Helps control wind erosion.

17. Provides quick ground cover. Rhizomatous, tall. Seed available.

18. A bunchgrass of medium height. Adaptable to sandy sites. Good for grazing. Fair seed availability.

19. Bunchgrass, rapid early growth. Grows well on infertile soils. Good root system. Low palatability. Short-lived in Northeast.

20. Requires warm weather during the growing season. Cannot tolerate drought. Good seedbed preparation important.

21. Bunch forming. Winter cover. Requires nitrogen for good growth.

22. Short-lived perennial bunchgrass, matures early in the spring. Less heat tolerant than orchardgrass except in Northeast. Good on sandy and shallow shale sites.

23. Tall-growing bunchgrass. Matures early. Good fertilizer response. More summer growth than timothy or bromegrass.

24. Tolerant of a wide range of soil fertility, pH, and moisture conditions. Can withstand drought; good for wet conditions. Spreads by rhizomes.

25. Winter hardy. Good root system. Survives on coarse, sandy spoil. Temporary cover.

26. Excellent for temporary cover. Can be established under dry and unfavorable conditions. Quick germination; rapid seeding growth.

27. Short-lived perennial bunchgrass. More resistant than weeping love or tall oatgrass.

28. Tall, drought tolerant. Can be used on sandy sites. Rhizomatous. Seed availability poor.

29. Summer annual for temporary cover. Drought tolerant. Good feed value. Cannot withstand cool, wet soils.

30. Withstands eroded, acid and low fertility soils. Kanlow and Blackwell varieties most often used. Rhizomatous. Seed available. Drainageways, terrace outlets.

31. Stands are maintained perennially by vegetative reproduction. Shallow, fibrous root system. Usually sown in a mixture with alfalfa and clover.

32. Requires nutrients. Poor growth in sandy and poorly drained soils. Use for temporary cover.

33. Good for wet, alkaline areas. Tolerant of saline conditions. Sod forming. Easy to establish.

34. Sod forming, spreads rapidly, slow germination. Valuable for erosion control. Drought resistant.

SUMMARY OF CHEMICAL BINDERS AND TACKS

Name	USES: Temporary soil stabilizer	USES: Mulch	USES: Mulch tack	Description	Application method	Manufacturer or product information
AEROSPRAY R 52 BINDER	X	X		Water dispersible, alkyd emulsion. Nontoxic. Nonphytotoxic. pH 8-9	Any nonair entraining equipment (as for liquid fertilizer, asphalt emulsions, and water).	American Cyanimid Company Industrial Chemicals and Plastics Division Wayne, New Jersey 07970
AEROSPRAY R 70 BINDER	X	X	X	Water dispersible, liquid polyvinyl acetate emulsion.	Hydroseeder. Seed, fertilizer, and wood fiber may be applied with product.	American Cyanimid Company Industrial Chemicals and Plastics Division Wayne, New Jersey 07970
AEROSPRAY R 72 BINDER	X	X	X	Water dispersible, liquid alkyd emulsion resin.	Hydroseeder. Seed, fertilizer, and wood fiber may be applied with product.	American Cyanimid Company Industrial Chemicals and Plastics Division Wayne, New Jersey 07970
AQUATAIN	X	X		Water dispersible. Nontoxic. Nonflammable.	Hydroseeder or any nonair entraining equipment. Seed and fertilizer may be applied with product.	The Larutan Corporation 1424 South Allec Avenue Anaheim, California 92805
CURASOL R AE	X	X	X	Water dispersible, polyvinyl acetate copolymer emulsion. Nontoxic. Nonphytotoxic. pH 4-5	Hydroseeder or any nonair entraining equipment.	American Hoechst Corporation 1041 Route 202-206 North Bridgewater, New Jersey 08876
CURASOL R AH	X		X	Water dispersible, high polymer synthetic resin. Nontoxic. Nonphytotoxic. pH 4-5.	Hydroseeder or any nonair entraining equipment. Seed and fertilizer may be applied with product.	American Hoechst Corporation 1041 Route 202-206 North Bridgewater, New Jersey 08876
DCA-70	X	X	X	Water dispersible, polyvinyl acetate emulsion. Nontoxic. Nonphytotoxic. Nonflammable. pH 4-6.	Hydroseeder or any nonair entraining equipment	Union Carbide Corporation Chemicals and Plastics 270 Park Avenue New York, New York 10017
GENEQUA 169	X			Water dispersible, modified liquid acrylic resin.	Hydroseeder. Seed, fertilizer, and wood fiber may be applied with product.	The Delta Company Charleston, West Virginia
LIQUID ASPHALT			X	Asphalt cement that is dispersed or suspended in water or various solvents.	Hand-spray nozzle or an offset distributor bar attached to an asphalt distributor truck	Asphalt Institute Asphalt Institute Building University of Maryland College Park, Maryland 20740

Name	USES: Temporary soil stabilizer	Mulch	Mulch tack	Description	Application method	Manufacturer or product information
M-145	X			Water dispersible, liquid resin polymer.	Hydroseeder. Seed and fertilizer may be applied with product.	The Dow Chemical Company Midland, Michigan 48640
PETROSET[R] SB	X	X	X	Water dispersible oil emulsion. Nontoxic. Nonflammable. pH 6.0 ± 0.5	Any spraying equipment.	Phillips Petroleum Company Chemical Department Commercial Development Div'n. Bartlesville, Oklahoma 74003
TERRA TACK	X	X	X	Water dispersible, powdered vegetable gum.	Hydroseeder or, for dry application, standard hopper spreaders (as for fertilizers or lime).	Grass Growers, Inc. P.O. Box 584 Plainfield, New Jersey 07061
UREA-FORMALDEHYDE FOAM		X		Urea-formaldehyde resin plus a foaming agent, mixed and foamed with compressed air, then applied to soil. Wetting agent is then applied to the foam. Seed and fertilizer sprayed on top.	Nonair entraining equipment for resin and foam. Hydroseeder for seed and fertilizer.	U.F. Chemical Company 37-20 58th Street Woodside, New York 11377
XB-2386	X			Water dispersible, liquid reactive polymer.	Injected into slurry at the nozzle of a hydroseeder.	3M Company Adhesives Coatings and Sealers Division, 3M Center Saint Paul, Minnesota 55101

Source: Hittman Associates, 1976.