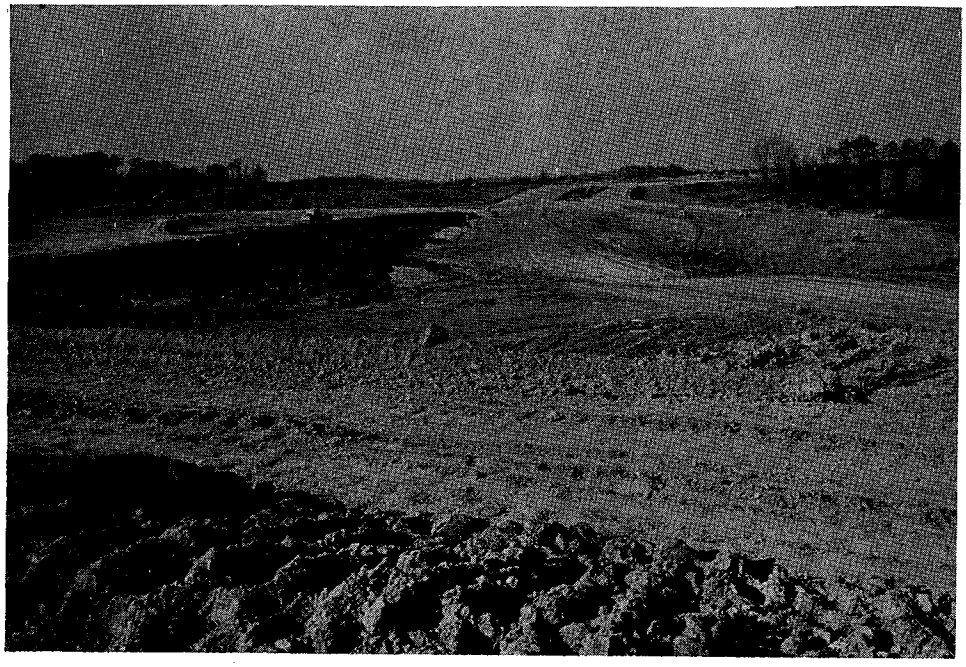


FIGURE 6

A LAYER OF SPOIL IS BEING SPREAD ON THE EMBANKMENT

Embankment Placement and Contractor Coordination

From a practical standpoint, the two most troublesome characteristics of the spoil during construction were its heterogeneous nature and its drastic deterioration in workability when wet.

Because of the variability of coal content within the spoil piles, the contractor found that some pans from these piles contained little or no coal or carbonaceous shale. However, rather than deciding pan by pan, which would be considered spoil or native material, it was agreed that all material obtained from these areas would be treated uniformly as spoil when spread on the embankment. This prevented many arguements over the course of the contract, and simplified handling and placement procedures as well.

Initially, the contractor experimented with several patterns for laying down, spreading and mixing spoil and native material. It was determined that the most efficient method was simply to place a layer of native material over a layer of spoil and mix them on grade. Figure 6 shows a layer of spoil being spread. By varying the number of pans (CAT 651s and 641s) hauling from the spoil piles, and pan hauling native material, the 5 percent coal control criteria could be met. Mixing was performed on grade by dozers (CAT D-8s) and 30-ton sheepsfoot rollers (CAT 824) were used for compaction.

The 5 percent limit on coal content in the embankment mixture was also a source of continued debate. Grading foremen often did not agree with the inspector's complaint that the coal content was too high. The hand separation method was used in conjunction with jar samples to determine the amount of coal. In cases where coal content was still too high after mixing on grade, additional loads of native material were spread and scarified. The mixture was then recompacted.

Experience showed that the ability to work with spoil decreased dramatically as its moisture content increases above optimum. A supply of shot rock was maintained for mixing when wet pockets of spoil were encountered. This allowed continuous placement. However, under wet weather conditions, the use of mine spoil was suspended and native materials were used exclusively. This proved to be an efficient system not only for control but also from the contractor's standpoint.

The contractor was also required to have sufficient native soil and rock available for the top 20 feet of the embankment. Quantitatively this created no great problem since spoil composed only about 10 percent of the total excavation, but logistically, it required close coordination. It was important to have that material available when needed and this was resolved by continued re-evaluation and adjustment of the scheduled work.

There were several other desirable effects caused by mixing the spoil with native materials. The variability of compaction control densities were placed more in line with those for the natural soils. The in-situ moisture content of the spoil could also be adjusted so that it could be placed immediately without the need for scarifying and air drying.

Conclusion

The most notable aspects of this project were the control and coordination measures used to place strip mine refuse within this embankment. None of the spoil was wasted, and the quality of the embankment was not affected. The unit cost for embankment-in-place was $1.15 per cubic yard, which was comparable to prices received on nearby projects not having mine spoil. In addition, the overall balance of available embankment materials has been maintained or improved for future grading projects at GPIA.

The minimum required thickness of surface cover was reduced for infield areas on the next contract because of the excellent results which were being obtained when using the spoil. It is anticipated that these reductions will be made for subsequent embankments.

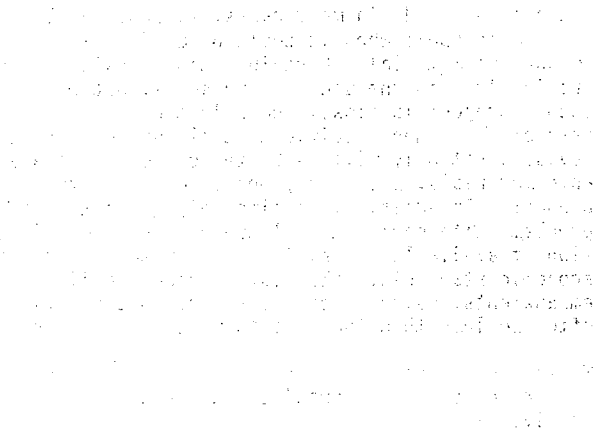
There is no need, in many cases, to arbitrarily waste mine spoil when it could be used an an embankment material. Certainly many considerations are involved in the use of mine spoil, but when such a project is closely coordinated with the contractor, and when specific control methods can be devised, not only will we better conserve our available materials, but we may end up with a better product. In addition, wasting only tends to shift problems with environmental control and the utilization of available materials to the future. From an economic standpoint, the cost to place spoil in embankments, using special care and control, will often be less than the cost to waste and replace it.

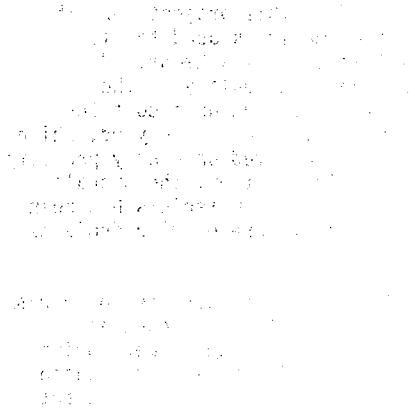
We should be especially sensitive to these factors as our sources of material dwindle and become more costly.

BIBLIOGRAPHY

(1) Brant, R. A., "Geological Description and Effects of Strip Mining on Coal Overburden Material," Ohio Journal of Science, Volume 64, No. 2, March, 1964, Pages 65-175.

(2) "2400 Series COMPAC Surface, Moisture-Density Gauge," March, 1972, Page 9, prepared by and published for Troxler Electronic Laboratories, Research Triangle Park, North Carolina.





OBSERVATION EVALUATIONS OF COAL REFUSE EMBANKMENT STABILITY

Wendell W. Parker and Richard E. Gray

SYNOPSIS

The stability of coal refuse embankments is dependent upon many factors. Included among these are the steepness of side slopes, strength of embankment materials, surface of seepage location, variability or zoning of embankment materials and embankment maintenance. Many of these factors can be qualitatively evaluated by direct observation. This paper discusses the visual evaluation of factors affecting the stability of coal refuse embankments. In addition, methods of quantitatively evaluating the stability of coal refuse embankments using data from approximate field measurements of physical properties such as slope angles are presented. Also included is a discussion of observed coal refuse disposal area construction and operation methods that are conducive to the development of unstable embankments.

INTRODUCTION

Coal refuse embankments have often been constructed with little engineering control. Coal refuse exhibits variable composition and, in addition, many coal refuse embankments are constructed randomly with significant amounts of other materials. Thus, engineering properties normally used to predict the stability of slopes may show large variations within an embankment. The quantitative evaluation of the stability of such embankments requires an appreciable amount of engineering work to determine the condition of the embankments interior and its associated engineering properties. However, the stability of coal refuse embankments can be qualitatively evaluated with some degree of reliability from field observations. In addition, rough quantitative estimates of stability can be made using approximate solutions. This paper discusses the qualitative evaluation of embankment stability based on visual field observations and the quantitative evaluation of embankment stability using approximate solutions.

This paper is based on work conducted in conjunction with the inspection and evaluation of 179 coal refuse disposal sites in Pennsylvania and the review and evaluation of about 40 coal company proposals for construction or modification of coal refuse disposal areas in Pennsylvania, Virginia, West Virginia and Kentucky. The qualitative and quantitative methods of stability evaluation discussed were used during the inspection and evaluation of these coal refuse disposal sites and the review and evaluation of coal company proposals. The site inspections and evaluations were oriented to embankment failure and the downstream flood and damage potential of any retained water or sediment. The review and evaluation of coal company proposals considered the overall engineering acceptability of the proposed construction.

OBSERVATIONAL STABILITY EVALUATIONS

The slope and crest surfaces of embankments often exhibit characteristics that are indicative of overall instability. However, similar characteristics are sometimes caused by other than unstable conditions. Also, observed characteristics are sometimes due to surficial instability with the major portion of the embankment being stable. Therefore, the use of observable surficial characteristics to establish the qualitative stability of embankments should be done with caution and by competent personnel. A list of embankment characteristics indicative of instability is presented in Table 1.

TABLE 1

EMBANKMENT CHARACTERISTICS INDICATIVE OF INSTABILITY

a) Embankment Crest and/or Slope Cracks

b) Embankment Crest and/or Slope Irregularities

c) Toe or Embankment Erosion

d) Seepage and Soft and/or Wet Areas

e) Zoning of Embankment Materials

f) Past or Current Landslide Scars

g) Evidence of Embankment Repairs

h) Malfunctioning Hydraulic Facilities

i) Toe and/or Foundation Bulges

j) Slope Angles and Embankment Material Types

k) Tree or Vegetation Attitude

Embankment crest and/or slope cracks often occur due to tension relief within the embankment. These cracks result in a net reduction of embankment stability which could propagate into a major failure problem. However, embankment crest and/or slope cracks may result from drying and shrinking of surface materials. In addition, the material on embankment slopes is often more loose than interior embankment material and the differential settlement of this loose material on embankment slopes often results in cracks at or near the crest of the embankment. Thus, embankment crest and/or slope cracks may not be indicative of overall embankment instability.

Embankment crest and/or slope irregularities are often a sign of haphazard construction. Thus, a weak zone may have been inadvertently included in the embankment. Irregularities may also indicate some past movement or sliding of embankment materials. However, embankment irregularities may only be surficial features unrelated to embankment stability. Thus, such irregularities should be used cautiously when qualitatively evaluating embankment stability.

Toe or embankment erosion tends to locally weaken the integrity of the embankment. Such localized weakness could propagate into a major embankment instability. Therefore, the repair and prevention of erosion is an important part of a good embankment maintenance program.

Seepage of water through an embankment has a direct effect on embankment stability. Embankment material beneath the surface of seepage is subjected to both bouyant and seepage forces which result in a net reduction of forces resisting sliding. Wet areas at the embankment toe and/or on the embankment slope generally indicate the location of the surface of seepage. (See Figure 1.) However, the common practice of constructing the embankment in stages often results in preferred zones of seepage being constructed into the embankment. Thus, wet areas may be due to seepage through porous and localized zones or sections of the embankment. In either case, the seepage usually results in the development of a weak area in the embankment that could propagate by sloughing and/or piping into a major instability problem. Therefore, wet areas on the embankment may be considered as potentially weak areas.

The past stability performance of the embankment is often indicated by scars remaining from past or current sloughing or sliding of materials. These movements generally result in oversteeping of the slope which could result in a major instability problem. The removal of slope materials also may cause a reduction in the length of the seepage path through the embankment with a decrease in embankment stability. Although landslide scars may be due to a surficial condition, they are often indicative of major embankment instability.

Embankment repairs are often evidenced by patches or zones composed of material differing in appearance from the general embankment materials. This difference in appearance is usually discernable regardless of the age of the repair work. Embankment repairs indicate past performance and may not be indicative of current weakness or instability. However, embankments that required past repairs need to be carefully evaluated relative to future performance.

Hydraulic control facilities that remove water from behind embankments that impound water often deteriorate with time and develop significant leaks. Such leaks result in the internal charging of the embankment with water which reduces the forces resisting sliding. A comparison of the amount of water entering hydraulic control facilities to that leaving is a good check for significant leaks. Leaks from hydraulic control facilities cause a localized weakening of embankment strength that could, if not repaired, result in a major instability problem.

Bulges in the embankment toe area are often indicative of some material movement within or beneath the embankment. Tension cracks in the embankment crest and/or slope generally accompany any significant bulges. However, many embankments are under continuous construction as coal waste material becomes available during operation of the mine or processing plant. Fresh material being placed continuously on the embankment crest tends to mask any tension

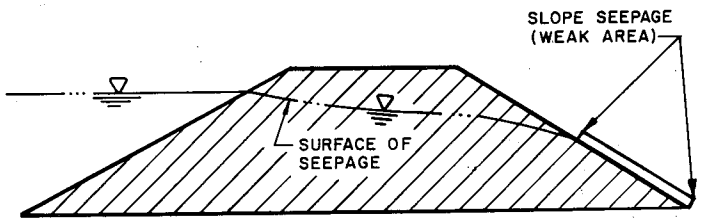


Figure 1. Surface of Seepage in a Uniform Embankment.

cracks that develop. Construction of embankments is sometimes performed in a haphazard fashion and bulges are constructed into the embankment. Therefore, embankment bulges are not positive signs of embankment instability, however, they should be viewed with suspicion.

Coal refuse embankments have been and are constructed using many different methods. The methods used in the past have often resulted in the coal refuse being placed at its angle of repose. Coal refuse being generally granular in nature tends to exhibit surficial ravelling and sloughing when placed at its angle of repose. An embankment at its angle of repose has a factor of safety for stability of approximately 1.0 near the slope surface with the factor of safety increasing with depth into the embankment. This assumes there is no significant change in material type with depth, no significant effect from seepage, and the foundation has sufficient strength to support the embankment. A weak foundation, variable embankment materials, a high surface of seepage coupled with steep slopes could reduce the general factor of safety for the overall embankment to less than 1.0 and result in failure. Therefore, embankments constructed with material placed at the angle of repose should be closely inspected relative to the detection of signs of internal instability.

The attitudes of trees and tall vegetation on the embankment are good indicators of instability. Embankments that have experienced movement often contain trees or tall vegetation that deviate from their normal vertical position. The attitudes of trees or tall vegetation should be evaluated along the entire length and width of the embankment in an effort to locate all unstable areas.

The visual inspection of embankments to determine their qualitative stability should be done with caution. Many of the instability signs discussed above may be indicative of minor and localized problems. Although minor and localized instability often develops into a serious problem, an embankment may reach a stable configuration through one of the following:

a) The embankment may become stable after sliding if the movement has alleviated the cause of the instability.

b) The condition(s) causing instability may no longer exist.

c) A combination of a) and b).

Numerous examples of the above situations come to mind, however, only two are discussed here due to space limitations.

a) (Example of a). A stable embankment composed of coal refuse is often blanketed with a layer of soil to support vegetation and to improve appearance. This blanket of soil may be placed in a loose condition and eventually slide down the slope of the embankment. However, the remaining embankment material, after the blanket cover has been removed by sliding, will be in a stable condition.

b) (Example of b). An embankment with an unstable downstream slope caused by a high surface of seepage may have been stabilized by lowering the surface of seepage level thru installation of a rock toe drain.

The signs of instability will remain for these two examples although the underlying cause(s) have been removed and the overall embankment is stable.

QUANTITATIVE STABILITY ESTIMATES

The stability of coal refuse embankments can be qualitatively evaluated with some degree of reliability from field observations. However, no quantitative evaluation, other than when an embankment exhibits surficial instability indicating a factor of safety for stability of approximately 1.0, can be made using visual observations. However, it is often possible to conclude from visual observations that embankment stability is significantly greater than one.

The stability of coal refuse embankments can be quantified using approximate methods such as stability charts and stability coefficients. Approximate methods have been developed by Taylor[1], Bishop and Morgenstern[2], Spencer[3] and others. However, design charts prepared by Hoek[4,5] are preferred by the authors for a quick estimate of the factor of safety of coal refuse embankments. The Hoek design charts are discussed in detail below.

Embankment dimensions can be suitably established for quantitative stability analyses using approximate field measurement techniques. Short distances, such as crest width, may be measured by pacing. Longer distances, where necessary, may be measured using a hand-operated range finder. Slope angles may be measured using a Brunton compass and heights of less than 25 feet may be measured using a Locke level. A precision altimeter may be used to determine heights in excess of 25 feet.

The slope failures depicted by the Hoek charts are classified as either plane failure or circular failure. The circular failure fits essentially all of the coal refuse embankments reviewed by the authors. Therefore, only the circular failure method of estimating

stability is discussed. It is emphasized that the stability analysis described is approximate and should be used only where an approximate analysis of stability is acceptable.

Hoek developed the stability charts with the factor of safety presented as functions of dimensionless slope parameters X and Y for circular failure surfaces in uniform slopes with various water and tension crack conditions. The parameter X reflects water conditions within the embankment by one of three idealized cases:

a) a drained embankment with no water;

b) a normal drawdown condition with the surface of seepage at a known height; and

c) a horizontal water table at a known height.

The parameter Y reflects tension crack conditions in the embankment crest by one of three idealized cases:

a) no tension crack;

b) a dry tension crack of known depth; and

c) a water filled tension crack of known depth.

The parameter X as a function of the internal angle of friction, ϕ, of the embankment material, total height of the embankment, water height in the embankment, and the slope angle is presented in Figure 2. The parameter Y is also presented in Figure 2 as a function of the cohesion, c, of the embankment material, the unit weight, γ, of the embankment material, depth of tension crack, total embankment height, and the slope angle, i. The ground-water level is reasonably assumed to be at a height H_w above the toe of slope at a distance of four times the total height of the embankment behind the toe of slope. The stability chart that relates the factor of safety for stability to parameters X and Y is presented in Figure 3. Additional details relative to the development of the stability chart are presented by Hoek in References 4 and 5.

Factors required to calculate parameters X and Y, other than material strength and density, are readily obtainable in the field using the measurement techniques discussed above. Typical strength and density values for coarse coal refuse are presented in Table 2. Typical strength and density values for soil types other than coal refuse are available in most soil mechanics texts. The establishment of strength and density values based on literature sources requires an accurate classification of soil types in the field. The Unified Soil Classification System is preferred.[6] Table 3 contains an example showing how the stability chart is used.

The average slopes of over 200 existing coal refuse embankments that were inspected at 179 coal refuse disposal areas varied from 30 to 37 degrees. These slope angles were angles of repose in many cases. Assuming these values are rough approximations of the internal angle of friction of coal refuse, no seepage or water buoyant forces are present and the underlying foundation material has adequate strength to support the embankment, the slope required for an embankment to have the normally required minimum factor of safety[8] of 1.5 can be established from the following equation:

$$F_s = \frac{\tan \phi_s}{\tan \phi_\beta} \qquad (1)$$

where

F_s = factor of safety for stability,

ϕ_s = internal angle of friction for embankment material, and

ϕ_β = embankment slope angle.

Based on the data and equation above, the embankment slope, on the average, necessary to obtain a satisfactory factor of safety for stability of 1.5 under the above assumptions should range between 22 and 27 degrees.

DETRIMENTAL EMBANKMENT CONSTRUCTION AND OPERATION PRACTICES

A variety of detrimental embankment construction and operation practices were observed during the site inspections of coal refuse disposal areas. These detrimental practices generally result in higher long-term maintenance costs and some dangerously weaken the embankments.

Many fine coal refuse disposal ponds are periodically cleaned by removing accumulated fine coal sediment. Some cleaning crews significantly weaken the embankment by removing portions of the interior of the embankment slope during cleaning operations. This results in oversteepening of the interior embankment slope and in a significant reduction in the length of flow path of water through the embankment which increases seepage.

Impounding facilities are often modified by coal companies to increase retention capacities. Retention capacity is sometimes increased by raising the embankment height by filling on the upstream side of the embankment. This added material is usually founded on saturated fine coal sediment. Resulting settlements may crack and weaken the embankment. A low shear strength in the saturated fine coal sediment may also

result in the overall embankment becoming unstable. Thus, the performance of the total embankment may be placed in jeopardy as the impoundment level rises to a new normal level of operation. However, this upstream filling method of embankment modification is acceptable if the design adequately accounts for settlement and low strength of the fine coal refuse.

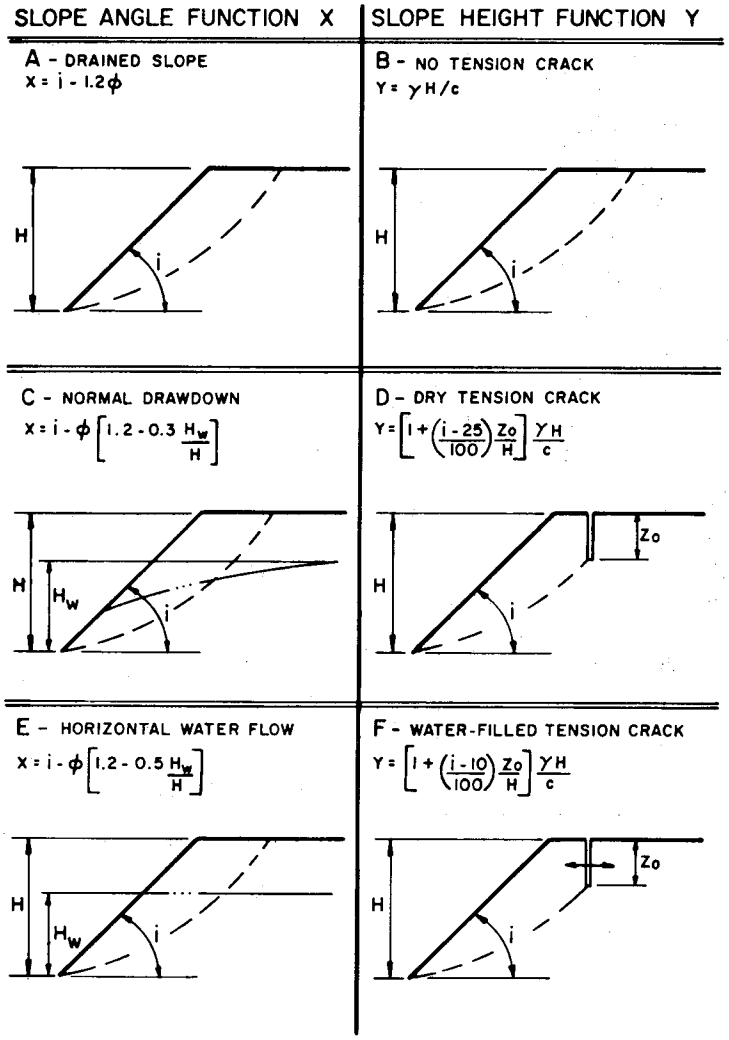


Figure 2. Slope Angle Functions X and Y (after Hoek).

Fine coal refuse slurry from coal washing plants is usually discharged into a sedimentation impoundment via a pipe. Such pipes are often allowed to discharge on the upstream slope and/or crest of the embankment resulting in extensive erosion which weakens the embankment.

Many coal refuse disposal areas are operated without any formal inspection and maintenance program. In addition, some impoundments have been abandoned with no provisions for inspection or maintenance and the facilities have not been properly prepared for abandonment. Thus, impoundment outlet works are allowed to partially or completely clog which may result in the embankment being overtopped in major storms. In a few cases, embankment crest and slope erosion due to minor overtopping and the discharge of abutment runoff onto the crest has dangerously weakened embankments.

The downstream toe of some embankments have been removed to allow for the construction of access roads and small structures, such as pumphouses. This practice usually results in a significant reduction in embankment stability.

In many cases, embankments are constructed with a flagrant lack of construction control. This results in a poorly compacted and layered or zoned embankment which exhibits localized weak areas and preferred zones of seepage that can result in embankment instability.

Many coal companies use a series of sedimentation impoundments to clarify preparation plant discharge and/or treated acid mine drainage water. Initially, the effluent from the preparation or treatment plant is discharged into the uppermost impoundment in the series. The effluent from the impoundment is then allowed to flow successively from one impoundment to another until the lowermost impoundment is reached. The discharge from the lowest impoundment in the series is then

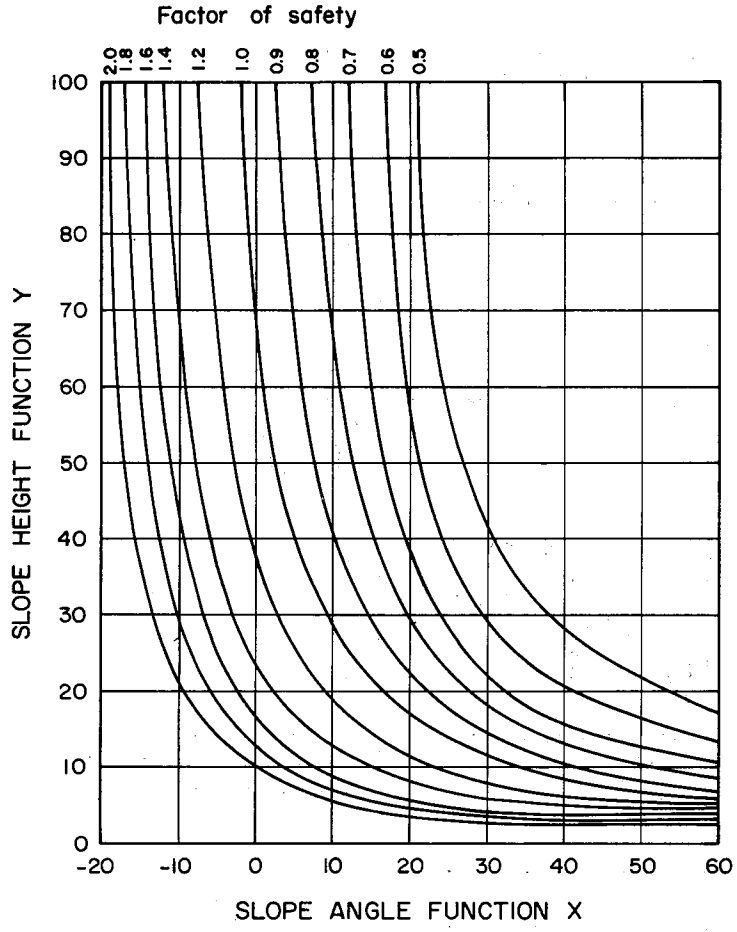


Figure 3. Slope Design Chart (after Hoek).

Table 2

TYPICAL STRENGTH AND DENSITY VALUES FOR COARSE COAL REFUSE

Item	Density (pcf)	Strength	
		Internal Angle of Friction (degrees)	Cohesion (psi)
Typical range of values[7]	80-110	33-40	1-2
Case 1*	95	27	2
Case 2*	95	33	0

*Two cases of recommended average values. Factor of safety should be determined for both cases. If the lower factor of safety is unacceptably low, a detailed analysis of embankment stability is recommended.

allowed to enter natural drainage channels. These impoundments are often constructed such that the pools of the lower impoundments inundate the toe areas of the upper impoundments. Also, water is often allowed to pond at the toe of the embankments. Saturation of the toes of embankments results in a significant strength loss and may cause local instability or affect general embankment stability.

Some embankments constructed predominantly of relatively pervious coarse coal refuse are covered on their downstream side with a blanket of relatively impervious clay. The relatively pervious refuse permits the development of a high surface of seepage in the embankment which applies an appreciable amount of pressure on the impervious blanket greatly reducing its stability.

Table 3

EXAMPLE OF
HOEK CHART USAGE

Embankment Condition

Embankment with normal surface of seepage drawdown and a dry tension crack.

Embankment Properties

Total embankment height, H, of 50 feet.

Slope angle, i, of 30 degrees.

Dry tension crack depth, Zo, of 1 foot.

Maximum surface of seepage depth above toe, H_w, of 35 feet.

Strength parameters from Table 2:

Case 1: γ = 95 pcf, ϕ = 27° and C = 2 psi

Case 2: γ = 95 pcf, ϕ = 33° and C = 0 psi

Slope Angle and Height Functions (Figure 2)

$$X = i - \phi\,[1.2 - 0.3\,H_w/H]$$

$$Y = 1 + [(i - 25)/100]\,[Zo/H]\;\gamma H/C$$

Case 1: $X = 30 - 27\,[1.2 - 0.3\,(35/50)] = 3.3$

$$Y = 1 + [(30 - 25)/100][1/50]\,(95)(50)/288 = 16.5$$

Case 2: $X = 30 - 33\,[1.2 - 0.3\,(35/50)] = -2.7$

$$Y = 1 + [(30 - 25)/100][1/50]\,(95)(50)/0 = > 100$$

Factors of Safety (Figure 3)

Case 1: F.S. = 1.5

Case 2: F.S. = 1.0 (unacceptable)

Conclusion: A detailed embankment stability analysis is required.

Embankment areas that exhibit instability are often masked by covering the slide mass with additional material. This results in continued instability by loading the slide mass and blockage of seepage and increased applied water

pressure.

A large majority of embankments impounding water are constructed without emergency spillways and diversion ditches. Thus, the primary outlet works are of extreme importance and must carry all flow that enters the impoundment in excess of storage capacity. However, many primary outlet works have not been designed or constructed to carry such flows.

SUMMARY

Field observations can be used with some degree of reliability to evaluate the qualitative stability of coal refuse embankments. A visual evaluation in the field of the physical items listed in Table 1 should provide sufficient data to indicate if an embankment is marginally stable. It is emphasized that no quantitative evaluation, other than when an embankment exhibits marginal stability (factor of safety for stability of approximately 1.0), can be made using visual observations. However, the stability of embankments may be approximated using stability charts and stability coefficients. Stability charts prepared by Hoek[4,5] are considered to be especially applicable for rapid evaluation of coal refuse embankments.

Coal mine owners and operators often use construction and operation methods detrimental to the long-term, and sometimes, short-term performance of embankments. The adequate design and control of embankment construction by a competent engineer experienced in the design and construction of earth and earth-rock dams, coupled with a good maintenance program, will significantly reduce the hazard from coal refuse embankments.

REFERENCES

1. Taylor, D. W., Fundamentals of Soil Mechanics, John Wiley and Sons, Inc., New York, 1948, pp. 455-462.

2. Bishop, A. W. and Morgenstern, N. R., "Stability Coefficients for Earth Slopes," Geotechnique, 1960, Volume 10, pp. 129-150.

3. Spencer, E., "A Method of Analysis of the Stability of Embankments Assuming Parallel Inter-Slice Forces," Geotechnique, 1967, Volume 17, pp. 11-26.

4. Hoek, E., "Estimating the Stability of Excavated Slopes in Open-Cast Mines," Transactions, Institution of Mining and Metallurgy, London, 1970, Volume 79, pp. A109-A132.

5. Hoek, E., "Design Charts for Excavated Slopes," Proceedings, Symposium on Open-Pit Mine Slope Stability, Johannesburg, 1970.

6. United States Bureau of Reclamation, Earth Manual, First Edition Revised, 1968, pp. 1-47.

7. United States Department of the Interior, Mining Enforcement and Safety Administration, Engineering and Design Manual Coal Refuse Disposal Facilities, undated, pp. 5.76 - 5.116.

8. Parker, W. W. and Gray, R. E., "Making the Grade in Coal Refuse Disposal," Coal Mining and Processing, Volume 12, No. 11, November 1975.

CONVENTIONAL AND UNCONVENTIONAL APPROACHES USED TO LOCATE AND ELIMINATE HAZARDOUS MINE WASTE DAMS IN OHIO

By

Vikram V. Rajadhyaksha, M.S., P.E.
Supervisor, Dam Inspection
Division of Water
Ohio Department of Natural Resources

SYNOPSIS

Immediately after the Buffalo Creek disaster in West Virginia in 1972, a joint Federal-State Task Force was convened under the direction of the U.S. Army Corps of Engineers to study the problem of hazardous coal waste dams in the coal mining states. The Task Force was directed to locate and identify potentially hazardous dams and embankments and make recommendations to the individual states for follow-up action to eliminate the hazardous conditions.

Mr. Rajadhyaksha was both a member of the Task Force and leader of the State team which implemented the Task Force recommendations. This paper describes the problems encountered first in locating and identifying the unsafe dams and then in working with the coal companies to eliminate the potentially hazardous conditions. Three case histories are presented with the aid of photographs covering the period from location to the time the hazardous conditions were eliminated.

INTRODUCTION

On February 26, 1972 the coal waste embankment on Middle Fork, a tributary of Buffalo Creek, West Virginia, failed, causing the death of 125 people and millions of dollars of property damage. Within 60 days of this disaster a joint Federal-State Task Force was convened under the direction of the U.S. Army Corps of Engineers to locate and identify hazardous coal mine refuse piles and impoundments, and bring this information to the attention of local officials with recommended remedial measures. For operational purposes, each state was assigned to a Corps District. The Ohio investigation was directed by the Pittsburgh District. Other Federal agencies cooperating in the study were the Soil Conservation Service, the Geological Survey and the Bureau of Mines. The State of Ohio was represented by the Division of Water, Department of Natural Resources.

A field team was organized in each state to identify and visually inspect all the potentially hazardous coal waste embankments in that state. The team in Ohio consisted of a representative each from the U.S. Army Corps of Engineers, the Ohio Division of Water and the U.S. Soil Conservation Service. Reports of the field team's inspections were reviewed by a Board of Review which recommended appropriate follow-up action to be taken by the state. The Board of Review included representatives from all of the cooperating agencies.

PROBLEMS WITH LOCATING EMBANKMENTS

To begin the study, information concerning the location and types of waste embankments was obtained from the U.S. Bureau of Mines. This list contained 13 coal waste embankments in Ohio of which only two impounded any water. However, the field team quickly learned that many embankments had not been included in the original list. The problem then became how to locate for inspection all the "unknown" coal waste embankments. A few embankments were located by scanning the U.S.G.S. 7.5 minute topographic maps of Eastern Ohio. Insufficient map detail in mined areas plus considerable mining activity which had occurred since the maps were compiled severely limited the number of waste piles and impoundments found on the U.S.G.S. maps. An attempt to use ERTS satellite imagery to locate the embankments also proved unsuccessful.

It was finally decided to personally interview officials of all the coal mining companies active in the area, explain the field team's mission, and ask for the locations of waste embankments in their area. At first it was feared that this approach would not work because the mining companies might not be willing to divulge such problematic information to government agents. However, this concern proved to be ill founded. The first few meetings with coal company officials revealed that the companies were just as, or perhaps more, concerned than the field team about having a Buffalo Creek event in their workings. The coal companies gave excellent cooperation in locating the embankments. The study located and inspected 56 embankments. Of these, 13 were declared potentially hazardous by the Board of Review. Recommendations were made to the State for follow-up action. In spite of this approach a few additional embankments were found by the State after the initial phase of field work was completed.

CASE HISTORIES

Three of the embankments found to be potentially hazardous are described along with the approaches used to correct the hazardous conditions.

1. Willow Grove No. 2

Willow Grove No. 2 was a 130-foot-high coal-waste (gob) pile across Kings Run, a tributary of Little McMahon Creek in Belmont County, Ohio. The drainage area at the site was about 680 acres. The surface area of the pond varied with pool fluctuations but was about 5 acres at the overflow or "spillway" level. The small community of Echo was located about 2000 yards downstream on the banks of Little McMahon Creek. The larger community of Neffs was located another mile farther downstream. Figure 1 shows a location sketch for both Willow Grove No. 2 and Willow Grove No. 3. The field team's initial inspection noted that sudden failure of this dam during a general flood would obviously cause significant property damage and loss of life in both Echo and Neffs.

Figure 2 shows an overall view of the lake and the embankment from the upstream side. Note that the only "spillway" is a 10-foot-wide channel at the left end of the embankment. Generally this "spillway" functioned only during times of high flows. The pool level was normally several feet below the spillway due to seepage through the dam which emerged on the downstream slope. Figure 3 shows the right bank of the spillway channel cut in the embankment. Note the nearly vertical side slope.

The field team concluded that the side slope of the embankment adjacent to the spillway channel was unstable, could fail, and block the spillway. Such an incident would probably happen during a cloudburst-type storm leading to unusually high water levels in the pond. Increased depth of water would result in excessive pore pressures near the toe of the embankment. The downstream slope would fail leading to the collapse of the "dam". The field team also noted that the "spillway" discharged directly onto the downstream slope of the embankment, causing significant erosion and adding to the instability of the slope.

The embankment was constructed prior to 1935 using bottom dumping methods. The Willow Grove mine was closed about 1940. In recent years the entire area adjacent to the embankment was leased as a landfill. The mine waste was used as cover in the operation. Because the dam was serving no useful purpose, the field team recommended to the State that the owner breach the embankment and eliminate the hazard.

As the State and the owner, a coal company, studied a plan for breaching the structure, it quickly became obvious that the combination of the narrow valley and the need to stabilize the side slopes of the breach would require removal of the entire embankment. Disposal of large quantities of the material to be removed was also a problem. Furthermore, the stability could be a problem if the material was placed along the steep valley walls. Because of these difficulties the breaching approach was soon abandoned.

A second approach was to improve the stability of the embankment, provide adequate seepage control

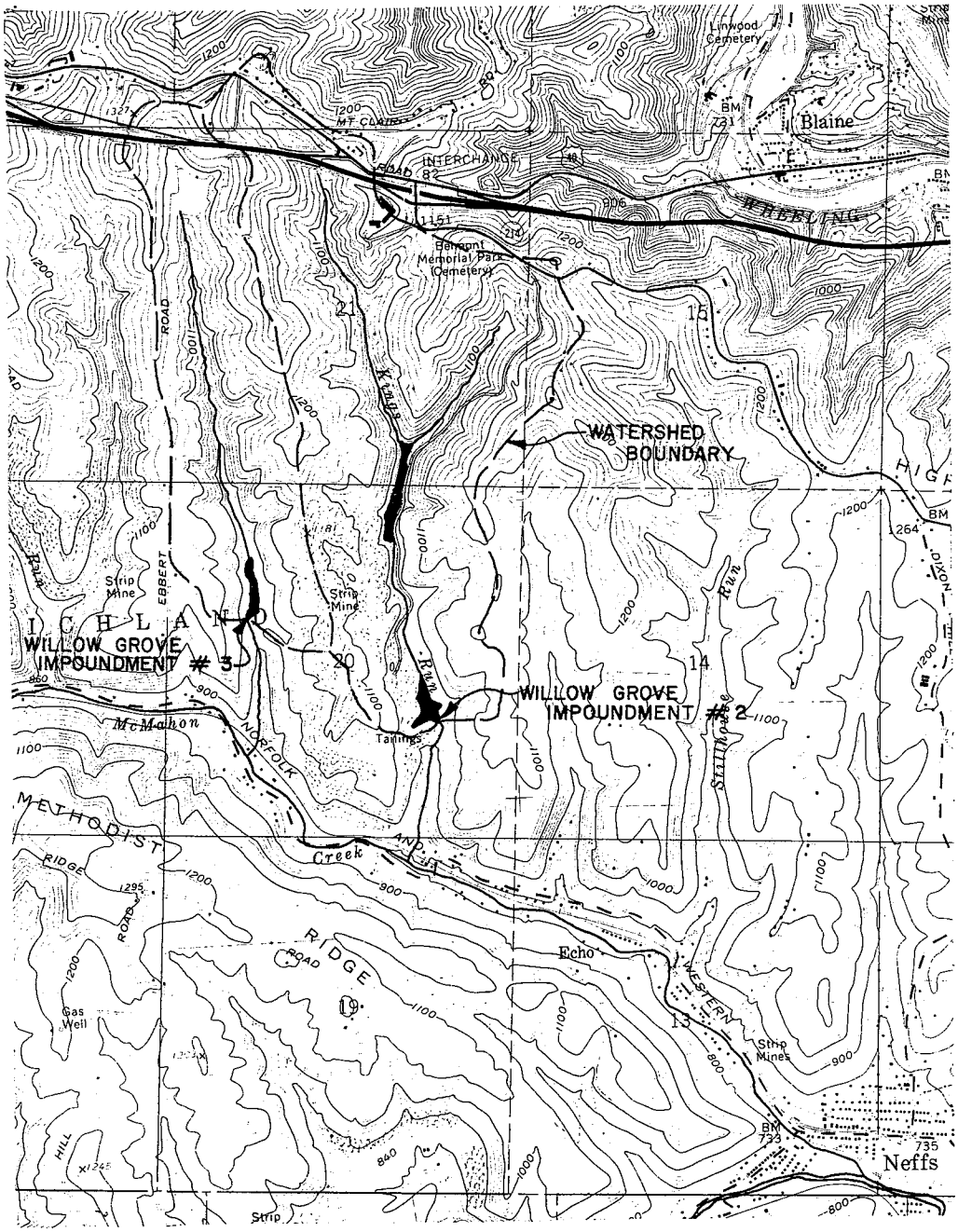

WATERSHED BOUNDARY
WILLOW GROVE IMPOUNDMENT # 3
WILLOW GROVE IMPOUNDMENT #2
Blaine
Linwood Cemetery
Belmont Memorial Park (Cemetery)
INTERCHANGE 82
WHEELING
McMahon
Creek
RIDGE ROAD
Echo
Neffs
Strip Mine
Strip Mines
Tailings
Gas Well
METHODIST
NORFOLK
WESTERN
EBBERT
ROAD
Run
Ridge
HILL
King
Stillhouse
BM 731
BM 733
BM 1264
1151
1295
1245

FIGURE 1

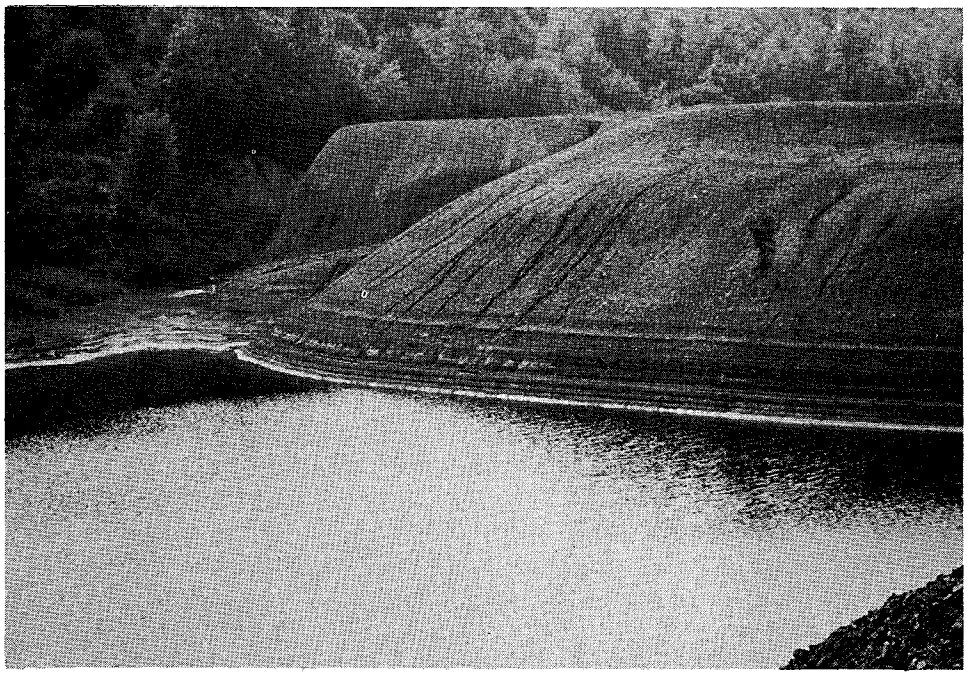


FIGURE 2

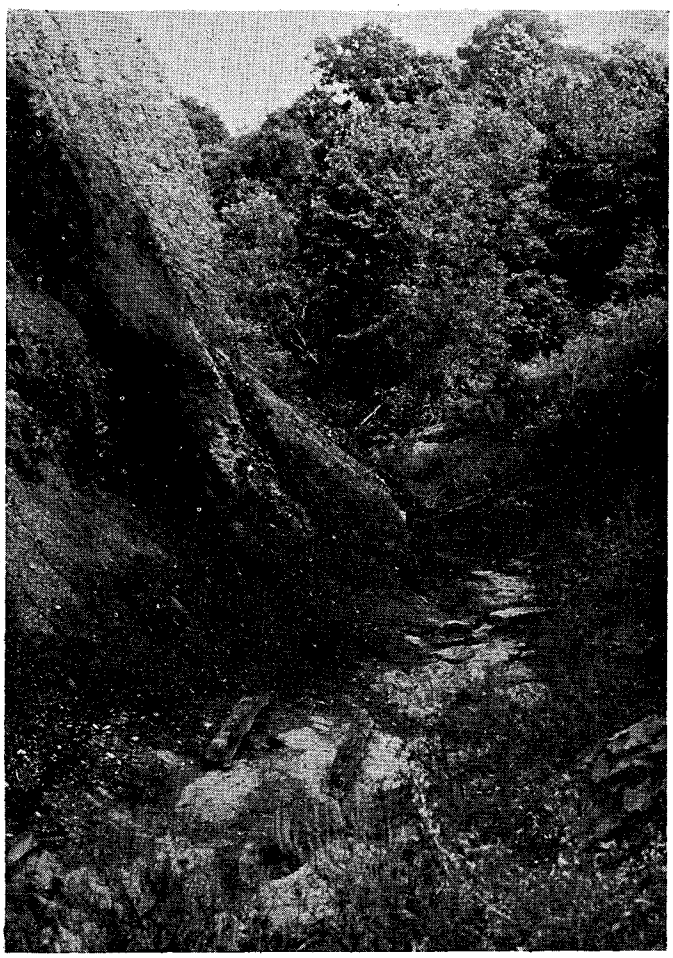


FIGURE 3

measures, and construct a new spillway. This approach would have required a detailed engineering study. It also involved the improvement of an abandoned structure which was of no use to the owner. The owner vehemently opposed this approach. The State engineers also felt that because of the heterogeneous nature of the embankment and the method by which it was placed, an engineering study in reality would be inconclusive and a waste of time and money.

After numerous meetings the owner and State engineers agreed upon an acceptable method of removing the hazard. The stream was relocated along the left hillside from the upstream end of the lake past the dam, thus completely bypassing the lake and the dam. The new channel was designed to carry the discharge from storms equivalent to 25 percent of probable maximum precipitation. The owner reached an agreement with the landfill operator that following construction of the bypass channel the lake would be used as landfill area. This would fill the entire lake area and eliminate the impoundment. The E.P.A. regulation requiring daily covering of land fill with a layer of clay was recognized by the State in approving this method because it would provide an upstream clay blanket. Figure 4 shows the completed channel in the left hillside. Note the land fill operation in progress.

2. Willow Grove No. 3

Willow Grove No. 3 was a 130 to 150-foot-high coal waste embankment located on a small stream immediately to the west of Willow Grove No. 2 (see Figure 1). The drainage area at the embankment was 340 acres and the surface area of the pond varied between 2 and 5 acres. The deficiencies and the hazard potential of this embankment were similar to Willow Grove No. 2, having unstable spillway sideslopes and significant seepage on the downstream slope which also normally kept the pool level below "spillway" elevation. Figure 5 shows a view of the downstream slope of the dam. Figure 6 shows a view of the toe of the embankment from the top. Failure would endanger the homes along Little McMahon Creek at Echo and Neffs. While there were many similarities between the two Willow Grove dams, there were two important differences: the upper portion of Willow Grove No. 3 was burning; and there was no active landfill operation in the area.

The State recommended that the hazard from this "dam" be eliminated by using an approach similar to the one used for Willow Grove No. 2. The stream was to be relocated into the hillside and the lake filled with material from the upper portion of the dam. Because the fire was confined to the upper portion of the embankment this scheme was expected to remove most of the burning material and place it in the lake area thus solving the fire problem also.

While this was the most economical scheme, it still involved moving over 100,000 cubic yards of material at considerable cost to the owner. Furthermore, there was no land fill operation to absorb much of the cost. Since the drainage area and surface runoff at the site were much less than that at Willow Grove No. 2, the owner suggested studying alternative schemes. All alternatives involved providing a bigger spillway, but ignored the problem of seepage and the fire in the embankment. For this reason none of the alternatives were acceptable to the State.

During the negotiations between the owner and the State, news of the State's concern about safety of this dam had reached the local news media. The residents of Echo and Neffs became concerned about living downstream from a potentially hazardous dam.

A significant, but not unusual, storm over the watershed in early 1975 resulted in considerable discharge through the spillway. With higher water levels the seepage at the toe increased alarmingly. The large discharge through the spillway led to a rumor in Neffs that the dam was about to fail. Fortunately the storm subsided without failure occurring. The aroused local citizenry demanded that the coal company remove the dam. The coal company also realized that they may have been underestimating the hazard from this dam. Action by the coal company was immediate. The state's plan was implemented within six months. Figure 7 shows the view of the top of the remaining embankment from the upstream side after all work was completed. Note that the lake has been filled and the channel bypass constructed into the right hillside.

3. Miller Creek Haul Road Dam

The first two case histories described experiences with dams built out of coal waste residue from deep mining operations. The Miller Creek Haul Road Dam is a 50-foot-high embankment constructed from strip mine spoil, which was essentially a heterogeneous mixture of clean soil and rock. This dam, which impounds a 36-acre lake, is one of several similar structures in Central Ohio Coal Company's Muskingum Mine in Muskingum County, Ohio. The top of the embankment is about 50 feet wide and serves as a haul road for mining equipment. The 48-inch corrugated metal pipe spillway at the left end of the embankment carries runoff from the 1.64-square-mile drainage area. The upstream and downstream slopes were a relatively steep 1 1/2H to 1V.

In late summer of 1973 the Division of Water received an inquiry from the Division of Reclamation on what would be required of the coal company to renovate an existing dam so that it could be used as a sediment retention basin. A field inspection by State engineers revealed a badly deteriorated structure. Several sinkholes were observed on top of the spillway pipe (Figure 8). Several large tension cracks on top (Figure 9) indicated a slide in the downstream slope. Furthermore, the spillway pipe had collapsed and was almost completely plugged. The water level in the lake had risen to 5 feet above the normal pool elevation.

Further inspection revealed that the spillway pipe collapse was at about the middle of its length. Piping was in progress around the pipe as evidenced by the sinkholes. The downstream slope, which was originally marginally stable, had become unstable because of the new pore pressures induced near the downstream slope by the break in the pipe and higher pool level.

Fortunately, failure of this dam would result in no significant damages downstream. A 65-foot-high haul road fill crossed the valley about one mile downstream. There was no development between

FIGURE 4

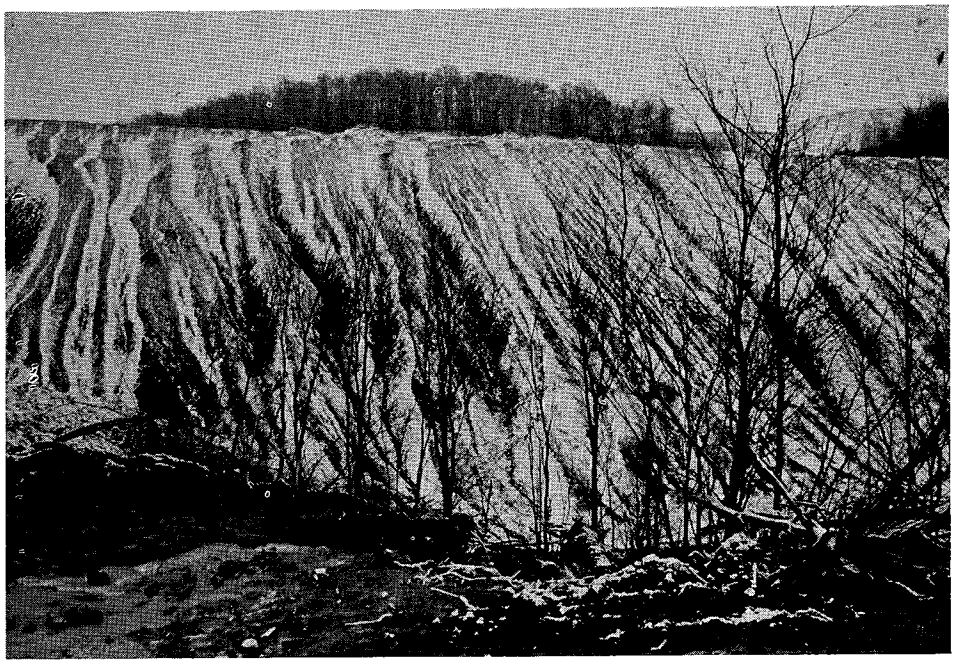


FIGURE 5

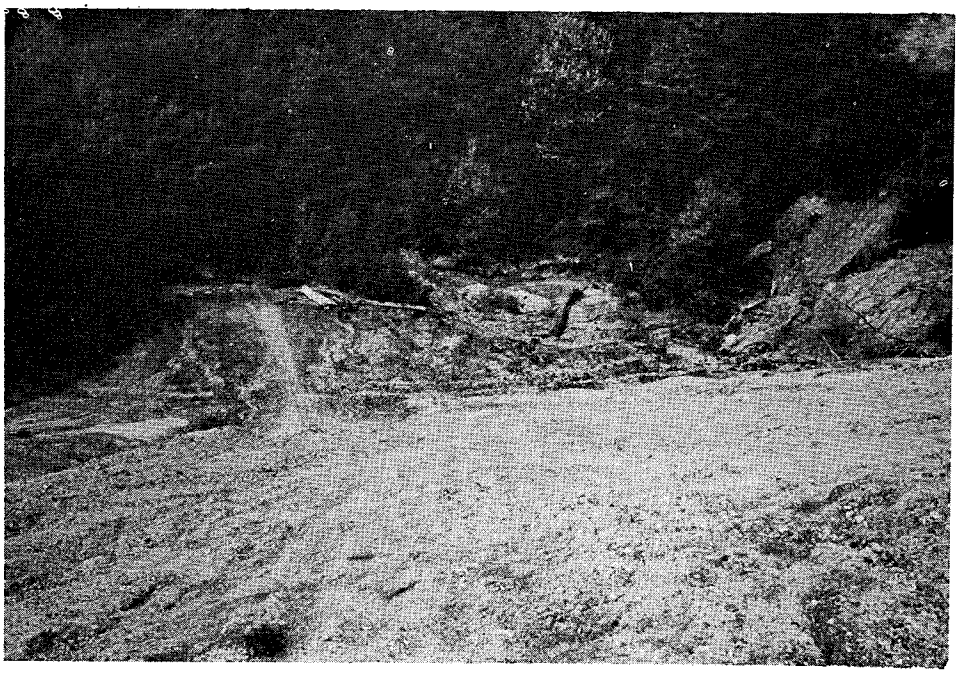


FIGURE 6

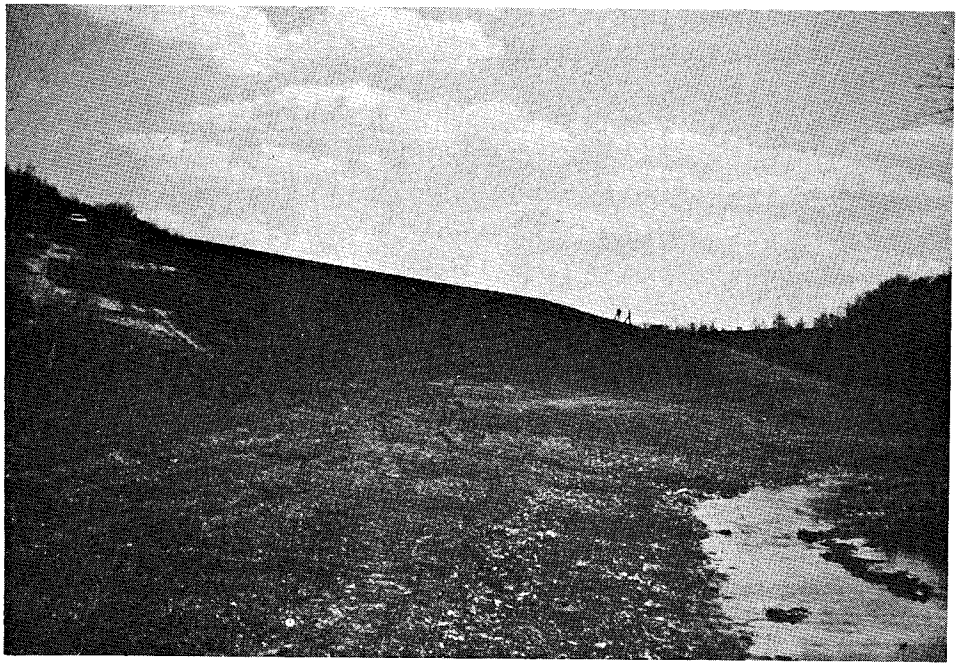


FIGURE 7

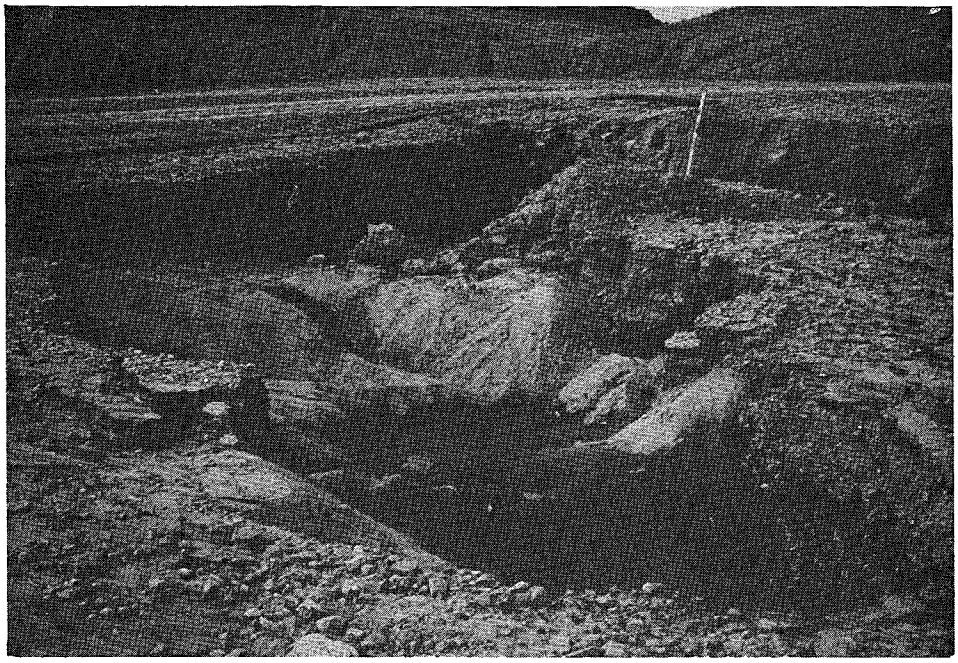


FIGURE 8

FIGURE 9

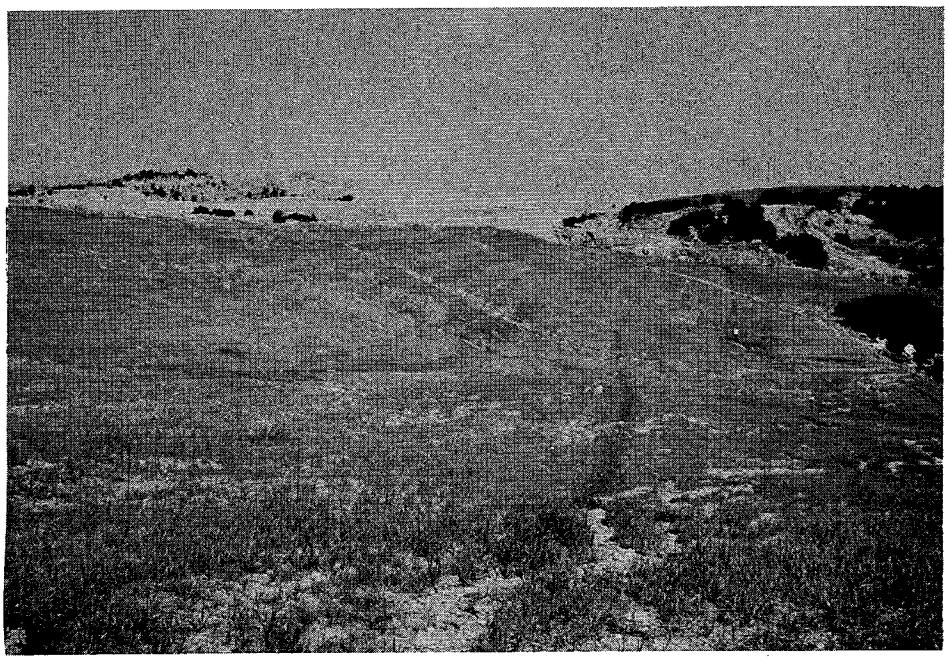


FIGURE 10

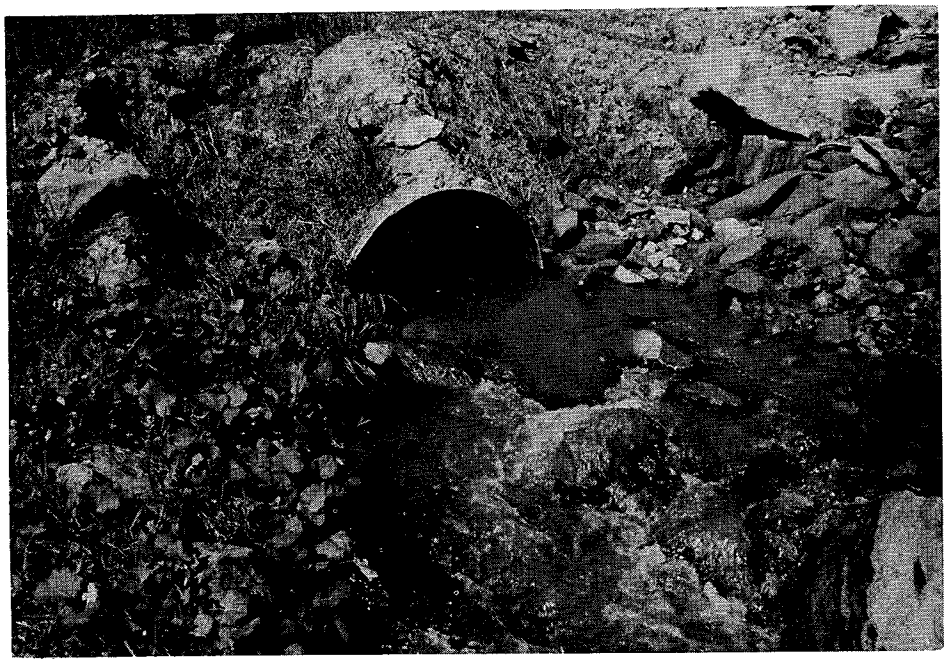


FIGURE 11

the two structures. The valley between them would hold about 20 inches of runoff from the entire drainage area before overtopping the embankments.

To correct this situation a coffer dam was immediately constructed from the entrance to the spillway pipe to the left abutment. This permitted the old spillway pipe to be removed and a new pipe installed in the left abutment. The embankment was refilled and compacted at these two locations. The downstream slope was flattened to 3H to 1V and keyed into the existing slope. A rock berm with a filter blanket was added at the downstream toe. The coal company also agreed to provide a vegetated emergency spillway in the right abutment after mining activities were completed and the dam was not needed as a haul road.

Figure 10 shows the overall view of the downstream slope after the construction was completed in 1974. Figure 11 shows a view of the downstream end of the new spillway pipe. The reconstructed dam has performed satisfactorily to date.

CONCLUSION

These three case histories are typical of the many coal mine dams that were investigated in Ohio. One of the lessons learned from this experience was that conventional engineering approaches to solving the problems of coal waste dams must be supplemented with practical economic and political considerations. Cost, safety, and public concern must be considered by enforcement agencies when issuing directives to coal companies. For example, in the case of the Willow Grove embankments the State could have chosen to enforce conventional solutions such as complete breaching of the embankments, or strengthening the embankments and providing adequate spillways. Such approaches would surely have been rejected by the coal company because of their cost and because the embankments and the ponds they created were completely useless to the company. The result would have been a legal battle between the coal company and the State. Such litigation normally requires years to be resolved. Cost to the coal company and the State probably would have exceeded the cost to remove the hazard by the approach used.

A second lesson was that coal mine dams have to be treated differently from conventional dams because of the material from which they are constructed, the way they are built, and their relatively short useful lives. The material used is extremely heterogeneous. Conventional soils testing will only give approximate results. Design or repair of such dams must take these factors into account using a great deal of judgement. A third factor to be considered in coal mine dams is the fact that while most coal companies have enormous earth moving capabilities, they have very little in-house engineering personnel experienced in the design of conventional dams. As one mining engineer succinctly pointed out to the author, "We are not dam builders, we are dirt movers." The author found most coal companies generally unwilling to use a great deal of drilling and testing and a sophisticated design. However, they were more than willing to use a conservative and simple design even though it might require more material.

The author found in most cases that after the unsafe conditions and engineering problems were understood by the coal companies, they were extremely cooperative in eliminating the hazardous dams.

ACKNOWLEDGEMENTS

Many people contributed their time and energy to this effort. Foremost among them were Stuart Long of U.S. Army Corps of Engineers, Pittsburgh District and Kyle Moran, U.S. Soil Conservation Service, Columbus, Ohio who along with the author were the field team. Charles Hahn, who was a member of the Board of Review and Roy Winkle, Chief of the Ohio Division of Water, provided general guidance and invaluable advice in effecting the repair of the embankments and in reviewing this paper.

GEOTECHNICAL OVERSIGHT NULLIFIES PROPER PROCEDURES

by

Bill Rosen
Marks-Rosen, Inc.
Knoxville, Tennessee

ABSTRACT

During late 1974 and early 1975 heavy periods, of precipitation combined with a series of common mining practices to develop landslide problems on a surface mine in Scott County, Tennessee. While land movement did not endanger life directly it did produce damage to property and pollution of a nearby stream.

The unique aspect of this situation is that the mining operator and/or his representatives complied with local surface mine regulations. Because certain geotechnical properties not uncommon to this area were overlooked, massive landmovements were possible. Liquifaction of mining spoil and saturation of colluvium produced shear failure in existing berms.

Reclamation of the surface mine required two phases of operation to prevent future movement. Temporary seeding of exposed soil and reshaping of the mining berm to aid drainage were recommended. Additional work would be performed after moisture contents could be reduced during summer months.

Detailed analysis of the problem, slide mechanisms, and possible theories to prevent future movements are discussed.

INTRODUCTION

Mining operations had proceeded approximately 3600 feet along the 1500 foot contour. Nearly one-half of the coal to be mined had been removed when temperatures started to fluctuate between brisk, cool days and cold, winter nights. Mining continued as the typical late fall - early winter rainstorms dampened the operations periodically. With the rains came not only operating problems, but landslides. Landslides occurred which blocked a local stream whereby the state halted mining operations. Although the regulations governing contour mining had been adhered to carefully, material movement stopped mining operations causing a sizeable loss of income to the mining operator as well as endangering aquatic life. The obvious question is whether the landslides could have been prevented.

During development of the mining and reclamation plant for this area, a small berm, in the form of an old jeep trail or access road, downslope from the coal seam outcrop was to be utilized as a natural buttress or berm for mining spoils. This berm was within the limits for slope spoiling and provided an excellent spoil limit boundary.

During the site investigation conducted by this author in May, 1975, five major slide areas were inspected. Three of the five slide areas inspected were associated with relatively large natural soil movements which altered the course of stream. The other two slides were composed primarily of mining spoil movement. Movement of material occurred from December, 1974 through March, 1975, during periods of heavy precipitation.

REGULATIONS

To understand conditions a brief review of mining regulations is in order. In addition to the general provisions described under the "Tennessee Surface Mining Law" specific regulations govern the removal of coal mined under contour methods.

The Law provides for several procedures for spoil deposit or storage. One of the oldest, most common, and most abused methods is downslope spoil banks. Attempts at controlling downslope overburden spoils have been regulated over the years by various means in the state of Tennessee. Suitable alternatives to downslope storage such as the modified block cut, head-of-hollow fills, or offsite storage have become increasingly popular due to strict requirements of spoil banks and the large volumes of wastes encountered. Variances and unusual conditions have often been the rule rather than the exception to deposition of spoil material; thus, making enforcement of restrictions difficult and susceptible to increased pressures on inspectors for involved engineering decisions based on limited knowledge and time.

Where the slope of the original ground covering the coal seam or lying below outcroppings of coal seams exceed fifteen degrees special provisions govern. With regard to depositing mining spoil downslope two basic regulations apply:

1) original contour slope greater than twenty-eight degrees - no downslope spoil allowed.
2) original contour slope less than twenty-eight degrees - spoil slope less than thirty-five degrees with downslope spoil not to exceed 125 feet. (This requirement changed to fifty feet effective July 1, 1975).

Final grading requires a maximum of twenty feet of highwall left in previously unmined areas (revised July 1, 1975 to eliminate highwall). Spoil placed against the highwall to reduce its height should have a maximum slope of thirty-five degrees with a drainage ditch along the mining bench at the toe of the sloping spoil. The fill bench should be graded to roll gently in with the natural ground below the toe of the outslope. This requirement not only develops an aesthetically pleasing reclaimed slope but also aids drainage and erosion control.

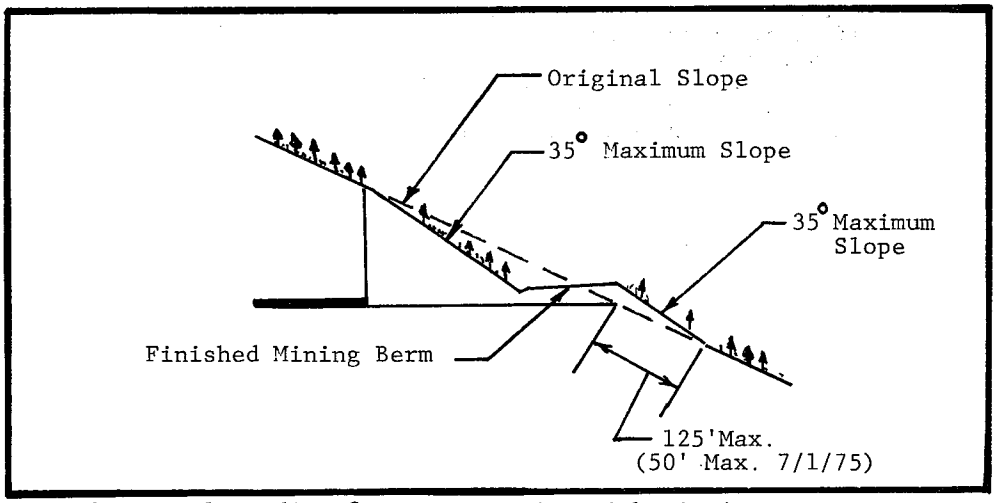


Fig. 1. Final grading for contour mine with virgin cut.

EXISTING CONDITIONS

The mining area is located on the east side of Arch Mountain in Scott County, Tennessee. This area is very prevalent in coal seams and mining operations at different elevations. Geology of the area shale, and sandstone overlaid by varying depths of colluvial and residuum soil. Coal seams mined by surface means range from sixteen to thirty-six inches in thickness, which necessitates removal, storage, and restoration of large volumes of overburden per ton of coal extracted.

Virgin slopes in the vicinity of the mining operation ranged from twenty-one to twenty-seven per cent. Vegetation consisted mainly of broad-leaf forest and scrub brush.

The mining bench was approximately one hundred feet wide with high walls less than forty feet high and associated slopes of less than thirty-five degrees. Drainage was directed away from the slope berm and spoil material had been retained behind the old jeep trail berm well within the then existing spoil limit boundary requirement. With the exception of high walls which were above the requirements stated for final reclaimed mining areas, mining operations had been conducted in accordance with regulations in effect when the permit for surface mining had been issued.

In December, 1974, a small slide consisting of spoil material occurred. Slippage continued to occur until the berm consisting of colluvial material underwent shear failure and a major slide developed which produced a course alteration in the Straight Fork Creek. During the period of January through March, 1974, four additional slides developed. Two of the slides caused blockage and reorientation of Straight Fork Creek.

In order to avoid excessive costs to the operator, the mining activities were allowed to continue after the first slide occurred since its location was very near the most recent spoil storage area. Attempts were made to alleviate sedimentation of the stream; however, continued rains caused progressive failures at various locations along the mining berm and spoil banks. At this point all mining activities were suspended and efforts directed toward analyzation and correction of the landslide problems.

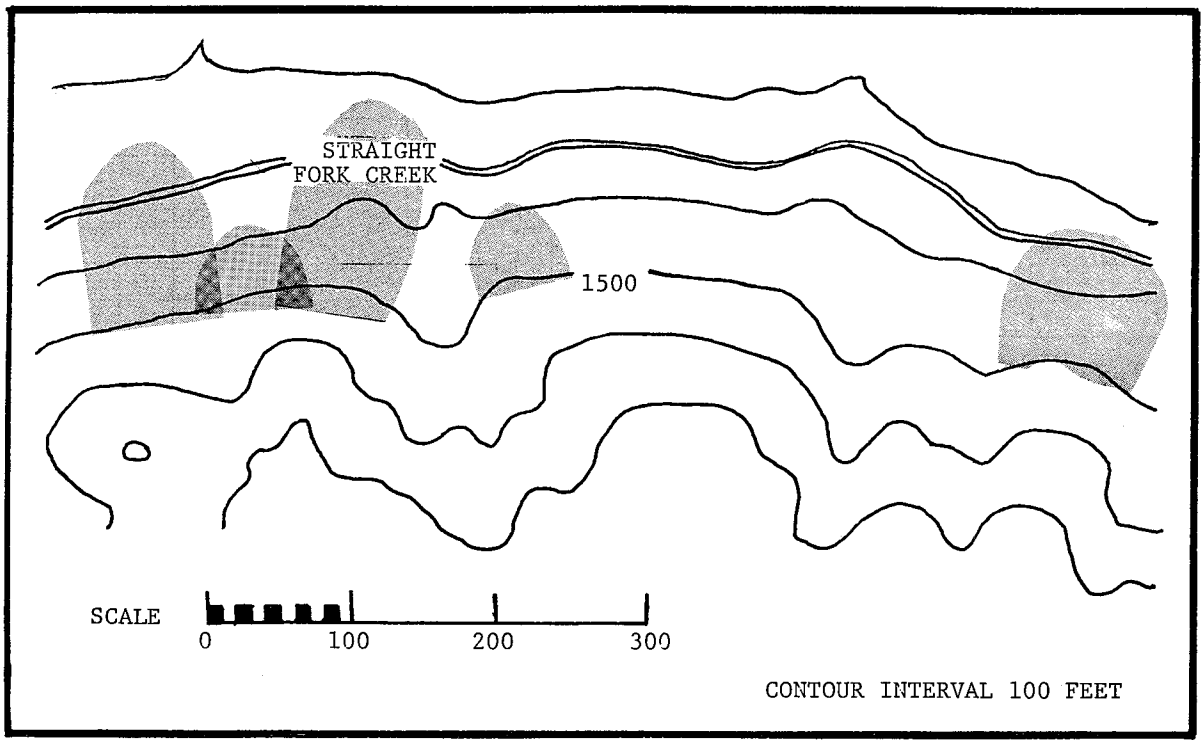


Fig. 2. Contour map of mining site and slide areas.

ANALYSIS OF SLIDE MECHANISMS

The land movements occurring at this particular mining operation were not different from slides occurring at other mining locations. Two mechanisms combined to cause significant land movements at this contour mine. These mechanisms did not involve mining practices which are not uncommon for this type of mining.

Spoil material stored on the side slopes and the small jeep berm became saturated during periods of heavy rains. The conglomerate of shale, sandstone, and topsoil was able to absorb and retain a considerable amount of water. Retention of moisture produced a high potential for liquifaction or saturated flow of the material.

In addition to the liquifaction potential of the spoil material the colluvial material in which the berm was cut is highly unstable and exists in nature in a metastable condition. Loading of the berm with well-drained or dry spoil material would most probably have produced no stability problem; however, saturation of the spoil material increased loading on the berm sufficiently to produce failure of the berm which relieved lateral support of the spoil material.

As may be seen from the following cross section, the spoil material has undergone sufficient flow to be very near the natural slope. Where natural material underwent movement, the mass has moved into a relatively stable position by movement into the adjacent creek bed. As with most soils undergoing movement it is seeking an equilibrium condition with slopes very near their original grade.

However, as a result of initial liquifaction and flow, the mass of the slides are generally in a saturated to supersaturated condition. This condition could not be immediately relieved because the toe of the slide resided in the stream bed and any addition of moisture or disturbance of the slide material could have reactivated slide mechanisms.

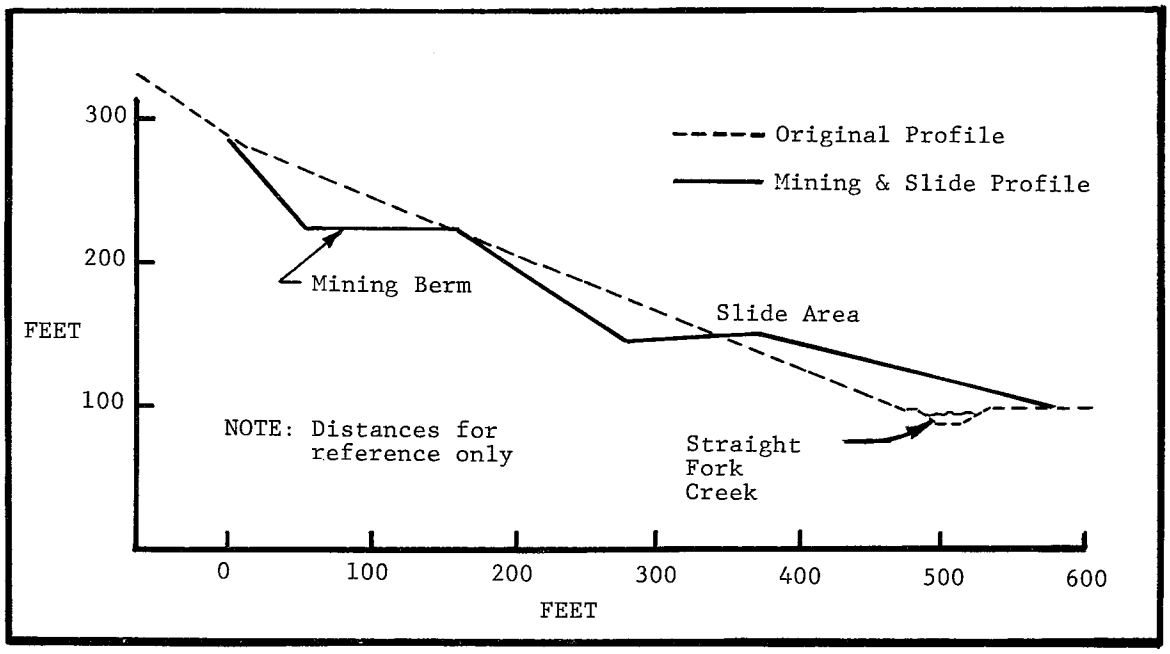


Fig. 3. Typical cross section of major slide

CORRECTIVE PROCEDURES CONDUCTED

Primary concern in reclamation of existing slides was not considered to be prevention of liquifaction of the spoil material held by the natural material. Existing drainage conditions were not condusive to prevention of future liquifaction of the spoil material. Surface drainage conditions existing after the slides allowed large volumes of water to flow onto and through the spoil slide debris. The saturated condition of slide debris prevented the immediate reshaping and grading of the slide mass to drain surface run-off away from the slide areas.

Of primary concern was the correction of flow problems associated with Straight Fork Creek in a permanent manner. Shortly before the author's association with the slide problems, the mine operator had submitted and gained approval for a plan which consisted of diverting the creek around the slide areas into a new channel. This action provided two positive correction procedures: 1) a new channel would avoid further sedimentation and pollution of the creek, and 2) the re-routing of the stream provided some stabilizing effects as a result of lateral drainage from the toe of the slides and utilization of the old creek bed for an area to toe out the slides. Consolidation and densification of the spoil slide debris as a result of drainage helped to stabilize the slides.

Correction procedures associated with reclaiming the slide areas were developed with the most plausible and feasible measures available to increase the stability and erosion resistance of existing slide and spoil debris. Two phases of correction were conducted: 1) immediate or temporary and 2) final or permanent.

Immediate action involved the reshaping of the bench area where necessary to insure that ponding of water did not occur, surface run-off did not drain outward over the bench slope, and bench drainage ditches were adequate to transfer surface water parallel to the bench into natural drainage channels. During the immediate action, no disturbance of the slide debris was attempted except for some handwork in drainage of areas where severe ponding was occurring on the body of the slide masses. Subsequent to reshaping and drainage of the bench area, the entire disturbed area, both the mining bench and slides, were temporarily seeded with a mixture of English Rye and Sericea Lespedeza and mulched. No attempt was made to plant trees or heavy cover vegetation since final reclamation efforts would disturb existing seedlings.

Temporary actions allowed the slide debris to undergo increased stability as a result of a decrease in moisture contents and dryer summer periods. Also, erosion of slopes was decreased by the grass vegetation.

During late September of 1974 the stability of the slide masses began with the use of small dozers. The slide areas were sloped and contoured in such a manner as to insure that no surface runoff ponded or seeped into the spoil material. The benched area was checked to insure that proper drainage had been provided by the immediate action previously taken. In conjunction with final grading, appropriate species of trees were selected for planting in the area. In addition to tree planting, areas which were disturbed by final correction procedures were reseeded.

DISCUSSION

The facts presented in this paper represent incidents which are not uncommon. While regulations are broad enough to allow flexibility and judgement on behalf of both the mining operator and mining inspectors, they also allow considerable leeway for a multitude of possible events to happen which create problems. Several areas of possible preventive measures should be investigated.

Limits on the slope angle that can be mined, the extent to which spoil can be placed downslope, a critical slope for spoil material, and a critical slope that can be mined with no discharge of spoil downslope are all areas under investigation and research. In Tennessee, our limit for maximum spoil slope is thirty-five degrees with a maximum mining slope of twenty-eight degrees. However, it is realistic to conceive of spoil material in a relatively loose condition to remain at an angle of repose much greater than its deposition in nature.

In order to eliminate the possibility of slope movement two requirements of the soil-rock slope can be adjusted to aid stability. The maximum slope of spoil material and the maximum distance downslope material may be deposited. To achieve some degree of correction or at least limit the potential slope movement, the state of Tennessee has limited the placement of spoil downslope to fifty feet. This regulation change on July 1, 1975, by the state adjusted the second alternative. This was a convenient way to eliminate the area disturbed by mining activities and produce two seemingly positive solutions to slope stability associated with spoil material: 1) the amount of spoil material stockpiled downslope would be reduced drastically, and 2) if movement occurred damage to the surrounding area would be far less severe. While these two objectives are certainly beneficial compared with previous situations, the requirement does not necessarily prevent slides.

Whereas, this concept would certainly seem to decrease the possibility of overloading slopes, the fifty feet limit severely reduces the area for spoiling material. Consequently, mining operators are forced to use other acceptable but more costly means of stockpiling refuse. Many of these methods only remove the slide problem to another location or increase the potential of problems other than stability.

Reduction of the maximum slope angle could be just as beneficial. Less material would be stored as a result of the decrease in angle. This would enable the spoil material to more closely resemble its

naturally deposited state and reduce the loading condition on semi-stable colluvial soils. Lengths of downslope spoil could be extended to some reasonable distance; thus, allowing some relief to mining operations. This adjustment would enable an increased volume of material to be stockpiled over the slope while reducing the potential for spoil movement.

Another important aspect of reducing the spoil bank slope angle is the reduction of the spoil erodibility potential. The steepness of the slope is an important factor in the amount of spoil material which may be transported into natural drainage channels. Although vegetative cover effects the amount of material subject to erosion, the slope length and slope angle are probably more easily controlled. Reduction of the slope angle would not only decrease soil erodibility directly; but would also improve conditions for establishing grass and seedlings. Current state regulations have not dealt with compaction requirements or densities mainly because of the variety of spoil material. The densification of material has a direct bearing on the geotechnical properties of the material and consequently, the stability of slopes constructed. Since inspections of surface mines are very seldom on a daily basis, a large part of the responsibility of spoil storage lies on the mining operators. Frequently, spoil is spilled over the slopes with the only compactive effort coming from densification by its own weight. Unfortunately, this process allows soft, overburden soil to be discarded first, followed by heavy and often larger amounts of rock. This produces a condition very susceptible to movement without additional forces required. It is foolish to expect mining operators to bear the cost of material separation and storage when not required by law. Even if required it is doubtful separation of soil and rock and proper compaction of this material on steep slopes could feasibly be accomplished or controlled.

SUMMARY

It has been the intent of this paper to stimulate thought towards research and development of sound engineering practices in the reclamation of contour mining slopes. Two important aspects of slope stability, slope angle and slope compaction, have been presented which are very often overlooked by ruling bodies when considering stability of spoil slopes. Other contributing factors such as proper drainage, vegetation, and spoil limit have been thoroughly covered by existing regulations because these areas have had quantitative as well as qualitative input from their respected technical fields. Neglect of the engineering properties of mining wastes can no longer be tolerated. Environmental and economic consideration of the storage and use of mining refuse has become of paramount importance. Haphazard and arbitrarily determined regulations based on a try-and-hope philosophy perpetuates general abuse of already established procedures.

The development of engineering concepts with respect to the interactions of all aspects of the problem requires a concentrated effort to collect available data, regulations based on this data, and strict enforcement of those regulations.

Slope stability has long been studied relative to the highway and construction industries. It is imperative to progressive legislation and control of mining operations that information obtained from these other industries be converted to contour mining operations. It is anticipated that further emphasis will be placed on the role of the geotechnical engineer in developing guidelines and specifications for spoil slopes.

WEST VIRGINIA EXPERIENCES WITH REVIEW OF COAL REFUSE DISPOSAL FACILITIES

George A. Hall, Ph.D., P.E.
Geotechnical Control Engineer
Coal Refuse & Dam Control Section
West Virginia Department of Natural Resources
Nitro, West Virginia

Ronald E. Smith, Ph.D., P.E.
Vice President
Woodward-Clyde Consultants
Rockville, Maryland

Edward R. Underwood, M.S.C.E.
Geotechnical Engineer
Coal Refuse & Dam Control Section
West Virginia Department of Natural Resources
Nitro, West Virginia

Following the Buffalo Creek disaster of February, 1972, the State of West Virginia enacted the "Dam Control Act" and the "Coal Refuse Disposal Control Act" to regulate and control coal refuse piles and all structures capable of retaining water. The implementation of these two acts was delegated to the Department of Natural Resources, and this subsequently led to the formation of the Coal Refuse and Dam Control Section. This group has been given the responsibility for establishing minimum standards for both refuse piles and the design of existing and new facilities having water retention capabilities.

During the past several years the authors have reviewed a large number of design studies submitted to the state for approval. Based on these experiences, the State is currently preparing a modification to its guidelines for engineers involved in these projects. It is hoped that these guidelines will simplify the design and review process while retaining the emphasis on the safety of the facilities. In addition to the discussion of guidelines and design concepts, the paper also includes a review of geotechnical engineering properties both from the technical literature and as reported in a number of the design reports submitted to the State.

INTRODUCTION

Many governmental agencies have been created as a result of long standing problems which are suddenly brought to public prominence. Unfortunately, this prominence is sometimes forced by tragedy. The Buffalo Creek disaster was one such tragedy, and the Coal Refuse and Dam Control (CR & DC) Section now has legal responsibility for the safety of dams and coal refuse embankments throughout West Virginia.

A governmental agency charged with responsibility for public safety has two divergent concerns, especially when engineering disciplines are involved. First, the agency must act in a manner to truly protect the public interest and, second, the agency must not be unduly cautious at the expense of businesses and individual citizens. Since engineering is not an exact science and depends partly on judgement, the state of the art becomes a very important part of governmental regulation. Consequently, it is paramount that governmental agencies and engineers submitting plans to those agencies share their knowledge and experience in an effort to optimize engineering practice. This paper is an outgrowth of design and construction guidelines currently being utilized and upgraded by the CR & DC Section. The authors have all contributed preliminary information to these guidelines.

While compiling experience and technical data for assistance in preparing the guidelines, it became apparent that information on coal refuse structures and coal refuse properties was available which might be of interest to the geotechnical engineering profession in general. This paper is an attempt to present some of that information.

BACKGROUND

The two main pieces of legislation pertaining to dams and coal refuse disposal piles in the State of West Virginia are Article 5D, entitled "Dam Control Act", and Article 6D, entitled "Coal Refuse Disposal Control Act", of Chapter 20 of the West Virginia Code. Collectively these laws impower the Director of the Department of the Department of Natural Resources to regulate and control the safety of dams and coal refuse disposal piles within the State by adopting and enforcing rules and regulations governing the safety of dams.

Under the Dam Control Act, owners of existing dams, more than 15 feet high, or creating reservoirs covering an area of 10 acres or more, or persons Proposing to place, construct, enlarge, alter,

repair or remove such facilities must apply for and obtain a certificate of approval from the Department of Natural Resources. Under the Coal Refuse Disposal Control Act, operators of existing coal refuse piles that are deemed to constitute safety hazards, shall be ordered by the Director to eliminate such hazards.

One of the basic differences between these laws is that action is required on the part of all dam owners; whereas action by operators of coal refuse piles that do not constitute dams, is required only upon receipt of an order or notice from the Department. It should be noted, however, that because most coal refuse piles "impound or divert water" they are classified as dams and require a certificate of approval.

The Coal Refuse Disposal Control Act was enacted in 1972 followed in 1973 by the Dam Control Act. Following enactment of this legislation, inventories of such facilities were made and the results indicated that there was in excess of 1400 dams and coal refuse sites in the State. In October 1973 a set of design criteria was formulated and published by CR & DC Section to serve as a basis for evaluating new and existing facilities. During 1974 the Section received so many applications and design submittals that it became necessary to engage the services of a geotechnical consulting firm to participate in the reviews and to help set up a systematic approach for dealing with each application on an equal, fair and timely basis. To that end "review checklists" were established.

Review Checklists

Review checklists were developed by the Section for in-house use but equally as important, for general distribution to individuals and organizations submitting applications to the State. The first group of applications received varied from comprehensive investigative and design reports to very limited statements of "facts" relative to the stability and adequacy of certain facilities. Based on these differences, it was believed to be to the advantage of all concerned to provide owners and their consultants with a set of ground rules to be used in judging application submittals. The public availability of the checklist has helped to meet this need.

Checklists carry with them an inherent potential for abuse when applied in a rigid manner by persons lacking depth of understanding of the technical areas being considered. This problem was recognized at the time of formulation of the first checklists. To help lessen the possible abuses each checklists question was formulated to require more than a simple "yes" or "no" check-off answer. Each question was followed by a check-off space marked "not applicable", and a similar space to reference detail comments that could be made at the end of the review sheet. It was hoped that these additional options would encourage the individuals making the reviews to make or seek judgements of appropriateness of each question.

To date the most extensive use of the checklist applies to structures that either do or have the potential to "impound or divert water". This checklist consists of 15 different sections with individual sections containing between three and fifteen questions. Given below is a listing of the major sections and a summary of the kinds of information sought:

1. General - Information relative to the nature of the facility and overall appraisal of the design.
2. Hazard Potential - Has a realistic appraisal been made of potential failures and are emergency procedures considered?
3. Subsurface and Geologic Investigation - Has an adequate subsurface investigation been made of all areas that could reasonably be expected to influence the safety of the facility?
4. Laboratory Investigation - Have appropriate tests and interpretations been made of pertinent materials?
5. Hydrology - Have the watershed and design storm been adequately assessed?
6. Hydraulics - Are the spillway and freeboard adequate to accommodate the design rainfall? Are the hydraulic structures stable?
7. Slope Stability and Protection - In this section questions are addressed to selection and analyses of critical conditions postulated during the life of the structure including variation of material properties and phreatic surfaces with time.
8. Seepage Analyses - Is the structure safe against internal erosion (piping) and seepage forces?
9. Settlement Analyses - Questions relative to settlements due to compressible materials and mine subsidence.
10. Foundation Analysis - Questions the suitability of the soils underlying the planned or existing dam.
11. Liquefaction - Is there a potential for liquefaction (especially of fine waste) that could compromise safety of the structure?
12. Quality Assurance - What provisions have been made to assure that the facility will be constructed in accordance with the design recommendations?
13. Monitoring Program - Are monitoring programs necessary? If so, are they specified during the construction or operational phase of the project?
14. Specifications - Do the construction specifications and other contract documents incorporate the design recommendations?
15. Maintenance and Inspection - By whom and how will the facility be inspected and maintained during its design life?

Earthen structures capable of impounding water are potentially dangerous items. Hundreds of thousands of people have died as a direct result of the failure of such facilities. The design of these facilities should be approached with thoroughness and caution. Even so-called "ultra-conservative designs" can prove fatal if conservation is in fact a cover for inattentiveness to detail. These checklists are not meant to be punitive. They are meant to remind both the reviewer and the designer of their professional obligations to provide due consideration.

GEOTECHNICAL PROBLEMS

Properties of Coal Refuse

The engineering properties assigned to coal refuse for purposes of making geotechnical analyses are an important part of assessing potential problems. When an analysis is based on limited testing it is important to know whether the results obtained are consistent with experience for that type of material. If results appear extreme, a check of testing techniques or the representativeness of the samples may be in order.

While the engineering properties of coal refuse vary considerably, depending on the source, the handling, and many other factors, the average

values and range in values seem to be surprisingly consistent. A summary of test results for coarse, fine, and closed circuit (mixed coarse and fine) refuse for disposal areas throughout West Virginia is presented in Table I. These results were obtained by various consulting engineering firms submitting plans to the Coal Refuse and Dam Control Section. For comparison purposes, a summary of data from the literature is provided in Table II.

The effective stress strength parameters are among the most interesting of the engineering properties of coal refuse. A material with little or no plasticity and often little or no clay sized particles would normally be expected to be "cohesionless". Although 32 of the 57 test results for coarse refuse shown in Table I yielded zero cohesion several values in the order of 1000 psf (0.5 kg/cm^2) were submitted. Carefully conducted tests on stiff West Virginia clays seldom yield values of c' greater than 500 psf (0.25 kg/cm^2)-see reference(6). Why are such high values of c' obtained for coarse refuse? In addition to sample scatter, the authors view two possibilities as the more plausible explanations:

1. In view of the low permeabilities exhibited by some coal refuse samples, negative pore water pressures may develop if samples dilate during shear at low confining pressures; if the rate of testing is too fast, incomplete drainage or improper pore pressure measurements will lead to an overestimation of cohesion intercepts (1,2).
2. The strength envelope of many coal refuse materials shows a marked curvature, usually with zero cohesion and a higher value of ϕ' the origin (7,9). Depending on the effective compressive stress range used in testing, variable values of ϕ' and c' can be obtained.

The first possibility can be eliminated by using proper testing rates (1,2,5,8). The curvature of the strength envelope implies that the stress range in testing must correspond to the stress range in the field, which may require more than three tests per envelope.

The foregoing discussion of c' also applies to fine coal refuse, which is often predominently silt sized. It is interesting that values of ϕ' for fine refuse are of the same order of magnitude as those for coarse refuse.

The specific gravity and unit weight of coal refuse vary considerably and are often much lower than typical values for soil (7). This fact becomes quite significant in terms of effective stress, and implies that high pore water pressures and pore pressure gradients can be more dangerous in coal refuse than in soil. Thus, with respect to potential problems such as slope stability and piping, care should be excercised in the selection of both the strength parameters and unit weights used in analyses.

Slope Stability - Case Histories

During the inventory of coal refuse embankments it was found that the "angle of repose" of dumped embankments was usually between 33 and 38 degrees. An average angle would be about 36 degrees, with angles as great as 40 degrees being rare. It is interesting that the average slope angle corresponds closely to the average angle of shearing resistance, ϕ', given in Table I. This is the condition expected for an infinite slope of cohesionless material at limiting equilibrium without seepage.

There are few case histories in the CR & DC files which yield direct observations of conditions at the time of the failure or incipient failure of coal refuse embankments. Two cases are exceptional in regard to the completeness and timeliness of observations, and these cases are described in the following paragraphs.

An incipient failure, as evidenced by a small scarp at the crest and bulging at the downstream toe, of an impounding coal refuse embankment was observed in February 1974. The embankment was the Wharton Number 2 refuse area in Boone County, West Virginia. E. D'Appolonia Consulting Engineers, Inc. had been retained by the coal company to design remedial measures for the embankment, and preliminary investigations were under way. The consultant was able to both obtain good documentation of conditions and to suggest emergency measures which may have averted a complete failure. The documentation allows estimates of the average strength parameters of the coal refuse. By assuming a factor of safety of 1.0, values of ϕ' and c' which will satisfy equilibrium can be obtained from stability analyses. The failure was apparently relatively deep seated. The consultant chose to use the values $\phi'=27^\circ$ and c'=450 psf (0.22 Kg/cm^2) in subsequent analyses. Analyses by CR & DC, assuming c'=0, gave $\phi'=33^\circ$.

A failure of an impounding coarse refuse embankment occurred on March 16, 1973 near Caples in McDowell County, West Virginia. In contrast to the first case, the failure was relatively narrow and shallow and developed into a flow. The flow slide led to the failure of three catchment ponds beyond the downstream tow of the embankment, and portions of the combined mass moved about 2000 feet downstream. Only minor property damage resulted. Measurements of the intact and failed slopes and of the highest observed seepage level in the failure area permitted an approximate reconstruction of the failure condition. The phreatic surface was assumed to correspond to the highest level of observed seepage. Measurements were made on March 20, 1973 by Department of Natural Resources personnel. Assuming c'=0, $\phi'=43.7^\circ$ gave a factor of safety of 1.0 using a circular arc which closely approximated the observed failure surface.

Both of the case histories just described involved slopes of considerable height (Wharton No.2 about 500 ft and Caples about 200 ft). On the other hand, the Caples failure was much more shallow and represents a condition of low effective compressive stress. While no general conclusions can be drawn from only two case histories, the results are in line with test data showing a curved strength envelope for coarse coal refuse.

Seepage Estimation Problems

Two of the most difficult problems in geotechnical engineering pertain to the estimation of seepage conditions. First, how do you measure and properly represent seepage effects at an existing site, especially for the purpose of conducting a stability analysis? Second, how do you predict the eventual conditions which will obtain at a proposed site? The authors have no clear-cut answers to these questions, but a commentary based on our experience in West Virginia seems in order.

With respect to the first question, it is first judicious to discuss the problem. Seepage through coal refuse is often quite heterogeneous. This condition has been observed on numerous occasions, whether the source of seepage was impounded water or natural seepage from a hillside. In view of the diverse values of measured permeability for refuse, the condition is not difficult to understand. It is, however, quite difficult to measure

Table I. Coal Refuse Properties (from CR & DC files)

Type Refuse		% < #200 sieve	LL(%)*	PI(%)*	ϕ' (Degrees)	C' (PSF)	S.G.	Inplace Density (γ_d) PCF	Inplace Density (γ_d) %Standard Proctor	Standard Proctor γ_d(PCF)	Standard Proctor Opt.Mois.	Permeability (CM/SEC)
Coarse	AVG	16.5	27	5	35.3	241.3	2.16	97.7	88.9	106.5	8.0	3.78×10^{-3}
	HIGH	53	32	7	47.0	1080	2.70	117.4	107	115.8	11.2	1.41×10^{-2}
	LOW	0	21	1	25.0	0	1.68	75.3	64	92.7	1.5	8.00×10^{-7}
	NO.	45	8	8	57	57	34	22	15	22	20	16
Fine	AVG	57.5	32	8	34.7	131	1.64	60.2	88.7	72.2	16.7	1.94×10^{-5}
	HIGH	95	43	14	42.5	1224	2.46	84	97	119.5	23	1.04×10^{-4}
	LOW	13	20	2	23	0	1.39	48.9	81	56.8	10.5	6.10×10^{-7}
	NO.	29	11	11	29	29	20	11	6	10	5	9
Combined	AVG	28.3	27	5	33.2	259.2	2.15	120	114	106.7	10	7.31×10^{-4}
	HIGH	44	30	8	40	500	2.30			116.2	11.2	1.39×10^{-3}
	LOW	12	23	4	29	0	1.87			95.7	8	7.22×10^{-5}
	NO.	6	3	3	9	9	4	1	1	5	4	

*Several samples were nonplastic and Atterburg limits were omitted in most reports.

Table II. Typical Coal Refuse Properties from the Literature

Engineering Property	Values	Reference	Type Reference	Comments
Ø' (degrees	29.5 - 36.9	(3)	Coarse	Triaxial and Direct Shear
	25.5 - 41.5	(9)	Coarse	Triaxial
	25 - 39	(10)	Coarse	Triaxial
	21.5 - 38	(9)	Fine	Triaxial
c' (psi)	-0.5 - 12.6	(3)	Coarse	Triaxial and Direct Shear
	0	(9)	Coarse	Triaxial
	0	(10)	Coarse	Triaxial
Specific Gravity	1.62 - 2.28	(3)	Coarse	
	1.50 - 2.75	(9)	Coarse	
	1.58 - 1.87	(4)	Fine	
	1.53 - 2.71	(9)	Fine	
Permeability (cm/sec)	5.02×10^{-5} - 1.10×10^{-3}	(3)	Coarse	
	9×20^{-7} - 3×10^{-2}	(9)	Coarse	
	7.76×10^{-7} - 3.75×10^{-3}	(4)	Fine	
	3×10^{-8} - 4×10^{-3}	(9)	Fine	

and assess. The use of standpipes and piezometers, unless properly placed and used in sufficient numbers, can be quite misleading on the nonconservative side. When measurements are being made for the purpose of assessing slope stability, it must be borne in mine that a pore pressure distribution along a potential failure surface is an alternative representation for the effect of seepage forces through the potentially failing mass. When wet zones in borings or seepage on the slope surface are observed at an elevation which is higher than the measured water level, it is probable that all seepage forces through the slope mass are not accounted for with the use of the measured water level. If the mass of material influenced by the additional seepage and the hydraulic gradient of that seepage are sufficiently great, the factor of safety of the slope will be significantly overestimated.

The problem of predicting eventual maximum seepage conditions for proposed refuse sites is even more difficult, except for the fact that drainage facilities can be built into a new site. As a general rule, uncertainties are greater for non-impounding sites since sources and quantities of seepage are more problematic. The Coal Refuse and Dam Control Section encourages the use of subsurface drainage systems whenever seepage seems likely. Since the Section is also responsible for abandoned sites, opportunities for future evaluation of subsurface drainage should be available. It should be noted that some coal companies have questioned the use of subsurface drainage because air could enter the refuse through the drainage system and produce spontaneous combustion. As yet, no burning problems have arisen, and, if compaction of the refuse is adequate, no problems are anticipated.

A panacea for the problems of measuring, estimating, and controlling seepage through coal refuse is not at hand. Future experience should help, but, in the interim, a conservative approach is suggested. Experience shows that "assuming the worst" is often not overly conservative.

CONCLUSIONS

A brief summary of some of the experience gained from coal refuse site reviews in West Virginia has been submitted. It is hoped that the information provided will be of value to other regulatory agencies and to engineers in general. It is also hoped that this paper will encourage further research and further accumulation of field experience pertaining to problems in coal refuse disposal.

REFERENCES

(1) American Society for Testing and Materials, "Laboratory Shear Testing of Soils," Special Technical Publication No. 361, Philadelphia,

(2) Bishop, A. W., and D. J. Henkel, The Measurement of Soil Properties in the Triaxial Test, Edward Arnold, London, 1962.

(3) Busch, R. A., R. R. Backer, and L. A. Atkins, Physical Property Data on Coal Waste Embankment Materials, U. S. Bureau of Mines, Washington, 1974.

(4) Busch, R. A., R. R. Backer, L. A. Atkins, and C. D. Kealy, Physical Property Data on Fine Coal Refuse, U. S. Bureau of Mines, Washington, 1974.

(5) Department of the Army, Laboratory Soils Testing, E M 111-2-1906, Washington, D. C., 1965.

(6) Hall, G. A., The Development of Design Criteria for Soil Slopes on West Virginia Highways, Ph. D. Dissertation, West Virginia University, 1974.

(7) Holubec, Igor, "Geotechnical Aspects of Coal Waste Embankments," Canadian Geotechnical Journal, Vol. 13, 1976.

(8) Lambe, T. W., Soil Testing for Engineers, John Wiley and Sons, New York, 1951.

(9) Thomson, G. M., and S. Rodin, Colliery Spoil Tips -- After Aberfan, The Inst. of Civil Engineers, London, 1972.

(10) Zook, R. L., B. J. Olup, Jr., and J. J. Pierre, "Engineering Evaluation of Coal Refuse Slurry Impoundments," Society of Mining Engineers of AIME, 1974.

CLOSED CIRCUIT COAL REFUSE AS A STRUCTURAL FILL

By David C. Cowherd

This paper discusses the use of closed circuit coal refuse as a structural fill material. Several triaxial tests, permeability tests, etc., have been performed on this material in conjunction with a project. The project required the development of a valley fill embankment 500 feet in height. The coal refuse when manufactured could be quite wet and unstable so a method was devised utilizing the properties of the material to make it stable in a large fill, without handling and without interruption of the daily process. This paper presents in detail the strength values, permeability characteristics, index properties, etc., of the combined closed circuit refuse material as well as the method for handling this material in order to achieve a safe, stable valley fill.

The purpose of this paper is to outline the data obtained in connection with a project utilizing combined coal refuse. By the term "combined coal refuse" is meant the combination of coarse coal refuse and fine coal refuse (commonly called "slurry"). The nature of this material characterized by index properties, strength parameters at various densities and moisture contents, permeabilities, etc., and its application to the handling of this material in reference to for a particular project will be outlined and discussed.

The requirements of the particular project in connection with which the data presented in this paper were developed are as follows: The coal cleaning plant at this project was to produce a combined coal refuse to be drained by a vacuum type dewatering system. The project was to be designed for a twenty-five year life of mine as a refuse valley fill composed of both the fine and coarse refuse as a single unit. The requirements of the engineering project were to determine the characteristics of this material and to design a valley fill which would exhibit the proper strength requirements. A particular valley was selected for this project. Volumes were calculated and the height of the valley fill to provide for a twenty-five year life of mine refuse disposal was determined. The overall scheme of this fill is shown in Figure 1.

Basically, at this site the toe of the embankment was to be around elevation 1190 feet, with the top being around 1705 feet, dictating a front face slope of about 500 feet in height.

In order to determine if this project could be constructed of the combined refuse material, it was necessary to know several characteristics of the refuse. It was not known at what moisture content the new plant might produce the refuse. It was, therefore, necessary that the strength values at different moisture contents be determined and the slope of the front face be designed accordingly. It also was obvious that if this material were produced in a wet state, a certain amount of drainage would occur. It was, therefore, desirable to note the permeability at various different degrees of density and moisture content of the material. It had to be determined whether this material would be stable or whether it might flow at the moisture contents produced. If it were to flow, provisions would have to be made for containment.

An existing plant, somewhat similar to the one to be constructed, was found to be producing combined refuse at a moisture content of around 12%. The improvements made with the new plant, it was felt, would permit the production of the refuse at a moisture content of around 9%. It was decided, however, that provisions should be made in the overall design of this project for handling the refuse if it happened to be at 12% moisture. It was decided, therefore, to test this material at moisture contents of 9% to 12% and to design a method of placement around the 12% moisture content.

In addition, it is desirable on a project of this size to do as little handling of the material as possible, so it was decided to determine what would happen if this material were hauled in and placed in 2 foot layers with the only compaction being that achieved by drainage and by equipment traversing the area. It was also of interest to note whether the first layer would be stable enough for placement of a second layer after about twenty-four hours.

The testing program was designed to evaluate all of these ideas. The gradation curves of the combined refuse material are included as Figure 2. The material varied in classification from "SP-SM" to "GW-GM" (in accordance with the Unified Soil Classification System) with the silt and clay sizes varying between 7% and 12%.

A strength testing program was

TABLE 1: TRIAXIAL TESTS ON COAL REFUSE MIXTURE

A. Unconsolidated-Undrained Tests on Laboratory Compacted Samples

Test No.	Sample No.	Dry Unit Weight (pcf)	Moisture Content (%)	Degree of Saturation (%)	Lateral Pressure (psi)	Average % of Standard Proctor	C (psi)	ϕ (°)
1	1	94.9	6.7	41.5	40.0			
	2	100.6	7.7	62.3	40.0	91.6	1.0	30.7
	3	94.7	6.6	41.2	60.0			
	4	96.7	5.7	38.5	80.0			
2	1	90.9	8.4	44.2	45.0			
	2	93.2	8.3	48.7	60.0	87.1	2.3	28.0
	3	91.6	8.6	46.9	80.0			
	4	92.1	9.0	50.1	55.0			
3	1	93.4	12.0	70.3	35.0			
	2	96.2	11.5	76.3	40.0	88.0	8.6	9.8
	3	89.8	11.6	58.7	60.0			
	4	92.2	11.7	64.9	80.0			

B. Consolidated-Undrained Test on Laboratory Compacted Samples

Test No.	Sample No.	Dry Unit Weight (pcf)	Moisture Content (%)	Degree of Saturation (%)	Lateral Pressure (psi)	Back Pressure (psi)	% of Standard Proctor	C' (psi)	ϕ' (°)
As Molded:									
4	1	98.4	11.6				93.2		
	2	96.5	10.3				91.4		
	3	94.5	8.8				89.5		
Consolidated:									
4	1	99.9	9.7	85.5	40.0	20.0			
	2	99.4	10.5	68.8	60.0	20.0		0.0	39.8
	3	98.3	10.6	53.9	80.0	20.0			

embarked upon to evaluate this material both for total and effective strength parameters. These strength parameters were determined by triaxial testing. It was decided that the samples should be 4 inch diameter samples, 8 inches high. Samples of the combined refuse material were compacted to dry unit weights varying between 89.8 and 100.6 pounds per cubic foot, at moisture contents varying between 5.7% and 12.0%. Table 1 above indicates the values of C and ϕ obtained on these samples. The data for these tests are included in Figures 3 and 4.

These tests indicate that the strength parameters vary considerably with the molding moisture content of the material. The moisture content seemed to have more effect on the strength parameters than did the compaction density.

It was somewhat debatable at this point as to whether a total stress analysis or an effective stress analysis would be most appropriate for consideration of the fill. In the first place, provisions were made for drainage internally and externally so that a phreatic line would not be developed in the embankment. The material would not, therefore, be saturated. In addition, effective strength parameters can give results that are not indicative of the actual strength of material in a short term strength analysis, depending on how quickly the material might consolidate to the consolidation pressures used in the triaxial tests and how quickly the material is placed. With this project being constructed over a period of twenty-five years, and with the material being relatively permeable and having a relatively quick rate of consolidation, it is probable that the consolidation would take place rather rapidly. With this project, even though the material is placed loose, by the time any appreciable material is placed density is gained and strength increases. Because of all these factors it was decided to analyze the slope using both total stress and effective stress analyses.

TABLE 2: STABILITY ANALYSES

Slope	Type of Analysis	Soil Parameters				Factor of Safety	
		ϕ (°)	C (psi)	ϕ' (°)	C' (psi)	Static	Dynamic
Total	Total Stress	28.0	330.0			1.69	1.46
Total	Effective Stress			40.0	0.0	2.46	2.13
Phase I	Total Stress	28.0	330.0			1.56	1.36
Phase I	Effective Stress			40.0	0.0	2.13	1.86
Phase II	Total Stress	28.0	330.0			1.63	1.44
Phase II	Total Stress	28.0	330.0			1.76	1.54
Phase II	Effective Stress			40.0	0.0	2.07	1.83
Phase II	Effective Stress			40.0	0.0	2.01	1.77
Individual 2:1 Slope	Total Stress	28.0	330.0			2.05	1.83
Individual 2:1 Slope	Effective Stress			40.0	0.0	1.87	1.66
Total at 12% Moisture	Total Stress	10.1	7.6			0.86	0.76

Obviously, using a total stress analysis based on soil parameters determined from quick tests performed on samples of material prepared at high moisture content and quite loose density is a very conservative assumption. The factor of safety computed in such a manner would be much lower than would ever be approached by the actual construction techniques and methods utilized. This analysis was performed, however, to simply try to determine what could would happen if the entire pile were to be at the very loose density, at a high moisture content all at one time. The results of these analyses are included in Table 2 above.

It was found that if, in fact, the material could be placed at a density of 90% of Standard Proctor at 9% moisture content, the pile would be quite safe with factors of safety of 1.8 and 2.0 for the total and effective stress analyses respectively. However, considering a situation where the pile were placed at 12% moisture, the material would become unstable and problems would develop. This was true obviously only on the total stress analysis.

Avenues were then explored to determine what could be done in the way of draining this material if it were in fact placed at 12% moisture content and what densities might reasonably be achieved in placement. In order to do this, a permeability testing program was instituted. The permeability was determined in the following manner. Samples were compacted in standard Proctor molds. A porous stone was placed at one end of each specimen and water at a constant head was flushed through the samples until a stabilized flow was achieved. The coefficients of permeability so determined are listed below in Table 3.

The permeability tests revealed that the material in a loose state had a permeability of around 10^{-3} centimeters per second but in a dense state the material had a permeability of around 10^{-5} centimeters per second. It was felt that these characteristics could be used to advantage. It was determined that by placing

TABLE 3: PERMEABILITY TESTS

Test No.	Dry Unit Weight (pcf)	% of Standard Optimum Dry Density	Coefficient of Permeability (cm/sec)
1	61.7	58.4	1.39×10^{-3}
2	90.8	86.0	1.83×10^{-3}
3	98.7	93.5	1.03×10^{-4}
4	95.7	90.6	7.22×10^{-5}

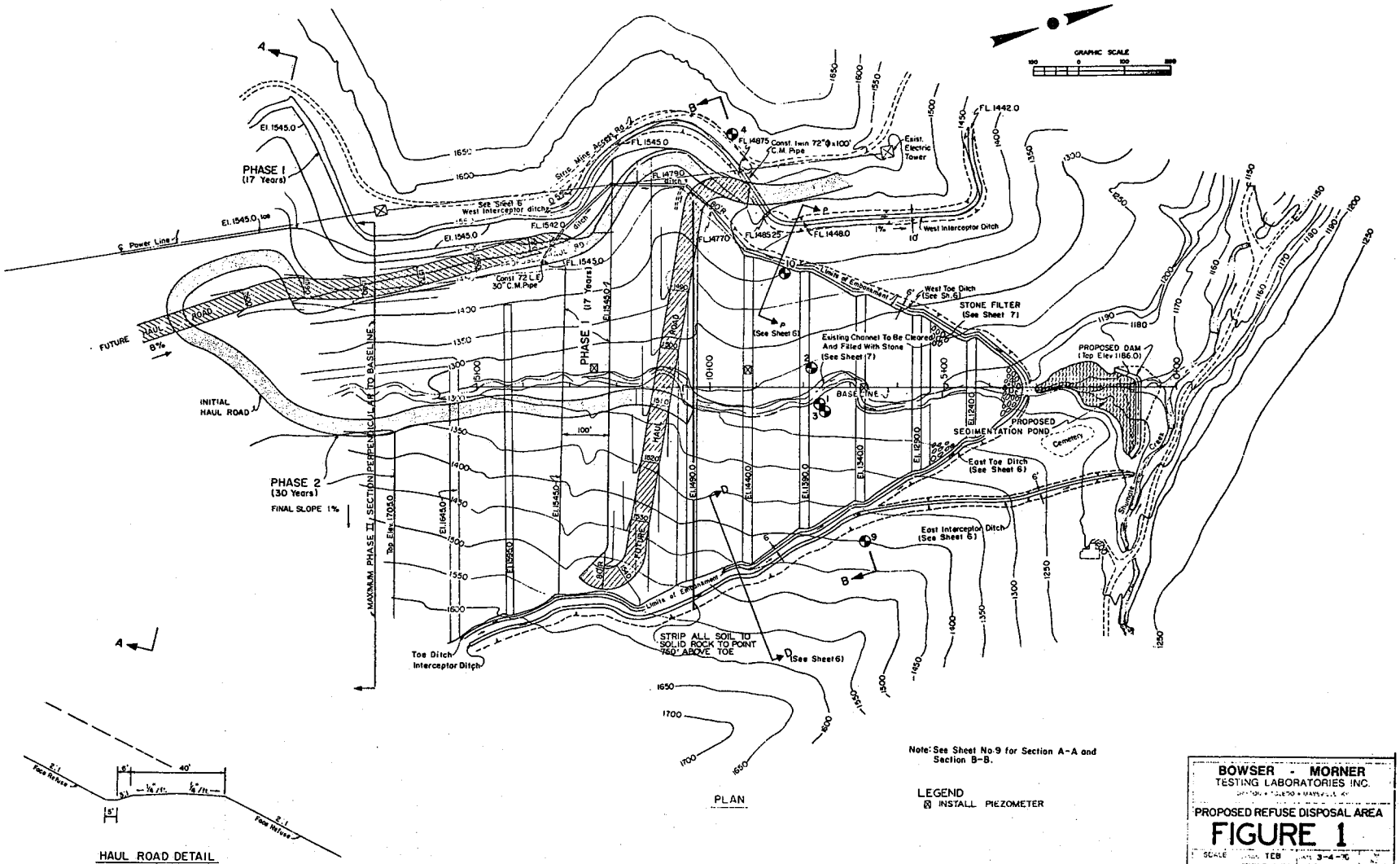
the material on a slope to provide an exit for drainage, the material would be self-draining at the permeability of 10^{-3} centimeters per second. As the material drained, it would in essence compact itself to some degree.

A system was designed for the refuse to be placed on a grade up the valley (the bottom grade of the valley being about 10%) and on a grade toward the side of the valley of 3% with a notch to the side of the valley acting as a drainage ditch. In order to provide containment of the material a series of small, coarse dikes were to be constructed. Drainage would be provided through the coarse dikes and also to the side of the valley toward the notch. In this manner a self-draining system was designed. It was determined that the material would drain sufficiently to be stable within about one days time, therefore, a layer that would take two days to place was scheduled to be placed up and across the valley with the second layer to be placed two days later.

It was determined that the method of compaction after placement was simply to be the haul trucks, which would evenly spread the material over the area and would compact the area as they cross it distributing their material. It was felt that the degree of compaction achieved by the trucks in this manner would be in excess of 90% Standard Proctor and that the moisture content would have drained to less than 9% by the second day, thereby providing for the strengths determined in the triaxial testing procedures at 90% of Standard Proctor and 9% moisture content.

The fact that the permeability decreased to 10^{-5} centimeters per second at this density and moisture content is used to advantage. When the second layer is placed the water drains down through the layer, hits the previously compacted layer (which is now relatively impervious) and drains toward the front and side of the valley, thus providing for drainage of the second and succeeding layers.

In effect, the entire refuse disposal valley fill is constructed by a series of small dikes composed of coarse refuse and compacted on the front face to permit proper drainage, and then by simply placing the combined refuse behind the coarse dikes on a grade toward the dikes and toward one side of the valley to provide drainage. The only compaction necessary is that produced by trucks hauling in the refuse and dozers spreading this material. Good compaction is achieved in this manner and a stable valley fill is produced, with surface water diverted around the pile and internal drainage provided.



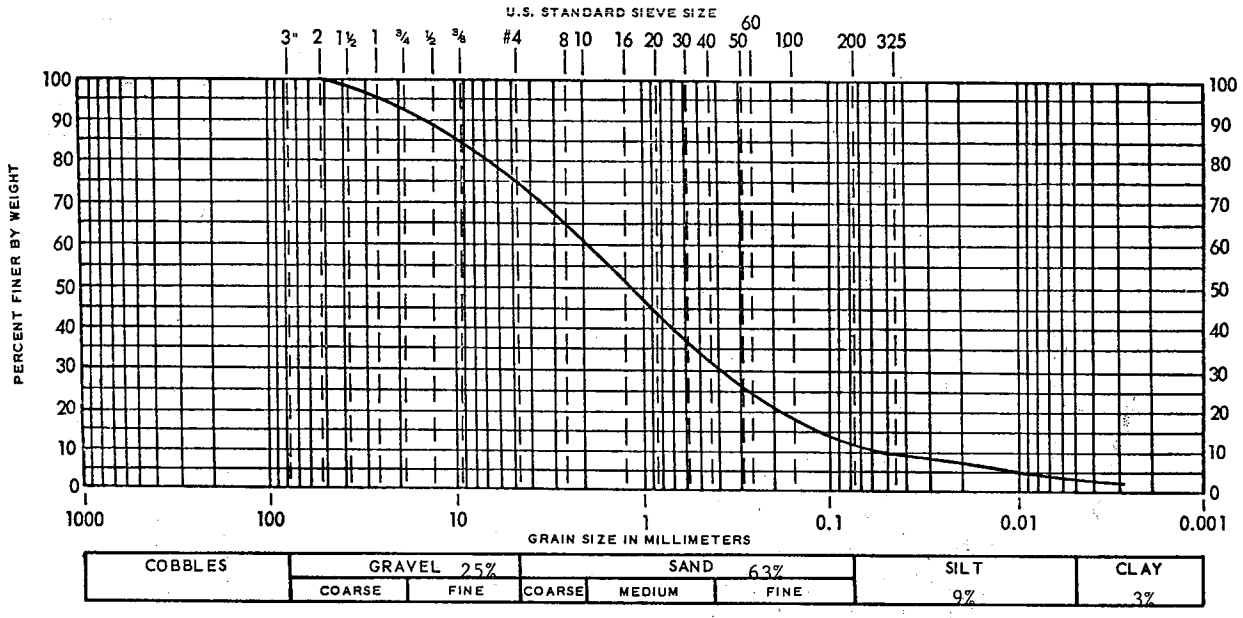
BOWSER - MORNER
TESTING LABORATORIES INC.

PROPOSED REFUSE DISPOSAL AREA

FIGURE 1

SCALE ... TEB ... 3-4-76
... DCC ... 3-4-76 05273

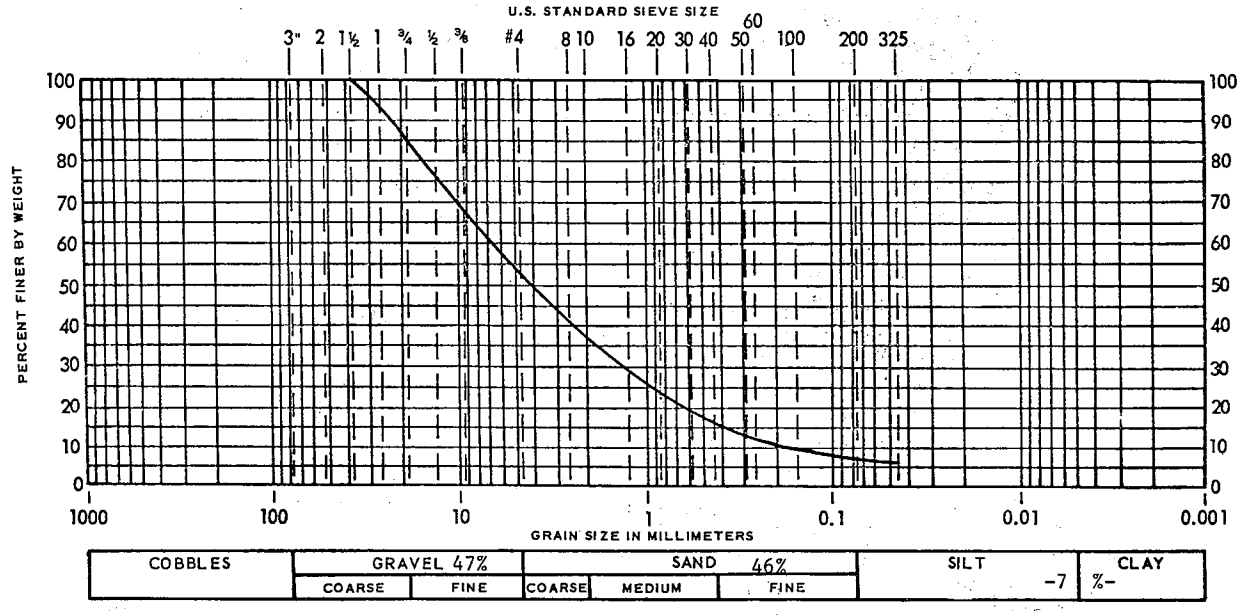
PLAN



CLASSIFICATION "SP-SM"
poorly graded, silty SAND

COLOR Black
REMARKS "U.S.C.S."

LIQUID LIMIT
PLASTIC LIMIT Visually Non-plastic
PLASTICITY INDEX



CLASSIFICATION "GW-GM"
well-graded, sandy, silty GRAVEL

COLOR Black
REMARKS "U.S.C.S."

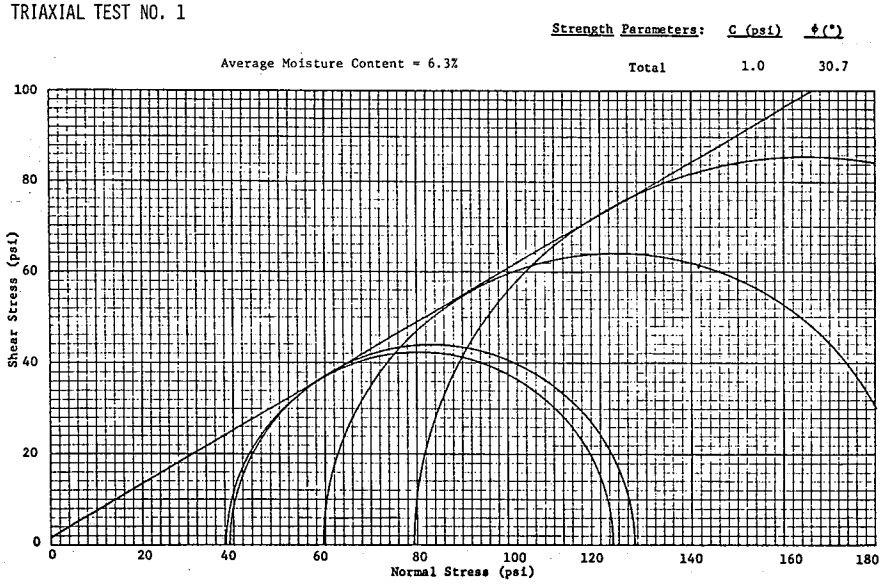
LIQUID LIMIT
PLASTIC LIMIT Visually Non-plastic
PLASTICITY INDEX

BOWSER-MORNER
TESTING LABORATORIES, INC.

FIGURE 2: GRADATIONS OF COMBINED COAL REFUSE

UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST
MOHR PLOTS OF FAILURE STATES

TRIAXIAL TEST NO. 1



UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST
MOHR PLOTS OF FAILURE STATES

TRIAXIAL TEST NO. 2

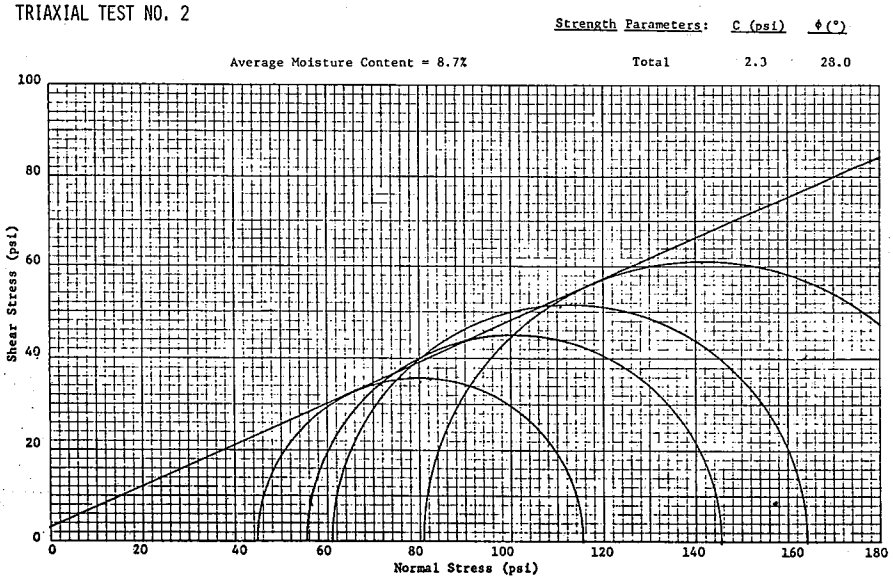


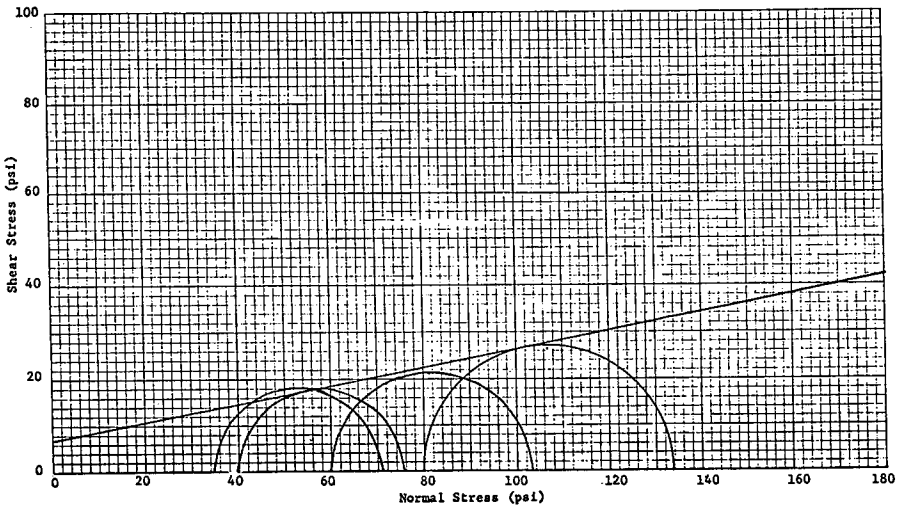
FIGURE 3: TRIAXIAL TEST DATA ON COMBINED COAL REFUSE

UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST
MOHR PLOTS OF FAILURE STATES

TRIAXIAL TEST NO. 3

Strength Parameters:	C (psi)	ϕ(°)
Total	8.6	9.8

Average Moisture Content = 12.0%



CONSOLIDATED-UNDRAINED TRIAXIAL STRESS PATHS

TRIAXIAL TEST NO. 4

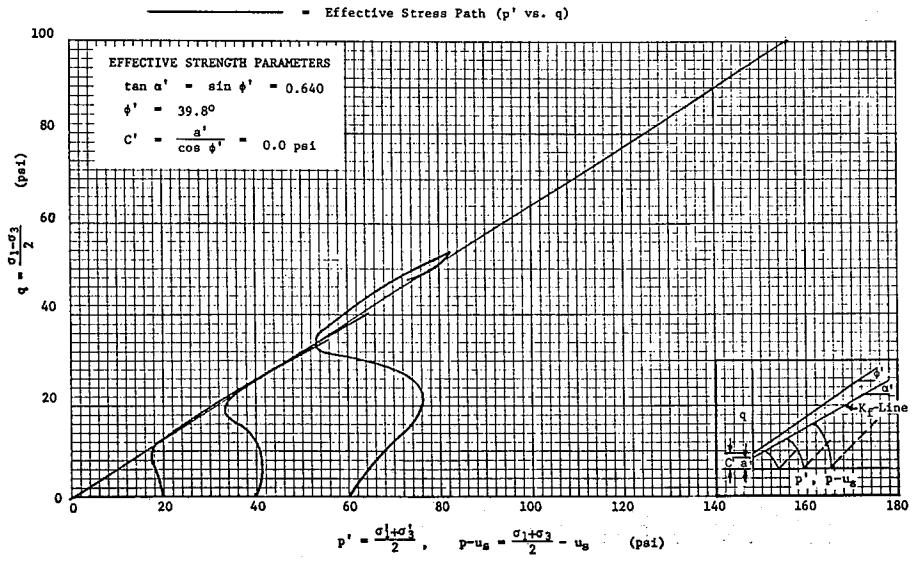


FIGURE 4: TRIAXIAL TEST DATA ON COMBINED COAL REFUSE

PAPER NOT RECEIVED IN TIME FOR PUBLICATION

DYNAMIC DESIGN CONSIDERATIONS OF LOOSE FINE COAL REFUSE

by

Dr. Richard D. Ellison
Vice President
D'Appolonia Consulting Engineers, Inc.
Pittsburgh, Pennsylvania

and

Yo. Y. Cho
Project Engineer
D'Appolonia Consulting Engineers, Inc.
Pittsburgh, Pennsylvania

ABSTRACT

This paper discusses design considerations of loose fine coal refuse when subjected to earthquake loading conditions. It is concluded that standard soil mechanics testing and analysis procedures can be applied. The relatively low density of fine coal refuse must be considered, however. Suggestions for design restrictions which allow material to be used as a structural part of an embankment are presented.

Because of the material's low density, saturated loose fine coal refuse has a high ratio of dynamic shear stress to effective confining stress even at moderate seismic loading. The determination of material properties related to dynamic shear stress and of resistance to liquefaction are important in fine coal refuse design.

Results from resonant column tests indicate that low strain dynamic properties can be represented by Hardin-Drnevich expansion after modification with a reduction factor based on specific gravity. Also, the strain dependent characteristics were observed to be similar to comparable fine sands or silts.

Results from cyclic triaxial tests indicate relatively lower, but still manageable, dynamic strengths compared to soils with similar D_{50}'s. This condition was measured for well-graded fine refuse with a high fine content.

The dynamic test results for a variety of loading intensities and durations are also compared to static triaxial tests conducted on very loose to dense saturated fine refuse. These comparisons establish a basis for determining the most appropriate testing and analysis program for a variety of material characteristics and seismic loading conditions.

PROPERTIES OF COAL MINE FLOOR SHALES

Richard W. Stephenson
Civil Engineering
and
John D. Rockaway, Jr.
Mining, Petroleum and Geological Engineering
University of Missouri-Rolla

Pillar punching, ground squeezing and related rib and roof failures are significant problems in the mining of coal resources of the United States. Such problems can increase safety hazards, reduce the amount of recoverable coal and contribute to increased costs of mining and materials handling. Rib and roof failures have been and are being studied by many people because of the direct relationship between injuries and fatalities to these failures. However, ground squeezes and pillar failure caused by weak substrata have received little attention.

In the Fall of 1975 the University of Missouri-Rolla (under a grant from the U.S. Bureau of Mines) began a three-year study of the pillar-punching, floor-heave problem. The objectives of this investigation included: 1) the evaluation of the strength and stability parameters of the coal mine floor shale; 2) the study of the mechanism of failure; and 3) the application of the results of the investigation to a national design approach for the improvement of the procedures currently in practice for extracting and conserving coal resources.

The evaluation of the strength and stability parameters of the floor shale consisted of a sampling and testing program of approximately six coal mines in the Southern Illinois region. This paper will report on the findings of this phase of the program.

I. INTRODUCTION

A. Statement of Problem

An important geotechnical consideration in the coal mining industry today is the control of the newly exposed roof and floor. Pillar punching, ground squeezing, and related rib and roof failures are well documented as significant problems in the mining of Midcontinent and other coal resources of the United States. Such problems can increase safety hazards, reduce the amount of recoverable coal, and contribute to increased costs of mining and materials handling because the most efficient plan of mine exploitation must be modified. The significance of these problems is recognized in the coal industry. Rib and roof problems are well publicized because of the direct relationship between injuries and fatalities to these failures. Ground squeezes and pillar failures caused by weak substrata however, have received little publicity or attention because serious accidents have not been directly related to these failures. Therefore, very little information exists as to the mechanism of failure or the technical evaluation of mine conditions that have contributed to such failures.

In coal mine stability studies the underclay, pillar, and roof must be considered as a structural unit with varying relative strengths. As such, the failure of any component can initiate the failure of the entire system (Fig. 1). It is possible that the observed failure of one component might actually be the result of the subtle failure of another. For example, the effects of floor failure may be manifested such that: 1) the floor around the base of a support or of a pillar may be heaved up; 2) adjacent supports or pillars may become overloaded, possibly to failure and therefore lost as support elements for the roof and overburden; and 3) the immediate roof may then fail as a result of differential deflection with the result that the entire mine structure will become unstable. It is believed that the engineering properties of the strata underlying the coal measure may be of more significance to the total mine layout and design than has been recognized to date, particularly since, of the three components of roof, coal and pillar failure, the floor is generally the weakest.

B. Objectives

The fundamental objectives of this research were to study the physical properties of coal mine materials (underclay) to determine those factors which contribute to their development or loss of strength. In addition, the process by which floor failures occur was investigated and the applicability of bearing capacity analysis to solving floor stability problems evaluated.

These objectives were to be obtained through an investigation of the engineering properties of the underclays and an evaluation of the response of these materials to the stresses applied by the pillar loads. The results of the investigation include:

1) A summary of data regarding weak floor conditions and a statistical analysis of the physical properties of underclay materials as they are applicable to bearing capacity analysis;
2) A description of the floor failure process(s) and an evaluation of the significance of specific engineering properties of the substrata with respect to their relationship with bearing capacity;
3) An analysis of the engineering data required to determine the bearing capacity of the underclay and a comparison of predicted bearing capacity from field and laboratory data with actual conditions in specific mines; and,
4) Recommended practices for the design of coal pillars supported by weak floor conditions.

The bearing capacity of the floor of the mine therefore, must be considered as a fundamental factor in any program of strata control. Floor materials range from those which have very little cohesion and are dependent for their strength almost entirely upon internal friction to very strong rocks that are quite brittle in their nature. Some floor materials exhibit a high degree of plasticity, others are almost perfectly elastic. Lack of homogeniety is a feature of almost all coal mine floors. In addition, during the process of mining, the floors are exposed to a number of external factors such as the presence of water and considerable variations in the rate of loading. It is, however, imperative that the factors which affect bearing capacity of a floor be thoroughly understood and that the strength and stability parameters of floor materials be evaluated with respect to their capacity as bearing members. An approach to fundamentally similar problems has been established in the field of foundation engineering and it is anticipated that the concept of analyzing substrata as foundation materials, subject to column loads from the pillars will be applicable to coal mine strata control.

This project is nearing the completion of the first year of a projected three year study. In an effort to make the results of this work available to the profession as rapidly as possible this paper will concentrate on those phases of the project most nearly complete; i.e., parts 1) and 2). As more data is compiled and analyzed it will also be released.

C. Scope of Investigation

In recognition of the lack of substantive information on the floor heave of coal mines, the U.S. Bureau of Mines awarded a research grant (J0155153) to the University of Missouri-Rolla for the express purpose of analyzing the pillar stability-floor heave problem. The project is a joint effort between the Department of Mining, Petroleum and Geological Engineering and the Geotechnical Division of the Department of Civil Engineering.

The scope of the project is essentially laboratory oriented in that testing of the engineering properties were made on intact samples under laboratory conditions and a minimum of in-situ testing was included in this phase. The laboratory tests used were conventional procedures employed in soil and rock mechanics and the equations for bearing capacity analysis are commonly accepted formats. The entire investigative program was designed with the concept of establishing a fundamental data base of the engineering properties of underclay by standard procedures and applying this data, through conventional equations, to solving the problem of floor failure under coal pillars.

The scope of a research project of this nature must invariably be determined by the facilities made available and the cooperation offered by the mining industry. The project has been strongly supported in this report. Access to particular coal mines of interest was willingly provided and the project staff were able to arrange the necessary logistical and personnel support whenever necessary. As a further measure, many of the companies made appropriate files and historical records available for our review and expressed a sincere interest in the conduct of this research.

II. DESCRIPTION AND PHYSICAL CHARACTERISTICS OF UNDERCLAY

The strata immediately underlying a coal seam is referred to by many terms; "seat earth" or "seat rock", "ganister", "fire clay", "underclay" among them, but the term "underclay" is as commonly used as any other term in this country and will be used in this report. Underclay has been described as "a seat rock associated with a coal bed composed mainly of clay minerals, that is generally nonbedded, slickensided, irregularly fractured and contains traces of plant roots and concentrations of iron or calcium compounds in the lower parts" (1). It is greenish, blueish or brownish grey in color and is commonly one to four feet (.3 to 1.2 m) thick under the coal although it may vary widely in thickness under the same seam. It often grades downward and occasionally horizontally into sandstone, shale or limestone.

The origin of underclays has been discussed extensively in the literature. They are generally considered to have formed in association with deltaic deposits in interdistributary bays or on the flood plain faces of the delta although other interpretations have been proposed (2,3,4). Underclays are characterized by a lack of bedding or structure but there is a vertical zonation that is typical of most deposits and many writers (5,6) suggest that underclays have formed as soils in an environment of swamp or marshland. The upper zone appears as a dark, carbonaceous clay, commonly containing root films which give it a streaked appearance. This zone is non-calcareous, leached of iron

and calcium, and is believed to have been subject to weathering (7,2). It is seldom more than one foot thick and is frequently disturbed by mining activities. This layer is underlain by a leached, highly plastic clay that varies in thickness from a few inches to a few feet and contains abundant fissures and slickensides. This is the horizon that is noted for its plasticity when wet although locally it may be replaced by the less plastic flint clay or some combination of both. This middle zone is also free of carbonates. The lower zone is generally less plastic than the middle zone and gradationally changes into the strata underlying the underclay. It is generally enriched in iron compounds and carbonate and limestone or iron carbonate nodules and pyrite are often found in this zone.

There is almost a complete mineralogical gradation between the plastic and the flint underclays and there have been three different types of underclays defined on the basis of clay mineralogy and plasticity (1,8). Plastic underclays are mixtures mostly of illite, montmorillonite, mixed layer illite montmorillonite with minor amounts of kaolinite, quartz and small amounts of calcite or gypsum. This is the most common type of underclay and will vary from moderately hard to soft, but it becomes highly plastic when wet and is extensively fissured. The flint clays are mainly kaolinite with small amounts of illite or mixed layer clay minerals. It is hard, resistant, does not develop plasticity when wet and is not as extensively fissured as the plastic clays. Semiflint clay is the third type, and is intermediate in mineralogical content between the two, containing from 60 to 80 percent of kaolinite and up to 40 percent quartz. It becomes plastic when wet and is commonly fissured. The clay mineralogy characterizing an underclay deposit varies with respect to vertical zonation and in addition, mineralogical composition will vary from location to location and no defining mineralogical assemblage can be described (2,9,10).

The lack of bedding or structure, but the presence of a random pattern of fissures is a characteristic of most underclay deposits. Some writers believe that lack of bedding is the result of the deposition of the clay minerals in a flocculated state (11,12) or that it may be the result of bioturbation from either plant roots or animal life which destroyed the original bedding, or perhaps a combination of these factors (13,14).

Fissures, however, are common features of underclays. They are randomly oriented, discontinuous and seldom more than a few inches in length. They are generally concentrated in the upper, more plastic zones and often form along carbon films associated with the decay and compression of plant roots which provide planes of weakness in the upper part of the underclay. Slippage apparently takes place along the carbonaceous residues which act as a plane of sliding, although flocculation of clay during deposition together with slippage during compaction could also have caused the slickensides. Recent slickensides, as opposed to those caused during consolidation, and tectonic movements most probably have occurred as a result of post mining stress relief (15).

Although there is much information available regarding the geological characteristics of underclay (mineralogy, paleoecology, stratigraphy, etc.), very little information is available on the physical properties as they relate to coal mine floor squeezing. Most of the data describes physical and chemical characteristics as they relate to ceramics or other industrial purposes. Although this data can be used to support engineering studies, it does not present a complete description of the engineering properties. Additional data occasionally may be obtained from confidential files, but few coal companies have the facilities to carry out a thorough testing program.

III. EXPERIMENTAL PROGRAM

A. Field Procedures

The experimental program followed throughout this investigation was organized under the concepts of going from the general to the specific, i.e., from observations of actual case histories and documentation of the squeeze problem in-situ to detailed analysis of engineering data obtained from core samples by laboratory testing. In this respect, the first step of the study was to arrange visits to mines experiencing weak floor conditions for inspection and collection of block samples for lab analysis. Where conditions warranted further investigation, return visits were made with the core drilling apparatus to obtain undisturbed samples for strength testing. In individual cases, other tests such as plate loading and seismic velocity were carried out where it was felt that they would provide useful data on in-situ conditions.

1) Inspection Visits

During the course of this investigation, staff personnel visited a total of 20 underground mines. Not all the mines visited were experiencing floor squeeze problems. However, trips to these mines were scheduled to collect samples of "good" underclay and to document mining conditions for comparison purposes.

Standard procedures on the inspection trips to squeezing mines were to visit the areas of floor movement, record the extent of the floor squeeze, and map the room and pillar dimensions in the immediate vicinity of the squeeze problem. The form of floor movement was illustrated or photographed where possible, and block samples collected for laboratory analysis. Additional samples were normally taken from locations outside the squeeze prone areas for comparison purposes as were samples from mines not experiencing floor failure. Discussions with mine personnel usually provided information on the mine development plan, mining, operations and nature of the floor failure. In most cases, some form of geological log for evaluation of overburden conditions was available from company files.

In addition, during the early stages of this study, several surface mines were visited to obtain block samples for laboratory analysis of their physical properties.

2) Core Sampling

Additional information on the intact strength properties of the underclay was required for those mines selected for detailed investigation. To obtain this data, it was necessary to recover core samples of sufficient size and continuity such that they could be tested in laboratory triaxial apparatus.

The equipment selected for mine floor sampling was a Target Mini-Con concrete hole saw. The drill is driven by a 3 1/2 horsepower, 230 volt AC/DC electric motor. It is relatively lightweight weighing approximately 150 pounds when assembled. It may be dissembled into several component parts so that it is readily portable by a two or three man crew. It may be cooled by either air or water although the water cooling mode of operation was used since tapping into mine water lines provided more flexibility than was available when an air compressor was required.

Initially an NWG diamond bit with 19 chips/carrat was used for drilling. However, this proved to be very slow since the underclay would cake between the diamond chips and reduce their cutting efficiency. The diamond bit was replaced with an NWM tungston carbide sawtooth bit which was very effective for drilling the underclay. The diamond bit was used only when limestone strata was encountered.

The bit and a two-foot long, double tube, swivel type core barrel were extended down the hole by means of two foot sections of A rod. Two foot sections of N rod were used to stiffen the drill string at depths greater than 8 or 10 feet. The deepest hole drilled by this apparatus was 20 feet from the base of the coal, although additional footage could have been drilled since drilling was stopped when an underlying coal seam was encountered and not because of equipment limitations.

Room and pillar dimensions in the immediate vicinity of the boring sites were recorded as was any information regarding mining activities and squeeze history at the drilling site. In addition, a micro-seismic survey was run across the area to determine representative in-situ compression wave velocities for comparison with laboratory run sonic tests and to define changes in stratigraphy for correlation with drilling logs.

When the core was received at the laboratory, it was inspected for any alterations that might have occurred during shipment and samples were selected for water content analysis which were run immediately. The core was then logged in detail and further samples selected for the various tests of physical properties.

B. Laboratory Procedures

After the underclay was received in the lab and the lithology described in detail, samples were selected for testing. The tests included Atterberg limits, clay mineralogy, grain size distribution, natural water content, specific gravity, sonic velocity, triaxial strength and unconfined compressive strength.

All laboratory tests were conducted in the University of Missouri-Rolla Civil Engineering Soil Mechanics Research Laboratory and the Rock Mechanics Laboratory of the Department of Mining, Petroleum and Geological Engineering.

The selection of samples for the specific tests was based, to a certain extent, on variations in lithology in that an attempt was made to test specimens representative of all lithological units. When selecting samples from the core, however, first priority was given to allocating pieces of core at least 4 to 6 inches long to triaxial or unconfined strength testing. After failure, samples from the broken specimen were used for testing of the other physical properties. The testing sequence and organization of the testing program is illustrated in Figure 2.

1) Strength Testing

Sample Selection--All samples chosen for triaxial or unconfined compression testing were NWG (2 1/2 inch diameter) size cores obtained from mine floor surface borings. The cores were taken with a double tube core barrel which allows the outer tube to rotate while the inner tube remains stationary.

The core was removed from the barrell immediately, logged and sampled for water content. The core was placed in a specially constructed core box for transportation to the laboratory.

Upon arrival at the laboratory, the core was again logged. Samples were selected for the various tests and all the material marked for identification and placed in polyethylene bags for storage.

The highly fissured nature of the underclay material generated problems in sample preparation for strength testing. Many samples were fissured to such an extent that they could not be used. Other samples initially sound would, in a relatively short period of time, open cracks and deteriorate to such an extent to render them unusable. Obviously this occurred due to stress relief of the micro fissures existing in the material.

Consequently, the strength tests were conducted on those samples that were relatively sound and could be prepared for testing. This biased the data toward the strong side to an unknown degree.

Sample Preparation--Due to the fragile nature of the test material it was not possible, as a rule, to use normal sample preparation procedures. Rather than grinding the sample ends parallel, the specimen was first placed in a lucite cylinder. Plaster of paris was then placed on the specimen ends and struck smooth with a glass plate and allowed to air dry. The plaster dried in approximately 30 minutes. The minimum plaster thickness was of the order of 1/8 inch.

The test specimen had diameter of 2 1/8 inch. Height to diameter (H/D) ratios varied from 1.7 to 2.8.

Unconsolidated Undrained Triaxial Testing Procedure--Two 0.005 inch thick rubber membranes were placed over the specimen to prevent intrusion of the cell fluid into the sample. The specimen was then placed between the special lucite end caps that were built to reduce the load plate size and friction transmitted to the specimen. The specimen was placed in a triaxial cell and the specified confining pressure applied.

The tests were conducted at a constant rate of strain of 0.1% per minute. Readings of load and vertical deformation were taken until failure. The test was conducted in general accord with ASTM D-2664.

The resulting stress-strain curves were corrected in two manners. The first consisted of correcting the results for deformation of the cell and

lucite end caps at the relatively high confining pressures (150 and 300 psi) and axial loads (>10,000 lbs) used.

The second correction was a shifting of the axis. During sampling and trimming procedures micro-fissures develop due to stress relief. Consequently, during reloading in the triaxial cell, significant vertical deformations sometimes occurred before vertical load was of any appreciable magnitude. This vertical deformation at very low loads was therefore subtracted from the measured vertical deformations to yield a more descriptive representation of in-situ stress-strain behavior.

2) Results of Triaxial Tests

The triaxial testing program yielded three parameters of interest to this study. First was the ultimate compressive strength, q_{ult}. Secondly, the secant "modulus of elasticity" drawn to the ultimate compressive strength (E_{S100}). Thirdly, the secant "modulus of elasticity" drawn through 50% of the ultimate compressive strength (E_{S50}).

The strain at failure and hence the moduli value must be corrected for the influence of the plaster of paris end caps. This was done using a method presented by Ganow (15).

Unconfined Compression Testing Procedure--Unconfined compression tests were performed in general accordance with ASTM D-2938. The resulting parameters were the same as for the triaxial tests; i.e., q_{ult}, E_{S50}, and E_{S100}. Corrections were made for zero shift and the plaster end caps in the same manner as for the triaxial tests.

3) Tensile Strength Testing

Tensile strength testing was conducted on selected specimens for two basic reasons. First, one of the mine floor failure modes seemed to be associated with a tensile failure of a hard layer existing above a softer layer of material. Therefore, it was felt necessary to ascertain the tensile strength of the material.

Secondly, the test method selected allowed shorter samples to be used. This allowed strength testing of samples that could not be tested compressively due to low H/D ratios.

The tensile test method selected was proposed by Carneirs and independability by Akazawa and is independently referred to as the Brazilian Tensile test. The sample is placed horizontally between bearing plates of a testing machine and loaded to failure in compression. To prevent crushing along the loading line, the load is distributed over an area by applying the load to a narrow strip. If the diameter of the specimen is D and the length H, and a line load F is applied along the length of the specimen at AB and CD, a uniform tensile stress will develop according to the following formula:

$$\sigma = \frac{2F}{\pi DH} \qquad (1)$$

If $F = F_c$ = load at failure then

$$T_o = \text{tensile strength} = \frac{2F_c}{\pi DH} \qquad (2)$$

The results of the tensile strength testing are presented in Table 1 as strength type "B".

The point load method of strength testing was employed in the field on freshly removed and unprepared NX size core immediately upon removal from the core band. The objective of this test procedure was to obtain laboratory type test data on strength properties as soon as possible so that variations due to environmental changes would be minimized, and to determine if data from this method of strength indexing could be correlated with data on strength from other procedures. Thus, if the correlation is high, this test would provide a convenient method for the evaluation of mine floor strength properties during mine development operations.

The testing machine used for the point load was Terrametrics Point Load Tester, Model T-500. This mode is a hand held portable tester with a maximum load capacity of 10,000 lbF (45 KN). Because of the low strength of the floor materials being tested a pressure gauge with a load range of 0 to 1750 lbF was used to maximize the accuracy of the load readings.

For the purpose of this investigation, the point load test was not employed throughout the entire core length, but rather on samples which were considered to be too small for triaxial or unconfined testing. A further limitation on the point load test was that a portion of the core, especially some of the underclays, were too soft or fractured for even this low load type of "unconfined" testing. The point load strength was determined in orientations both parallel to bedding and perpendicular to bedding. In this way, the degree of strength arisotropy could be determined for the materials in question.

The Point Load Strength Index, IS, is calculated as the ratio P/D^2 where P is the load required to break the sample and D is the distance between platen contact points at failure. The point load strength data for this investigation is reported for NX size core (unadjusted for size variation), because it is felt that NX is the most reasonable standard to use. The point load data is also reported directly as IS instead of converting it into the correlated Uniaxial Compressive Strength in order to maintain this data as a discrete variable separate from the Uniaxial Compressive Strength data also obtained. Results of the point load tests are presented as strength type "P" in Table 1.

4) Ultrasonic Velocity Tests

The ultrasonic test for this testing program was employed as an approximate, but rapid determination of ultrasonic P-wave velocities of underclay samples. The stringent requirements on sample preparation and test procedure as stated in ASTM D2845-69, were modified by following the general test procedure as outlined by Deatherage (19). The rapid technique made it possible to apply this test procedure to a large number of samples in a reasonable amount of time so that the drill cores could have essentially ultrasonic data recorded for their entire length.

By performing this rapid evaluation of ultrasonic velocities through sections of core it was hoped that data from some of the more time consuming or expensive tests, such as strengths or plasticity indices, would be correlated to ultrasonic velocities and this correlation extended to por-

tions of the core covered by ultrasonic tests only. The ultrasonic velocity parameters considered in this correlation were:

1) velocity parallel to bedding;
2) velocity perpendicular to bedding; and
3) velocity parallel divided by velocity perpendicular to bedding.

The ratio of parallel velocity divided by perpendicular velocity was obtained to evaluate the anisotropic properties of the core samples.

IV. RESULTS

As of this writing the data is available only for the River King Coal Mine in St. Clair County, Illinois. This data will be reported below.

A. Borings

Seven borings were made at the River King Coal Mine, St. Clair County, Illinois. A typical subsurface profile is shown in Fig. 3 . The underclay was beneath the Herrin #6 coal which is itself below about 50 feet of overburden. About 11 feet separated the Herrin #6 coal from the Harrisburg-Springfield #5 coal position below.

The underclay sampled is generally dark greenish-gray in color (5Y5/1), poorly to moderately indurated, non-calcareous and the upper 6 to 25 inches, often fossiliferous throughout, and often very friable due to the presence of many slickensided fractures. The underclay encountered in 3 of the 7 holes had limestone nodules occurring throughout while the underclay of 3 of the holes was free of limestone nodules in the top 12 to 25 inches. A final hole had no limestone nodules.

In all but one hole limestone was found below the underclay at depths ranging from 21 to 36 inches. Claystone containing a large concentration of limestone nodules was found below the underclay in one hole. The limestone encountered was light to medium green-gray in color (5Y7/1), fine grained, well indurated, containing fossil hash along certain horizons, and often broken or brecciated by shear zones which typically are filled with a light gray-green, calcareous and non-calcareous, clay material.

A second hole passed into claystone at a depth of 90 inches. The claystone is light green-gray in color (5Y6/1), moderate to well indurated, and contains limestone nodules throughout.

A third hole at a depth of between 40 and 52 inches, contained a narrow lense of medium gray, non-calcareous, moderately indurated claystone containing no limestone nodules.

Hole four passes from limestone to claystone at 99 inches. This claystone contains a high percentage of limestone nodules, is well indurated, gray to dark greenish-gray in color, calcareous in its upper portions and has been sheared in several areas. Below the claystone at 134 inches is an underclay-like, poorly indurated, dark greenish-gray (5Y4/1), non-fossiliferous, non-calcareous, very friable, slickensided claystone. This claystone may well be the prospective underclay for a Springfield #5 coal. Claystone again appears at 166 inches to the bottom of the hole.

B. Clay Mineralogy

Examination of the clay mineralogy of underclay samples from seven borings at the River King mine and from one boring each at Ziegler #5 and the Murdock mine was carried out by Illinois Geological Survey, under the direction of Dr. Arthur White. A total of 50 samples were analyzed by the X-ray diffraction method for kaolinite, illite, montmorillonite, chlorite and mixed layer clay minerals and the results reported as "parts in ten" (Table 2) In addition, the relative percent of expandable crystal structures in the mixed layer clay minerals was reported as percent hydration.

The samples from the River King mine contained no chlorite, a minor amount of kaolinite and moderate amounts of illite and montmorillonite. The mixed layer clay minerals were the major constituent and apparently were composed of a combination of illite and montmorillonite crystal layers. Eight of the samples contained a few percent gypsum.

The underclay samples from the Ziegler 5 and Murdock had a significantly different distribution of clay minerals. For example, no discrete montmorillonite crystals were reported in either boring, whereas kaolinite was much more abundant and up to 1-2 parts in ten of chlorite was present. As with the River King mine, the mixed layer clay minerals were the most predominant mineral in these samples, however, test indicated that chlorite, as well as illite and montmorillonite, was a component of the expandable mixed layer structure. No gypsum was reported. All of the underclay samples from all borings contained from 10 to 15 percent quartz less than two microns in diameter included within the clay fraction.

Many authors have considered the presence of montmorillonite to be an important parameter in defining the strength properties of underclay. This is based in part upon its high values for specific surface swelling potential, plasticity and activity, although it is not felt that clay mineralogy alone is the determining factor. An estimate of the total percentage of "montmorillonite" crystal layers in clay fraction was computed by adding the "parts in ten" of discrete montmorillonite minerals (Col. 3, Table 2) to a value determined by multiplying the percentage of hydratable layers reported in the mixed layer clay minerals (Column 7, Table 2) times the "parts of ten" representing the relative abundance of the mixed layer clay minerals (Column 4, Table 2). In this way, an approximate value for the total amount of "montmorillonite" crystals was determined that would indicate the relative abundance of the expandable clay mineral in the clay fraction.

The percentages of the significant clay minerals have been plotted on a triangular diagram which permits the representation of trivariate data of a constant sum when two variables are independent and a third dependent. In this case the constant sum is ten and the relative amounts of the three variables, kaolinite, "montmorillonite" and illite parts in ten. The distribution of the clay mineral percentages is apparent by visual inspection of the table.

Table 1. Laboratory Test Data

Boring RK-2-12-76-1 — Coal Seam Herrin #6 — Site 1st Panel South off Submain West — Sheet 1 of 3

DEPTH BELOW COAL (in)	LITH.	STRENGTH (psi)	STRENGTH TYPE	SECANT MODULI E_{50}	SECANT MODULI E peak	WATER CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	SHRINKAGE LIMIT %	SHRINKAGE RATIO	P_{ds}
6-11	UC	262	U	26,150	24,670	10.6	60.1	20.5	39.6	16.8	1.79	1.93
11-16	UC	205	U	32,063	27,360	11.6	81.5	24.4	57.1	23.5	1.55	2.10
33.5-37.5	LS	712	B			9.0	66.0	21.3	44.7	16.3	1.88	2.37
37.5-42.5	LS	2.32 - 192 - 212	P			9.0	66.0	21.3	44.7	16.3	1.88	2.37
		38 - 46 - 42	P P									
43.5-50	CS	380 - 52 -	P P			6.7	43.8	21.5	22.3	18.5	1.81	3.21
		1338	U	180,811	178,400							
52.2-56	LS	240 135 195 190	B B B			6.9	* 43.8	* 21.5	* 22.3	* 18.5	* 1.81	3.12
60-65	LS	10,534	U	993,774	1,022,718	1.6						

Boring RK-2-12-76-1 — Coal Seam Herrin #6 — Site 1st Panel South off Submain West — Sheet 2 of 3

DEPTH BELOW COAL (in)	LITH.	STRENGTH (psi)	STRENGTH TYPE	SECANT MODULI E_{50}	SECANT MODULI E peak	WATER CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	SHRINKAGE LIMIT %	SHRINKAGE RATIO	P_{ds}
65-70.5	LS	7,533	U	1,141,515	1,77,031	1.5						
70.5-76.2	LS	5,464	M	666,341	515,472	1.5						
76.2-85.8	LS	11,761	U	639,130	864,779	1.5						
85.8-88.8	LS	385 9,705	P	915,472	1,128,488	8.0						
95.2-99.8	LS	5,365	U	724,865	766,429	1.8						
99.8-107.8	LS	1,446 1,002	B U	210,947	148,444	5.8	54.6	20.1	34.5	22.3	1.77	3.47
107.8-111.8	LS	1,785	U	364,082	309,091	2.2						

Boring RK-2-13-76-1 — Coal Seam Herrin #6 — Site 1st Panel South off Submain West — Sheet 1 of 3

DEPTH BELOW COAL (in)	LITH.	STRENGTH (psi)	STRENGTH TYPE	SECANT MODULI E_{50}	SECANT MODULI E peak	WATER CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	SHRINKAGE LIMIT %	SHRINKAGE RATIO	P_{ds}
4.5-12.5	UC	63	B			12.7	59.2	26.7	32.5	17.5	1.81	2.10
		366	T_{300}	9,541	9,625							
12.5-18.8	UC	58 108 78	B B			10.4	68.8	24.4	44.4	16.3	1.79	2.35
18.8-22.0	UC	99 31 65	B B			9.2	98.4	33.6	64.8	10.7	1.77	3.65
36.-40.5	LS	66	B									
42.5-51.5	LS	1814 677 1232 1241 1643	B B B T_{300}	281,656	227,553	9.9						
51.5-63.5	LS	1981 904 1443 6506	B B U	963,852	1,049,355	1.4						
63.5-69.5	LS	6948	U	643,333	643,333							

Boring RK-2-13-76-2 — Coal Seam Herrin #6 — Site 1st Panel South off Submain West — Sheet 1 of 2

DEPTH BELOW COAL (in)	LITH.	STRENGTH (psi)	STRENGTH TYPE	SECANT MODULI E_{50}	SECANT MODULI E peak	WATER CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	SHRINKAGE LIMIT %	SHRINKAGE RATIO	P_{ds}
9.0-16.5	UC	270	T_{300}	61,472	24,422	10.3	59.8	27.5	32.3	15.2	1.85	2.67
19.0-24.5	UC	290	T_{300}	98,835	35,321	11.5	101.4	26.8	* 74.6	* 20.3	* 1.75	* 2.33
24.5-28.5	UC	66	B			10.4	101.4	26.8	74.6	20.3	1.75	2.58
48.5-54	LS	1612 120 794	P P T_{300}	263,978	374,511	3.0						
54-60	LS	7320	U	580,952	580,952	3.3						
60-65.5	LS	8627	U	1,091,899	1,098,981	2.8						
65.5-71.0	LS	1655 305 9110	P P U	813,393	813,393	2.8						

Boring RK-2-13-76-2 — Coal Seam Herrin #6 — Site 1st Panel South off Submain West — Sheet 2 of 2

DEPTH BELOW COAL (in)	LITH.	STRENGTH (psi)	STRENGTH TYPE	SECANT MODULI E_{50}	SECANT MODULI E peak	WATER CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	SHRINKAGE LIMIT %	SHRINKAGE RATIO	P_{ds}
73-78	LS	980	B			2.4						
		8482	U	744,035	712,773							
82-87	LS	7790	U	378,155	509,150	3.5						
87-91.5	LS	1345 9758	B U	487,900	582,567	3.1						
96.5-102.5	LS	2181 310 629 470	T_{300} B B	91,054	118,437	6.0						
107.8-115.8	LS	621	B			3.4						
115.8-120	LS	836 10,603	B U	1,076,447	1,076,447	1.6						

Boring RK 12-18-75-1 — Coal Seam Herrin #6 — Site 6th Panel East off Submain North — Sheet 1 of 1

DEPTH BELOW COAL (in)	LITH.	STRENGTH (psi)	STRENGTH TYPE	SECANT MODULI E_{50}	SECANT MODULI E peak	WATER CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	SHRINKAGE LIMIT %	SHRINKAGE RATIO	P_{ds}
0-15.0	UC						85.4 87.6 86.5	26.5 27.1 26.8	58.9 60.5 59.7	13.7 10.7 12.2	1.94 2.02 1.98	
		35	T_{300}	3,405	1,793							
19.0-23.0	UC	340	T_{300}	79,120	55,104		62.7	23.2	39.5	15.7	1.91	
24.8-27.0	UC	177 126 101	B B			* 13.0	* 69.6	* 20.0	* 49.6	* 12.4	* 1.99	* 1.54
48.5-51.0	UC	39 144 91	B B			* 9.4	* 76.6	* 16.6	* 60.0	* 14.4	* 2.00	* 1.77

Boring RK-12-19-75-1 — Coal Seam Herrin #6 — Site 6th Panel East off Submain North — Sheet 1 of 1

DEPTH BELOW COAL (in)	LITH.	STRENGTH (psi)	STRENGTH TYPE	SECANT MODULI E_{50}	SECANT MODULI E peak	WATER CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	SHRINKAGE LIMIT %	SHRINKAGE RATIO	P_{ds}
13.2-19.5	UC	74	B			9.1	70.6	* 22.7	* 47.9	* 19.1	* 1.78	* 2.49
24-27	UC	38	B			6.9	* 70.6	* 22.7	* 47.9	* 19.1	* 1.78	* 3.29
32.5-44.4	UC	95	B			8.5	81.2	60.2 21.0	20.0		1.92	2.47
		938	T_{300}	114,675	93,422							
44.5-50	UC	36 166 101	B B			3.5	51.6	33.3 18.3	19.0		1.77	5.23

Boring RK-12-19-75-2 — Coal Seam Herrin #6 — Site 6th East off Submain North — Sheet 1 of 1

DEPTH BELOW COAL (in)	LITH.	STRENGTH (psi)	STRENGTH TYPE	SECANT MODULI E_{50}	SECANT MODULI E peak	WATER CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	SHRINKAGE LIMIT %	SHRINKAGE RATIO	P_{ds}
18.5-26	UC	32	B			11.5	* 65.7	* 26.5	* 39.2	* 21.7	* 1.75	* 2.35
	UC	2,261	T_{300}	215,857	133,608	9.2	40.5	19.0	21.5	22.0	1.74	2.07
30.75-37.25	UC	2,170	u	692,708	489,376	* 9.2	* 40.5	* 19.0	* 21.5	* 22.0	* 1.74	* 2.07
37.25-41.25	UC	14.4	B			9.7	* 61.5	* 25.5	* 36.0	* 20.8	* 1.82	* 2.63
41.25-45.75	UC	1,264	T_{300}	41,435	51,760	7.3	61.5	25.5	36.0	20.8	1.82	3.49

Boring RK-21-31-76-1 — Coal Seam Herrin #6 — Site 1st Panel South off Submain West — Sheet 2 of 3

DEPTH BELOW COAL (in)	LITH.	STRENGTH (psi)	STRENGTH TYPE	SECANT MODULI E_{50}	SECANT MODULI E peak	WATER CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	SHRINKAGE LIMIT %	SHRINKAGE RATIO	P_{ds}
69.5-75.5	LS	1024	B			2.9						
78.5-84.5	LS	1500	T_{300}	746,195	380,163	3.7						
84.5-89.0	LS	5351	U	421,339	385,036	1.1						
89.0-95.0	CS	737	B			2.8						
95-100.8	CS	1870	T_{300}	545,122	352,634	7.0						
100.8-110.2	CS	1431 1050	T_{300} T_{300}	122,656 164,451	131,130 93,755	9.4						
113.5-120.2	CS	1092	T_{45}	80,298	83,326	7.0						

Table 2. Clay Minerals, X-Ray Diffraction Effects Parts In 10

Depth Below Coal (Inches)	I (2)	M (3)	Mx (4)	K (5)	Ch (6)	Percent Hydration (7)	"I" (8)	"M" (9)
RK 12-18-75-1								
16	2	1-2	5-6	1-2	-	47	5	4
21	1-2	3	5	0-1	-	72	3	6-7
24	1-2	3	5	0-1	-	63	3	6
26	1-2	2	6	0-1	-	58	4	5-6
30	2-3	2	5	1	-	54	5	4-5
36	2-3	1-2	5-6	1	-	50	4-5	4
RK 12-19-75-1								
10	1	1	7-8	t	-	63	4	5-6
12	2	2	5-6	0-1	-	58	4-5	5
16	1-2	1-2	7	0-1	-	63	4	6
22	1-2	1	7	0-1	-	57	4-5	5
26	2	1-2	6	0-1	-	54	5	4-5
30	2-3	2	3	1	-	42	5	3-4
36	2	1-2	6	0-1	-	47	5	4-5
K 12-19-75-2								
7	2	2	5-6	0-1	-	63	4	5-6
10	2	2	5-6	0-1	-	58	4-5	5
12	2-3	2	5	0-1	-	58	4-5	4-5
15	1-2	1-2	6	0-1	-	50	4-5	4-5
22	2	2	5-6	1	-	54	4-5	5
28	3	0-1	5-6	1	-	54	5-6	3-4
34	3	1	5-6	0-1	-	47	6	3-4
RK 2-12-76-1								
6	2-3	5	2-3	-	-	90	3	7
8	2-3	5	2-3	t	-	90	3	7
13	2	4-5	4	-	-	72	3	7-8
17	3-4	5	2	-	-	72	4	6-7
RK 2-13-76-1								
2	1-2	3	5-6	5	-	72	3	7
16	1	3	5-6	0-1	-	80	2	6-7
20	1	1	8	-	-	72	3	7
23	2	2	5-6	0-1	-	54	4-5	5
26	2	3	4-5	0-1	-	58	4	5-6
RK 2-13-76-2								
4	1-2	2-3	6	t	-	72	3	7
13	1-2	2-3	6	0-1	-	72	3	7
18	2-3	5-6	1-2	0-1	-	72	3	6-7
21	1-2	2-3	6	0-1	-	72	3	6-7
26	2-3	3-4	3-4	0-1	-	72	3-4	6
30	2	2-3	5	0-1	-	63	4	5-6
34	2	2	5	1	-	54	3-4	4-5
ZE 3-3-76-1								
6	2-3	-	5	2-3	-	38	5-6	1-2
15	2-3	-	5	2-3	-	40	5-6	2
21	1-2	-	4-5	3-4	-	34	4-5	1-2
25	3	-	5	1	1	34	6-7	1-2
27	2-3	-	5	1-2	1	34	6	1-2
33	2-3	-	3-4	4	-	40	4-5	1-2
37	2	-	5	3	-	35	5-6	1-2
41	5	-	3-4	1-2	1-2	32	6-7	1
45	4	-	3	1-2	1	42	6-7	1
52	3	-	6	0-1	0-1	35	5	2
76	4-5	-	2-3	1-2	1-2	S	-	-
81	4	-	4	1	1-2	S	-	-
5ZE 4-9-76-1	1	-	6	3	-	34	5	2
20	1	-	6	3	-	50	4	3
24	1-2	-	5	3	-	32	5	1-2
29	2	-	4-5	2-3	1-2	38	5	1-2
38	1-2	-	4	3	1	37	4	1-2
52	1	-	5	4	-	40	4	2
57	2-3	-	5-6	1-2	0-1	35	6	2
120	4-5	-	2-3	1	1-2	S	-	-

K = Kaolinite, I = Illite, M = Montmorillonite, Mx = Mixed layer clay minerals, Ch = Chlorite, t = Trace,

Data from Table 2 shows that for the plastic limit, R^2 = .67 which indicates that approximately two-thirds of the variability in triaxial strength may be "explained" by the plastic limit.. The importance of these statistical parameters is not so much in their values, however, but that they indicate that the values of triaxial strength for the underclay samples obtained from the River King Mine are most closely related in a negative manner to the Plastic Limit and the water content.

Table 4. Correlation of Triaxial Strength with Other Physical Parameters

River King Mine

Parameter	Corr. Coef. (r)	Coef. of Det. (R^2)	No. of Obs.
Plastic limit	- 0.82	.67	16
Water content	- 0.69	.45	23
Shrinkage limit	+ 0.55	.30	16
Liquid limit	- 0.31	.10	16
Median grain size (ϕ)	- 0.26	.07	15
Specific gravity	- 0.26	.07	16
Percent clay (< 2μ)	- 0.23	.05	15
Median grain size (mm)	+ 0.22	.05	15
Plastic index	- 0.14	.02	16

The results of the stepwise regression procedure indicate that the best fitting multiple regression model has the form $y = a - b_1 x_1 - b_2 x_2$ where y represents triaxial strength, x_1 Plastic Limit and x_2 Plastic Index and b_1 and b_2 the respective regression coefficients. This equation explains a total of 78 percent of the variability in the triaxial data. If the manufactured variable PL/w is included as an independent variable a total of 83 percent of the variation is accounted for by the new model.

C. Bearing Capacity

The bearing capacity analysis was carried out for three mines that were studied in detail during the course of this investigation. In addition, two areas of apparently different subcoal conditions were selected in one of the mines. The data required for this evaluation included depth of coal below the surface and pillar size and extraction ratios obtained from mine records, underclay thickness and the sequence of subcoal strata from the test borings, and the shear strength of the bearing materials from laboratory tests of selected samples. It was anticipated that this data could be applied with conventional procedures of bearing capacity analysis to determine an estimate of the factor of safety for mine floor stability. This factor of safety could be evaluated with respect to observed conditions of the mine floor.

The methods of analysis deemed most suitable to the in situ conditions were those suggested by Vesic (1970) and by Mandel and Salencon (1969). Both of these techniques for bearing capacity computation take into account the layered nature of the bearing stratum. The results of these analyses are given in Table 5. It is apparent that although the method of Mandel and Salencon is slightly more conservative, both methods provide similar results. Comparison of the calculated factors of safety with observed mine floor stability indicated that they also were appropriate for insitu conditons. For example, for the mines investigated Mine B was experiencing the most severe floor failure problems, whereas Mine A had no indication of floor problems.

In Mine C_2 the area C_1 had more problem with floor heave than area C_2.

The intact strength of the underclay (c_1) was reduced to account for the influence of fissuring and sample selection bias. A value of 0.35 was used in the calculations as suggested by both Lo (1975) and Ganow (1975). However, the results of the plate loading tests conducted in Mine B indicated that no reduction in shear strength was required in this case. The plate load tests yielded failure strengths of 117 and 135 psi and the laboratory strength values were in the range of 40 - 125 psi. Since Mines A and B have similar geologic conditions, the results of the plate load test were arbitrarily extended to include Mine A and a strength reduction factor of 1.0 (no reduction) was used in the calculations.

The bearing capacity results for Mines A and B indicated a direct relationship between the thickness of the underclay (H) and the Pillar stability factor of safety. This association is not as obvious in Mine C however, but there is also a wide variation in strengths for the underclay of this mine that may obscure the relationship.

The data scatter and lack of correlation of Mine C seems to be related to the more complex subsurface profile at this mine. The underclay grades trans tionally into a firm limestone layer at a relatively shallow depth. Beneath the firm layer there is another soft underclay-like material with very low strength values. This three-layer system results in a failure mode that is a combination of the simplified types discussed previously. It is hypothesised that the soft material initially is squeezed from beneath the pillar until enough shear strength is developed to distribute the load to the more rigid layer. At this point the pillars seemingly remain stable until either: 1) the tensile strength of its hard layer is exceeded; 2) the shear strength of the bearing material is reduced by wetting, or 3) the pillar stresses are increased by further extraction.

The underclays of Mine B are significantly less strong that those of Mine C. This fact is the most important reason for the comparatively low factors of safety. Where the thickest deposit of this material exists, the computed factors of safety against bearing capacity failure are the smallest. Also, the failure mode is consistently one of general shear failure with large quantities of floor and subfloor material being displaced from beneath the pillar, outward and upward into the haulageway.

Table 5. Bearing Capacity

Mine	Pillar size (ft)	Pillar stress (psi)	Underclay Thickness (ft)	q_{ult}	Avg F.S.
A	55x55	375-400	4	562-938	1.96
B	62x62	350-375	9	272-340	0.85
C_1	40x40	250-300	3	328-394	1.34
C_2	40x40	250-300	5	368-858	2.33

V. SUMMARY AND CONCLUSIONS

Since this research is only approximately 30% completed at the time of this writing only incomplete conclusions can be reached. Among these conclusions are: 1) the plasticity of the underclay seems to be the most closely correlated parameters associated with strength; 2) the concept of layered bearing capacity analysis seem to have usefulness for the analysis of the bearing capacity of coal mine pillars.

The results of this research will be made available to the profession as they are completed.

IV. TEST RESULTS

A. Clay Mineralogy

Correlation of clay mineralogy to underclay strength however did not reveal any statistically significant relationships, although some important concepts can be noted. The values of the correlation coefficient "r" and the coefficients of determination R^2 for the River King and Ziegler Mines samples are presented in Table 3, although the River King data are relatively inconclusive in that they are drawn from any seven observations. The relative ranking of the correlation coefficients with respect to mineralogy is of interest however, indicating that, for the River King Mine illite has the highest positive relationship with strength, montmorillonite a negative relationship and that the percentage of the mixed layer minerals and kaolinite apparently have no relationship with strength values. It is also interesting to note that the adjusted values of the clay mineral percentages for "montmorillonite" and "illite" have a weaker relationship than the nonadjusted values, suggesting that the procedure used to subdivide the mixed clay minerals was of no benefit.

Table 3. Correlation of Triaxial Strength with Clay Mineralogy of Samples

	River King Mine		
Clay Mineral	Corr. Coef. (r)	Coef. of Det.	No. of Obs.
Illite	+ 0.84	0.71	7
Montmorillonite	- 0.79	0.62	7
"Illite"	+ 0.66	0.44	7
"Montmorillonite"	- 0.65	0.42	7
Mixed Layer	- 0.10	0.01	7
Kaolinite	< .01	0.42	7

B. Triaxial Strength

The triaxial strength of the underclay varied considerably, ranging from a low value of 31 psi to a high value of 2267 psi. Considering the difficulties of collection and preparation of weak or poorly indurated samples, it can be expected that the lower values are probably more representative of most underclays than would be indicated by their frequency of occurrence in the test data. It can also be expected that the higher values, particularly those over 800 psi, have been obtained from samples with the properties of a claystone and from those which contained limestone nodules in the matrix.

It is apparent that the strength of underclay is notably less than that of the other subcoal strata, but that strength values over 2000 psi for any of the material is a very infrequent occurrence. Claystone values range from 59 psi to 2677 psi but generally may be expected to be about 1000 psi. The strength values for the limestone cannot be defined in any quantitative way since only 5 samples were tested, and the strength value will be dependent upon the nature of the limestone nodules in the clay matrix.

The development of these strength values is

dependent upon many interrelated physical properties of the material (Judd and Huber, 1962; D'Andrea, Fisher and Fogelson, 1965). Some of these properties may have a direct relationship with strength, others an indirect or secondary relationship, but if these relationships are known, they may be used to estimate the strength of the rock with a specified level of certainty. Since the tests for these parameters are often simpler to run and interpret than strength tests on underclays, the evaluation of these properties can provide readily available information useful for the investigation of potential floor failure problems. In addition, an evaluation of the correlation between strength and the other individual properties identifies those factors that are most closely related to strength, and provides information about how changes in these properties, either geological or mining induced, will effect underclay strength values.

These potential effects were analyzed by using a multiple linear regression model to define the form of and degree of relationship between triaxial strength and the other physical properties tested. This procedure determined which variables of a collection of independent variables should be most likely included in a regression model. In this case, triaxial strength was considered as the dependent variable y, and the other physical properties as the independent variables x . The technique is useful for data screening or preliminary evaluations of the relative strength of the relationships between proposed independent variables and a dependent variable.

The regression used was a stepwise procedure which is a component of the Statistical Analysis System (SAS) of the computer network of the University of Missouri. The regression is a generalized least squares program used for simple or multiple regression, analysis of variance and correlation. The input data for this program are a set of observations of the several independent variables and one dependent variable. The program output lists the mean, standard deviation and coefficient of variation for all variables and a matrix of correlation coefficients for all possible pairs of variables. It then lists the analysis of variance table, the regression coefficients and the statistics of fit for a series of prediction equations, each one containing one more independent variable than the last. The independent variables are selected in order of their highest degree of correlation with the dependent variable and the procedure continues by adding new independent variables to the variables of the preceding step until some final prediction equation is obtained.

The correlation coefficients and coefficients of determination for triaxial strength vs. the other physical parameters from the River King Mine samples are presented in Table 4 in order of their highest values. The correlation coefficient (r) is an estimate of the relationship between the random variables. It does not necessarily imply a causal relationship, but does indicate degree of linear association between the variables of interest, in that the closer to + 1.0, the higher the assocation. The coefficient of determination (R^2) indicates the proportion of total variation about the mean value of the dependent variable that is explained by the regression equation. It is often stated as a percentage 100 R^2 and, the larger it is the better the fitted equation explains the variation in the data.

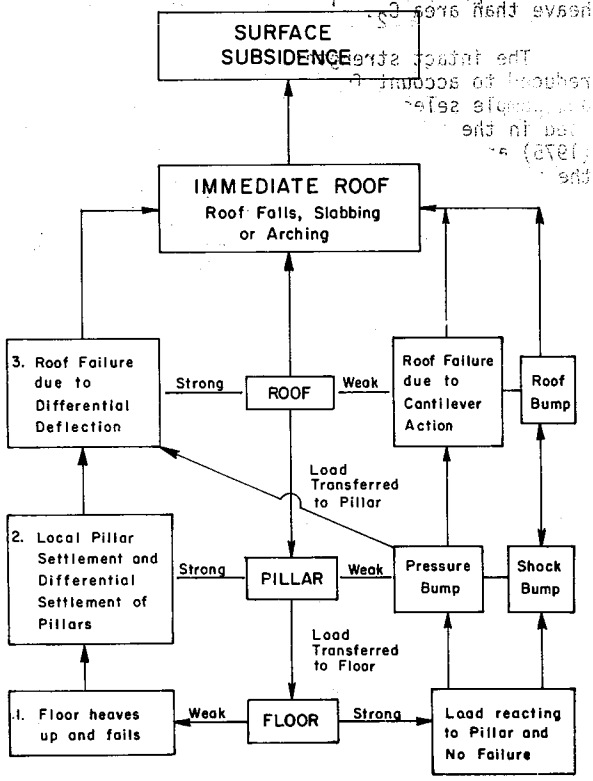


Figure 1. Interaction of Mine Roof, Pillar and Floor

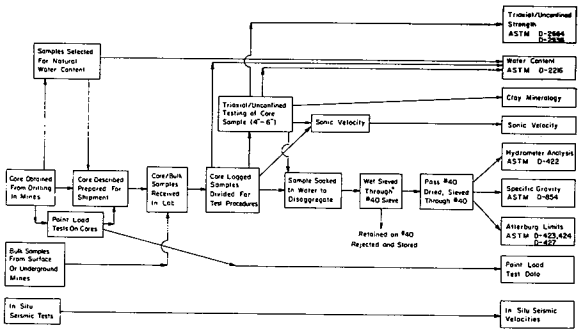


Figure 2. Laboratory Testing Flow Chart

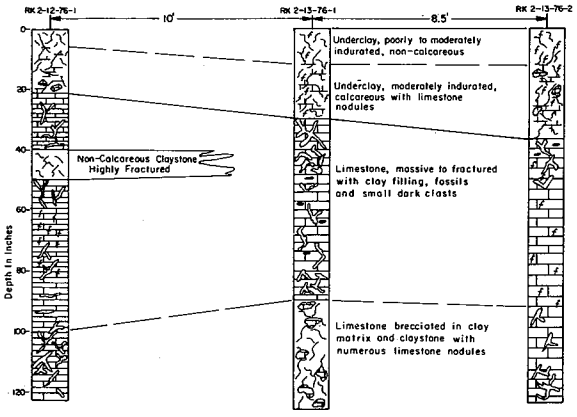


Figure 3. Subcoal Profile-River King Mine

SELECTED BIBLIOGRAPHY

1. Huddle, J.W., and Patterson, J.H., 1961, "Origin of Pennsylvanian Underclay and Related Seat Rocks", Geol. Soc. Am. Bull., V. 72, No. 11.

2. Schultz, L.G., 1958, "Petrology of Underclays", Geol. Soc. Am. Bull., V. 69, No. 4.

3. Obrien, H.R., 1964, "Origin of Pennsylvanian Underclays in the Illinois Basin", Geol. Soc. Am. Bull., V. 75, No. 9.

4. Wanless, H.R., 1970, "Late Paleozoic Deltas in the Central and Eastern United States", Deltaic Sedimentations--Modern and Current: Soc. Econ. Pol. and Mineral. Spec. Publ. No.15.

5. Waller, J.M., 1957, "Paleoecology of the Pennsylvanian Period in Illinois and Adjacent States", Treatise on Marine Ecology and Polcoecolgy, V. 2, Paleoecology: Geol. Soc. Am. Memoir, No. 67.

6. Wilson, M.J., 1965, "The Origin and Geological Significance of the South Wales Underclays", Journ. Sed. Pet. V. 35, No. 1.

7. Grim, R.E., and Allen, V.T., 1938, "Petrology of the Pennsylvanian Underclays of Illinois", Geol. Soc. Am. Bull., V. 49, No. 10.

8. Hostermery, J.W., 1972, "Underclay Deposits of Summerset and Eastern Fayette Counties, Pennsylvania", J. S. Geol. Surv. Bull. 1363.

9. White, W.A., 1954, "Properties of Clay as Related to Coal Mining Problems," Ill. Mining Inst. Proc.

10. Parnham, W.E., 1964, "Lateral Clay Mineral Variation in Certain Pennsylvanian Underclays", Proc. 12th Natl. Conf. on Clays and Clay Minerals.

11. O'Brien, N.R., 1963,"Clay Mineral Variation in a Pennsylvanian Underclay", Ill. State Acad. Sci. Trans. V. 56.

12. O'Brien, N.R., 1970, "The Fabric of Shale--An Election-Microscopic Study", Sedimentology, V. 15, No. 3-4.

13. Rutten, M.G., 1963, "Biological Homogenization in Fossil Underclays and Seat Rocks", Geol. Soc. Am. Bull. V. 74, No. 1.

14. Byers, C.W., 1974, "Shale Fissibility: Relation to Bioturbation", Sedimentology, V. 21, No. 3.

15. Ganow, H.C., 1975, "A Geotechnical Study of the Squeeze Problem Associated with the Underground Mining of Coal", Unpub. Ph.D. Dissertation, Dept. Geol., Univ. Illinois at Urbana-Champaign.

16. Freer, J., 1892, "Mine Creeps", Illinois Mining Institute Journal, Vol. 1, No. 3.

17. Jones, E., 1897, "The Control fo Creeps", Colliery Engr., and Metal Mines, Vol. 18, No. 3.

18. Nelson, A., 1947, "The Constitution and Behavior of Underclays," Colliery Engr., Vol. 24, No. 281.

19. Holland, C.T., 1942, "Physical Properties of Coal on Associated Rock as Related to the Course of Bumps in Coal Mines", Trans. AIME, Vol. 149.

20. Jenkins, J.C., "The Bearing Capacity of Mine Floors", Colliery Guardian, The Institute of Mining Engineers, Vol. 195, No. 5039.

21. Jenkins, J.C., "Some Investigations into the Bearing Capacities of Floors in the Northumberland and Durham Coal Field", Trans. of the Institution of Mining Engineers, Vol. 117, Part II, Aug. 1958.

22. Jenkins, J.D., "A Laboratory and Underground Study of the Bearing Capacity of Mine Floors", 3rd Int. Conf. of Strat. Control, May, 1960, Paris.

APPENDIX

PAST PROCEEDINGS
OF THE
OHIO RIVER VALLEY SOILS SEMINARS

ORVSS I
BUILDING FOUNDATION DESIGN AND CONSTRUCTION

October 16, 1970
Lexington, Kentucky

AND WHY DO WE DO IT THIS WAY? by R. C. Deen, Assistant Director of Research, Kentucky Bureau of Highways.

A CASE STUDY - PIPE PILING FOR A DEEP FOUNDATION PROBLEM by Don Fuller, P.E., L.S., Fuller, Mossbarger and Scott Civil Engineers, Inc.

DESIGN, CONSTRUCTION AND PERFORMANCE OF SPREAD FOOTINGS ON A NON-UNIFORM FOUNDATION by Robert Lennertz, M. ASCE, The H. C. Nutting Company.

DESIGN, CONSTRUCTION AND TESTING OF DRILLED PIERS ON SHALE AND THINLY BEDDED LIMESTONE by George J. Thelen, P.E.

BUILDING FOUNDATION FAILURE RELATED TO SOIL SHRINKAGE by Aubrey D. May, Chief Engineer, Stokley and Associates.

INNOVATIONS IN FOUNDATION DESIGN AND CONSTRUCTION by D. J. Hagerty, Assistant Professor, Civil and Environmental Engineering Department, University of Louisville.

ORVSS II
EARTHWORK ENGINEERING, START TO FINISH

October 15, 1971
Louisville, Kentucky

PRELIMINARY SITE SELECTION STUDIES WITH REMOTE SENSING TECHNIQUES by David J. Barr, Department of Civil Engineering, University of Cincinnati.

SITE EVALUATION WITH REMOTE SENSING TECHNIQUES APPLIED TO SUSPECT KARST TOPOGRAPHY by Melville D. Hensey, Engineering Division, The Proctor and Gamble Company.

EXPLORATION METHODS by Dr. Vincent P. Drnevich, P.E., Assistant Professor of Civil Engineering, University of Kentucky.

THE SELECTION OF STRENGTH PARAMETERS AND THEIR USE IN THE STABILITY ANALYSIS OF EARTH ROCK EMBANKMENTS by Woody McGraw and Don Tupman, Louisville District, U.S. Army Corps of Engineers.

CONTRACTOR'S PROBLEMS by Richard Geottle.

ENVIRONMENTAL CONSIDERATIONS IN SOILS ENGINEERING by D. Joseph Hagerty, Asst. Professor of Civil Engineering, University of Louisville and Joseph L. Pavoni, Asst. Professor of Civil and Environmental Engineering, University of Louisville.

ORVSS III
LATERAL EARTH PRESSURES

October 27, 1972
Fort Mitchell, Kentucky

INSTRUMENTATION FOR STRESS AND STRAIN MEASUREMENTS IN SOILS by Stanley D. Wilson, Executive Vice-President, Shannon and Wilson, Inc.

DEEP EXCAVATION AND BUILDINGS SUPPORTED BY TIE-BACK SYSTEM by Richard Geottle, P.E., Richard Geottle, Inc., James J. Flaig, P.E., H. C. Nutting Company, Arthur J. Miller, P.E., Miller, Tallarico, McNinch, and Hoeffel Consulting Engineers, and Steven E. Schaefer, EIT, Miller, Tallarico, McNinch, and Hoeffel Consulting Engineers.

LATERAL PRESSURES AND MOVEMENTS CAUSED BY PILE DRIVING by D. Joseph Hagerty, A.M. ASCE, Assistant Professor of Civil Engineering, University of Louisville.

FACTORS TO BE CONSIDERED IN THE DESIGN OF BACKFILL FOR LARGE DIAMETER FLEXIBLE METAL CONDUITS by David C. Cowherd, M.S., P.E.

GENERALIZED SLIDING WEDGE METHOD FOR SLOPE STABILITY AND EARTH PRESSURES ANALYSIS by Dr. Vincent P. Drnevich, Assistant Professor of Civil Engineering, University of Kentucky.

LATERAL PRESSURES AND PRESTRESSED TIE-BACK WALLS by P. C. Rizzo and R. D. Ellison, E. D'Appolonia Consulting Engineers, Inc., and R. J. Shafer, Herbert F. Darling, Inc.

DESIGN AND CONSTRUCTION OF SUPPORTED TEMPORARY EXCAVATIONS IN URBAN ENVIRONMENT by Yves Lacroix and Walter T. Jackson, Woodward-Moorhouse and Associates, Inc.

ORVSS IV
GEOTECHNICS IN TRANSPORTATION ENGINEERING

October 5, 1973
Lexington, Kentucky

RETRIEVAL AND USE OF GEOTECHNICAL INFORMATION by D. Joseph Hagerty, Assistant Professor of Civil Engineering, University of Louisville, Nicholas G. Schmitt, Soils Engineer, Law Engineering Testing, William J. Pfalzer, Research Engineer Assistant, Kentucky Bureau of Highways.

IN SITU SHEAR STRENGTH PARAMETERS BY DUTCH CONE PENETRATION TESTS by C. T. Gorman, Research Engineer Assistant, Kentucky Bureau of Highways, T. C. Hopkins, Research Engineer Principal, Kentucky Bureau of Highways, and V. P. Drnevich, Associate Professor of Civil Engineering, University of Kentucky.

REINFORCED EARTH - DESIGN AND CONSTRUCTION by David S. Gedney, Regional Engineer, Region 15, Federal Highway Administration and John L. Walkinshaw, Highway Engineer, Federal Highway Administration.

APPLICATIONS OF THE FINITE ELEMENT METHOD TO GEOTECHNICS AND TRANSPORTATION ENGINEERING by Yang H. Huang, Associate Professor of Civil Engineering, University of Kentucky.

RELIABILITY ANALYSIS AND COST OPTIMIZATION OF EMBANKMENTS by Warren J. Baker, Chrysler Professor and Dean, College of Engineering, University of Detroit.

STABILITY ANALYSIS - ITS POTENTIAL AND ITS LIMITATIONS by T. H. Wu, Professor of Civil Engineering, The Ohio State University.

ORVSS V
ROCK ENGINEERING

October 18, 1974
Clarksville, Indiana

EXPLORATION AND TESTING IN ROCK by Joseph Hagerty, Civil Engineering Department, University of Louisville, and James Coulson, Tennessee Valley Authority.

A ROCK CLASSIFICATION SCHEMA by R. C. Deen, Assistant Director, Kentucky Bureau of Highways, Division of Research, C. D. Tockstein, Research Engineer Assistant, Division of Research, and M. W. Palmer, Research Engineer Assistant, Division of Research.

FOUNDATIONS ON OR IN ROCK by E. D'Appolonia, E. D'Appolonia Consulting Engineers, Inc.

ROCK FOUNDATIONS (A Panel Discussion) Comments by: C. R. Lennertz, H. C. Nutting Company, Harry Thomas, US Army Corps of Engineers, and Milton M. Greenbaum, Greenbaum and Associates.

DESIGN OF OPEN EXCAVATIONS IN ROCK by D. Ross-Brown Dames & Moore.

ROCK EXCAVATION (A Panel Discussion) The Producer's Viewpoint by J. A. Waddell, Martin Marietta Aggregates, Consideration of Effects on Surroundings by D. J. Hagerty, University of Louisville, and Geologic and Support Considerations for Rock Excavations by J. Mahar, University of Illinois.

ROCK ENGINEERING ON SOME RECENT PROJECTS by Don U. Deere, Consultant in Engineering Geology and Rock Mechanics, University of Florida.

ORVSS VI
SLOPE STABILITY & LANDSLIDES

October 17, 1975
Fort Mitchell, Kentucky

GEOLOGIC PERSPECTIVES--THE CINCINNATI EXAMPLE by Robert W. Fleming, U.S. Geological Survey.

IMPORTANCE OF GEOLOGIC STRUCTURE IN STABILITY OF ROCK SLOPES by Robert L. Schuster, William K. Smith, and Fitzhugh T. Lee, U.S. Geological Survey.

CUT AND FILL ORDINANCE AS ADOPTED BY THE CITY OF CINCINNATI by James. R. Krusling, P.E.

DRILLED PIER RETAINING WALLS by Merle F. Nethero, Geotechnical Engineer - Vice President, H. C. Nutting Company.

REGRADING FAILED SLOPES by H. A. Mathis, Kentucky Department of Transportation.

EFFECTS OF WATER ON SLOPE STABILITY by Tom C. Hopkins, Research Engineer Chief, David L. Allen, Research Engineer Principal, and Robert C. Deen, Assistant Director, Kentucky Department of Transportation, Division of Research.

Availability and cost of past proceedings may be obtained by contacting
OHIO RIVER VALLEY SOILS SEMINAR
Engineering Continuing Education
University of Kentucky
Lexington, Kentucky 40506
Phone: (606) 258-5949