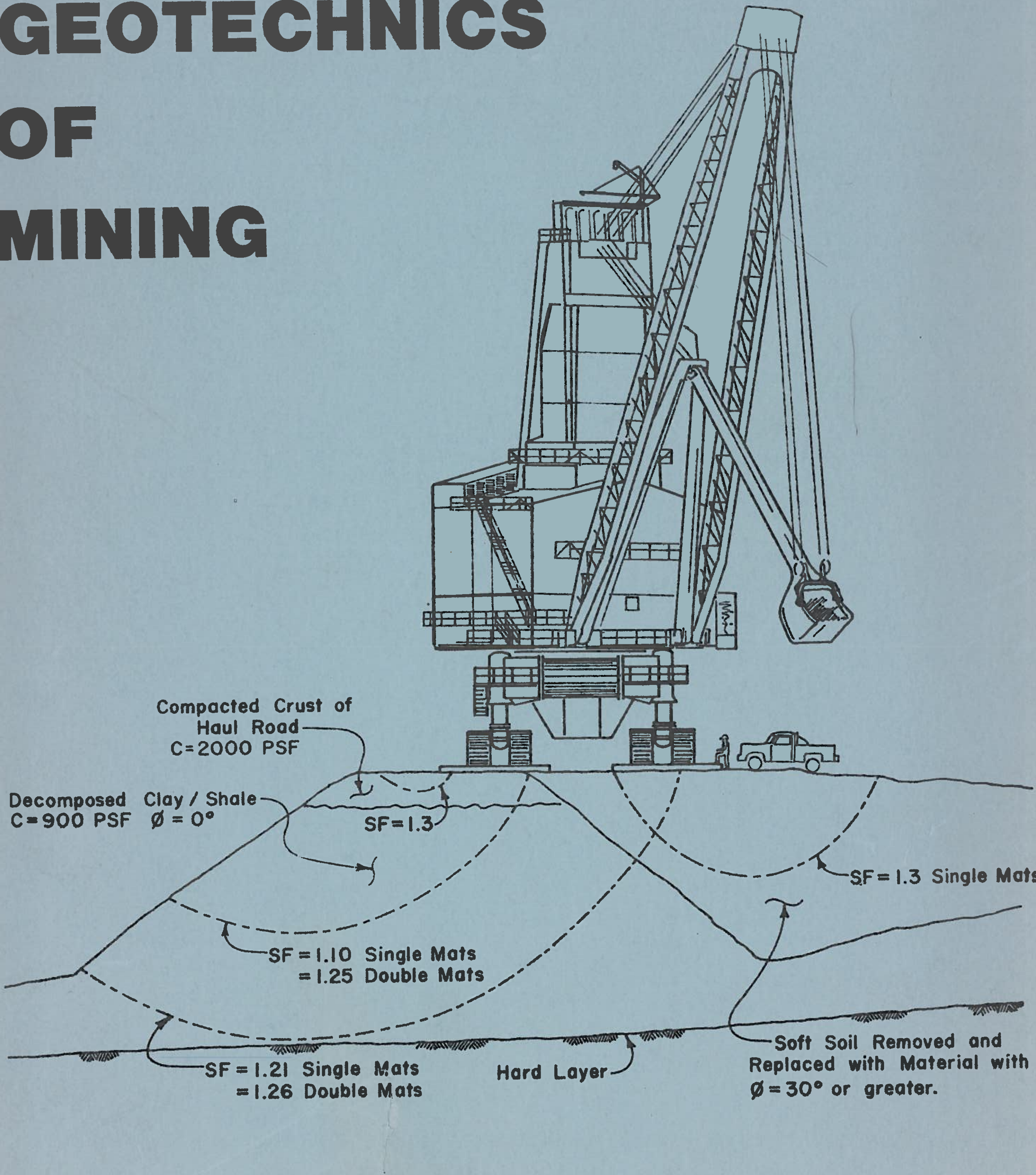
C. R. Ullrich

OHIO RIVER VALLEY SOILS SEMINAR X

Proceedings
October 5, 1979
Lexington, Kentucky

GEOTECHNICS OF MINING



PROGRAM and SCHEDULE
Friday, October 5, 1979

Kentucky Horse Center

8:30 am *Registration*

9:00 *Welcome*
Craig Avery, Fuller, Mossbarger, Scott, and May, (Chairman of Kentucky Geotechnical Engineering Group)

9:15 *A Critical Evaluation of Coal Mining Subsidence Patterns*
T. D. O'Rourke, Assistant Professor, and Susan M. Turner, Civil & Environmental Engineering, Cornell University

Evaluation of Damage Potential to Earth Dam by Subsurface Coal Mining at Rend Lake, Illinois
A. S. Nieto, Assistant Professor, In Charge, Engineering Geology Program, University of Illinois

Geotechnical Considerations for Deadheading a Marion 5761 Shovel
Kenneth E. Darnell, P.E., Geologic Associates, Inc., J. Richard Cheeks, P.E. Stokley-Cheeks & Associates, Dennis Albers, Amax Coal Company

10:25 *Coffee Break*

10:40 *Disposal of Coal Processing Waste at Sites of Limited Size*
David C. Cowherd, Chief Geotechnical Engineer, Bowser-Morner, Dayton, Ohio, Barry K. Thacker, Project Geotechnical Engineer, Bowser-Morner, Dayton, Ohio

Seepage and Stability Analysis for an Inundated Mill Tailing Impoundment, A Case Study
W. A. Charlie, Assistant Professor, Civil Engineering Department, Colorado State University, T. V. Edgar, Graduate Research Assistant, Colorado State University, B. Kilburn, Tailings Engineer, Climax Molybdenum, Climax, Colorado

Chemical Additives to Change the Durability of Shales
C. W. Lovell, Professor, School of Civil Engineering, Purdue University, M. Surendra, School of Civil Engineering, Purdue University

12:00 *Lunch*

Afternoon Session
Presiding: Dennis C. Brodbeck, Richard Goettle, Inc., (Cincinnati Geotechnical Group)

1:30 Panel Discussion *"Impacts of the Surface Mine Reclamation Act on Geotechnical Engineering"*
James E. Funk, Associate V.P. for Energy Research & Director, Institute for Mining and Minerals Research, University of Kentucky (Panel Moderator)

Clifford Vaughan, Civil Engineer, Region II Technical Services Branch, Office of Surface Mining, Department of the Interior, Knoxville, Tennessee

Gene Brandenburg, Commissioner, Bureau of Surface Mining Reclamation and Enforcement, Department for Natural Resources and Environmental Protection

James R. Jones, Director, Environmental Quality, Peabody Coal Company

Jerry Lombardo, Director, Environmental Affairs, Island Creek Coal Company

Richard Ellison, Vice-President, E. D'Appolonia, Consulting Engineers

3:00 *Break*

3:30 *Questions to Panelists, Discussions from Floor*

4:30 *Afternoon session ends*

Hilton Inn

6:00 pm *Social Hour*

7:00 *Dinner*

8:00 *Evening Session – Reclamation: The New Imperative*
Harry M. Caudill, Professor, Department of History, University of Kentucky (Noted author and Appalachian historian)

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Sponsored by
Kentucky Geotechnical Engineering Group, ASCE

University of Kentucky
Department of Civil Engineering
Office of Continuing Education and Extension
Institute for Mining and Minerals Research

Cincinnati Geotechnical Group, ASCE

University of Cincinnati
Department of Civil and Environmental Engineering

University of Louisville
Department of Civil Engineering

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Brainard-Kilman Drill Company

Celanese-Mirafi, Geotextiles
APA, Inc.
Ben-Tom Supply Company

Central Mine Equipment Company

E. I. DuPont de Nemours & Company, Inc., TYPAR Fabric

Troxler Electronics Laboratory, Nuclear Moisture-Density Meters

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O R V S S - X

We gratefully acknowledge the efforts of

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A CRITICAL EVALUATION OF COAL MINING SUBSIDENCE PATTERNS

by

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Abstract. This paper examines subsidence patterns caused by longwall mining in both the United States and the United Kingdom. Special attention is directed to the subsidence prediction charts that have been developed for mining conditions in the United Kingdom. The regional geologies in several coal producing areas are surveyed and compared. Case histories of mining subsidence are summarized with special emphasis on lateral ground strains and surface curvatures. Consideration is extended to subsidence associated with retreat pillar mining.

Introduction

The need to develop coal resources as an alternative to oil has generated interest in the United States regarding the environmental consequences of mining, particularly the subsidence caused by coal mining and its effect on surface structures. Subsidence engineering requires that the patterns of ground movement for a given mining method be reasonably predicted. Prediction capabilities can be developed only in accordance with obtaining reliable case histories of movement. Case histories establish a baseline from which empirical formulae can be derived and analytical techniques tested. In the United Kingdom the use of longwall systems is linked closely with empirical charts that were developed by the National Coal Board (NCB) from a comprehensive review of many case histories.[1] These charts relate the magnitude and pattern of surface subsidence with the thickness, depth, and width of the mined coal seam. The charts are used extensively for planning land use, developing structural designs for new buildings that minimize subsidence effects, and choosing remedial measures to protect existing structures.

Two questions are germane to the current problems of developing prediction capabilities in the United States. How do subsidence patterns in the United States compare with those typically forecast for the United Kingdom? How do subsidence patterns differ among various areas within the United States both on the basis of regional geology and method of mining? It is the intention of this paper to approach these questions by surveying the geologies associated with various coal producing areas and by examining records of movements to illustrate trends in the patterns of subsidence for the areas under consideration.

Longwall mining entails the complete removal of a tabular section of coal, referred to as a panel. Excavation is performed by mechanical breaking tools that are drawn continuously across the working face, leaving no pillars, and allowing the roof to cave behind the face. Retreat pillar mining refers to the systematic removal of coal pillars that are contained within a network of intersecting passageways. Each pillar generally is split by excavating through its center and subsequently removing as much of the pillar as local conditions of stability will allow. This paper deals with longwall mining and, to a limited extent, with retreat pillar mining.

Location of Longwall Mines

Longwall mining accounts for approximately 4 percent of the United States annual coal production. Figure 1 shows the distribution of longwall mines in the United States, which was developed from a recent U.S. Bureau of Mines survey.[2] Longwall mining is performed in at least nine states and four physiographic provinces. For the purposes of this paper three areas of longwall mining are identified as 1) the Allegheny Plateau in the eastern portion of the country, 2) the Illinois Basin, which covers a large segment of Illinois and smaller areas of western Indiana and Kentucky, and 3) the Western Reserves that occupy portions of Arizona, Colorado, New Mexico and Utah. An insert showing longwall locations in the Allegheny Plateau is provided in the figure.

In the United Kingdom longwall operations contribute approximately 94 percent of the annual coal production. Because of the high density of longwall mines, frequent observations of longwall subsidence have been made. The National Coal Board (NCB) has drawn upon 165 case histories of subsidence to develop charts for predicting the magnitude and distribution of both settlement and lateral strain likely to develop over longwall panels in the United Kingdom.[1] Figure 2 shows the approximate locations of the mines, or collieries, from which the case histories were developed. The great majority of the case histories are derived from sites within the Midlands and Yorkshire coalfields. This particular area has been isolated on the insert map of Great Britain and expanded to occupy the major portion of the figure.

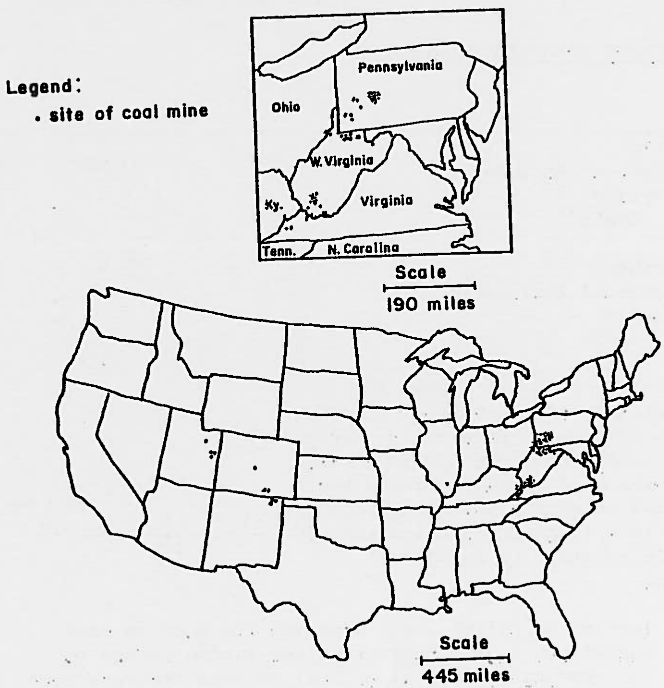

Figure 1. Location of Longwall Mines in the United States.

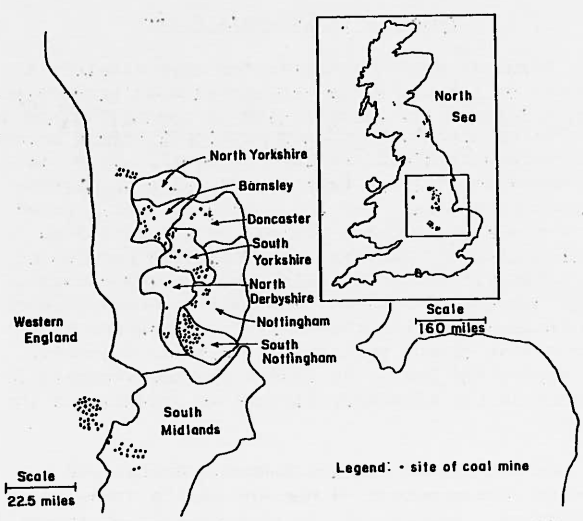

Figure 2. Location of Coal Mines Associated with Subsidence Case Histories in the United Kingdom.

Geologies of Coal Mining Areas

In this section a description of the geology in each of several coal producing areas is offered. It must be emphasized that significant local deviations are likely to occur. Furthermore, discussion is restricted to coal bearing strata of Pennsylvanian age. This precludes consideration of the Western Reserves because coal in this area is mined from strata of late Cretaceous and early Tertiary age.

United Kingdom

In the United Kingdom the most important coal bearing rocks are referred to as the Coal Measures and correspond in age to the Pennsylvanian System in the United States. Approximately three-quarters of the Coal Measures are composed of argillaceous rocks including shales, mudstones, siltstones and underclays.[3] Underclays, which are called seatearths in the United Kingdom, are found almost invariably beneath coal seams and vary in thickness from 2 to 6 ft. The distance between coal seams, as a rough average, is 30 to 50 ft, although individual shale beds may be as thick as 100 ft. Limestones are relatively scarce. Carbonates occur predominantly as sideretic claybands and blackband ironstones.[4] Sandstones are the most inconsistent members of the Coal Measures, generally being lenticular in shape and variable in thickness. In some areas thick sandstones form the immediate roof of workable seams. For example, the Tupton Sandstone has a thickness of 120 ft in the Rufford area of Nottinghamshire.[5] In the South Wales Coalfield the lower portion of the Coal Measures is mainly shale in contrast to the upper portion where thick feldspathic and micaceous sandstones, known as the Pennant Measures, are found.[6]

Illinois Basin

More than 99 percent of the mapped coal reserves in Illinois belong to the middle portion of the Pennsylvanian System which makes up the bedrock in about two-thirds of the area of the state.[7] Shales and underclays commonly form 65 to 70 percent of the upper portions of the system. Uninterrupted sequences of shale exceed 100 ft, but thicknesses of 20 to 40 ft are more common. In general, 5 to 10 percent and 1 to 2 percent of the upper portions of the system are limestones and coal, respectively. Underclays commonly are 2 to 5 ft thick. Most of the Pennsylvanian sequence shows great lateral persistence of all individual units except the sandstones. Sandstones frequently occur as channel deposits, and borehole records show that only two or three of these deposits are likely to be found in a given area.[8]

Allegheny Plateau

As the Pittsburgh coalbed is the largest and most extensively worked seam in the region, it serves as a good focal point for discussing the relatively complex geology of the Allegheny Plateau. Only a brief discussion is given here with the recognition that detailed, comprehensive treatments are offered elsewhere.[9,10]

The Pittsburgh coalbed is the basal member of the Monongahela Group, which is a series of coal bearing rocks occupying a sedimentary basin whose axis runs southwest from Pittsburgh, Pennsylvania to Huntington, West Virginia. The rocks of the basin have been folded into a series of anticlines and synclines that generally strike in a direction parallel to the basin axis. The Pittsburgh coalbed tends to reflect the regional structure, thickening near synclinal troughs and thinning near anticlinal axes.

It is difficult to generalize the Monongahela Group as there are wide variations in lithology, bed thickness, structure and surface relief. For the most part, the rocks are laterally discontinuous. In the lower portion of the group the strata are predominantly calcareous, whereas in the upper portion a high percentage of sandstone is found. It is not uncommon for 50 to 60 percent of the overburden at a given mine to be composed of limestones and sandstones. The rocks have been heavily dissected, and the maximum surface relief above individual mining areas frequently varies form 300 to 400 ft.

Discussion

The coal bearing strata of the Allegheny Plateau are subject to relatively complex variations in structure, stratigraphy and surface relief. For this reason it is difficult to make straightforward comparisons of the geology in this area with that of either the Illinois Basin or the Coal Measures of England. There are, however, some notable similarities between the geology of the English coalfields and those of the Illinois Basin. In both areas the coal bearing strata are composed predominantly of argillaceous rocks, usually shales and mudstones. Furthermore, faulting and folding in both the Midlands and southeastern Illinois are evident at roughly similar levels of intensity. In contrast to the United Kingdom, there is a greater concentration of marine-derived rocks in the coal bearing strata of Illinois. Limestones, in particular, are found in greater concentration.

Some of the most significant differences in the subsidence patterns among the various environments are likely to stem from the surficial geologies. Even within the United Kingdom there may be significant deviations from the lateral strains predicted by the NCB where coal workings are developed beneath near-surface limestones and sandstones of Permo-Triassic age. These rocks contain nearly vertical, open joints that have led to locally large concentrations of lateral movement.[11,12]

Subsidence patterns in the United Kingdom are affected almost invariably by previous mining activity. Much of the case histories used to predict subsidence patterns in the United Kingdom were developed in areas of past coal extraction, often at multiple levels above and below the panels under observation. The presence of caved longwalls, room-and-pillar workings, and filled open cast mines are likely to have a notable effect on the patterns of surface movement relative to those over mines in virgin strata. This particular aspect of British mining practice deserves special attention when comparing subsidence patterns in the United Kingdom with those in the United States where almost all coal mining currently is performed in virgin ground.

Case History Review

Four case histories of longwall subsidence are discussed in this section. The case histories have been chosen to illustrate several important characteristics of subsidence patterns as influenced by geology, previous workings and surface development. A summary figure for each case history is presented which contains a plan view of the panel, transverse profiles of settlement and strain, and information pertaining to mine geometry, percentage lithology, and the presence of previous workings. Each set of measurements shown was taken at a time when the panel face had been driven a distance in excess of 0.7 times the panel depth beyond the observation line.

Royston

Figure 3 summarizes information and observations pertaining to the Royston Drift Mine in West

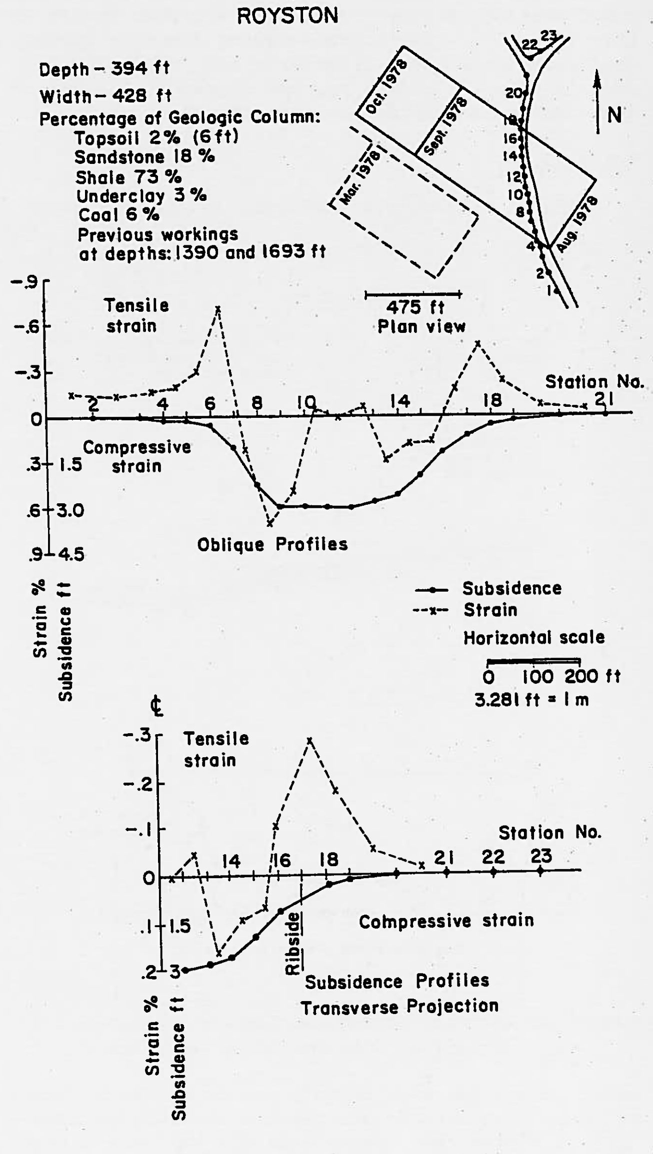

Figure 3. Summary of Information and Measured Movements for the Royston Colliery.

Yorkshire. The longwall panel was driven in a northwest direction beneath a road where markers had been established to measure vertical and horizontal movement. The site previously had been undermined by panels at depths of 1350 and 1693 ft.

Settlements and horizontal strains are shown along the road which intersects the axis of the panel at an oblique angle. In addition, the settlements and strains are shown in a transverse projection across the width of the panel to allow direct comparison with NCB predicted values. The observed settlement and strains are in excellent agreement with NCB predictions.

There is a pronounced asymmetry in strain magnitude across the road with both the maximum tensile and compressive strains occurring near the south corner of the panel. This orientation of maximum strain suggests that the panel corner has contributed to local support. There is some evidence in the literature that settlement contours exhibit maximum convergence along lines that roughly bisect the panel corners; however, there are no clear trends in this regard.[13] This characteristic deserves further consideration because transverse and longitudinal profiles, in all cases, may not represent the maximum distortion imposed by the advancing panel.

Rothwell

Figure 4 summarizes information and subsidence

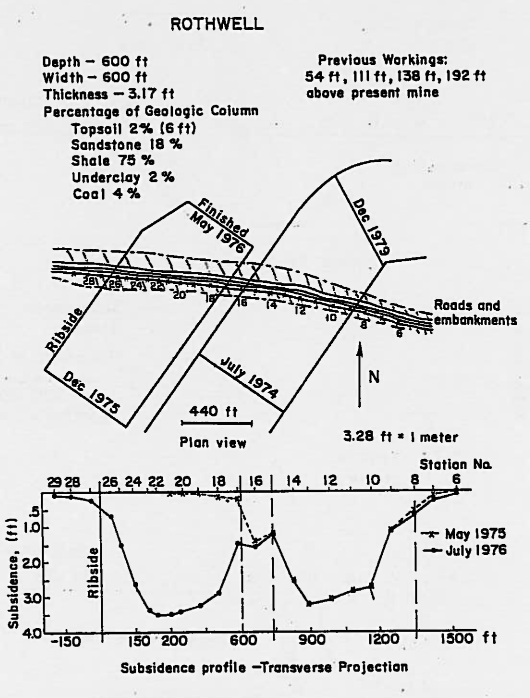

Figure 4. Summary of Information and Measured Movements for the Rothwell Colliery.

observations for two longwall panels of the Rothwell Colliery that were driven beneath the A639 Motorway in West Yorkshire. Longwall panels had been driven on four separate occasions at this site through coal seams located above the panels shown. Observation points were established along the crest of the motorway embankment, which is approximately 25 ft high. The embankment is founded on restored opencast coal and sand and gravel workings.

The settlements along the embankment in response to driving each panel are plotted on a transverse projection across the panel widths. The shape of the subsidence profile is reaonably well predicted by the NCB charts; however, the maximum settlement is approximately 40 percent larger than the amount forecast. Relatively large settlements, particularly at the crests of embankments, have been described for colliery spoil heaps by Forrester and Whittaker.[14] Their observations indicate that embankment deformations are primarily down slope and that a net horizontal spreading, especially near the embankment margins, may occur as the panel is driven by. At the Rothwell site the effect of mining on a reinforced concrete underpass has been reported by Jones and Spencer.[15] Their own observations confirm a net horizontal extension through the embankment as the joints between adjacent segments of the underpass box structure opened from 1 to 2.25 in. without subsequently closing.

Blaenavon

Figure 5 summarizes information and subsidence

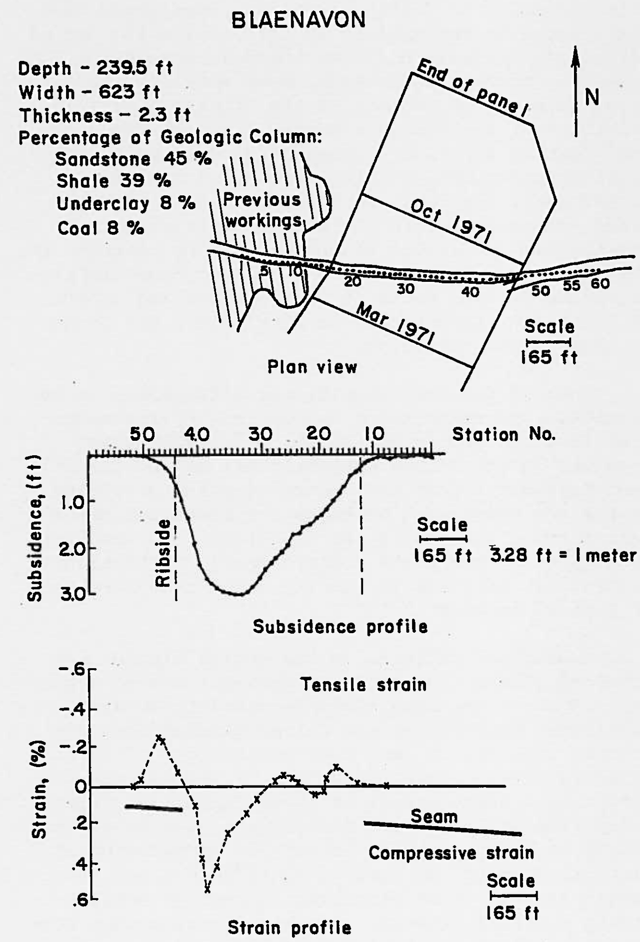

Figure 5. Summary of Information and Measured Movements for the Blaenavon Colliery.

observations for the Blaenavon Colliery in the South Wales Coalfield. The measurements at this site have been described by Collins as part of a study of residual mining movements.[16,17]

At this site, previous mining had been performed in three seams overlying and adjacent to the panel under observation. The approximate location of the known previous workings are shown on the plan view. Accurate records of the previous mines are not available, although the older workings in the relatively shallow seams are most likely to have been room-and-pillar mines.

The centerline settlement is approximately 2.6 ft, which corroborates previous centerline settlements that have been reported for the same panel by Orchard and Allen.[18] The settlement profile at this site is notably asymmetric with the maximum settlement being off center by a lateral distance of approximately 20 percent of the panel width. The maximum settlement corresponds with the point of maximum compressive strain and is approximately 50 percent in excess of the NCB prediction for similar panel geometry. In analyzing the measurements, Collins has noted that the previous workings most likely have contributed to the excessive subsidence movements.[16]

Old Ben No. 24

Figure 6 summarizes information and subsidence

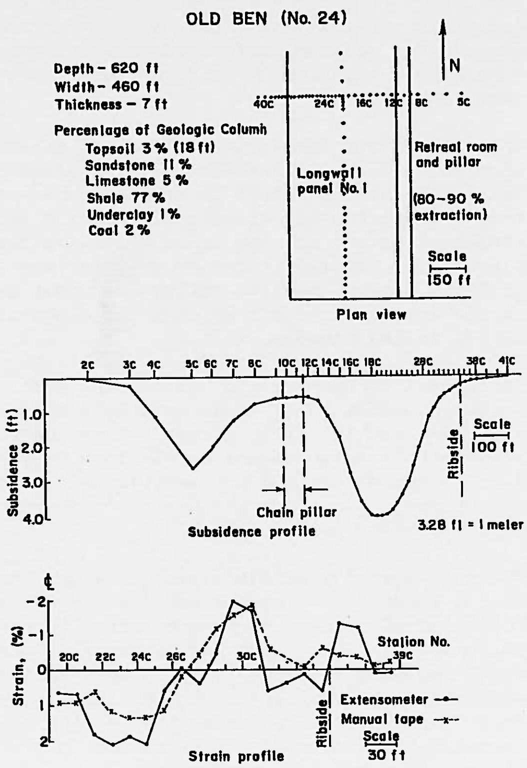

Figure 6. Summary of Information and Measured Movements for the Old Ben Number 24 Coal Mine.

observations over a longwall panel at the Old Ben No. 24 Mine in Benton, Illinois. The panel was driven from north to south in an area where no previous workings had been performed either above or below the mined seam. However, retreat pillar mining was performed adjacent to the longwall panel. The percentage of the coal seam extracted by this operation was approximately 80 to 90 percent. Chain pillars, roughly 60 ft wide, were left between the longwall and retreat pillar panels.

Along the observation line, the maximum settlement incurred by the retreat pillar mining was approximately 70 percent of that for the longwall. The maximum settlement over the longwall panel is well predicted by the NCB Handbook; however, there are notable differences between the shape of the subsidence profile forecast on the basis of British experience and the pattern that was observed. The subsidence profile at the Old Ben Mine was narrower than the typical profile in the United Kingdom, with the point of maximum curvature occurring at a horizontal distance, about 20 percent of the panel width, inward from the panel edge. The shape of the subsidence profile closely resembles the shape of other profiles over longwall panels in virgin strata reported by Gentry and Abel and by Dahl.[19,20]

The point of maximum curvature corresponds to the point of maximum horizontal tensile strain. The horizontal strains across the western edge, or ribside, of the panel are plotted in Figure 6. Horizontal strains were measured with a tape extensometer and by means of optical sighting and manual taping. The two methods of measurement serve as checks on each other. Both sets of measurements show maximum tensile strains of nearly 2 percent. This is roughly three to four times in excess of the values predicted by NCB charts for a similar mining geometry.

Subsidence and Mining Geometry

In Figure 7 the centerline settlements measured over longwall and retreat pillar panels in the United States are normalized with respect to the thicknesses of the excavated seams and plotted as a function of the ratio of panel width to depth. The data for the plots are taken from published case histories. In each case the settlement was measured at a point or averaged from the measurements among several points, each located a longitudinal distance of at least 0.7 times the panel depth from both the face and rear boundary of the panel. In this way, the observation points should have been sufficiently removed from any support contributed by the panel face and rear boundary to insure that panel width has been isolated as the geometric variable.

Figure 7a summarizes the dimensionless data for several longwall mines. A curve showing the national average for the United Kingdom also is presented in the figure. For mining in virgin strata, the NCB recommends that the normalized settlement determined by their charts be reduced through multiplication by 0.9.

Two characteristics emerge from the limited data: 1) the normalized settlements for the Illinois longwalls agree with the United Kingdom data, especially if a reduction in the NCB curve is made for virgin mining; and 2) even when reductions are used, the settlements from the Allegheny Plateau and New Mexico are significantly less than forecast by the NCB. With regard to the latter point, Gentry and Abel have noted that the overburden at the site of the New Mexico observations contained a significant proportion of thick sandstone and that the presence of thick, strong sandstone beds has been elsewhere associated with affecting the extent of the surface area influenced by mining.[19, 25] These correlations can be taken further to point out that the relatively small settlements over mines in the Allegheny Plateau are associated with the presence of thick limestone and sandstone beds. However, the data base is currently very small and trends of this nature need to

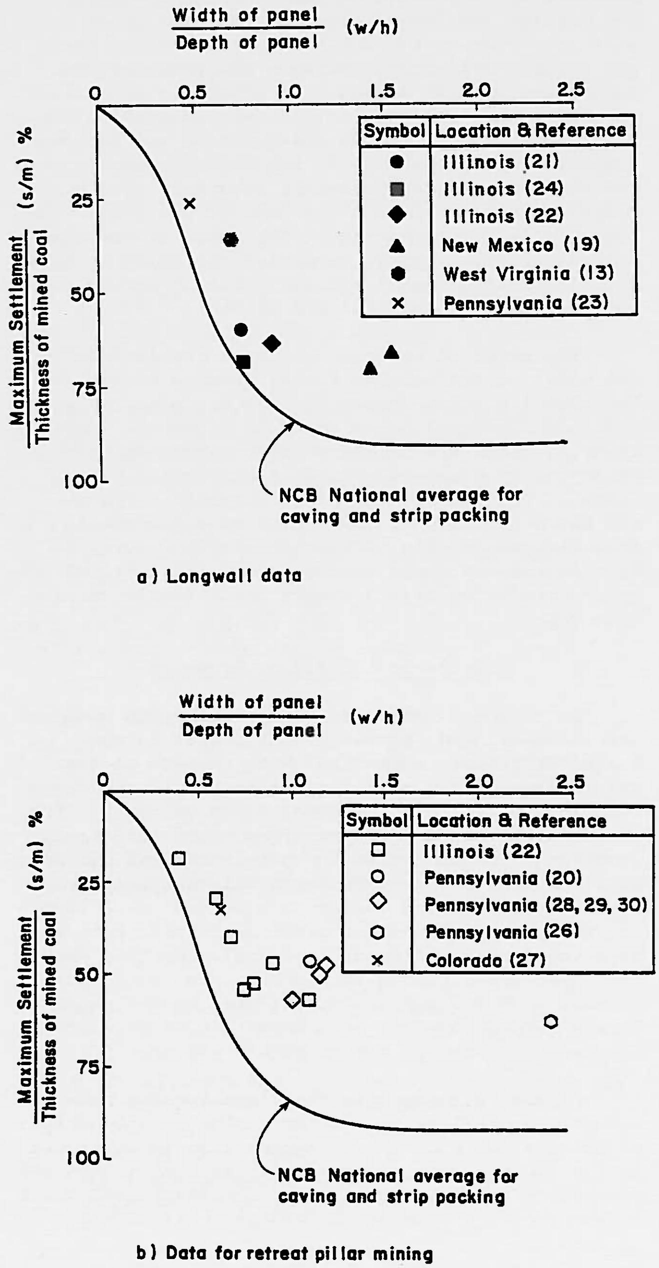

Figure 7. Normalized Settlements as a Function of Panel Width to Depth Ratio for Several United States Coal Mines

be substantiated by additional case histories.

With retreat pillar mining unexcavated zones of coal, referred to as stumps or fenders, are left in place for purposes of local stability. The caving mechanism over the mined area is affected by the unmined pockets. Although the percentage extraction for a given coal seam is higher than that of traditional room-and-pillar mining, it nevertheless is smaller than the percentage associated with longwall mining. In the Illinois Basin, typically 80 to 90 percent extraction is achieved by retreat pillar mining. In the Allegheny Plateau commonly 75 to 95 percent extraction is attained with the method.

Figure 7b summarizes the normalized settlements for several retreat pillar operations in various regions of the United States. The settlements are substantially smaller than those associated with longwall mining in the United Kingdom. They also are smaller than longwall settlements in the United States, although trends are difficult to distinguish for panel width to depth ratios less than 0.7.

Summary

On the basis of examining subsidence patterns in various regions of the United States and in the United Kingdom, several characteristics of the deformations and associated ground conditions can be identified:

1. Settlement profiles over United States longwalls in virgin strata differ from those typical of the United Kingdom. In general, the settlement profiles measured over United States longwalls are relatively narrow. In particular, the zone of maximum surface curvature and tensile strain tends to be shifted closer to the panel centerline.

2. Data summarized from several longwall panels show centerline settlements that are significantly smaller for mines in strata containing thick sandstones and limestones than the settlements typical for longwalls in the United Kingdom. A limited number of observations in Illinois show centerline settlements that are consistent with those predicted for conditions in the United Kingdom.

3. Horizontal surface strains associated with longwall mining in Illinois have been measured at values as high as 2 percent. These corroborate strain measurements at the York Canyon Mine in New Mexico and are substantially higher than those forecast on the basis of observations in the United Kingdom.[19, 2]

4. The presence of previous mining activity can have a large impact on the magnitude and distribution of surface movements caused by longwall mining. When comparing longwall observations for virgin mining in the United States with movements typical for the United Kingdom, it is important to recognize that mining in the United Kingdom commonly is performed in ground disturbed by previous mining.

5. Surface developments can have important repercussions on subsidence. In particular, the settlements on embankments over longwall panels may be large relative to those on level ground. In addition, high horizontal strains may occur in response to spreading at the embankment margins.

Acknowledgements

Deep gratitude is extended to Dr. Colin Jones of the West Yorkshire County Council for his assistance in providing case history information for the Royston and Rothwell sites. Dr. Steve Hunt of the Illinois Geological Survey provided many useful comments. Discussions with both Dr. Jones and Dr. Hunt were particularly rewarding, and their generosity is acknowledged. Discussions and correspondence with William Eichfeld of the U.S. Department of Energy and Dr. B.J. Collins of the Polytechnic of Wales were most helpful.

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EVALUATION OF DAMAGE POTENTIAL TO EARTH DAM BY SUBSURFACE COAL MINING AT REND LAKE, ILLINOIS

by

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Abstract. Presented are results of a study undertaken in 1975-1976 to evaluate probable effects of past, impending, and future coal-mining activity at Rend Lake Dam and Reservoir, near Benton, Illinois. Discussed is the conservative manner in which scarse Illinois longwall-subsidence data were used. Angles of influence defined in terms of critical deformations (damage criteria) were established. Results of study are evaluated in light of recently acquired longwall-subsidence data. Conservatism of study is confirmed. Future limited mining under dam, present and future mining under reservoir, and spontaneous subsidence of old mine under right abutment are also discussed. It is concluded that dam and appurtenances will not be affected by any of the types of mining activity discussed as long as such activity remains as outlined in this paper.

Introduction

This study was undertaken in late 1975 and early 1976 in order to evaluate the probable effects of past, impending, and future coal-mining activity near Rend Lake Dam and Reservoir (St. Louis Army Corp of Engineers). The dam is located along the Big Muddy River in Franklin County, about 5 miles northeast of Benton, Illinois and about 80 miles southeast of St. Louis, Missouri.

Figure 1 shows the axis of the dam, the outline of the reservoir, and the location of the extractive activities (coal mines and oil field) that could have some influence on the integrity of the dam and reservoir. Also shown is the surface trace of a fault zone that crosses the axis of the embankment under its right abutment. Figure 2 is an oblique air photograph of the dam and appurtenances. Specific items of concern were:

1) influence of impending longwall operations by Old Ben Coal Company, Chicago, Illinois, downstream from the dam at Old Ben No. 24 Mine.

2) probable reactivation of the fault because of future limited mining under the dam.

3) probable hydraulic communication between reservoir and high recovery (90-100%) mining areas under the reservoir.

4) long term stability of an abandoned room-and-pillar mine (Old Ben No. 19 Mine) under the right abutment.

5) influence of injection and withdrawal of fluids at an oil field (Benton North Oil Field) near the left abutment. The influence of this oil field was found to be non-existent and will be discussed no further.

Evaluation of subsidence and geologic data indicated that no problems would be anticipated in connection with any of the types of mining activities. Thus far two longwall panels have been completed downstream from the dam and some of the results of the surface deformation measurements show that significant deformations cease several hundred feet from the toe of the embankment. The main purpose of this paper is to illustrate the problems faced in forecasting surface damage induced by coal-mining subsidence in southern Illinois. As an example, a detailed discussion is presented of the manner in which the existing information was used to evaluate the probable influence of the downstream longwall mining. This evaluation is then compared with some of the results obtained from the instrumentation program conducted during and after the longwall operations.

General Geology

The topography in the area of the Rend Lake Dam and Reservoir is very gently rolling. The surficial deposits vary in thickness from approximately 10 ft or much less, in the rolling hills around the periphery of the reservoir, to about 50 to 60 ft under the reservoir, and overlie a relatively flat bedrock surface. Thin Illinoian drift caps the hills. The reservoir itself is underlain by about 30 ft of lacustrine silty clays (CL and CH) and about 20 to 30 ft of alluvial, silty, and clayey sands and discontinuous bodies of Illinoian till. The lacustrine deposits were laid down as a consequence of damming of the Big Muddy by the agrading Illinois River at the end of the Illinoian glaciation. These deposits play an important role in the water-tightness of the reservoir as will be discussed later.

The bedrock under the dam site consists predominately of shales and claystones and contains less than 10% of sandstone and limestone. The coal seam mined in the general area is the Illinois (Herrin)

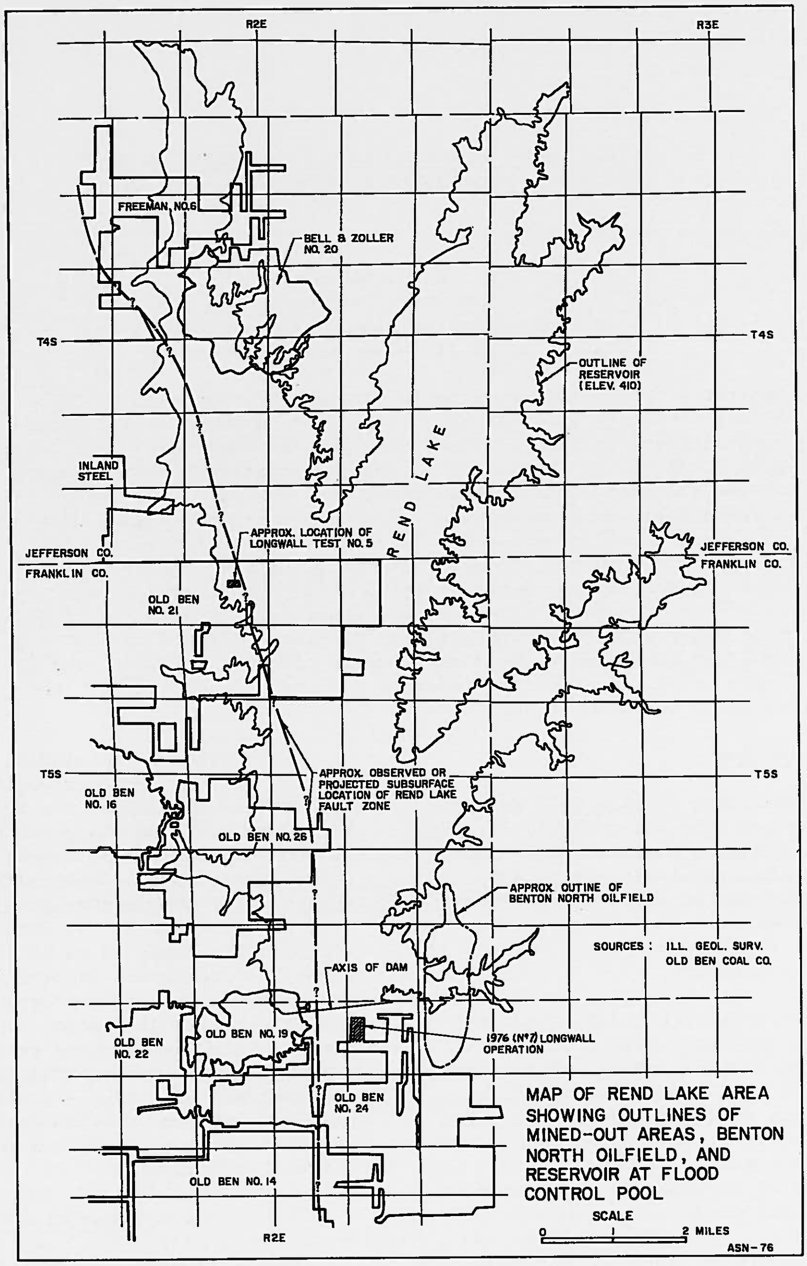

Figure 1. Map of area around Rend Lake Dam and Reservoir showing extractive industries (oil and coal-mining) as of 1976.

No. 6 coal seam which occurs at a depth of 620 ft under the dam. The roof and floor rocks are not known to present any of the stability problems (roof falls or squeezes) existing in other parts of the Illinois basin. Six to 12 in. of coal are customarily left at the roof and floor of openings to protect the rock from degradation.

A fault zone, recently named Rend Lake fault zone[1], has been mapped in the coal mines upstream and downstream from the dam. This fault zone was not uncovered by exploration for the dam nor was it disclosed during excavation. Thus, a question exists about its continuity to the surface. Subsurface control at two mines less than two miles from the dam site indicate that the fault zone at the site has a north-south strike and crosses the dam axis at approximately station 74 + 00 (see Figure 3). Under the site, this fault zone should have a total width of about 100 ft, a net vertical displacement (downthrown to the east) between 25 and 40 ft, and probably is composed of three or more steeply dipping (65-90°) faults which contain a few to several inches of clay gouge. The nature of the gouge suggests a field permeability of about 10^{-5} to 10^{-6} cm/sec. Observations in coal mines outside and under the reservoir show that the individual faults and associated fractures are extremely dry. Thus it appears that regardless of the vertical extent of the fault zone, an effective seal presently exists

Figure 2. Oblique air photograph of Rend Lake dam and appurtenances, looking east.

between the reservoir and possible hydraulic conduits in the bedrock.

Description of the Dam and Appurtenances

The Rend Lake Dam is a zoned earthfill embankment with a length of approximately 10,000 ft and a maximum height of 54 ft. The crown is 30 ft wide and has an elevation of 424 ft. M.S.L. Figure 3 shows an outline of the dam, and the location of the main and auxillary spillways, and the outlet works. Other features in Figure 3 will be discussed later. The main spillway is an ungated concrete structure located on the left abutment and has a crest length of 430 ft. The auxillary spillway is also located on the left abutment and has a crest length of 500 ft and an elevation of 415 ft. The outlet works consist of two 4 x 6 ft conduits regulated by four slide gates. Three important design aspects of the dam, in connection with this study, are the seepage-control system, the nature of the select fill, and the freeboard. Seepage is controlled by a 5-ft thick chimney located 10 ft downstream from the dam centerline, and 3-ft thick horizontal sand drain which extends to within 1 ft from the toe; seepage flow is collected by an 8-in. perforated pipe in a gravel filter. This collector system connects with precast concrete manholes, which in turn drain through 12-in. pipes into a collection ditch. The select fill extends from the chimney drain to the upstream slope and consists of inorganic clays (CL) of medium plasticity (Lw = 35-45%; PI = 20-25%). Finally, the freeboard is approximately 14 ft with respect to flood-control pool elevation (410 ft). Instrumentation relevant to this study includes open and closed system piezometers located in the select fill, the chimney and horizontal sand drains, and settlement gauges installed on bedrock downstream from the abutments of the dam.

Subsidence Background

The No. 6 coal seam was mined in the general area of Rend Lake many years before the construction of the dam and the creation of its reservoir. Figure 1 shows the extent of the reservoir at the flood-control pool elevation, 410 ft (this elevation will be assumed in all discussions concerning the reservoir). This Figure is current for the operations of the Old Ben Coal Company as of 1976; it does not reflect recent mine developments for the other coal companies. However, close to 40% of the mined-out area currently under water had already been developed prior to the first filling of the reservoir (1971). The recovery at production panels in the mined-out areas under the reservoir varies from about 90-93%, in areas where the Old Ben system of continuous miners is used (a majority of the areas being presently developed), to 40-60% in old areas such as the abandoned Old Ben No. 19 mine. Figure 1 also shows at least one area in which the recovery in a production panel reached 100%--namely, the Longwall Test No. 5 carried out by Old Ben Coal Company in 1966.

Ground surface deformation invariably accompanies coal mining as extensive as that shown in Figure 1 and recoveries as high as those mentioned above. Surface damage is caused by both vertical and horizontal displacements, but the extent of damage depends on the type of surface installation.[2,3,4] Masonry and concrete structures are affected by subsidence gradients which cause shear strains, as well as by tensile and compressive strains. Earth structures are primarily affected by tensile strains. The most undesirable type of

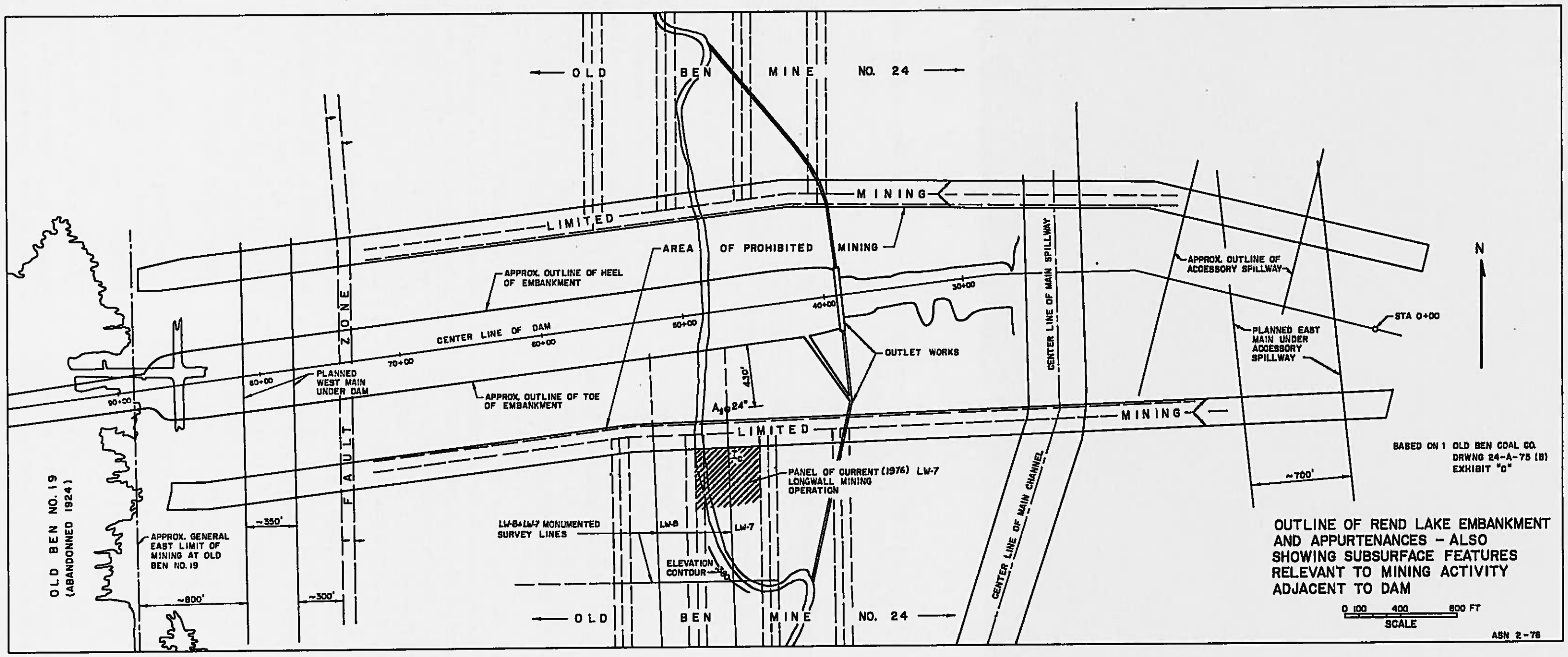

Figure 3. Map showing outline of Rend Lake and Appurtenances, and subsurfaces features relevant to minig activity adjacent to the embankment.

tensile strains in an earth embankment such as the Rend Lake dam is that occurring parallel to its axis.

Former Measurements of Surface Subsidence Around the Study Area

A comprehensive work on subsidence for Illinois, now available,[5] did not exist at the time of this study. Review of the geologic literature on the general area and gathering of unpublished reports were undertaken. As early as the beginning of the century, it was known, in the general area, that recoveries greater than 40% at production panels may be accompanied by considerable surface subsidence. Young[6] reported maximum subsidence of 36 in. for a depth of 500 ft, seam thickness of 8 ft, and an extimated recovery of 50-60% at the production panels.

Results of level surveys conducted by Old Ben Coal Company under their modern mining operations were made available for this study. The data for Old Ben No. 24 mine (No. 6 coal, 7 ft thick, 600 ft deep) indicated a maximum subsidence of 3.5 ft for maximum recoveries of about 93% at production panels. The most useful subsidence data was from surveys over Longwall Test No. 5 at Old Ben No. 21 mine. This former longwall operation had a panel width, W, of 610 ft, a depth, D, of approximately 650 ft, and a mined height, m, of 7 ft. Some of these subsidence data are discussed in detail in the next section. The precision of the survey readings was 0.01 ft; stations were wooden stakes driven into the ground every 50 ft and tied up to bench marks located outside the area of influence. Closing errors were generally kept within less than 0.1 ft. The quality of Old Ben level surveys was believed to be generally good, bearing in mind, of course, that the use of stakes rather than survey monuments limited their accuracy. No horizontal displacements were measured in any of these surveys, nor were any found in the reviewed literature on mining subsidence for the entire state of Illinois.

Evaluation of Effects of Mining Downstream from Dam

Discussed in this section is the method that was used in evaluating, before mining, the probable effects of the downstream longwall operations. This discussion illustrates the problems one faces in evaluating subsidence damage in the Illinois basin. The problems arise because of the lack of a reliable data base and of previous studies. The position of the two longwall panels, Old Ben Long Wall Test No. 7 (LW-7) and Old Ben Long Wall Test No. 8 (LW-8), are shown in Figure 3 with respect to the embankment and appurtenances. The average horizontal distance from the north side of the Zone of Limited Mining to the toe of the drainage ditch (the downstream limit of the embankment) is about 450 ft; this distance allows for an angle of draw of about 35°. The north faces (ribsides) of the longwalls, however, are approximately 700 ft downstream from the toe of the embankment; this distance allows for angle of draw of 48°. The width of the panels is 460 ft (W/D = 0.75), their length is about 1500 ft, and the mined height is 7 ft. Mining began from the north; the LW-7 operation began in September, 1976 and was completed in May, 1977; the LW-8 operation began in August, 1977 and was completed early this year (1979).

Of concern was whether the future longwall operations would induce surface deformations that could produce tensile cracks in the embankment or appurtenances. Review of the published literature on mining subsidence had showed no information regarding angles of draw or measurements of longwall subsidence for the southern Illinois region and very little for the entire USA. However, a large amount of data for European, in particular British, coal fields did exist.[2,3,4] Figure 4, from Woodroof[7], shows, for example, the variation of angle of draw with W/D. The graph shows angles of draw of about

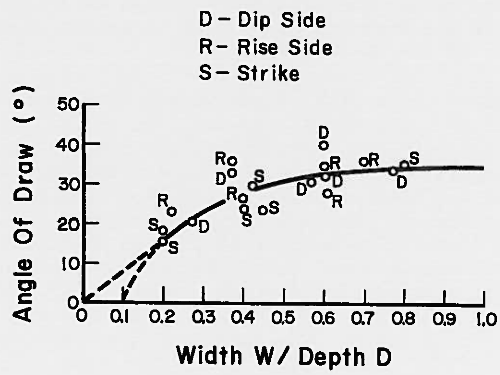

Figure 4. Plot of angle of draw versus width-to-depth ratio for British mines (Woodroof)[7]

35° for values of W/D greater than 0.6. Maximum tensile strains, $+\varepsilon$, reported were as high as 1.0% but common values usually ranged between 0.50 and 0.25%. More importantly, the zone of maximum tensile strain was reported to coincide with the position of the ribside for values of W/D greater than 1.35, but to lie outside the ribside for smaller values of W/D[3]. This latter observation raised the possibility of a displacement of relatively high tensile strains northward from the panels toward the dam.

The decision was made to evaluate the effects of the LW-7 and LW-8 operations on the basis of the subsidence measurements for LW-5 at Old Ben No. 21. Further, angles of influence were defined in terms of reasonable damage criteria and were used to express the extent of zones, outside the panels, where deformation might reach an unacceptable level. An influence angle was defined as:

$$\alpha = \tan^{-1}(d/D) \qquad (1)$$

where d is the horizontal distance from the projection of the ribside on the surface to a point where the value of the damage criterion (deformation) is exceeded.

The advantages of using the data from LW-5 was the close proximity between this and the future longwall operations and, hence, the similar geologic set-ups and expected ground responses. The disadvantage of using those data was the relatively low accuracy of the surveys and wide spacing between stations (50 and 100 ft) in zones where tensile strain were expected to be critical. Some slight differences existed between the longwall operations; the depth of overburden is 620 ft for LW-7 and LW-8, and 650 ft for LW-5; the width of the panels is 460ft for LW-7 and LW-8, and 610 ft for LW-5. Thus, W/D = 0.75 for LW-7 and LW-8, whereas W/D = 0.94 for LW-5.

Angles of influence between 10° and 30° were obtained using different assumptions and damage criteria. These results were obtained based on subsidence data surveyed 7 months after beginning of the LW-5 operation, and have been reported elsewhere.[7]

The following paragraphs describe the way in which an angle of influence was obtained, and illustrate the high degree of conservatism that can be reached because of the lack of adequate subsidence and strain data. The subsidence values used include results of measurements made up to 32 months after beginning of the LW-5 operation.

Figure 5 is a plot of s/m versus d/D, where s is subsidence, for the stationary face at LW-5 along the panel centerline. The individual data points are the average of the last three leveling surveys. Averaging was done in order to reduce the probable surveying errors. Portions of a logarithmic curve could be fitted reasonably well through the individual subsidence points. This curve was then extended up to its intersection with the line of 0.02 ft of subsidence; this intersection was defined as the point of "zero engineering subsidence", A_5, for the purposes of the study. It was believed that that amount of subsidence distributed over several hundred feet could be easily tolerated by the Rend Lake embankment. More importantly, the differential subsidence or slope, g, for the last two stations surveyed (1 + 00 W and 0 + 00 W) was only 0.0005. This value is considerably lower than the differential settlement that large concrete and masonry structures will accept before onset of cracking.[8]

Since there was no information on horizontal displacement, some assumptions have to be made regarding the probable magnitude and distribution of tensile strains. As a first step, a rough calculation of curvature and tensile strains was made for the smooth curve at station 0 + 00 W using the empirical relations given by the United Kingdom National Coal Board (NCB).[3] The results indicated negligible tensile strains (10^{-6}). However, for consistency, the value of "zero engineering tensile strain" was placed at A_5. The point of maximum tensile strain was placed at B, the point of maximum curvature for the subsidence profile.[2,3,9] As seen in Figure 5, this point is at a value of d/D = 0.15, inside the panel, rather than outside the panel as would be expected from the British literature[2,3] for a value of W/D = 0.75. (Some of the Australian data, however, show zones of maximum tensile drawn slightly inside the panels for values of W/D less than 1.0.[4]) The surveying stations had been spaced more widely than is recommended for accurate determinations of the point of maximum tensile strain;[10] nonetheless, the point thus determined was accepted and the difference between the locations of the point was explained in terms of geological factors. Calculations of maximum tensile strains from the curvature of the smooth curve using the NCB method gave values of strain of less than 0.4%. However, the value of maximum tensile strain was chosen as 2.0%, a value two to four times the common values reported for the British fields. The high tensile strains were chosen because field observations in the rim of the reservoir had disclosed the presence of tensile cracks compatible with such high strains; the cracks were associated to high extraction mining under Old Ben No. 26 mine. Figure 6 is a photograph of one of those cracks. Some of the cracks were isolated ones, a few to several inches wide and up to 100 ft long. In one instance, two parallel cracks were observed, about 80 ft apart, each one several inches wide. The cracks in all cases were observed in areas of sandstone outcrops or with thin (few feet) soil veneers.

Next, points A_5 and B in Figure 5 were connected by a straight line which was (conservatively) assumed to be the distribution of tensile strains. It was also assumed that tensile strains greater than 0.25% would probably cause cracks in the types of soils used in the embankment. Evidence indicates that this is a realistic damage criterion.[11,12] The intersection of line $\overline{AB}$ with a horizontal line representing tensile strain levels of 0.25% defined a distance outward from the ribside in which tensile strains had values greater than 0.25%. This distance is 0.44 D and corresponds to an angle of influence of 24°. It was clearly understood, however, that the real tensile strain distribution was not linear and that it probably resulted in an angle of influence of considerably less than 24° as shown by the dashed line in Figure 5.

This information could now be applied to the future longwall operations downstream from the dam. The horizontal distance corresponding to the angle of influence (24°) was plotted from the stationary walls for LW-7 and LW-8 along the panel centerlines, (see Figure 3) and a conservative limit was obtained for damage by tensile strains (430 ft downstream from the toe of the embankment). A further element of conservatism was introduced by the use of a subsidence profile for a panel with a value of W/D = 0.94 (LW-5) to predict subsidence displacements induced by individual panels with values of W/D = 0.75 (LW-7 and LW-8). The chain pillar between LW-7 and LW-8 was considered strong enough to prevent the two individual panels from combining into a single panel with twice the value of W/D. Therefore, it could be concluded quite safely that no damage to the embankment would be caused by the planned longwall operations. The main and accessory spillways were further away from the planned operations and were less likely to be affected in any manner.

The method outlined above is rather conservative and its use is not advocated to predict realistic zones of damage. However, in connection with this study, it was found useful in demonstrating that it was highly unlikely that surface damage could exist beyond a certain point.

Result of Instrumentation Program Downstream from Dam

An extensive instrumentation program was undertaken jointly by Old Ben Coal Company and the US Bureau of Mines (subsequently by the US Department of Energy) to monitor ground response to the LW-7 and LW-8 operations. Some of the results of this program have been presented by Conroy[13] and Schmechel and others.[14]

One line of subsidence monuments was constructed over each of the two panel centerlines and a third one was placed perpendicular to the first two at the midpoints of the panels (see Figure 3). The spacing of the monuments is 30 ft for the most critical areas of the subsidence profiles (near the ribsides), 60 ft for less critical areas, and 120 ft at average distances of about 400 ft from the ribsides. The surveys were made to Second Order, Class II (vertical), or Third Order, Class II (horizontal)

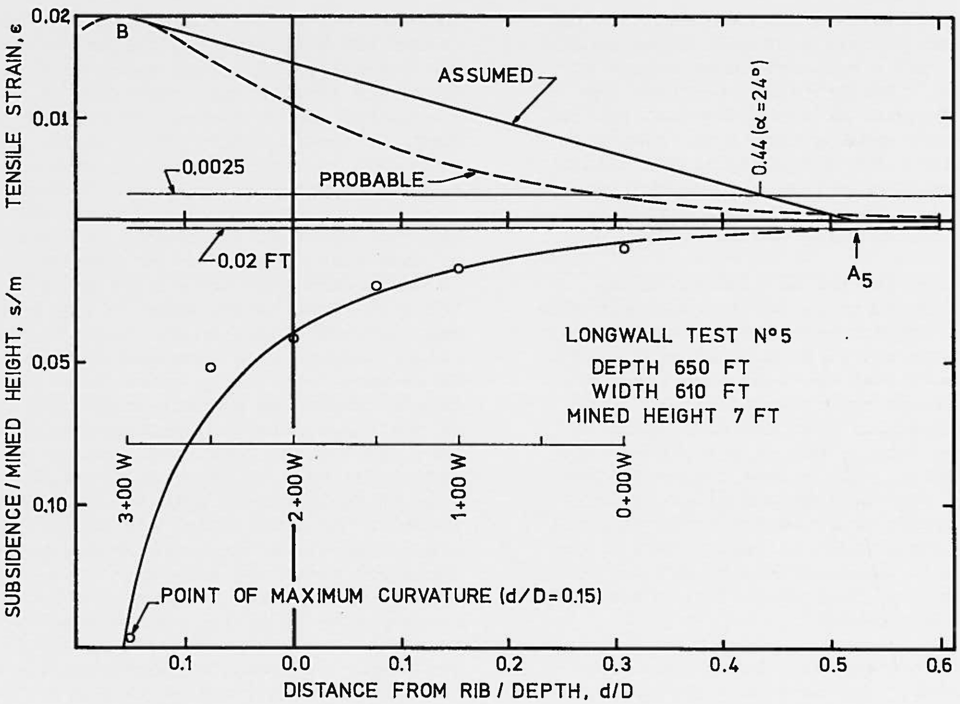

Figure 5. Subsidence data from Longwall Test No. 5 (1966) Old Ben No. 21 Mine and assumptions made in damage evaluation.

accuracy standards.[14] In addition to these surveys, the program included the monitoring of a "traveling surface strain wave" by electronic (Automated Data Acquisition System or ADAS) and conventional surveying (precision tilting level, invar level rod, and steel tape) techniques.[14] Observations were made on five monuments spaced at 15-ft intervals and located near the center of the LW-8 panel, along its centerline.

Figure 6. Crack (dark area in grass) in thin veneer of soil over rock along west rim of reservoir near Old Ben No. 26.

Figure 7 shows data from this instrumentation program available to the writer at the present time. Values of s/m and tensile strains (ε) are plotted against d/D. The subsidence data were obtained by the monthly surveys of vertical displacements performed along the longitudinal monument lines and the tensile strain data were obtained by the monitoring of the traveling subsidence wave. Both the results of the ADAS and of the field tape measurements are included and show very good agreement. The point of zero engineering subsidence for LW-5, A_5, the assumed linear tensile strain distribution for LW-5, and the subsidence at ribside for LW-5, (s_5), have also been included for comparison. The subsidence points are for the stationary wall (north ribside) of the panels and are the average of the last three surveys, the last one of which was performed in December, 1978 (27 months after the start of LW-7 and 16 months after the start of LW-8). The location of the points of maximum curvature for LW-5, LW-7, LW-8 and the ADAS subsidence profile (not shown) are presented along the horizontal axis of Figure 7. Note that the average distance for these points seem to be centered at about a distance from the rib of 0.15 D inside the panels.

Points A_7 and A_8 (subsidence less than 0.02 ft) essentially coincide and occur fairly close to A_5. The distance from A_7 and A_8 to the ribside define an angle of influence of 29°. Also shown in Figure 7 for both subsidence profiles are the points at which the ground slopes, g, exceed values of 0.005 and 0.002. The average distance (from the ribside) at which these slopes exceed the above values define angles influence of 14° ($g > 0.005$) and 18° ($g > 0.002$). It should be noted that there is ample evidence that earthfill dams can easily tolerate differential settlement as large as those, particularly if such settlements occur parallel to the embankment

Calculations to evaluate the stability of the roof, floor and long pillars were made using recommended methods[2,7] and a range of known values of material properties. Results indicated that the three structural elements of the individual entries had factors of safety greater than 2.0. However, the strongest evidence for stability of the mains, as currently designed, came from the observation that no subsidence was known to have occurred in mined areas where the recovery was as low as 20%.

A second problem related to limited mining under the dam was the possible stress redistribution that could trigger displacements on the east side of the abandoned Old Ben No. 19 mine or along the fault zone. Figure 3 shows that the west main under the dam lies 300 ft west of the first fault and about 800 ft east of the general east end of Old Ben No. 19. Because of the large width of the pillar ribs relative to the entries, the stress concentrations on each entry wall will be independent of the concentrations on adjacent entry walls. Thus at about 10-20 ft from the easternmost or westernmost entry walls the increase in stress caused by driving of the entries should have dissipated to the state of stress existing prior to mining.

It was therefore concluded that ground movements would not occur in connection with the proposed limited mining under the dam.

Mining Upstream from the Dam and Under the Reservoir

As shown in Figure 3, the planned upstream development of Old Ben No. 24 and the downstream development are symmetrical with respect to the axis of the dam. It is then reasonable to assume that any anticipated effects on the dam and appurtenances by upstream mining should be the same as those, namely none, caused by the downstream operations. The only difference in the two operations will be the weight applied by the reservoir at the level of the coal seam. At the depth of the coal seam, the increase in vertical stress because of the weight of the reservoir has a maximum value of 2%. An increment of 2% in the vertical stress is so small as to be insignificant in any stability considerations in areas of limited mining.

More important, however, were problems associated with the probable hydraulic connection between the reservoir and the underground works. This possibility needed to be examined, particularly in view of the presence, on the west margin of the reservoir, of the surface cracks. These cracks are known to be associated with thin soil covers and with sandstone outcrops. Thus, they probably reflect localized tensile strain concentrations in the shallow competent rock layers. Localization of strains in rocks along joints or other fractures is not uncommon.[10] On the other hand, surface tensile strains under the reservoir will have a more uniform distribution because there the unconsolidated sediments are about 60 ft thick. The upper 30 ft of this section is mostly CL-CH soils which can react plastically to the imposed deformations and may not develop cracks with any appreciable continuity. These lacustrine deposits should exist throughout the extent of the impoundment since the ancestrial lake was more areally extensive than Rend Lake but occuppied the same drainage area.

Figure 1 shows that the ground surface over the Longwall Test No. 5 is currently submerged, close to the west end of the reservoir. At this location, cracks had been reported during the development of the longwall, and it was known that the soil veneer there was thinner and comparable to that where cracks have been observed around the reservoir. Thus, it seemed likely that, at one place under the reservoir, surface cracks had been present. Whether surface cracking under the reservoir is extensive or exists only in localized areas, the incontrovertible fact was that all the mines visited by the writer, under the reservoir and outside it, were dry--an indication that, if cracks do exist, they do not transmit water to the level of the mine. If cracks had been continuous to the level of the mining operation, leakage from groundwater in the unconsolidated materials (silty sands below the clay sequence) above the bedrock would have been observed in the mines prior to the building of the reservoir. That was not the case. The reservoir contributes an additional head of about 30 ft which, at the elevation of the mine, amounts to only a maximum increase of 5% of the total head. It was believed that the extra hydrostatic pressure in any postulated cracks would not cause any widening: the width of the cracks was throught to be controlled by ground movements caused by mining operations, not by a slight increase in water pressure. It was concluded that the danger of hydraulic communication with the underground works, and particularly that of a sudden connection, was very remote.

Subsidence Caused by Old Ben No. 19 Mine

Spontaneous ground surface subsidence caused by time-dependent failure of old abandoned mines, up to 80 years after closing of operations, is a known and troublesome occurrence in some areas of southern Illinois. The main factors believed to be responsible for this erratic behavior of old mines are progressive failure of pillars and deterioration of the mechanical properties of floor materials, with their eventual bearing capacity failure. Old Ben No. 19 certainly meets the criteria of an old mine. It could not be shown that the works have subsided shortly after abandonment, thus it had to be assumed that the mines could conceivably develop such behavior in the near or distant future. From the evaluation of the data of LW-5 it was conservatively concluded that tensile strains could reach levels harmful to the embankment anywhere between the west end of the embankment, approximately at station 106 + 00 and station 85 + 00. For a pool elevation of 410 ft, water is impounded behind the embankment up to approximately opposite station 103 + 00. Thus, tensile strains along this zone, between stations 103 + 00 and 85 + 00, could create cracks perpendicular to the embankment. The Corp of Engineers was cognizant of this problem and incorporated into their instrumentation program some rock settlement plates to detect movement caused by past mining. The design also incorporated considerable amount of freeboard (14 ft at flood-control pool elevation).

The likelihood of subsidence under the right abutment was considered low because it was known that delayed subsidence of old mines (40 years or more) usually occurs in areas with a history of continuing subsidence elsewhere in the area. This was not the case for the immediate area of the dam. However, it was reasoned that even if subsidence would occur, the event would not be catastrophic and the dam could withstand it. First, the settlement of the crest could not be greater than the freeboard (14 ft) because room-and-pillar operations, of the Old Ben No. 19 type, do not subside more than

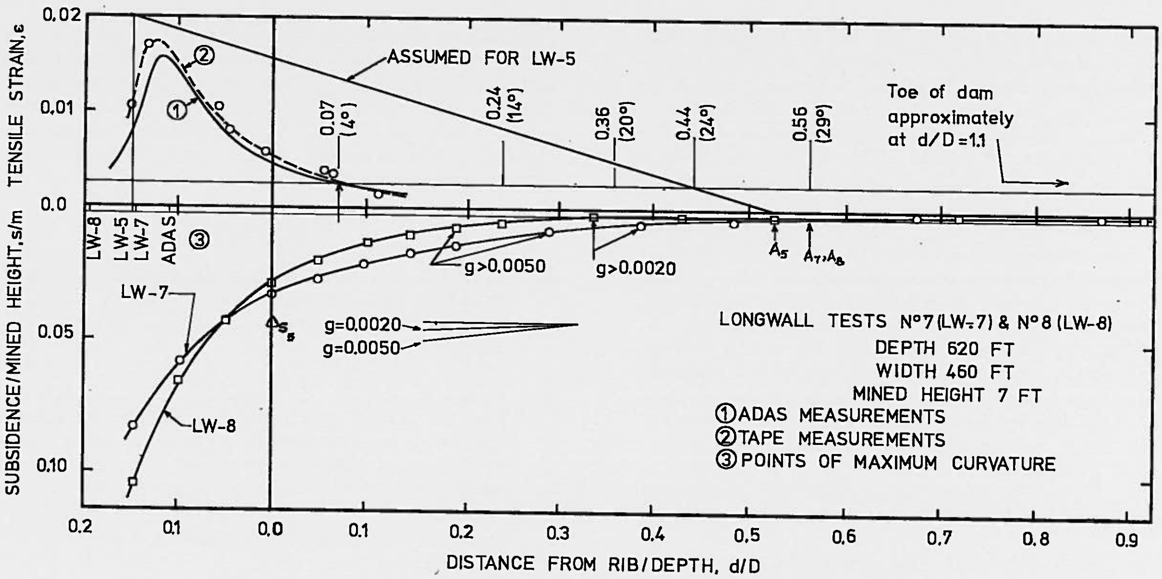

Figure 7. Recent (1979) subsidence and tensile strain data for longwall panels 7 and 8, Old Ben No. 24 mine; information from Longwall Test No. 5 also plotted for comparison.

axis.[11] However, damage to appurtenant structures (outlet works, drainage pipes) may occur.[3,9] Thus, the problem at these relatively low angles of influence for differential subsidence is not one of safety of the embankment but one of economic and legal nature involving the repairs that may be incurred to restitute the appurtenant structures to their original condition.

The larger subsidence values for LW-7 are not believed to be a reflection of a time effect but are believed to be caused by geologic differences. The larger value of subsidence for the point over the ribside in LW-5 (s_5) is compatible with the greater W/D value for that panel.

Three points are noteworthy concerning tensile strain. The value of the maximum tensile strain, approximately 1.7%, is greater than expected from a review of the British data, but close to the value estimated from field observations. Calculations of maximum tensile strain by the NCB methods yield values of less than 0.4% for both of the subsidence profiles shown. Second, the location of the point of maximum tensile strain occurs close to a distance of 0.15 D inside the panel, as expected from the LW-5 data. This location of the point of maximum tensile strain is confirmed by the location of points of maximum curvature for LW-7, LW-8, as well as for the ADAS profile (the average of these points occurs at 0.15 D). Third, the level of tensile strain decreases rapidly outward toward the unmined area. The angle of influence for tensile strains greater than 0.25% is only 4°. The angle of influence obtained by the conservative method outlined above is 24°.

Supporting evidence for higher tensile strains, higher surface slopes, and the position of the points of maximum tensile strain inside panels of high extraction in the Illinois basin, as compared to the British coalfield data, as been presented recently by Hunt.[5] It appears that the differences between the longwall subsidence profiles of the Illinois basin and those of the British coalfields are a function of rock mass properties. The Illinoian rock masses appear to respond as stronger materials. This difference in rock-mass quality may be partly caused by inherent geological factors and partly by the fact that a majority of the measurements from the United Kingdom have been made in rock masses disturbed by previous mining activity.

In summary, the data gathered from the instrumentation program confirm that the Rend Lake Dam and appurtenances are quite safe regarding surface deformation caused by the LW-7 and LW-8 operations. The data also show that the angles of influence for different damage criteria can vary from 20° to much smaller values.

Limited Mining Under Rend Lake Dam

Figure 3 shows the outlines of two main haulageways or mains under both sides of the Rend Lake Dam. These mains will eventually connect the upstream and downstream operations at the Old Ben No. 24 mine. The west main will have six individual entries and the east main will have eleven entries. Their total widths are approximately 350 ft and 700 ft, respectively. The overall recovery is to be approximately 20%. Each entry will be machine-driven and will be 7 ft high; the width will depend on the type of machine used: the continuous miner will bore a 12.6 ft-wide entry; with individual rippers, the width will vary between 13 and 14 ft. The entries will be roughly eliptical in cross section. All of these entries are to be driven with a minimum spacing of 60 ft between centerlines. Thus, if a maximum width of 14 ft is assumed for the entries, intervening pillar ribs will be 46 ft wide.

Of concern in connection with the east main was whether such limited mining could result in ground movements that would eventually produce a trough of subsidence under the dam, perpendicular to its axis. The stability of the roof, pillars, and rock under the pillars needed to be evaluated as these were the elements which could create ground displacements.

50% of their mined height (9 ft, if it assumed that the entire seam was mined.[15] Second, if subsidence were to occur, the resulting tensile cracks could not be nearly as wide as those observed in the rim of the reservoir. The maximum tensile strains associated with subsidence of mines with 40-60% recoveries (at comparable depth) could only be approximately half of the maximum tensile strains for high recovery and longwall mines. More importantly, the soil under the dam between stations 103 + 00 and 85 + 00 has an average thickness of more than 50 ft. Thus, the tensile strains would probably create small but frequent cracks rather than large, widely spaced cracks. Finally, if such small cracks would begin to transmit water, the flows would be small because of the relatively small head (10 ft maximum) existing behind the dam between those stations. These small flows could be handled by the chimney and blanket sand drains until remedial measures were undertaken. No rapid piping could be foreseen given the nature of the select fill material.

It was concluded that the likelihood of a sudden failure because of spontaneous subsidence was very remote. However, it was recommended that a monitoring system be installed to supplement the existing settlement plates. This system consisted of one line of Second Order survey monuments, 100 ft apart, along the toe of the embankment, downstream from stations 103 + 00 and 80 + 00. It was recommended that surveys be run at the same times as the periodical inspections of the dam.

Conclusions

1) Present mining activities downstream from Rend Lake Dam, future limited mining under the embankment, and mining upstream from the dam under the reservoir most likely will not cause any deleterous effects to the dam and appurtenances.

2) An impervious seal exists between the level of the workings and the reservoir, and hydraulic communication is quite unlikely. This conclusion and the preceding one assume, of course, that both the actual extent and the methods of mining are essentially as understood by the writer and as outlined in the paper.

3) The likelihood of spontaneous subsidence under the right abutment is remote. If such an event were to occur, design provisions and geologic conditions would render the subsidence event noncatastrophic and remedial action could be undertaken.

4) This case history exemplifies the difficulties encountered in evaluating subsidence damage in the Illinois basin. These difficulties arise from the lack of subsidence measurement and previous studies. It also underlines the problems incurred in attempting to extrapolate the empirical principles from British coalfields. These principles, which are sound for given geologic conditions and mining history, need a great deal of adaptation[10] to be used effectively and safely in this country.

Acknowledgements

Some of the material presented was generated as part of a study for the St. Louis US Army Corps of Engineers in 1975-1976.

The writer is grateful to Steve Hunt, Illinois State Geological Survey, for his help and many illuminating discussions; to Michell Van Sint Jan, Department of Civil Engineering, UIUC, for his kind help in preparing the manuscript; and to Cecil Bailie, Old Ben Coal Company and Peter Conroy, Dames and Moore Inc., for providing the subsidence data for the original study. Ms. Judy Rouse diligently typed the manuscript.

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GEOTECHNICAL CONSIDERATIONS FOR DEADHEADING A MARION 5761 SHOVEL

by

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Abstract. Amax Coal Company moved a Marion 5761 Shovel weighing 8,000,000 pounds cross-country a distance of 3.2 miles. This is equivalent to placing a twenty story building on steel tracks imparting a unit load of 10 KSF. One of the primary considerations of such a move was to provide a stable subgrade on which the shovel could travel. Consequently, a geotechnical study was performed which analyzed the soil bearing conditions from which conclusions were drawn regarding wooden matting configurations, remedial earthwork, stability analyses, etc. Using the results of the study Amax executed a safe, economical move.

Introduction

In the summer of 1977, Amax Coal Company's Management instructed their operations personnel at Ayrgem Mine near Central City, Kentucky to investigate moving their Marion 5761 shovel to a new mining pit, about three miles away. Accordingly, the staff at Ayrgem began considering the ramifications of the move from both a production and technical standpoint. Because the 5761 shovel is one of the world's largest pieces of earth moving equipment, weighing about 8,000,000 pounds, the most obvious technical problem with such a move is the ability of the ground along the proposed route to support the shovel's weight. Therefore, Amax engaged a geotechnical engineering consultant, Law Engineering Testing Company, to perform a comprehensive geotechnical study along the alignment of the proposed route.

Several features of the move and study were non-routine:

1. The most obvious aspect is the size of the shovel. The weight vs. plan area is equivalent to placing a twenty story building on four pair of tracks and moving it cross-country. The loads are extremely large for earth bearing.

2. The subsurface conditions were complex and not "geologically predictable" because the route primarily traversed spoil generated by coal mining operations.

3. None of the conventional stability/bearing capacity analyses were directly applicable to most of the potential failure mechanisms.

4. The foundation bearing pressures could be economically altered and effectively distributed along the route by constructing various arrangements of wooden mats upon which the shovel traveled.

5. And, on the positive side, the Amax operations personnel are knowledgeable civil engineers who understand and were able to weigh the various geotechnical risks against the cost of remedial earthwork and other operations necessary to safely accommodate the moving shovel.

Information Known At The Beginning Of The Project

The machine is a Type 5761 Electric Shovel manufactured by the Marion Power Shovel Company, Inc. (See figures 1 and 2) at an original cost of over eight million dollars. The weight of the machine is 5,875,000 pounds. Including the additional ballast load of 1,900,000 pounds, the total weight of the shovel approximates 8,000 Kips. The plan area of the shovel is 50 feet by 50 feet as measured to the extremities of the exterior tracks. The machine stands 160 feet in height. The capacity of the excavating bucket is 65 cubic yards. Maximum cutting radius is 191 feet and maximum vertical cutting height is 165 feet. The shovel travels on four pairs of steel tracks, each driven by its own 250 h.p. motor. Each track has a ground contact area of about 100 square feet resulting in an applied bearing pressure of 10,000 PSF.

Typically, the shovel is operated in a box cut, removing overburden and re-depositing it after

Figure 1. Marion 5761 Shovel.

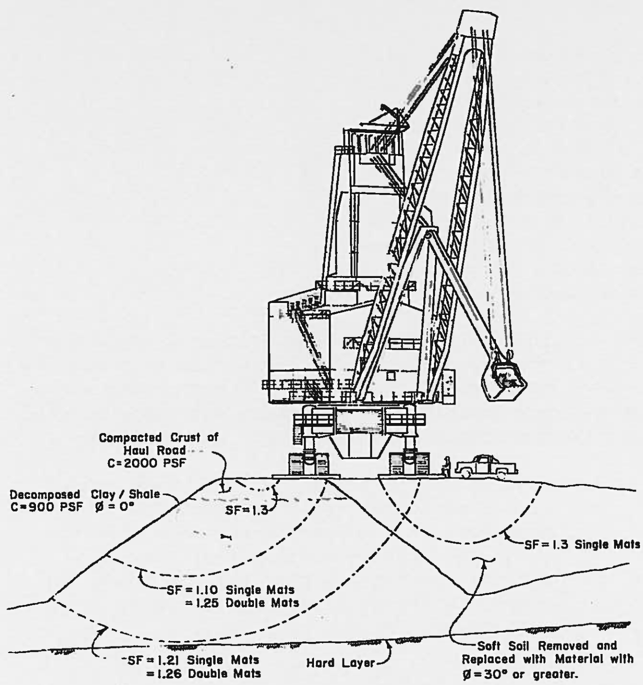


Figure 2. Diagrammatic Embankment Section showing potential failure surfaces with safety factors, removal and replacement of soft soil, and moving the route away from the edge of an embankment slope.

a 180 degree turn. The mechanics of a shovel operation dictate that the stripped overburden be spoiled indiscriminately and later shaped for reclamation by draglines and dozers. The only exception to the undifferentiated re-deposition of overburden is that a "buckwall" consisting of large boulders is usually placed at the base of the spoil for stability purposes.

Amax personnel selected three alternate deadhead* route locations. All of the proposed alignments traversed existing haul roads and previously reclaimed areas underlain by the mine spoil. As planned, the proposed route would extend slightly over three miles regardless of the alignment selected. Along each of the alternate routes, there were several "topographic hazards", consisting of silt ponds and narrow embankments.

It was known that a sister shovel at a nearby mine had "sunk" during a deadhead several years previously, but limited data was available regarding this failure. Additionally, a similar shovel in Illinois experienced subgrade failure causing it to sink about forty feet into the ground. This shovel was recovered at a reported cost of over one million dollars. Both of these previous failures seemed to be progressive wherein surficial failures were experienced first which immobilized the machine, and then the failure surface began to migrate downward (along with the machine) until a stable horizon was encountered.

A smaller shovel weighing 3,000 Kips had traversed the main proposed route earlier and had caused a partial embankment failure along an existing haul road. A failure bulge at the toe of the embankment had developed and the mine personnel noticed a visible deflection of the haul road surface. The smaller shovel was quickly moved past this weak section. The prompt moving was probably instrumental in averting complete failure.

Amax personnel recalled several areas along the proposed route which they felt were underlain by weak soils, or which had experienced problems during deadhead moves of smaller equipment. However, this information was often conflicting between different persons memories and could only serve to alert the consultants to areas which required careful examination.

The Study

Data Gathered

The routes for the shovel were investigated by making nearly one hundred Standard Penetration Test borings varying in depth from about seventy-five to twenty feet. The majority of these borings were extended to the bedrock surface. Also, detailed topographic information and route cross sections were developed by Amax personnel. Laboratory testing was extensive, consisting of over thirty shear strength tests as well as density tests and tests for Atterberg limits.

During the study it became apparent that there were several distinct soil types which would affect the move. Along the northern one-half of the route the subsurface consists of silty sand spoil which is derived from mechanical alteration of sandstone during the mining process. This spoil varies in thickness from fifteen to over seventy feet, but generally is about fifty feet in thickness. Along the southern one-half of the alignment, the disturbed subsurface is clay/ shale spoil. This spoil is principally shale boulders and fragments in various forms of decomposition ranging from large unweathered boulders, to soft clay.

* "Deadhead is a term used by the mining industry to describe non-productive moving of a piece of equipment.

The third soil type which presented problems to the move is soft alluvial clay and silt. These soils were found both in an undisturbed condition where mining had not occurred and also as re-deposited spoil. Additionally, along a portion of the alignment, there was an undisturbed area consisting of stiff and very stiff residual clays overlying a relatively shallow bedrock surface.

Where the proposed alternate routes coincided with existing haul roads, a "crust" of well compacted soils was discovered. This compaction is attributable to the constant flow of heavy traffic, principally "Dart" coal haulers. (Darts are heavy rubber tired coal haulers weighing about 200 tons when loaded). This crust extended to a depth of about five feet in the haul roads underlain by silty sand, and to only a depth of about two and one-half feet for the haul roads underlain by clay/shale. Beneath the crust, the character of the subgrades did not differ materially from that encountered in the reclaimed areas.

Because of the nature of the load application, undrained soil shear strength was the principal factor to the analysis. Compressibility and settlement were secondary concerns because vertical movement posed minor operational problems during the move.

Shear strength determination for the cohesionless spoil was a straightforward analysis wherein correlations were made between varying densities of the sand and its internal friction angle as measured by "quick" triaxial shear tests. Additionally, several inspection pits were excavated at borehole locations to allow measurements of the in-situ densities of the sand for comparison with the standard penetration "N" values. Accordingly, a direct correlation of "N" values and internal friction angle was developed. These data are presented in Figures 3 and 4.

Shear strength determination in the clay/shale spoil was difficult because the "N" values of this material were often greatly inflated by the large shale fragments. Numerous unconfined compression tests were performed on undisturbed samples of this soil and the measured shear

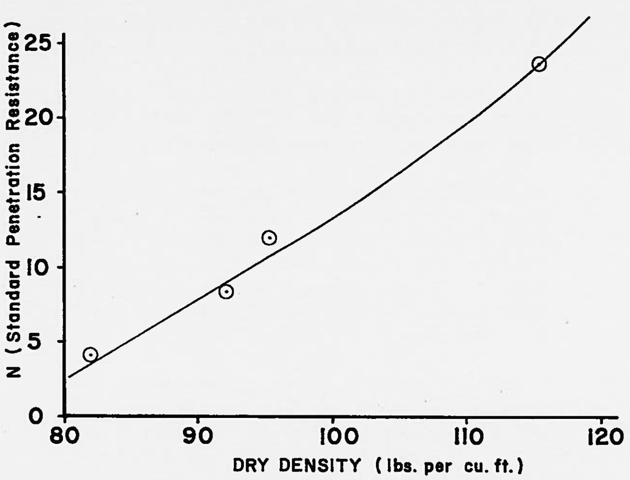


Figure 3. Dry Density vs "N" as measured by borings and subsequent in situ sand cone density tests.

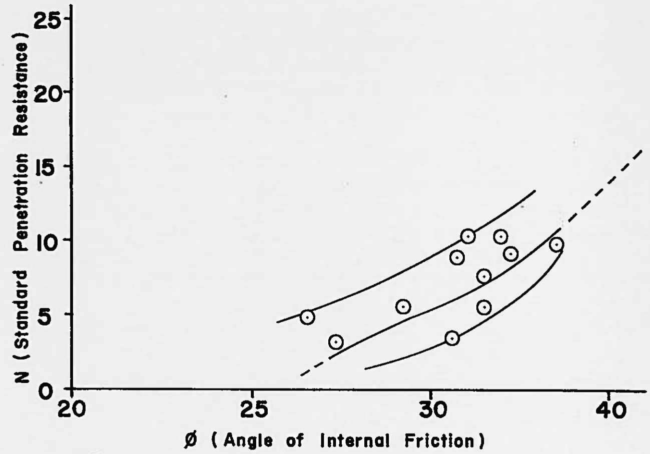


Figure 4. "N" vs ϕ obtained from triaxial quick tests using varying densities.

strength was plotted against the "N" values obtained from the borings. As can be seen in figure 5, this data was inconclusive. As a consequence, a great deal of judgment, based on close sample inspection, was necessary to permit selection of shear strength parameters for use in stability and bearing capacity analyses for the clay/shale.

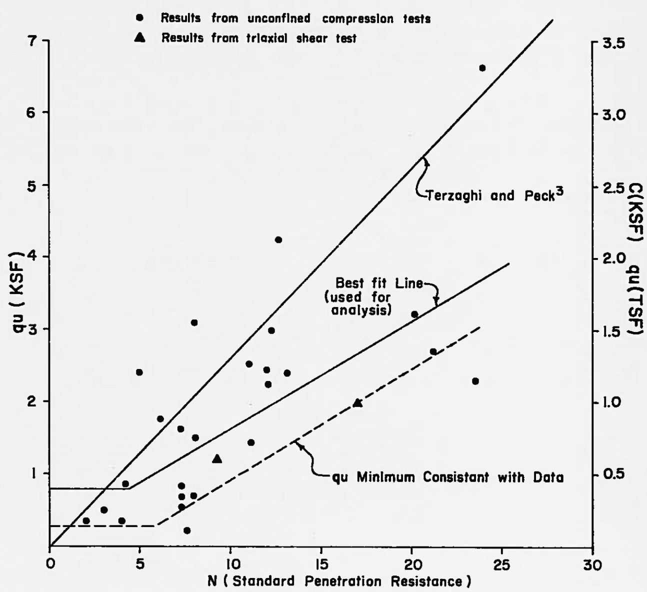


Figure 5. "N" vs Shear Strength for cohesive soil (clay/shale).

Methods of Analysis

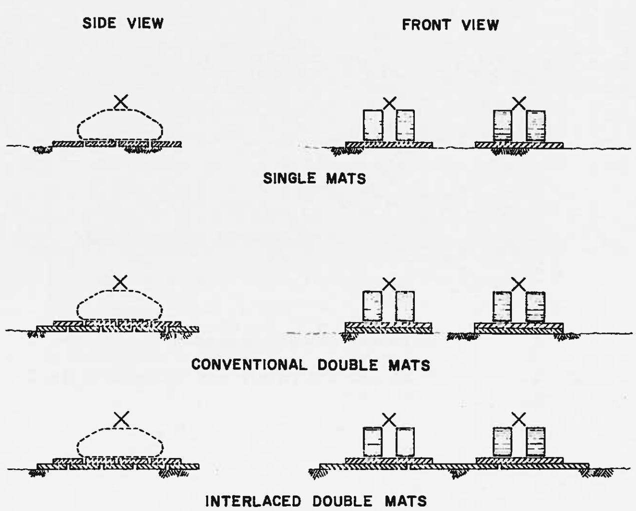
One of the most difficult decisions confronting the engineers during this study was selection of the failure mechanism for use in the analysis. Ultimately, the methods considered included conventional bearing capacity analysis, sliding wedge slope failures, and conventional circular arc failure analysis. After much deliberation, it was decided that the circular arc failure analysis is the most applicable based on both the writers' judgment and on descriptions of past failures. Figure 2 illustrates some typical potential failure surfaces. It was concluded that this analysis is applicable for flat ground, cut sections, embankments, and sloping road sections.

The principle shortcoming of the circular arc analyses is the fact that it is only two dimensional and there are no provisions included to consider the end of the affected section. Such an assumption is generally acceptable in slope stability analysis where the width of the slide is considered infinite and the principal driving force is the soil itself. However, in analyzing stability beneath the shovel the principle "driving force" is the weight of the shovel and this terminates abruptly at the edges of mats. The writers believe that the end effect is a significant consideration for this type of analysis. But, because there was an absence of state of the art data regarding such a failure mode, rational consideration of this aspect was not possible.

Matting Arrangements

Because of the extremely high unit load applied by the tracks, the shovel must travel on wooden mats except when it is on relatively sound bedrock. (Refer to figure 7). These mats are typically eight feet wide by twenty-four feet long by eighteen inches thick. They are made of six inch by twelve inch oak timbers bolted together, and their cost is about $800 each. For the analysis, the mats were considered to be rigid and to have a nominal shear strength.

When the shovel is traveling on a single layer of mats the ground contact pressure is reduced to about 4800 PSF. Two variations of a double layer of matting were studied (Figure 6); these



TYPE OF ARRANGEMENT	BEARING PRESSURE	RELATIVE STABILITY* SHALLOW	DEEP
SINGLE MATS	4800	0.9	0.9
CONVENTIONAL DOUBLE MATS	3700	1.0-1.1	1.3
INTERLACED DOUBLE MATS	2400	1.3	1.3

* For Hypothetical Surface and Subsurface Conditions Constant for all Mat Types

Figure 6. Matting Arrangements and how they affect stability safety factors.

Figure 7. Wooden Mats.

arrangements lower the contact pressures to about 3700 PSF and 2400 PSF, respectively. For a uniform soil condition, the different matting arrangments were considered and the relative safety factor against failure was computed for each. The results of this analysis are shown by Figure 6. Close perusal of this data indicates that stability considering a shallow failure is primarily related to contact pressure while stability considering a deep failure is related more to total load.

Study Conclusions

Based on the results of the field and laboratory testing and the analysis, recommendations were developed and a risk analysis was made for presentation to the Amax personnel. The three mile route was divided into several sections and comprehensive, alternative recommendations were provided for each section. Generally, the geotechnical recommendations consisted of the following:

1. Across the areas where the route traversed, natural residual soil, the sand spoil, and competent clay/shale spoil, the shovel could proceed on single mats.

2. All embankments on which the shovel traveled must be at least one hundred twenty feet wide and the centerline should be staked so that the shovel traversed the center of the embankment.

3. Additional alternate routes were suggested which were underlain by more stable ground and required less remedial earth work.

4. Depending upon subsurface conditions, remedial earthwork was sometimes recommended. This remedial work ranged from complete removal and replacement of the soil profile to simply excavating and replacing soft ground in ditches and other drainage features.

At each critical section, the safety factors were computed for each matting arrangement and based on various degrees of remedial earthwork. A discussion of what the safety factors actually meant, considering the degree of accuracy and built-in conservatisms, was presented to the Amax engineers. It was recommended that risks associated with a safety factor less than 1.15 be considered as unacceptable. Acceptance of safety factors greater than 1.15 was a discretionary decision by Amax where the merits of decreased risk versus increased costs were evaluated by their staff using the data provided by the consultants.

Because the safety factors were significantly below those considered reasonable, for "normal" geotechnical engineering, practice, several other back up or secondary safety measures were recommended. One important aspect of previous failures was the fact that the shovels "bogged down" at the surface first and then the immobility allowed slow, deep failure. Therefore, it was recommended that one full shovel length of mats be left in place behind the shovel at all times to allow a rapid "back up" if necessary. Also, to minimize rapid load redistribution, the bucket was not used to move the mats except during extreme circumstances. Finally, a standby plan was advocated in case problems developed so that personnel involved in the move could take positive, immediate action, should subgrade failure appear eminent.

The Move

A few days prior to beginning the move a large dragline and a fleet of D-9 dozers were sent ahead to begin preparing the route for the shovel. Several meetings were held between the consultant, Amax's Engineering personnel, and the operations personnel actually responsible for the execution of the move. Additionally, the Amax engineers prepared a detailed map for the move foreman which was color coded in green, yellow, and red to delineate low, medium, and high risk areas.

During the preparation of the route, further exploration was performed by making test excavations with the dozers and draglines at areas delineated by the study as potentially unstable ground. Further decisions regarding remedial earthwork and matting arrangements were made based on 1) general conditions revealed by the excavations, 2) prior analysis, and 3) engineering judgment. Thus some critical decisions were necessarily made in the field concurrent with the move.

The move generally progressed according to plan and was successfully completed in eighteen days. In an area near the beginning of the move where the consultant predicted that double mats would be required, only single mats were used on the first attempt. A shallow bearing capacity failure occurred under one pair of the front tracks; however, the shovel was quickly backed up, the failed area was repaired, and the shovel then proceeded successfully, across the critical area on double mats. Thus, as a result of prior planning and quick execution, a potentially disastrous situation was averted.

Conclusion

Because of space limitations some aspects of the study could not be discussed in great detail but are mentioned here for reference. In the sand spoils, the stability analysis predicted that the mats would sink two to three feet before adequate overburden confinement was developed to achieve stability. This prediction proved accurate during the move. Additionally, the spoil was laced with numerous boulders which should "break-up" uniform shearing surfaces, and this aspect was considered when judging the adequacy of safely factors. However, no analytical numbers could be derived which reflected the magnitude of this increased shearing resistance. Moreover, both the consultants and Amax recognized the probability that exploratory borings might not reflect spoil conditions even ten feet away; therefore, the "back-up" procedures and the exploration by field excavation were certainly an integral part of risk reduction measures implemented during the move. Finally, the Amax engineers realized that the geotechnical study represented only a risk reduction technique and both economic and time constraints prevented the detailed exploration required to virtually eliminate potential soil stability problems.

Additionally, the study revealed two areas where the authors believe that further research might be appropriate. Initially, we encourage development of a straightforward, generally accepted, three dimensional stability analysis technique that will allow evaluation of strength derived by end effects. While the technology is currently available to perform this type of analysis using finite elements, both economic and time limitations will generally limit highly detailed, individual computer solutions. Secondly, research to develop usable analytic or empirical data regarding shale decomposition in embankments, and the effect of such degradation on shear strength will be useful in evaluating long-term stability of clay/shale spoils.

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Acknowledgments

The writers would like to thank Law Engineering Testing Company and Amax Coal Company for the opportunity to investigate and study this unique project. Special recognition is given to Mr. David L. Thomasson, who assisted in the work. The efforts of Mr. Steve Anderson and Mr. John Hageman, Geologic Associates, Inc., in assisting with the artwork for this presentation are greatly appreciated by the authors.

DISPOSAL OF COAL PROCESSING WASTES AT SITES OF LIMITED SIZE

by

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Abstract. Large quantities of disposable wastes are being produced in the processing (cleaning) of raw coal. Depletion of current disposal areas at many coal processing sites requires that additional capacity be found. Many times, difficulties arise in obtaining future disposal area adjacent to the processing plant because of physical site limitations (i.e., residential development, power lines, cemeteries, etc.). Two cases are presented where development adjacent to the disposal area required special techniques for the design and construction of additional disposal capacity. This paper presents the limiting criteria, results of field and laboratory examinations, conceptual design, and construction details of the facilities.

Introduction

Separate coarse and fine coal wastes are generally produced in the coal cleaning process. The coarse refuse materials (shales mined along with the coal) are trucked or conveyed to disposal areas. Fine coal refuse (residue from coal washing) is often slurried and pumped into an impoundment. The coarse refuse is also commonly used to construct or raise an embankment to impound the slurried fine coal wastes.

At Site I, in Marshall County, West Virginia, coarse refuse is conveyed and trucked to the head of an existing disposal hollow. Slurried fine coal refuse is pumped into an impoundment downstream of the coarse refuse disposal area. Coarse refuse disposal was available for several years at the site, however, fine coal refuse storage was nearly depleted. Development downstream of the embankment required that further storage be created by raising the embankment using the upstream construction method (building the embankment out over the settled refuse fines).

Coarse coal refuse disposal area at Site II, in Harrison County, West Virginia, was also available for many years while storage for fine coal refuse was nearly depleted. A new valley was selected for fine coal refuse storage, however, downstream and upstream constraints limited the size of this structure. Maximum fine coal refuse storage was obtained by contour strip mining the Pittsburgh Coal seam outcropping in the valley and using the resulting spoil material to construct the embankment. Thus, the valley was enlarged economically and material was obtained in the process for construction of the embankment.

Site I (Marshall County, West Virginia)

Background

Coarse and fine coal refuse materials had been deposited at this site for several years. Slurried fine coal refuse was pumped upstream of a coarse refuse embankment and excess coarse coal refuse was deposited in an upstream disposal hollow. A refuse conveyor system was located near the toe of the facility and a fan for deep mine ventilation exited near the center of the embankment at one of abutments. Surface runoff from the site was conveyed away from the coarse refuse disposal area by a diversion ditch, while a 60-foot wide concrete spillway provided outlet for the impoundment. Accumulated fine coal refuse was close to the elevation of the spillway when plans for future disposal were initiated.

The upstream construction method was selected for future disposal because of the downstream constraints at the site and the need to maximize fine coal refuse storage. Upstream construction also results in the minimum disturbed downstream area. This item is extremely important in meeting effluent criteria for runoff from disturbed areas according to new federal regulations of the Office of Surface Mining. The downstream face of the embankment can be regraded and seeded immediately after construction in the upstream method, thus, permanently reclaiming the disturbed area. Several stages of the future embankment were designed with slurry accumulating in the impoundment of a stage before the next stage was constructed. In this manner, more fine coal refuse could be disposed in the several smaller stages than in one large stage as shown in Figure 1.

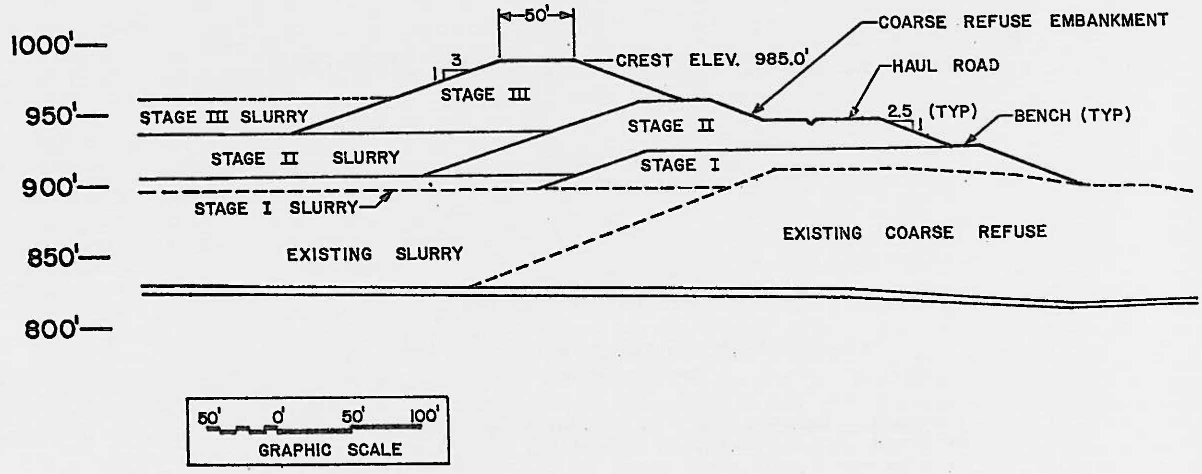

Figure 1. Cross-section of final embankment (Site 1).

Problems were anticipated due to the large drainage area at the site and the composition of the settled fine coal refuse adjacent to the embankment. Much of the capacity of future stages would be taken in storage and passage of storm runoff, thus reducing the amount of slurry which could be deposited. Also, the slurry discharge was at the rear of the impoundment causing the coarsest fine coal refuse to deposit at the discharge point with progressively finer wastes being deposited next to the embankment. This segregation would make dewatering of the slurry prior to construction almost impossible.

Design and Construction

The most important geotechnical engineering aspect of upstream construction is the dewatering ability of the fine coal refuse. If the slurry is not dewatered properly before the first lift of embankment material (coarse refuse) is placed, the slurry will laterally displace instead of consolidating. Results of consolidation tests and previous experience indicate that fine coal refuse consolidates quite rapidly.[1,3] If the material can be dewatered properly to eliminate lateral displacement, then the pressure of the first lift of embankment material will consolidate the fine coal refuse to a more stable deposit. Note that placement of the coarse coal refuse in relatively thin lifts over the fine coal refuse is essential for the proper dissipation of pore pressures in the loaded fine coal refuse.

The manner in which the slurried fine coal refuse deposits when discharged can often be utilized to aid in the dewatering process. The best location for the slurry discharge line prior to upstream construction is at the embankment because a delta deposit of fine coal refuse will occur at the slurry discharge point. The water elevation can be lowered at the rear of the impoundment by pumping and the slurry deposit adjacent to the embankment will dewater by gravity flow. The first lift of coarse refuse embankment material can then be placed as a construction pad over this more stable fine coal refuse deposit. At this project, however, the discharge point had always been at the rear of the impoundment, causing a delta deposit at the discharge point and a "pond" near the embankment (see Figure 2).

Figure 2. Slurry impoundment prior to current construction. Slurry line discharge is at rear of impoundment with pond next to embankment (Site 1).

A method of dewatering was developed for the initial stage in which a small dike was constructed across an upstream hollow and the slurry line was rerouted to discharge into this created impoundment. After the first stage of construction, the slurry line would be relocated next to the embankment so that future stages of upstream construction could be built on dewatered slurry as previously described.

Index testing and visual observations were used to classify the materials recovered during the field exploration into respective groups. Index properties of the foundation-construction materials are included in Table 1.

Table 1. Index Properties (Site 1).

Material	Grain Size Distribution >4.76 (mm)	4.76-0.074 (mm)	0.074-0.005 (mm)	<0.005 (mm)	Atterberg Limits w_L	w_P
Original clay	10%	26%	19%	45%	50%	28%
Coarse coal refuse	43%	47%	5%	5%	-NP-	
Fine coal refuse	0%	35%	42%	23%	-NP-	

Representative undisturbed samples of the coarse refuse, fine refuse, and original soil underlying the embankment were subjected to consolidated-undrained triaxial compression testing with pore pressure measurements to determine in-place effective strength parameters. Remolded coarse coal refuse was tested to predict effective strength parameters of the new embankment material. Results of these tests are given in Table 2.

Table 2. Effective Strength Parameters (Site 1).

Material	Total Unit Weight (pcf)	Effective Strength Parameters ϕ' (°)	c'(psi)
Undisturbed original soil	124.8	34.2	0.0
Undisturbed fine coal refuse	91.1	34.8	0.0
Undisturbed coarse refuse	109.4	34.7	0.0
Remolded coarse refuse	124.8	34.8	0.0

Stability analyses were performed on the critical sections of the proposed embankment stages. Effective strength analyses were considered applicable due to the granular nature of the waste materials and the gradual loading of the underlying foundation soils. Results of these analyses are given in Table 3.

Table 3. Stability Analyses (Site 1).

Section	Location of Minimum Circle	Minimum Factor of Safety Static	Earthquake
A-A	Total upstream slope	1.98	1.57
A-A	Individual downstream slope between benches	1.85	1.61
A-A	Total downstream slope	2.67	2.09

A computerized version of the Modified Bishop Method of Slices slope stability program was utilized for examining both static and dynamic conditions. The site is in a Zone 1 earthquake region (low risk) and an earthquake coefficient of 0.05 was utilized. A pseudostatic method of dynamic analysis was used with the earthquake force being taken as 0.05 times the weight of the slice applied horizontally to the centroid of each slice.

One item of major concern in the design of a slurry impoundment is the placement of the phreatic surface within the embankment for stability analyses. The coefficient of permeability of coarse refuse is usually between 10 and 100 times that of the fine refuse.[1] The fine refuse remains saturated even when covered over with later stages of an embankment, however, the water falls out almost immediately when coming in contact with the coarse refuse. Piezometer results of the original embankment conditions indicated that the phreatic surface was less than 10 feet from the bottom of the valley throughout the downstream slope.

The phreatic surfaces for future stages of the impoundment were calculated using Casagrande's method of construction for a homogeneous embankment. This was a conservative approximation, however, the method was selected because the permeability of the newly placed coarse refuse would be greater than that of the existing coarse refuse.[4] Thus, water seeping through the embankment would have a tendency to flow along the interface of the existing and the new coarse refuse materials. Note that a drainage blanket was installed at this interface to collect any seepage along the surface.

One other item of major concern in the design of the project was the passage of the design storm runoff. A dry dam constructed of coarse refuse was designed upstream of the impoundment to hold and slowly decant storm runoff. In this manner, the created slurry impoundment area could be used for storage of fine coal refuse instead of storm runoff. A concrete trough structure was designed to pass storm runoff from both the dry dam decant and the drainage area between the dry dam and the main embankment.

While the dry dam was being constructed, a small dike was constructed for the temporary storage of slurry during dewatering of the slurry impoundment. It was impossible to completely dewater the slurry because the impoundment kept filling with surface runoff. Construction of the embankment had to begin before dewatering was completed due to the depletion of slurry storage in the original impoundment and the temporary diked impoundment.

Much more slurry was laterally displaced than originally anticipated. It was necessary to modify the Stage I embankment during construction to reduce the amount of coarse refuse embankment built over the slurry. Once Stage I had been completed, the slurry discharge line was relocated to discharge near the upstream slope of the embankment. In this manner, a delta will form near the embankment and later stages can be constructed by lowering the water elevation at the rear of the impoundment to dewater the accumulated fine coal refuse adjacent to the embankment. Figure 3 shows the delta accumulating adjacent to the Stage I embankment.

Conclusions

Upstream construction was necessary at this site because of physical downstream constraints.

Figure 3. Slurry impoundment after Stage 1 upstream construction. Slurry line is relocated to discharge at embankment (Site 1).

The upstream construction method also had the added benefit of disturbing only a minimum area. New federal regulations regarding runoff from disturbed areas require more stringent design of refuse embankment facilities. Upstream construction enables all previous downstream areas to be regraded and seeded with only a small area of disturbance occurring.

The following conclusions are made based on results of design and construction at Site I in Marshall County, West Virginia.

1. Detailed analyses and planning are required in the design of any waste disposal facility.

2. Designs involving upstream construction should include positive methods of slurry dewatering before construction.

3. Designs should be flexible enough for changes to be made due to conditions discovered during construction.

Site II (Harrison County, West Virginia)

Background

Fine coal refuse produced at the Site II cleaning plant currently is being pumped into an impoundment. Coarse coal refuse storage capacity will be available for many years at an adjacent disposal hollow, however, storage capacity at the existing fine coal refuse disposal impoundment will soon be depleted. Area is not available at the existing disposal impoundment for raising the height of the embankment, thus new area is required. Since the plant is already oriented toward a facility where fine coal refuse (slurry) is pumped into an impoundment, the most economically feasible method of continued disposal is a fine coal refuse disposal impoundment.

The most accessible hollow was selected for the site of the future disposal impoundment. Development (dwellings) in this hollow and a cemetery near the head of the hollow limited the size and maximum height of slurry within the impoundment. The size of the impoundment could have been increased by constructing the embankment with borrow material from the upstream area, however, test borings indicated that sufficient soil borrow material was not available.

The alternative selected for obtaining the required storage capacity involved excavating soil and rock material from within the impoundment, thus artificially creating the additional storage capacity. The out-cropping of the Pittsburgh Coal Seam (6 feet thick) at the site was of prime importance in the selection of this concept. The final approach to constructing the refuse disposal system included contour strip mining the valley upstream of the embankment for coal and then using the overburden (spoil) material to construct the embankment (see Figures 5 and 6). This concept included a facility having the required fine coal refuse storage capacity plus the added benefit of paying for itself with the sale of coal removed during construction. Excess spoil material not required in the embankment construction was designated for placement in a valley fill just downstream of the proposed embankment.

Figure 5. Strip mining and embankment construction operation (Site II).

Figure 6. Borrow material being hauled to embankment. Note filter fabrick between earth and rockfill zones of embankment in background (Site II).

Design and Construction

Results of test borings performed in the area of the proposed embankment and in the upstream borrow area indicated that bedrock was relatively shallow at the site. Approximately 4 to 5 feet of clayey silt was underlain by alternating strata of sandstone, coal, and shale. Index properties of the foundation and construction soils at the site are included in Table 4.

Table 4. Index Porperties (Site II).

Material	Grain Size Distribution >4.76 (mm)	4.76–0.074 (mm)	0.074–0.005 (mm)	<0.005 (mm)	Atterberg Limits w_L	w_P
Brown and gray silt, some clay (foundation soil)	9%	25%	36%	30%	36%	28%
Brown silt, some clay (borrow material)	5%	32%	39%	24%	37%	25%

Laboratory tests were performed on undisturbed and remolded soil samples to determine original soil and embankment material strength parameters, respectively. Results of triaxial compression tests for each of the respective soil types encountered are included in Table 5.

Table 5. Strength Parameters (Site II).

Material	Wet Unit Weight (pcf)	Total Strength Parameters ϕ (°)	c (psi)	Effective Strength Parameters ϕ' (°)	c' (psi)
Brown and gray silt, some clay (undisturbed)	129.0	16.0	3.4	31.0	0.0
Brown silt, some clay (95% standard Proctor laboratory compacted)	125.0	21.6	3.5	30.0	0.0

The strength parameters for compacted rockfill were obtained from a literature survey with $\phi' = 36°$ and $c' = 0$ psi being used in the stability analyses.[5,6]

Since the quantity of suitable soil (4 to 5 feet) was limited at the site, the embankment was designed to be an earth and rockfill structure. The upstream third of the structure would consist of compacted cohesive material and the remaining embankment would consist of compacted rockfill (see Figure 4). Durable rock would be used in the rockfill portion of the embankment and the excess spoil would be placed in the designated valley fill. Stability analyses were performed on critical sections of the embankment. The factors of safety for the proposed slopes are given in Table 6.

Table 6. Stability Analyses (Site II).

Slope Section	Location of Minimum Circle	Minimum Factor of Safety: Short Term (Total Strength) Static	Short Term Earthquake	Long Term (Effective Strength) Static	Long Term Earthquake
A-A	Total upstream	1.54	1.34	2.41	1.96
A-A	Total downstream	1.61	1.43	1.80	1.53
B-B	Total upstream	1.53	1.34	2.93	1.84
B-B	Total downstream	1.62	1.43	1.62	1.43

A filter material was required between the cohesive soil and rockfill within the embankment to keep the cohesive material from piping through the rockfill as water seeped through the embankment. There was no natural filter material available at the site. Preliminary scheduling estimates indicated that construction would last nearly two years, thus presenting a problem for trucking in the filter material from off-site. It was decided to examine substituting commercial filter fabric for the natural graded filter material. In this manner, the filter fabric could be stored on site and placed as required.

The purpose of a graded filter is two-fold: 1) to allow free drainage from the material in question; and 2) to prevent piping of fines as the drainage is allowed. The filter material must have these characteristics in addition to not piping itself. Experience has shown that graded filters meeting standard grain size relations perform adequately.[2] A graded filter was designed for the soils at the site. A suitable graded filter was obtained and different filter fabrics were compared to the graded filter.

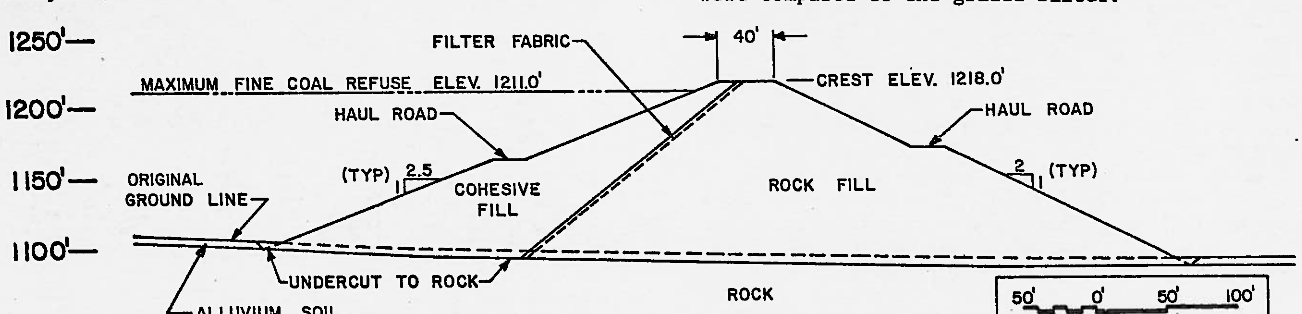

Figure 4. Cross-section of final embankment (Site II).

A solution of water and soil was allowed to drain through the granular graded filter. The flow and the amount of soil carried with the flowing water were measured. Three commercial filter fabrics were also subjected to this test. Included as Figure 7 are results of these tests. Filter A exceeded graded filter criteria, Filter B matched graded filter criteria, and Filter C did not meet graded filter criteria. Since the impoundment will be functioning for only thirty years before being drained and abandoned, one of the acceptable filter fabrics was selected for use at the site.

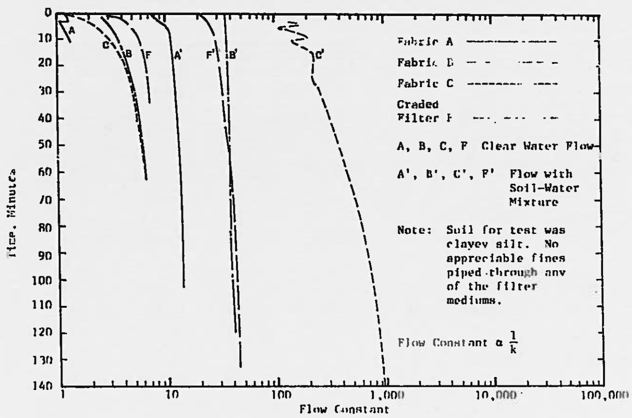

Figure 7. Flow tests for filter favrics vs graded filter.

A construction pattern was developed in which overburden material was excavated and placed within the embankment or valley fill. A cofferdam was initially constructed and an 8-inch drain pipe was installed through the embankment for drainage during construction. This facility provided for sediment control of the mining-construction area. With the new federal criteria requiring large capacity for sediment control, the system described enables mining to occur using the embankment to provide sedimentation capacity. Also the new federal criteria requires that highwalls formed during contour mining be eliminated. In this case, the highwall will be eliminated economically by the storage of fine coal refuse waste.

Conclusions

A strip mining-slurry impoundment construction operation not only provides income from the sale of the coal, but also increases the fine refuse disposal capacity and provides for borrow material to build the structure. The impoundment can provide sediment control during mining, thus providing an efficient means for meeting new federal effluent criteria. Geotechnical engineering expertise should be used in all mining related structures to meet state and federal mining regulations in an economical manner.

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SEEPAGE AND STABILITY ANALYSIS FOR AN

INUNDATED MILL TAILING IMPOUNDMENT

- A CASE STUDY -

by

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Abstract. An investigation was conducted to analyze the probable behavior of Tenmile, an existing 350 feet (100 m) high mill tailing impoundment, during construction of an adjacent tailing impoundment named Mayflower. A suggested method to control wave erosion on the downstream face of Tenmile is to hydraulically deposit coarse free draining cycloned tailing on the existing downstream face of Tenmile. This method was also suggested to control the phreatic surface and increase stability. Computer models were used to analyze the effects of infiltration, the encroaching water pool and slime deposits, and the proposed coarse cycloned tailing facing on the location of the phreatic surface and on the stability of the Tenmile tailing impoundment. The results indicate that the proposed cycloned facing will significantly increase stability and have little effect on the phreatic surface.

Introduction

Increasing volumes of mining waste material must be disposed of as the demand for minerals from low grade ore bodies increases. Environmental and economic considerations usually dictate above ground storage of these wastes behind embankments. For economic reasons, tailing embankments are generally hydraulically constructed from the coarse fraction of the waste material. Tailing is the waste products from mill operations where the crude ore is crushed and ground to free the valuable materials. Some tailing embankments rank among the highest and largest hydraulic-fill structures in the world and may impound millions of tons of material. Governmental regulatory agencies and the mining industry are employing Geotechnical Engineering methods for the economic and safe design of new tailing impoundments.

This paper discusses the use of Geotechnical Engineering principles to analyze the probable behavior of Tenmile, an existing mill tailing impoundment located near Climax, Colorado. The possible effects of the encroaching water pool and slimes on stability and on the location of the phreatic surface, with and without a coarse free draining cycloned tailing deposit on Tenmile's face, were analyzed and are presented on the following pages.

Climax Tailing Impoundments

The four tailing impoundments of the Climax Molybdenum Company are located above an elevation of 10,700 feet (3260 meters) and are on the western drainage of the Continental Divide near Climax, Colorado as shown in Figure 1. These impoundments

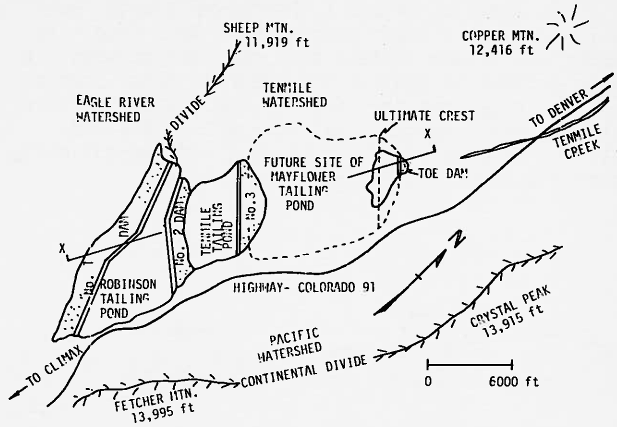

Figure 1. Location of Climax Molybdenum Company tailing impoundments.

are lower in elevation than the mill and are situated in the watersheds of the Tenmile Creek which flows to the north and the Eagle River which flows to the west.

Duggan (1939) reported that prior to 1936 the tailing at Climax was deposited in an impoundment located within one mile (1.6 km) of the mill. In the mid-thirties, increased production and further delineation of ore reserves forced the construction of a much larger impoundment, called Robinson, which now straddles the divide between the Eagle and Tenmile watersheds. The original tailing impoundment has been utilized as land on which to expand the milling complex.

Windolph (1961 and 1973) reported that in 1958 a third impoundment, named Tenmile, was constructed by the hydraulic upstream method further down the Tenmile watershed and below Robinson. A relatively impervious earthen starter dam 35 feet (10.7 m) high was constructed above generally strong glacial subsoil. The tailing deposited behind the starter dam is relatively coarse (5% retained on the No. 35 or 0.50 mm mesh) when compared to most mill tailing. The downstream slope of the starter dam and lower portions of the embankment are four horizontal to one vertical (4:1) and three horizontal to one vertical (3:1) for the middle and upper portions. During the winter months, the tailing was deposited at points about 200 feet (60 m) from the impoundment crest to minimize the development of ice lenses. By 1977 about 45,000 tons (41,000 metric tons) were being deposited daily and Tenmile covered much of the downstream face of the Robinson impoundment. Today, Tenmile impoundment is over 350 feet (100 m) high and contains approximately 175 million tons (160 million metric tons) of mill tailing.

In 1977 the fourth tailing impoundment, named Mayflower, was started 3 miles (4.8 kilometers) downstream of the Tenmile impoundment. The projected height is over 550 feet (170 m) and will contain 570 million tons (520 million metric tons) of mill tailing. Since the known ore reserves are expected to be depleted by the time this height is reached, Mayflower is planned to be the last impoundment constructed. Therefore, a significant engineering effort has been expended on Mayflower to control seepage, maximize stability and provide for future reclamation. Over 2 million dollars were spent on the design, planning and construction of the underdrains and the 70 foot (21 m) high starter dam before any tailing was placed.

As shown in Figure 2, Mayflower's water pool will eventually cover much of Tenmile's face with water and slimes. Since this may lead to possible wave erosion of Tenmile's downstream face, changes in stability, and the level of the seepage surface, it has been suggested that coarse free draining tailing be separated by cycloning and hydraulically placed on the existing face of Tenmile.

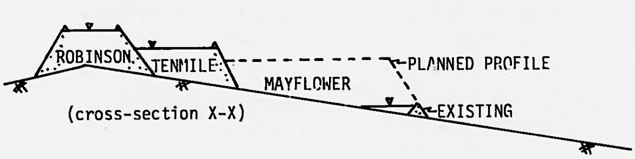

Figure 2. Simplified cross-section of Robinson, Tenmile and Mayflower tailing impoundments.

Engineering Properties of Mill Tailing

Many existing tailing impoundments have been investigated for both short term and long term behavior. However, the results of only a few investigations have been published. Kealy and Busch (1971) reported using permeability data and finite-element computer methods to predict the phreatic surface within tailing impoundments. Kealy, Busch, and McDonald (1974) investigated seepage in the slime zone and concluded from field observations that a zone of low permeability will develop at the bottom of the impoundment in the slime zone. Mittal and Morgenstern (1975) studied five coarse tailings and reported that shear strength is comparable to or greater than natural sands. Pettibone and Kealy (1971) investigated eight tailing impoundments and reported cohesion ranged from zero to 5 psi and the angle of internal friction ranged from 34° to 42°. Tests show that tailing from Tenmile have similar shear strength parameters.

Seepage Analysis of Tenmile

The simplified cross section of the Tenmile Tailing Impoundment used in the analysis is shown in Figure 3. The change in soil properties throughout the impoundment is represented by zones of different permeabilities. The phreatic line was located using a finite element computer program based on that presented by Kealy and Busch (1971) and also used by Nelson, Shepherd and Charlie (1977). Various cases are considered to demonstrate the effects of the Mayflower impoundment as it encroaches upon the downstream face of Tenmile and the possible effects of the cycloned facing. The permeabilities used for the seepage study are summarized in Table 1 and are discussed below.

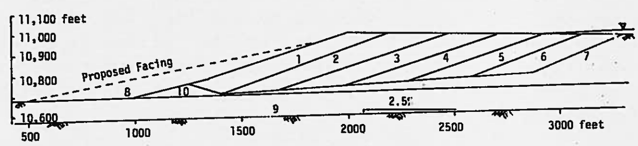

Figure 3. Simplified profile of the existing Tenmile tailing dam, Climax, Colorado.

TABLE 1. Permeabilities Used For Study.

Material (See Figure 3)	Number	Permeability Horizontal	Permeability Vertical
Tailing	1	2.6×10^{-3} cm/sec	8.8×10^{-4} cm/sec
"	2	1.9×10^{-3}	6.4×10^{-4}
"	3	1.4×10^{-3}	4.6×10^{-4}
"	4	9.5×10^{-4}	3.2×10^{-4}
"	5	7.4×10^{-4}	2.5×10^{-4}
"	6	5.3×10^{-4}	1.8×10^{-4}
Slimes	7	3.2×10^{-4}	1.1×10^{-4}
Cycloned Tailing	8	5.3×10^{-3}	5.3×10^{-3}
Foundation	9	1.1×10^{-5}	3.5×10^{-6}
Toe Dam	10	1.1×10^{-5}	3.5×10^{-6}
Slimes in Lower Impoundment	11	3.2×10^{-4}	1.1×10^{-4}

Tailing Material

Laboratory permeability tests on reconstituted Tenmile tailing gave permeabilities of 3×10^{-3} to 4×10^{-5} cm/sec, which are in the general range of permeabilities expected for fine sands and silts. The permeability decreases with increasing distance away from the crest as is to be expected.

Foundation Material

The foundation material is glacial and was assumed for this study to have a low horizontal permeability of 1×10^{-5} cm/sec. Kealy and Busch (1971) report that permeability of the foundation only has a significant effect on the location of the phreatic surface when the foundation material has a permeability larger than the tailing.

Stratification of Tailing

At Tenmile, as in most upstream dams, thin horizontal layers of fine tailing seem to be interspersed in the coarser tailing. This type of stratification is felt to have considerable influence on the vertical permeabilities but have little effect on the horizontal permeabilities. Kealy and Busch (1971) measured the ratio to be about 0.3 outside the slime zone for the Von Stone tailings impoundment. Nelson, Shepherd and Charlie (1977) expected that the vertical permeabilities would be about 0.4 of the horizontal permeabilities and Soderberg and Busch (1977) measured the ratio to be about 0.2 to 0.4 for tailings similar to Tenmile's. A ratio of 0.33 was assumed for this study.

Cycloned Coarse Tailing

Separation of tailing by hydraulic cyclones removes a portion of the fine particles. Tests indicate that the permeabilities of the proposed free draining layer to be deposited on the face of Tenmile range from 1×10^{-3} to 6×10^{-3} cm/sec. These values were found to decrease somewhat at higher overburden pressures. A value of 5.3×10^{-3} cm/sec was selected for this study.

Infiltration During Deposition

Infiltration of water from the surface occurs during deposition of the tailing over the beach areas. Computer modeling and field data have shown that infiltration may cause a significant rise in the phreatic surface. If infiltration raises the phreatic surface too high, wet spots may occur on the impoundment's face and instability of the impoundment may result. This phenomenon is the primary factor for not depositing tailing over any given area for long periods of time. Piezometers are used in the field to monitor the phreatic surface fluctuations to determine when to stop or start depositing tailing over any given area.

The quantity of infiltration for homogeneous soils was estimated from a variation of Darcy's law.

$$q = c\ k\ i\ A$$

q = quantity of vertical infiltration over Area A

A = area over which infiltration is occurring

k = vertical coefficient of permeability

i = hydraulic gradient and equals unity for infiltration

c = coefficient of reduction.

$c = 1$ for saturated flow

$c < 1$ for unsaturated flow

The quantity of infiltration was varied in the model to force the phreatic surface and outflow seepage to match typical field results.

Findings

Seepage

The effect of infiltration during deposition of tailing over the beach is shown in Figure 4. Case 1-a is the model prediction for the existing impoundment for no infiltration and Case 1-b-I is the prediction with infiltration. Infiltration caused a rise of 20 to 60 feet in the phreatic surface and the point where the phreatic surface intersects the face to move approximately 50 feet up the face. Therefore, if deposition is continued too long over a given section, "wet spots" are likely to develop in this zone. Also shown in Figure 4 are actual piezometer readings taken on August 11, 1977 at the locations shown. These and other readings fall in between the predictions of Cases 1-a and 1-b-I. The actual elevations depend on time of year and time since surface deposition. The results of the analysis are summarized in Table 2 and shown in Figures 4 to 8.

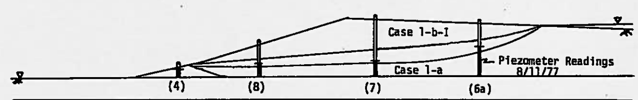

Figure 4. Actual piezometer readings and effect of infiltration.

Figure 5 shows Case D-0%-I which includes the effect of the proposed free draining cyclone tailing facing along with infiltration on the location of the phreatic surface. The location of the phreatic surface as compared to Case 1-b-I shows less than a 5 foot rise. Figures 6 to 8 show the effect of the cyclone facing, without infiltration, on the location of the phreatic surface for various degrees of inundation. As in Figure 5, the effect of the proposed free draining cyclone facing on the location of the phreatic surface is minor.

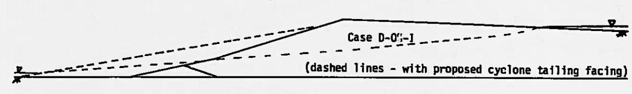

Figure 5. Effect of proposed cyclone tailing facing with infiltration -- lower pool elevation at 10,700 feet.

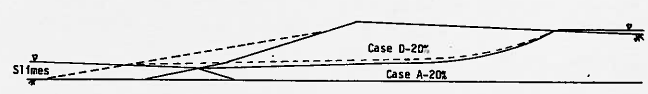

Figure 6. Effect of proposed cyclone tailing facing -- lower pool elevation at 10,774 feet.

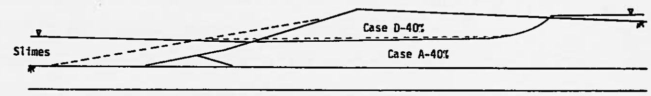

Figure 7. Effect of proposed cyclone tailing facing -- lower pool elevation at 10,842 feet.

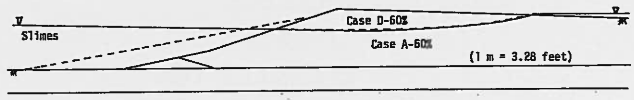

Figure 8. Effect of proposed cyclone tailing facing -- lower pool elevation at 10,911 feet.

Stability

A slope stability computer program, using circular failure surfaces, developed by Siegel (1975), was employed to investigate the slope stability of Cases 1-a, 1-b-I, D-20% and D-60% for the phreatic surfaces given in the previous section and Table 2. The relative values as compared to Case 1-b-I without the facing, for the factor of safety for stability, for the above cases are presented in Table 2. Table 2 also presents the relative safety factor for Case D-60%, with and without the facing, for the unlikely rapid drawdown of Mayflower's pond from 911 feet to 774 feet. For all the above cases, the factor of safety appears to be quite adequate.

Conclusions

1. The use of cycloned free draining tailing for a facing on the existing Tenmile tailing impoundment will increase stability and not significantly effect the level of the phreatic surface. In addition, the proposed facing will also reduce possible erosion caused by wave action.

2. Infiltration of water during deposition of tailing on the surface will cause a rise in the phreatic surface, increased seepage, and a reduction in stability. Piezometers are monitored at Tenmile to ensure that deposition of tailing is moved to another area before the phreatic surface rises above a given elevation.

3. Slope stability of Tenmile appears to be quite adequate for all conditions analyzed.

Acknowledgments

The writers gratefully acknowledge the cooperation and advice of Roy Soderberg and Dan Kealy of the Spokane Mining Research Center, of John Nelson and Thomas Shepherd of Colorado State University, and of John Welsh of Robertson-Pincock, Inc. The writers also wish to thank Climax Molybdenum Company for permission to publish this paper.

TABLE 2. Existing and Predicted Phreatic Surfaces within Tenmile.

Lower Water Pool Elevation[1]	Case[3]	Seepage[2] per foot (cfd.)	Phreatic Surface Elevation (All Elevations are plus 10,000 feet)				Slope Stability Percent of Existing Safety Factor	
			No. 6a	No. 7	No. 8	No. 4	w/o Facing	w/Facing
No Water Pool (Field Piezometer Readings)	8/11/77	-	898 (3a&1)	820	795	715	-	-
No Water Pool (Model of Existing Dam)	1-a	25	870	817	779	720	115%	150%
	1-b-I	85	937	873	801	720	100%	-
10,700 feet	D-0%-I	-	944	879	817	775	-	-
10,774 feet	A-20%	30	870	821	785	774		
	B-20%	30	871	822	788	774		
	C-20%	-	874	827	796	781		
	D-20%	30	874	827	796	781	-	165%
10,842 feet	A-40%	-	895	863	844	842		
	B-40%	25	896	863	842	842		
	C-40%	-	897	866	846	842		
	D-40%	25	897	866	846	842	-	-
10,911 feet	A-60%	20	937	919	911	911		
	B-60%	20	936	919	911	911		
	C-60%	-	938	920	911	911		
	D-60%	20	937	919	911	911	-	180%
Rapid Drawdown from 911 feet to 774 feet	D-60%-20%						90%	115%

Notes:
(1) Upper pool elevation is at 11,033.5 feet for all cases.
(2) Seepage per foot of dam width calculated at maximum section of dam (centerline).
(3) I. Vertical infiltration of water from beach area during spigoting (depositing tailing).
A. No cyclone facing - water w/o slimes adjacent to face.
B. No cyclone facing - slimes adjacent to face.
C. Cyclone facing deposited on face - water w/o slimes adjacent to facing.
D. Cyclone facing deposited on face - slimes adjacent to facing.

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CHEMICAL ADDITIVES TO CHANGE THE DURABILITY OF SHALES

by

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Abstract

Given the need to place non-durable shales in a fill, the popular approach is mechanical stabilization through degradation and compaction. The classification systems used to group shales into durable and non-durable categories are based on the slaking properties of the shales. These systems do not consider the hardness, degradability or the physico-chemical properties of the shales. Hence we encounter many combinations of hardness and durability. The hard and non-durable shales pose particular problems in embankments often developing settlement and even slope failures. These hard and non-durable shales are difficult to stabilize by mechanical means, such as increased compactive effort.

The intent of this study was to identify chemical additives that could either: (a) help break down the shales during placement or (b) reduce the deterioration caused by slaking during the service life of the compacted shale embankments. The slake durability test and the simple slaking test were used to evaluate the durability and the change in durability effected by various chemical additives. The chemicals used were various salt solutions at one level of concentration. The type of shales studied were: hard and non-durable, soft and non-durable and hard and durable; all from Indiana.

The paper reports the laboratory research and the conclusions reached relative to the potential usefulness of additives to modify the durability of shales.

Introduction

Shale is one of the most common rocks used in embankment construction. The engineering behavior of these shales varies depending on the factors of history and composition viz., geological formation and age, chemical composition, and service environment. Two extremely important properties of shales in embankments are (a) degradability in placement and compaction and (b) durability in the service environment. This paper concentrates on the problems posed by low durability.

Shales are classified as to durability by the Indiana State Highway Commission (ISHC) by the system proposed by Deo.[1] This system has four categories: Soil-like, Intermediate 1 and 2, and Rock-like. Shales which belong to the soil-like category are ordinarily placed in thin lifts and compacted as a "soil", while the rock-like ones may be placed in thick layers as rockfills. Some shales are mechanically hard and hence difficult to degrade, but also non-durable. These hard and non-durable shales pose maximum problems during the service life of a highway embankment.

This paper describes efforts to modify the durability of shales with chemical additives. The additives might either reduce slaking in the service environment or accelerate slaking during the construction process. In the former case, a material could be safely placed as a rockfill; in the latter, it could be more economically used in an ordinary soil fill.

Shales Studied

Three Indiana shales were selected for study, namely, New Providence, Mansfield, and New Albany. [These shales have similar slake durabilities as those of New Providence (Kentucky 6), Osgood (Kentucky 1), and New Albany (Kentucky 18), respectively, as reported by Shamburger, et. al.[2]] Slaking properties and point load strength data for these shales are given in Table 1. Point load strength (PLS) was used as a measure of hardness and slake durability and slaking index were used as indices of durability.

The New Providence shale belongs to the Borden group of the Mississippian epoch. The Mansfield shale belongs to the Mansfield formation of the Pennsylvanian-Pottsville series and was found above the Mississippian-Pennsylvanian contact. The New Albany shale forms a transition between Devonian and Mississippian rocks in Indiana, with some portions being a part of the Mississippian-Kinderhookian series.

Properties of the Shales Studied

The soil index values of the shales studied are given in Table 2. New Providence and Mansfield shales are classified as 'CL', whereas New Albany is classified as 'ML and OL' according to the Unified Soil Classification System. The clay minerals present in these shales were estimated by x-ray diffraction analysis, and are presented in Table 3. Results of chemical analyses of the

Table 1. Shale Properties.

SHALE	Slaking * Index (S.I.)	Slake Durability Index (Id)	Deo's Classification	Point Load Strength (PLS)	
				PSI	MN/m^2
1. New Providence	50.81	58	SOIL-LIKE	2347.1	16.18
2. Mansfield	40.78	66	SOIL-LIKE	1352.1	9.32
3. New Albany	0.14	99	ROCK-LIKE	3434.5	23.68

* Results from 5-Cycle Slaking Test.

Table 2. Soil Index Values.*

SHALE	Atterberg Limits			Unified Soil Classification
	w_L	w_P	I_P	
1. New Providence	33.6	22.6	11.0	CL
2. Mansfield	32.3	21.7	10.6	CL
3. New Albany	34	28.4	5.6	ML & OL

* From ISHC Laboratory test results.

Table 3. Percentage of Clay Minerals in Different Shales.

SHALE TYPE	Kaolinite %	Illite %	Chlorite %
1. New Providence	4.13 4.43	93.57 92.28	2.30 3.29
2. Mansfield	24.76 27.74	71.32 70.22	3.92 2.04
3. New Albany	1.22 9.83	98.78 90.17	- -

shales from the same formation as reported by Shaffer and Murray are compiled in Table 4.[3,4] The values from this table are approximations only, since they are merely for samples from the same formation as the samples of this study.

Chemical analyses for the shale samples studied are presented in Table 5. The cation exchange capacity (CEC), exchangeable sodium percentage (ESP), sodium absorption ratio (SAR), percentage sodium in the saturation extract, and exchangeable sodium ratio (ESR) for the three shales examined are given in Table 6.

Table 4. Chemical Analysis of the Shales from the Same Formation as that of Test Shales.

SHALE TYPE	S_iO_2	H_2O_3	Fe_2O_3	FeO	MgO	M_nO	CaO	Na_2O	K_2O	T_1O_2
New Providence ‡	60.2	16.3	1.98	4.23	1.98	0.06	1.78	1.11	3.78	0.87
	66.1	15.6	5.73	-	0.81	0.06	0.25	0.16	3.71	2.22
Mansfield ‡	55.1	19.6	0.11	6.65	1.83	0.18	0.55	0.17	3.31	0.06
	59.1	16.6	0.18	5.78	1.87	0.16	2.02	0.52	2.97	0.67
	53.9	21.7	2.65	4.30	1.49	0.11	0.26	0.10	3.20	0.81
New Albany *	56.7	12.5	4.04	-	2.23	0.098	2.42	0.53	3.44	0.67
	62.9	15.6	3.70	-	2.18	0.10	2.15	0.49	4.55	0.81
	57.5	16.5	4.29	-	2.48	0.11	3.49	0.37	3.9	0.65
	50.3	13.5	3.57	-	2.42	0.089	5.72	0.19	3.2	0.5

‡ Results from Shaffer.[3]

* Results from Murray.[4]

Table 5. Chemical Analyses.

Chemical Analysis of Shale Samples:

SHALE TYPE	Ca meq/100gm	Mg meq/100gm	K meq/100gm	Na meq/100gm
1. New Providence	5.94	4.2	1.24	3.02
2. Mansfield	4.24	5.0	1.18	0.59
3. New Albany	4.63	1.71	0.76	0.22

Chemical Analysis of Saturation Extract:

SHALE TYPE	Ca meq/liter	Mg meq/liter	K meq/liter	Na meq/liter
1. New Providence	13.0	9.17	2.73	13.96
2. Mansfield	5.1	6.33	1.77	2.22
3. New Albany	5.2	2.75	0.74	0.77

Chemical Analysis of Shale Samples:

SHALE TYPE	Organic Matter %	Soluble Salts μmho/cm	Loss On Ignition (LOI) %	pH
1. New Providence	1.10	3.2	4.87	5.58
2. Mansfield	1.59	1.39	7.11	5.18
3. New Albany	5.36	0.89	20.71	6.45

Table 6. Chemical Analyses Results.

SHALE TYPE	New Providence	Mansfield	New Albany
CEC meq/100gm	16.61	16.36	7.92
$ESP = \frac{Na}{CEC}\ \%$	18.2	3.6	2.8
$SAR^* = \frac{Na}{\sqrt{\frac{Ca + Mg}{2}}}$	4.19	0.93	0.39
$\%\ Sodium^* = \frac{Na}{(Ca + Mg + Na + K)}$	36	14	8
$ESR = \frac{Na}{CEC - Na}\ \%$	22.2	3.74	2.86

* The ratios are from saturation extract.

Slaking Mechanisms

Terzaghi (1967) attributed the slaking phenomenon to the compression of entrapped air in the pores as water enters these pores.[5] This behavior can be recognized in the case of soil aggregates and poorly cemented shales and mudstones. There have been cases where this mechanism did not satisfactorily explain the shale response.[6,7]

Ion absorption on clay particle surfaces has been suggested as the second mechanism causing slaking through swelling of illite, chlorite or montmorillonitic mineral content.[8] Dispersion of clay materials is caused by the swelling and disaggregation of clay minerals followed by erosion and removal of the dispersed material. The exchangeable cations of the clay are responsible for producing dispersion. Sherard, et. al. in the study of piping of earthern dams presented Figure 1, which correlates the expected erosion performance of clay in earth dams with the chemical environment.[9] Zones 1 and 2 in the figure would include nearly all the samples from dams which failed by breaching in Oklahoma and Mississippi, viz., highly erodible clays. Zone 3 includes erosion resistant clays. Zone 4 is also called the transition zone and contains clays which have low dispersion characters. The lower boundary of this zone was not well established by the data. The shales studied, namely New Providence, Mansfield and New Albany, plot in zone 3 and would be expected to be erosion resistant. Therefore, it appears that any breakdown of these shales would not be of dispersive clay origin.

Removal of the cementing agents in case of shales, siltstones, and mudstones by the solution action of the moving ground water is considered as the third mechanism causing slaking.[6,10]

The authors believe that no single mechanism can be considered as the dominant cause for slaking of Indiana shales. A combination of the above mentioned mechanisms by either one triggering the other or each occurring at the same time is likely. The composition of the shale and the environment in which the shale is placed determines the principal mechanism causing the failure.

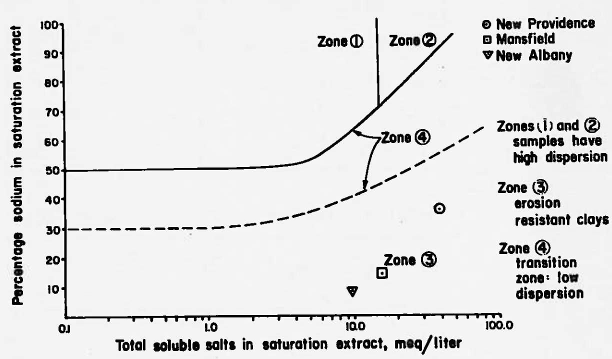

Figure 1. Test shales plotted on the summary of correlation between test results and dam experience (from Sherard et al., 1972).

Laboratory Testing

Slake Durability Test

This test was one of the two chosen to conveniently compare the effect of different slaking fluids on the durability of the shales. The slaking fluids were solutions of different salts. The slake durability apparatus (developed by Franklin) consists of a drum with a screen opening of 2 mm (No. 10) shown in Figure 2.[11] The drum is rotated in a bath of slaking fluid (0.1N salt solution) at a constant rate (20 r.p.m.), driven by an electric motor. The sample charge consisted of ten equidimensional pieces of shale, each weighing between 40 and 60 gm. These pieces were oven dried to constant weight at 110±5°C and cooled to room temperature before use. The drum was immersed in the bath containing the slaking fluid and was rotated for 200 revolutions. At the end of the test, the material retained in the drum was oven dried and weighed. The retained material was then returned to the drum and the 200 revolutions were repeated. The Slake Durability Index (I_d) was calculated at the end of the second cycle as,

$$I_d = \frac{\text{Oven dry wt. of material retained at the end of the second cycle}}{\text{Oven dry sample wt. before test}} \times 100$$

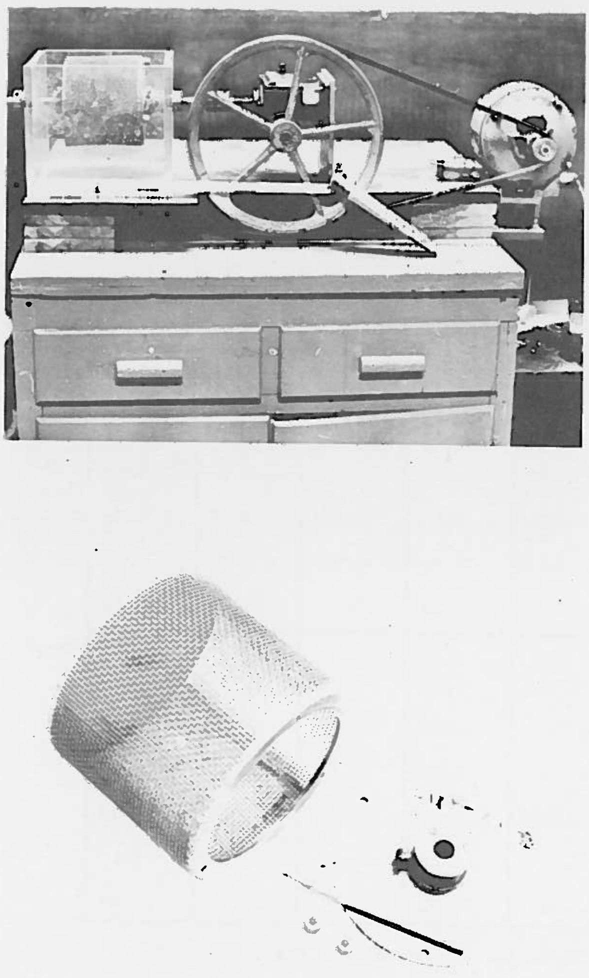

Figure 2. Slake Durability Apparatus

At least four repetitions were accomplished for each set of test variables, and average values are reported.

Slaking Index Test

Slaking of the shales was also studied in a simple (one-cycle) slaking test of discrete pieces of shale in a beaker containing the slaking fluid. In preparation for this test, two chunks of shale weighing approximately 150 gm each were oven dried to constant weight at 110±°C. Each chunk was then soaked in a 600 ml beaker containing the slaking fluid (0.1N salt solution), such that the shale was covered by at least 12.7 mm (0.5 in) of solution for 24 hours. The sample was then drained, washed over a No. 10 sieve, and the retained material oven dried to constant weight. The Slaking Index (S.I.) was calculated as,

$$\text{S.I.} = \frac{\text{Oven dry wt. of material lost at the end of test}}{\text{Oven dry wt. of the sample before test}} \times 100$$

The breakdown of the shales during the test was also visually observed and described.

Results and Discussion

The chemical additives used consisted of 0.1N solution of inorganic salts, some of which are commonly used in civil engineering practice as a stabilizing agent or as a catalyst with lime stabilization. Sodium chloride, calcium chloride, ferric chloride, calcium sulfate, aluminum sulfate, and ferrous sulfate were used. The results from the slake durability tests are presented in Table 7. Table 8 gives the slaking index from one-cycle slaking tests in the above slaking fluids.

Because of the inherent variations that exist within the results for each set of test variables for the slake durability and slaking tests, it was necessary to analyze the data with statistical hypothesis testing. The hypothesis consisted of comparing the mean value of the index obtained from testing for each variable against the mean value of the slake durability and slaking index of the shales in water. The 't' distribution was used to determine the variable which gave an identifiable difference in the indices (compared to that of water) at both 95% and 98% levels of confidence.

From the test for difference in the means of the slake durability index, using the t-distribution [$t_{0.02,5}=3.365$, $t_{0.02,3}=4.541$, $t_{0.02,1}=31.821$, $t_{0.05,5}=2.571$, $t_{0.05,3}=3.182$, $t_{0.05,1}=12.706$], it can be seen that sodium chloride and calcium chloride have no effect at the 98% level of confidence. Sodium chloride does have an effect at the 95% confidence level. Ferric chloride increases slaking of Mansfield shale but not of New Providence shale at a 98% level. Calcium sulfate and ferrous sulfate reduce slaking at a 98% level for both shales. The slaking of Mansfield shale with the addition of aluminum sulfate is different from that of New Providence shale. Aluminum sulfate produced improved durability as measured by both slake durability and one-cycle slaking tests at a 98% level of confidence for Mansfield shale, which is rich in kaolinite content compared to the New Providence shale. Sodium chloride and calcium sulfate increased the durability in both slake durability and one-cycle slaking tests for New Providence shale,

Table 7. Slake Durability Test Results.

Shale Type \ Slaking Fluid		WATER	0.1 N $NaCl$	0.1 N $CaCl_2$	0.1 N $FeCl_3$	0.1 N $CaSO_4$	0.1 N $Al_2(SO_4)_3$	0.1 N $FeSO_4$
NEW PROVIDENCE	MEAN	58	69	60.5	57.4	69.3	34.5	71.3
	Δ MEAN		+11	+2.5	-0.6	+11.3	-23.5	+13.3
	% Δ MEAN		19	4.3	-1	19.5	-40.5	22.9
	Std. Dev.	5.85	8.12	8.15	1.04	3.52	7.06	3.95
	t		3.32	0.751	-1.41	7.86	-8.15	8.27
	Coeff. of Var.	10.08	11.77	13.47	1.81	5.08	20.46	5.54
MANSFIELD	MEAN	66	64.8	66.2	56.1	77.2*	88.5*	78.6
	Δ MEAN		-1.2	+0.2	-9.9	+11.2	+16.5	+12.6
	% Δ MEAN		-1.8	0.3	-15	17	25	19.1
	Std. Dev.	2.55	0.79	0.66	0.58	3.52	1.22	2.96
	t		-3.40	0.75	-41.7	6.36	27.05	9.52
	Coeff. of Var.	3.86	1.22	1.00	1.03	4.56	1.48	3.77

NOTE: There was no measurable difference in the Slaking Durability Index for the New Albany Shale
+ve sign indicates improved Slaking Durability
All the indices are mean of six tests
*Mean of four tests

Table 8. One-Cycle Slaking Test Results.

Shale Type \ Slaking Fluid		WATER	0.1 N $NaCl$	0.1 N $CaCl_2$	0.1 N $FeCl_3$	0.1 N $CaSO_4$	0.1 N $Al_2(SO_4)_3$	0.1 N $FeSO_4$
NEW PROVIDENCE	MEAN	3.16*	0.84	0.47	1.06	0.79	1.52	0.65
	Δ MEAN		+2.32	+2.69	+2.10	+2.37	+1.64	+2.51
	% Δ MEAN		73.42	85.13	66.46	75	51.9	79.43
	Std. Dev.	1.48	0.19	0.064	0.11	0.099	0.92	0.36
	t		17.18	59.44	27.94	33.86	2.52	9.84
	Coeff. of Var.	46.84	22.74	13.62	10.03	12.53	60.47	55.49
MANSFIELD	MEAN	5.41*	3.32	5.75	5.72	2.64	1.24	2.20
	Δ MEAN		+2.09	-0.34	-0.31	+2.77	+4.17	+3.21
	% Δ MEAN		38.63	-6.28	-5.73	51.2	77.08	59.33
	Std. Dev.	1.23	0.4596	3.2881	5.332	2.375	0.24	0.48
	t		6.43	-0.15	-0.08	1.65	24.57	9.46
	Coeff. of Var.	23.97	13.84	57.18	93.22	89.96	19.35	21.82

NOTE: There are no measurable differences in the Slaking Index for the New Albany Shale
+ve sign indicates improved Slaking Index
All the indices are mean of two tests
*Mean of five tests

which has a high percentage of sodium in the saturation extract.

New Albany shale, which is a durable shale, was unaffected by different slaking fluids. This shale is well cemented and has an appreciable amount (approximately 5%) of organic matter.

The behavior of shales subjected to the slake durability test with different pore fluids considered is represented graphically in Figure 3.

The effect of chemicals on each shale needs to be studied more thoroughly before field use and some measure of alteration in long term slaking tendency is also needed.

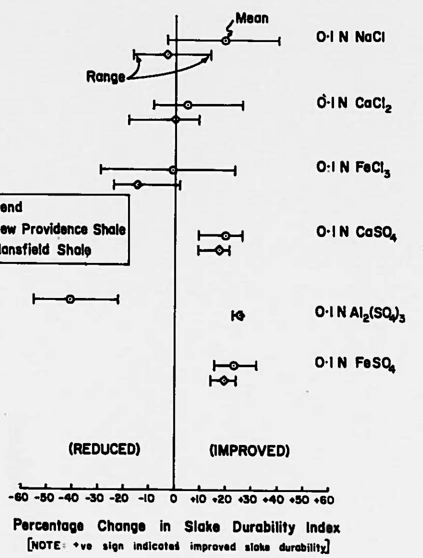

Figure 3. Percentage change in slake durability index of shale with different slaking fluids.

Summary

The slake durability of New Providence shale was improved when sodium chloride, calcium sulfate or ferrous sulfate was used as the slaking fluid. Calcium sulfate, aluminum sulfate and ferrous sulfate produced improved durability of Mansfield shale as measured by both the slake durability and one-cycle slaking tests. The slaking fluid used in these tests contained 0.1N salt solutions. Similar changes in durability of shales of similar origin and chemical composition can be expected.

These preliminary results are sufficiently positive to encourage further laboratory study of the chemical alteration of shale durability. Such chemicals could be incorporated with the shales during placement and water addition prior to the compaction process, as water is essential to the chemical reaction.

Alteration of durability in the long term has not been investigated and will require both laboratory testing and field verification. Other chemicals may be more effective for shales of differing compositions.

Acknowledgements

The work reported here is part of Joint Highway Research Project No. C-36-SL, entitled "Design and Construction Guidelines for Shale Embankments". Funds for the study were supplied by the Federal Highway Administration and Indiana State Highway Commission (ISHC). Personnel from the Division of Materials and Tests, ISHC were most helpful throughout the study. The advice of Prof. L. E. Wood of Purdue University is much appreciated.

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POINT LOAD STRENGTH TESTING OF COAL SPOIL

by

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ABSTRACT

The point load strength test is finding increased use in the rating of coals, shales, siltstones and similar soft rocks. It has the major advantages of simplicity and rapidity, especially since geometric shaping of the samples is not required. The usual dependent variable is the nominal failure stress, but reasonable estimates of the energy required to produce the tensile failure and the stress-strain modulus are also readily obtained. The major independent variables are material type, sample size, and water content.

The test has been used to compare a number of material capabilities, but the ones of greatest interest, with the soft rocks of mine wastes, are degradability and durability. The former is highly important to the density of the waste fill and its as compacted behavior, while the latter can control the long term stability. Degradability is usually measured directly by a laboratory compaction-degradation test, the results of which can be correlated with the point load strength data.

This paper reports on the development of a research experience with the point load strength at Purdue University, particularly in the prediction of the degradability of soft rocks as encountered in mine waste fills.

INTRODUCTION

The shales, siltstones, and other soft rocks associated with coal spoil present special engineering problems. Because the low durability of shales and siltstones plays a major role in the performance of refuse embankments, special compaction procedures are generally required which consist of construction controls on lift thickness and compactive effort. During the compaction process, densification is obtained through both rearrangement and breakdown of the individual pieces. As part of a study on shale degradation during compaction, the point load strength (PLS) test was considered as a possible index test for the prediction of shale degradability.

SPECIAL PROBLEMS WHEN TESTING SHALES

Abeyesekera[1] (1978) discussed the problems related to the use of shales as embankment construction materials. The physical properties of shales which cause these difficulties also create problems in the PLS test. Sample moisture content is one such problem. Previous work by Bailey[2] (1976) shown in Figure 1 indicated that over a small range of moisture contents, shale strength (as

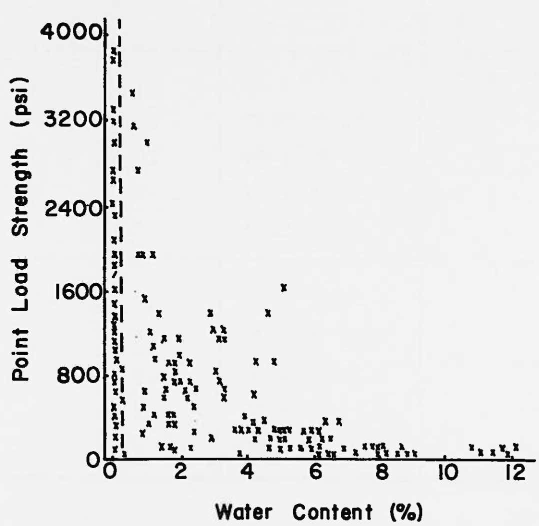

Figure 1. Variation in point load strength with water content for shales (after Bailey, 1976).

measured by the PLS test) increased as the moisture decreased from the natural value to oven dry. Further increase of moisture was impractical due to the slaking of shale samples upon the addition of water. Conversely, Bailey found that drying out of samples resulted in the development of weakening cracks. These problems favor the evaluation of the PLS index at the natural moisture content.

Shales exhibit stratification and bedding planes. The type of stratification and the thickness of the bedding planes affect both the fissility of the shale and the strength characteristics during the PLS test. Based on the sedimentary structure, shales may display spalling or even a form of successive failure as thin beds near the loading points failed individually. For such shales, the definition of failure load is arbitrary.

Samples for the PLS test require no special preparation. However, experience has shown that sample size has a major effect on the PLS index. The pieces selected for testing should be roughly equidimensional. Since the fissility of a shale tends to produce flaggy or platey, rather than bulky, pieces, obtaining samples of uniform size becomes difficult.

To evaluate the size effect in shales, approximately 150 samples were prepared from the New Providence and Attica shales of Indiana (similar to the New Providence and Tradwater shales of Kentucky) for PLS testing. The relevant properties of these shales are given in Table 1. More information on Indiana shales is given by van Zyl[5] (1977).

The samples varied in thickness from 0.19 inches to 1.50 inches. The data from the tests were grouped into size ranges of 0.25 inches and are presented in Table 2.

TABLE 1 PHYSICAL PROPERTIES OF NEW PROVIDENCE AND ATTICA SHALE

NEW PROVIDENCE		
GENERAL PHYSICAL DESCRIPTION		
COLOR-GRAY	HARDNESS-MEDIUM	FISSILITY-FLAKY
SHALE CLASSIFICATION		
DEGRADABILITY		SOIL-LIKE
SLAKING INDEX - 1 CYCLE		7.9%
5 CYCLE		46.3%
SLAKE DURABILITY INDEX		
DRY - 200 REVOLUTIONS		90.7%
500 REVOLUTIONS		82.9%
SOAKED - 200 REVOLUTIONS		66.9%
500 REVOLUTIONS		45.1%
NATURAL WET DENSITY		150.9 PCF
NATURAL MOISTURE CONTENT		8.3%
ATTICA		
GENERAL PHYSICAL DESCRIPTION		
COLOR-GRAY	HARDNESS-MEDIUM	FISSILITY-FLAGGY
SHALE CLASSIFICATION		
DEGRADABILITY		SOIL-LIKE
SLAKING INDEX - 1 CYCLE		0.9%
5 CYCLE		10.6%
SLAKE DURABILITY INDEX		
DRY - 200 REVOLUTIONS		95.6%
500 REVOLUTIONS		90.4%
SOAKED - 200 REVOLUTIONS		90.3%
500 REVOLUTIONS		67.6%
NATURAL WET DENSITY		154.2 PCF
NATURAL MOISTURE CONTENT		5.6%

TABLE 2 SIZE EFFECT ON POINT LOAD STRENGTH VALUES

SHALE	SIZE RANGE (IN.)	NUMBER OF SAMPLES	PLS VALUES (psi)		
			MEAN VALUE	STANDARD DEVIATION	COEFFICIENT OF VARIATION (%)
NEW PROVIDENCE	0.19 TO 0.50	25	2724	956	35
	0.51 TO 0.75	29	1277	515	40
	0.76 TO 1.00	14	604	256	42
	1.10 TO 1.25	5	439	185	42
	1.26 TO 1.50	7	262	79	30
ATTICA	0.26 TO 0.50	12	1190	368	31
	0.51 TO 0.75	22	687	305	44
	0.76 TO 1.00	19	409	127	31
	1.10 TO 1.25	9	283	56	20
	1.26 TO 1.50	5	207	59	29

As shown in Figure 2, the mean value of the PLS index from each size group decreased as the sample size increased. The variability measured by the coefficient of variation and shown in Figure 3 was generally lower for the larger sizes. These results

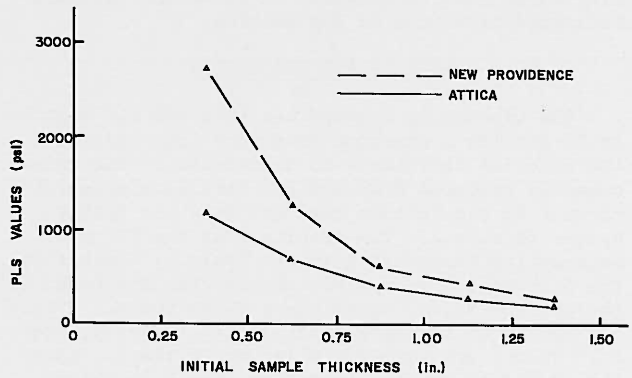

Figure 2. Size effect on point load strength values.

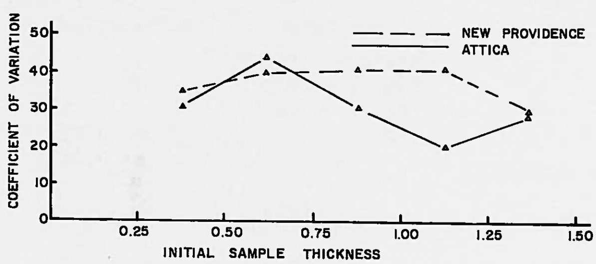

Figure 3. Size effect on variation in results.

demonstrate that both the PLS index and the variation among samples are affected by size. The results also show that the difference in the PLS values among the various size groups of one shale decreased as the sample size increased. These differences along with the observed variability indicate that the significance of the size effect would tend to decrease with larger samples.

One method of overcoming size effect is to select samples as large and uniform as possible. Brook[3] (1977) suggested still another method for avoiding size effect when he observed that the log - log plot of the failure load vs. the sample fracture gave a linear relationship. Brook's strength index was based on the PLS value for a standard area.

LABORATORY TESTING

POINT LOAD STRENGTH TESTS

The New Providence, Osgood and Palestine shales of Indiana (similar to the New Providence, Breathitt and Osgood shales of Kentucky) were used in a study on shale degradation. Relevant properties of these shales are given in Tables 3, 4, and 5. As part of the project, PLS tests were performed on samples from each shale.

TABLE 3 PHYSICAL PROPERTIES OF NEW PROVIDENCE SHALE

GENERAL PHYSICAL DESCRIPTION		
COLOR-GRAY	HARDNESS-MEDIUM	FISSILITY-FLAGGY
SHALE CLASSIFICATION		
DEGRADABILITY		SOIL-LIKE
SLAKING INDEX - 1 CYCLE		3.8%
5 CYCLE		63.6%
SLAKE DURABILITY INDEX		
DRY - 200 REVOLUTIONS		96.1%
500 REVOLUTIONS		79.8%
SOAKED - 200 REVOLUTIONS		72.8%
500 REVOLUTIONS		59.2%
NATURAL WET DENSITY		152.3 PCF
NATURAL MOISTURE CONTENT		4.4%

TABLE 4 PHYSICAL PROPERTIES OF OSGOOD SHALE

GENERAL PHYSICAL DESCRIPTION		
COLOR-GRAY	HARDNESS-HARD	FISSILITY-FLAGGY
SHALE CLASSIFICATION		
DEGRADABILITY		SOIL-LIKE
SLAKING INDEX - 1 CYCLE		4.5%
5 CYCLE		52.7%
SLAKE DURABILITY INDEX		
DRY - 200 REVOLUTIONS		89.6%
500 REVOLUTIONS		79.4%
SOAKED - 200 REVOLUTIONS		79.5%
500 REVOLUTIONS		62.3%
NATURAL WET DENSITY		164.3 PCF
NATURAL MOISTURE CONTENT		4.7%

TABLE 5 PHYSICAL PROPERTIES OF PALESTINE SHALE

GENERAL PHYSICAL DESCRIPTION		
COLOR-BROWN GRAY	HARDNESS-SOFT	FISSILITY-FLAKY
SHALE CLASSIFICATION		
DEGRADABILITY		SOIL-LIKE
SLAKING INDEX - 1 CYCLE		31.9%
5 CYCLE		68.5%
SLAKE DURABILITY INDEX		
DRY - 200 REVOLUTIONS		72.1%
500 REVOLUTIONS		30.6%
SOAKED - 200 REVOLUTIONS		27.0%
500 REVOLUTIONS		17.5%
NATURAL WET DENSITY		141.8 PCF
NATURAL MOISTURE CONTENT		7.6%

Since PLS tests on West Virginia siltstone conducted by the Waterways Experiment Station[4] (1978) showed considerable scatter in the log - log plot of failure load and fracture area, Brook's method of overcoming size effect was not used. Instead, the Indiana shale samples were prepared as large and uniform as possible.

The PLS test apparatus used in the study is shown in Figure 4. The entire apparatus was placed between the platens of a compression testing machine for loading. After placing a sample between the points, a small seating load was applied, and an initial reading of the apparatus dial gage was used to calculate the sample thickness. The samples were tested at their natural moisture contents and subjected to a loading rate of 0.01 inches per minute. Using readings from the load gage of the compression machine and from the dial gage of the PLS apparatus, both load and deformation were monitored at regular time intervals.

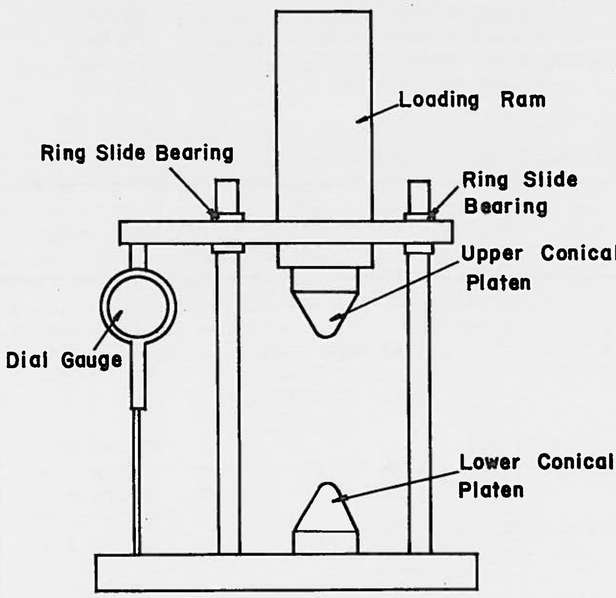

Figure 4. Side view of point load test apparatus (after Bailey, 1976).

All of the shale pieces tested eventually broke completely. However, to account for the successive failure displayed by some samples, the failure load was taken as the maximum load that had occurred any time during the loading process. The deformation corresponding to the maximum load value was either obtained directly or estimated from the recorded data.

DEGRADATION TESTS

The degradation tests for the shales used a method of impact compaction similar to the AASHTO compaction test. Material from each shale was blended to fit a gradation curve following the equation:

$$P = \left(\frac{d}{D}\right)^{1} \qquad (1)$$

where P is the percent passing any sieve, d is the sieve mesh size, and D is the top aggregate size. The sample gradations had a top size of 1.5 inches and were tested at their natural moisture contents. The material was placed in a 6 inch steel mold in three layers. Thirty blows of a 10 pound hammer having an 18 inch drop were applied to each layer for a total nominal compactive effort of 16,500 foot-pounds/cubic foot.

A sieve analysis was performed on the sample material after compaction. By knowing the initial and final gradations, the shale degradation due to compaction was evaluated.

The Index of Crushing (IC) discussed by Bailey[2] (1976) was used to quantitatively represent the amount of degradation due to compaction. Briefly stated, the IC is a comparison of the mean aggregate size in the initial and final gradations. The reduction in the mean aggregate size is expressed as a percentage of the mean size in the initial gradation. Therefore, a greater reduction in mean size would lead to a higher IC value and indicate increased breakdown or degradation.

RESULTS AND DISCUSSION

The hypothesis adopted was that the PLS test could provide a strength parameter indicative of the material resistance to degradation. The value commonly produced from the PLS test is simple P/D^2 where P is the failure load and D is the initial sample thickness. The results from the PLS and degradation tests are given in Table 6. Note that the P/D^2 values do not correspond with the order of degradation values among these three shales. This may be due to a sample size effect. However, the P/D^2 values are rather similar while the IC values show a larger range. Therefore, the P/D^2 values were judged to be insufficient.

TABLE 6 RESULTS OF POINT LOAD STRENGTH AND DEGRADATION TESTS

SHALE	PLS TESTS		DEGRADATION
	AVERAGE SAMPLE SIZE (IN)	INDEX P/D^2 (psi)	IC (%)
NEW PROVIDENCE	1.8	50.8	37.6
OSGOOD	2.0	44.7	25.1
PALESTINE	1.4	42.9	51.8

With the rejection of the P/D^2 values, another form of strength variable was necessary. Since both load and deformation were monitored during the PLS testing, the work required for failure could be calculated. The data showed the load-deformation relation to be essentially linear, and the work input was then calculated as:

$$WI = \frac{1}{2}\,(P \times Def.) \qquad (2)$$

where P is the failure load and Def. is the sample deformation at that load. The results of the calculations for work input are compared with the shale degradation values in Table 7. The similar values of work input for the New Providence and Palestine shales did not accurately reflect the difference in degradation properties of these materials.

TABLE 7 COMPARISON OF WORK INPUT AND SHALE DEGRADATION VALUES

SHALE	WORK INPUT (in.-lb.)	IC (%)
NEW PROVIDENCE	5.83	37.6
OSGOOD	3.15	25.1
PALESTINE	5.87	51.8

Materials with very different failure loads and deformation characteristics may ultimately include equivalent areas under their load-deformation curves. Observation of the load-deformation relationships for each shale indicated that the concept of stress-strain modulus could provide a suitable parameter. Using the P/D^2 value as a stress term and Def./D as a strain term, a general form of modulus was established as a material characteristic. A comparison of the modulus and degradation values is given in Table 8.

TABLE 8 COMPARISON OF MODULUS AND SHALE DEGRADATION VALUES

SHALE	MODULUS (psi)	IC (%)
NEW PROVIDENCE	1369	37.6
OSGOOD	2681	25.1
PALESTINE	490	51.8

The modulus term matches both the order and range of degradation values. A plot of the Index of Crushing vs. Modulus is shown in Figure 5. The best fit for the data was an exponential curve with the equation:

$$IC = 60.15\ e^{(-3.289 \times 10^{-4})(\text{modulus})} \qquad (3)$$

which gave an R^2 value of 0.998.

Although the curve is based on limited data, predictions of the IC from some limiting modulus values seem reasonable. The upper bound of the IC as the modulus approaches zero is appropriate for the set of conditions used in the degradation tests. The equation also gives reasonable lower levels of the IC for the higher modulus values possible in stronger shales.

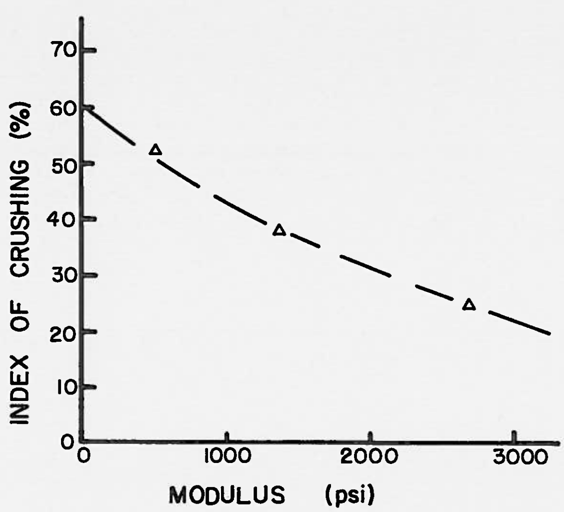

Figure 5. Comparison of modulus and index of crushing.

SUMMARY

The point load strength (PLS) test can be used as an indicator of aggregate strength for shales. The stress-strain modulus from this test provided the best correlation with degradation values and led to the formulation of a reasonable prediction equation for the Index of Crushing.

The PLS test is sufficiently simple and rapid to be performed conveniently in the field. Further experience in the use of this point load strength test will aid the prediction of degradation in soft rocks. Increased prediction capabilities could ultimately lead to the improvement of procedures and specifications for compacted coal mine refuse embankments.

ACKNOWLEDGEMENTS

The authors are pleased to acknowledge the financial support of the Indiana State Highway Commission (ISHC) and the U. S. Federal Highway Administration through the Joint Highway Research Project. The assistance of Mr. W. J. Sisiliano and Mr. Chris Andrews of the ISHC in obtaining the test shales is greatly appreciated. The valuable advice given by Prof. L. E. Wood of Purdue was significant in the performance and interpretation of the PLS tests.

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THE REVIEW AND REGULATION OF SLOPE STABILITY
A Technical Perspective

by

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Abstract. The design of waste embankment slopes is regulated by means of performance standards, rules of practice, and technical review. Mining regulations are administrated by a multiplicity of agencies functioning at different levels and operating with different perspectives and standards within local, state, and the Federal government.

The required performance of waste embankments is twofold: (1) they should not endanger life, property, or natural resources and (2) they should accommodate reclamation that provides for land use compatible with that of the surrounding lands. Mining waste embankments have a poor performance history. Many slope movements in the past have resulted from the failure to apply engineering practice rather than inadequacies of technology.

The major slope stability technical issues are the shear strength determination of embankment materials, the long term performance capabilities of end-dump embankments, and the achievement of a normalized perspective of safety. The major regulatory issue is the comparative values of flexibility versus standardization. Politically, the major issues are environmental protection and the values of competing resources.

This paper (1) classifies and examines slope failures common to non-water impounding waste embankments, (2) identifies design features which affect slope stability and design factors which provide assurances of safety, (3) discusses the use of TanØ-Cohesion shear strength charts for the review of slope design, and (4) identifies criteria used to make political, engineering, business, and administrative decisions.

Introduction

The design of stable non-water impounding waste embankments is not considered to be a difficult geotechnical problem. However, government regulation of these structures causes considerable disagreement among the mining industry, government regulatory agencies, and the engineering profession. This paper attempts to identify the major points of disagreement and explore the circumstances leading to the differences. A tolerance of different perspectives and less criticism will benefit all parties concerned. This paper does not make judgments or recommend solutions to problems.

Overview of Mining Regulation

Government regulation of mining has been in existence for over 100 years. Most of the lands being mined are subject to government control. Private mining rights have been acquired on government lands by claim, lease, or patent subject to continuing regulatory provisions. The Mineral Location Act of 1872[1] declared all valuable mineral deposits in lands belonging to the Federal government . . . to be free and open to exploration and purchase. This Act also identifies work assessment procedures needed to preserve the acquired mining rights.

Recent mining and related environmental laws address conservation and reclamation of the affected lands, protection of wildlife, prevention of stream pollution, etc., or, in short, environmental protection.

Regulatory enforcement provisions include revocation of mining rights, forfeiture of bonds, criminal or civil penalties, and injunctive relief.

The National Environmental Policy Act of 1969 (NEPA)[2] has significantly affected exploration and mining operations on Federal lands. NEPA requires a detailed environmental analysis for all major or controversial mining operation on Federal lands and provides for public review and response to the proposed actions. The analysis should identify slope stability hazards, if any, associated with the mining operations. The environmental analysis may be in the form of either an environmental assessment report or an environmental impact statement. Judicially, the environmental analysis has been interpreted to be a procedural requirement rather than a substantive restriction on administrative actions.[3] This means that decisions regarding mining activities do not, as a matter of law, have to follow the optimum course of action set out in the environmental analysis documents.

The Clean Water Act Amendments of 1977[4] is an example of Federal legislation which indirectly controls slope design by authorizing environmental standards which may in effect prohibit landslides.

The Surface Mining Control and Reclamation Act of 1977[5] and the Mine Safety and Health Act Amendments of 1977[6] directly control slope design by authorizing guides or rules of practice for embankment construction.

The Mineral Lands Leasing Act of 1920[7] authorizes government agency review and approval of design of waste embankments to be located on leased lands. Waste embankments frequently are placed on Federal lands located outside of the lease area. For example, the embankment design would be subject to review and permit application to the U. S. Forest Service, if the underlying land was being administered by that agency.[8] The jurisdiction of the regulatory agency may vary not only with the location of the mining activity but also with the nature of mining, such as coal.

Mining regulation involves a multiplicity of agencies functioning at different levels and operating with different perspectives and standards within local, state, and Federal government. Agency perspectives vary from single purpose such as environmental or health safety protection to multiple land use or maximum utilization of resources.

The need for some government regulation is generally recognized and accepted by the mining industry. However, the details and specifics of the regulation are often disputed.[9] Industry generally advocates the need for flexibility in regulation, goal or end product oriented standards and limits on specific direction.

The apparent reasons for favoring flexibility in regulation are: national standards may be diluted to be generally applicable and in the end they may identify industry norms rather than maximum limits; standards or codes may freeze practice at the current state of the art and tend to discourage innovation in methods; or rules that are followed uncritically without question may as a result create safety hazards for the exceptions.

The apparent reasons for favoring standardization and rules are: that objective standards result in efficient administration; that detailed provisions provide a means for conveying current practices to the less experienced; that general provisions require personal discretion and may be enforced with selectivity and bias; and finally that all geographic areas would have parity at the market place. This last item would apply to coal mining, but not necessarily to phosphate or copper mining which have considerable foreign competition.

Waste Embankment Failures

Mining waste embankments have a poor performance history. In 1969 it was estimated that 20-35 percent of the mine waste embankments in Canada experienced slope stability problems.[10] In 1964, an analysis of aerial photos indicated that 12 percent of the eastern Kentucky acreage affected by surface mining had landslides.[11] The problem of stability of mine waste embankments was dramatically brought to public attention in 1966 at Aberfan, Wales, and in 1972 at Buffalo Creek, West Virginia. At Aberfan, a slide involving 140,000 yd^3 of colliery rubbish resulted in the loss of 144 lives.[12] At Buffalo Creek, a water impounding coal refuse facility failed, devastating several communities and causing more than 100 deaths.[13]

Slope failures common to non-water impounding waste embankments may be classified as slumps, shallow flow slides, or foundation slides.

Because of the granular nature of the waste material, slumping is frequently limited to shallow depths. Edge slumps in side cast embankments are due to the oversteepening of the upper portion of the dump slope.[14] This oversteepening can be caused by a temporary accumulation of fines or to an apparent cohesion associated with moisture. Deep seated slumps or fully developed rotational slides are not common. Their occurrence can generally be attributed to an excessive embankment height in cohesive materials or more frequently to a reduction in toe support caused by groundwater or excavation. Without groundwater, sliding rates are generally slow and ground disruption is limited to a distance from the slope of 1/3 to 1/2 the slope height. The slump failure surface generally does not extend below the embankment if the foundation soil's shear strength is frictional in nature and exceeds that of the embankment soils.

Shallow flow slides are initiated by rain or snow melt, Figure 1. The important factors which affect stability are infiltration duration and rate, soil density, soil moisture content, and slope ratio. Snow melt or rainfall intensities vary geographically and are difficult to predict.

Figure 1. A typical shallow Flow Slide.

Flow slides occur because of the shear failure of the soil or the collapse of the soil structure. In soil collapse, the support of the soil grains is transferred to the pore water which has no shear strength. The writer believes there is an identifiable relationship between collapse susceptibility, hydraulic gradient, and density.[15] Groundwater conditions which cause collapse may differ from those which decrease effective stress to the point that the shear stress exceeds shear strength. However, this distinction may be academic with regards to the design of stable slopes. Shallow flow slides disrupt reclamation activities and may impact locations at considerable distances away from the slope.

Foundation slides involve the shear failure of the soil at the contact with or below the embankment slope. Sliding of this type generally is associated with end-dump placement methods or fill placement on steep slopes. Slope movements initiated below the embankment frequently cause a slope wedge to translate out from the embankment and the foundation soils spread or are squeezed ahead of the advancing toe. As the embankment construction is continued, the foundation failure progresses in the direction of dumping. If the movement of the foundation soils is restricted horizontally, movement will tend to become vertical, as shown in Figure 2.

Figure 2. Foundation spreading.

End-dumping causes a rapid unconsolidated undrained loading of the foundation soils.[16] In the west, overburden waste may be end-dumped from heights up to 1000 feet above the toe of the slope.[17] Most soils, if saturated, will fail under this extreme load. To engineer slope safety, limitations are generally imposed on the embankment height or lift thickness. Mine embankments installed in layers rarely experience foundation spreading type failures.

Embankments placed on mountain slopes may translate downslope along the contact between the embankment and the foundation. The size of the direct impact area below the slide will vary with the slope of the natural ground; the steeper the slope, the greater the potential for damage. To avoid this type of failure, maximum foundation steepness is limited and minimum foundation preparation requirements are identified. The steepness of the natural ground determines both the potential for sliding and the consequences of sliding.

Simply stated, landslide potential can be reduced at a specific site if limitations are placed on the embankment height, the slope ratios, the foundation steepness, and the placement methods. Also, provisions for drainage need to be constructed to assure the validity of the groundwater assumptions used in the stability analysis.

Most of the mine waste embankments that have experienced landslides either have not been engineered and/or have not considered geotechnical design criteria. It is this lack of engineering, not poor engineering, that has contributed to a majority of past waste embankment failures.

Waste Embankment Safety

Waste embankments can fail because of inadequate engineering technology or human error. Inadequate technology is defined to be the lack of capability or opportunity to determine all the adverse factors affecting slope stability. Technology improves with time through practice and research efforts. For example, statistics indicate that the average annual rate of dam failure decreased by a factor of approximately four for the period 1940-1972 compared to that of 1900-1939.[18] The apparent reason for this decrease is improved technology.

The design safety factor allows for a margin of error between the parameters used in design and those that may actually exist in the field. The magnitude of the safety factor considers (1) the consequences of failure, (2) the reliability of the design assumptions with respect to the most unfavorable conditions, and (3) the possibility of construction variations from design. In engineering practice, the slope safety factor is applied to the shear strength and it is that number by which the shear strength must be divided to bring the potential sliding mass into a state of limiting equilibrium.

The magnitude of designed safety may also be related to the design assumptions such as shear strength, drainage, and earthquake loading. Some design guides suggest that the applied safety factor should be 10 to 20 percent higher if there is a recognizable risk of personal injury or 10 percent higher if peak shear strength is used in the stability analysis rather than the residual strength.[19] The design safety factors generally are slightly greater than 1.0 for the extreme adverse conditions such as the instantaneous drawdown of a submerged slope, malfunctions of a drainage system, or maximum earthquake acceleration. Extreme adverse conditions are rarely compounded for minimum safety factor determinations. For example, it is not common to assume that instantaneous drawdown and the maximum predictable earthquake acceleration will occur simultaneously. This compound condition is conceivable but not probable, nor is it historically recorded. There is a need to temper that which is conceivable with experience.

It is recognized among engineers that all constructed facilities, particularly dams and earth structures, involve some degree of risk of failure. Most recently, the determination of acceptable risk has perplexed engineers designing nuclear power plants and dams. The problem is created by the situation that generally neither the engineers nor the facility owners will be the prime casualties if failure occurs. Also, safety hazards may be imposed on individuals without their consent.

The definition of acceptable risk may vary according to the decision maker's responsibility for failure. Acceptable risk is most definable on a personal basis. Engineering decision makers tend to avoid the acceptable risk issue and seek optimization.

For non-water impounding earth embankments, the potential for slope failures appears to be primarily attributable to human error rather than inadequate technology. Examples of human error would include lack of engineering or neglect of design details during construction. Also, slides involving non-water impounding structures generally do not threaten life or, if so, the hazard area is highly localized.

Shear Strength of Waste Materials

The shear strength values used to design waste embankments are approximate. The important material properties which affect mine waste shear strength are (1) gradation, (2) maximum particle size, (3) density, (4) durability, and (5) moisture content. The testing of representative undisturbed samples for design is not possible. Samples are prepared in the laboratory to model the embankment material. Both the modeling problems and the material variability make the design shear strength values highly uncertain.

There are two basic alternative approaches to sample preparation. They may be prepared by either removing the larger size particles (scalping), or by proportional grading to a reduced maximum size. Research investigations on granular soils indicate there is a tendency for the effective angle of internal friction to decrease as the fines content (minus No. 200 sieve) increases and the gravel content and maximum particle size decreases.[20,21,22]

Scalping and proportional grading methods reduce the maximum particle size and gravel content and may significantly increase the fines content. The validity of proportional grading generally has been demonstrated on samples having a nominal fines content or for particle sizes greater than the No. 30 sieve.[23] Errors in shear strength determination resulting from both scalping and proportional grading appear to have safe consequences relative to slope stability. An additional check for safety would be to verify that densities or void ratios of the model gradation do not exceed that expected in the embankment.[24] From studies of mine waste embankments it has been concluded that reasonable design strength values may be obtained from tests on loose samples.[25] Loose samples generally consolidate to a final density of the same order as that of the more compact specimens tested at the same cell pressure.

With regard to durability, limited evidence suggests that a significant reduction in shear strength does not occur in coarse coal discard embankments.[26] Durability tests, like the slaking index, are not used to adjust design shear strengths but are used to control placement methods and lift thickness.[27]

The shear strength used in analyzing safety with respect to slumps may differ from that used for shallow flow slide evaluation. It is well established that the angle of internal friction decreases with increasing confining or normal pressure.[28] Thus, the shear strength available to resist shallow flow slides may be greater than that available to resist deep slumping in high embankments. Also, back pressure saturation of samples may lead to the use of a conservative shear strength when designing slopes to avoid shallow flow slides. Infiltrating water rarely results in 100% saturation.

The use of residual shear strength may result in a more normalized perspective of design safety. The sensitivity of residual strength to variations in gradation, particle size, and density is presently being investigated.[29]

End-Dump Construction

There is considerable uncertainty regarding the performance capabilities or long term stability of end-dumped embankments. These embankments are constructed by dumping material on slopes which assume the angle of repose for the materials.

The primary advantage of end-dump construction compared to layer placement is economics. In steep terrain, building from the bottom up in layers costs more than end-dumping from the mine level. Some advantages of end-dumping can also be identified with regard to slope stability. As a result of end-dumping, the materials at the base of the embankment are coarser due to gravity sorting and consequently more permeable. Layer placement results in a finer embankment material overall because of mechanical breakdown under the placement and compaction equipment. Finally, layer placement of fill may result in perched groundwater conditions in the upper portions of the embankment.[30]

Many reasons are apparent for favoring layer placement and prohibiting end-dumping. The reasons most repeated are (1) 10 to 20 percent more material can be located in a given area because of higher compaction and lower void space, (2) the safety factor of a slope at its angle of repose cannot be much greater than 1.0, (3) reclamation and shaping can proceed concurrently with construction, (4) there is less risk of shear strength deterioration with time, (5) better quality control of drainage features can be maintained, (6) settlement may be more predictable, and (7) for coal waste there is less risk of fire.

The primary stability concern associated with end-dump embankments results from the presumption of the occurrence of a loose, collapsible particle structure within the fill. If this occurs, deformation can cause localized areas of arching, thereby reducing normal pressures and shear strength at those locations. Also, increases in moisture content or earthquake shocks may initiate collapse. If the fill is saturated, massive failures may occur. Without groundwater, these slope failures are expected to be shallow and would have limited mobility.

Stability Charts

The stability analysis of waste embankments is well suited to chart use. Generally, the waste materials are assigned a single shear strength value. Also, the slopes are generally designed for complete drainage or are assumed to be saturated. Intermediate groundwater conditions cannot be reliably predicted.

The TanØ-Cohesion chart is particularly useful for the review of slope designs. This format has also been used in landslide studies and for preliminary geotechnical investigations of dams.[31, 32] Figure 3 is an example of this type of chart used to evaluate slope safety relative to a circular arc type failure. This chart was prepared from Spencer's Stability Charts.[33]

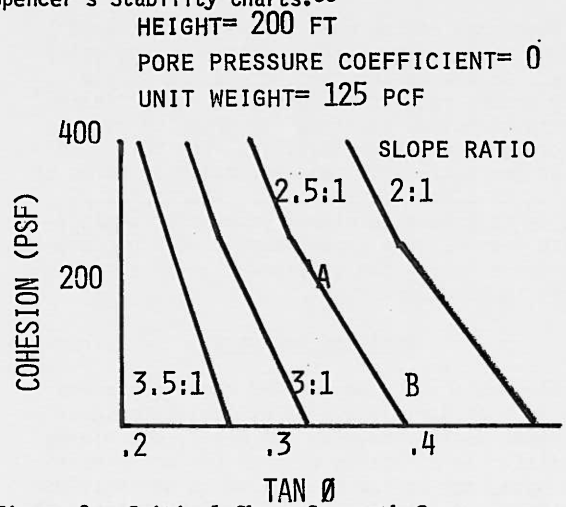

Figure 3. Critical Shear Strength Curves: Circular Arc Slide.

The critical shear strength for specific slopes are plotted as curves. The 2-1/2:1 slope has a safety factor of 1.0 at all points on the curve such as point A. The use of Figure 3 is limited to a specific pore pressure coefficient, embankment height, and unit weight, all of which are generally indicated on the chart. When the pore pressure coefficient is zero, this chart may be used for total stress analysis. This chart will also indicate if a given slope has a minimum safety factor. This is accomplished by dividing the shear strength by the desired safety factor and plotting the divided parameters, tan Ø and cohesion, as a point on the chart. If the point lies above a given curve, the safety factor is equal to or greater than the shear strength denominator. For example, if the shear strength parameters are tan Ø equals 0.6 and cohesion equals 90 psf, the shear strength coordinates (point B) would be 0.4 and 60 psf for a safety factor equal to 1.5. The 2-1/2:1 slope has a safety factor greater than 1.5 and the 2:1 slope has a safety factor less than 1.5.

The TanØ-Cohesion stability charts may also be used to evaluate the safety of slopes with respect to shallow flow slides. Figure 4 was developed by comparing the driving and resisting forces of an infinite slope in which the upper five feet is saturated.

Figure 5 is an example of a stability chart which can be used to evaluate the safety of a side hill fill with respect to base sliding.

Figure 6 is an example of a shear strength chart which can be used to evaluate the safety of slopes with respect to foundation failures caused by end-dumping. A safety factor can be applied to the undrained shear strength in similar fashion to that applied to the TanØ-Cohesion charts.

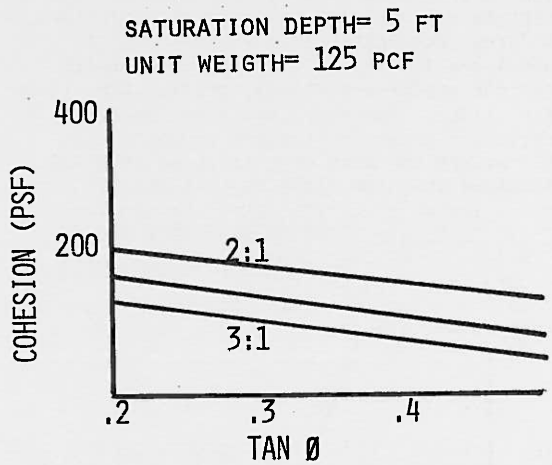

Figure 4. Critical Shear Strength Curves: Shallow Flow Slide.

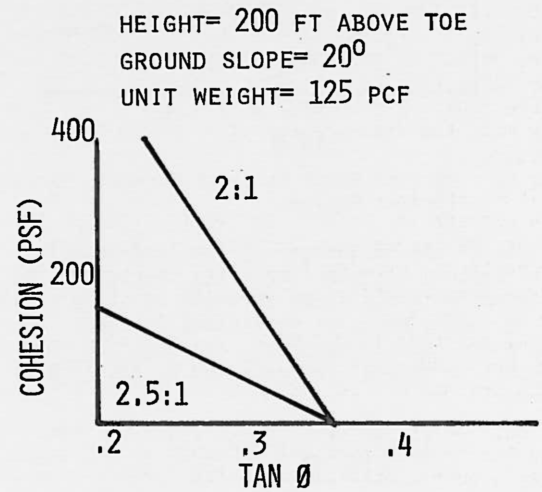

Figure 5. Critical Shear Strength Curves: Base Sliding.

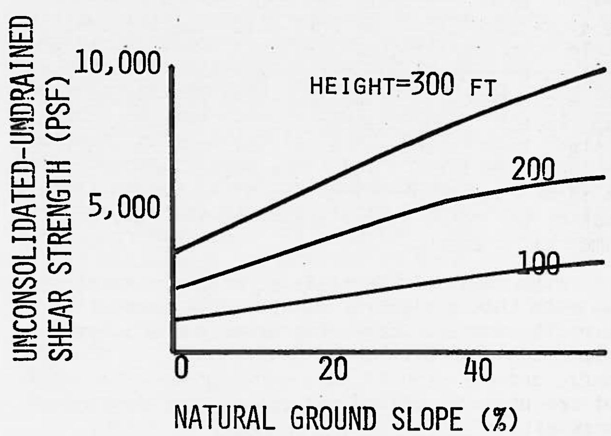

Figure 6. Critical Shear Strength Curves: Foundation Sliding Caused by End-Dumping.

The shear strength format may also be used to evaluate the most critical type of slope failure. Figure 7 compares the minimum shear strength required for stability of a 2-1/2:1 slope with respect to circular arc slides, shallow flow slides, and base slides. For the conditions identified in the previous figures, this chart suggests that base slides are the most critical type of slide for the shear strength range on this chart.

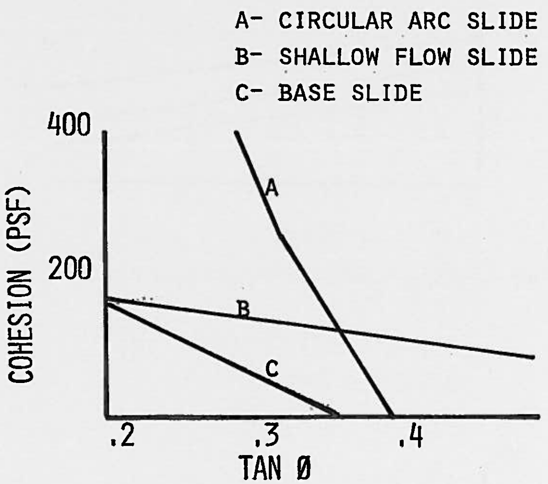

Figure 7. Critical Shear Strength Curves: Varying Types of Slides.

Some of the advantages of the TanØ-Cohesion format compared to other stability charts are that they directly relate shear strength to slope ratio, that the consequences of variations in shear strength are readily apparent, and that the shear strength from several projects can be easily considered.

Standards, Review, And Rules of Practice

The desired performance of waste embankments is twofold: (1) they should not endanger life, property, or natural resources and (2) they should accommodate reclamation that provides for land use compatible with that of the surrounding area. Waste embankments as structures have little utility in themselves. Their performance requirements are modest compared to highway embankments or embankments supporting structures. Engineering standards generally stipulate a purpose that must be accomplished rather than prescribing the means to accomplish the purpose. As a rule, these standards need to be based on performance criteria, design limit states, recommended detail and procedures, and provisions for small projects.[34] The primary purpose to be accomplished by engineering regulation is safety assurance. The means of accomplishment are the use of design safety factors, the control of design assumptions regarding adverse conditions, and peer review.

Engineering review and approval of operation plans is a highly flexible approach to safety regulation. With this approach, guidelines may be issued that identify investigation and design methods which will generally demonstrate embankment safety to the satisfaction of the reviewing agency. Compliance with the guides is not required and methods or solutions different from those set out in the guide would be acceptable provided their effectiveness can be substantiated and documented. A guide can be readily revised to accommodate comments and suggestions for improvements and to reflect new information or experience. These guides provide for efficiency in review and convey current practices to the less experienced.

Technical review may require an increased level of specialist staffing within a regulatory agency. If the review function is new to the agency or the regulatory agency is newly created, adequate technical staffing for detailed reviews may not be immediately feasible. For this condition or for uniformity reasons, codes or rules of practice have been implemented. When rules of practice have been developed because of administrative reasons, the establishment of a procedural mechanism allowing the consideration of variances appears to be needed.

Decision Analysis

Slope stability design and regulations are the result of decisions made by legislators, government administrators, engineers, and mining companies. This section reviews the criteria which have been or can be applied to making these decisions. Reportedly, several rules dominate all decisionmaking.[35]

The optimist rule directs decision makers to take the action with the largest possible gain. Mining companies apparently have followed this rule during construction of many of their waste embankments prior to 1970.

The reasonable rule is based on the premise that any reasonable man would do something because to do otherwise will appear stupid. This rule eliminates the do-nothing action. This rule may have affected the initiation or scope of some mining regulations.

The highly popular minimax rule, in contrast to the optimist rule, directs decision makers to select the action with the best of the worst outcomes. Use of this rule presumes that the comparatively bad occurrence will happen. Probabilities are not considered. The decision maker becomes more influenced by this rule as his responsibility or the magnitude of the consequences increase. As long as the economic expenditures are small, this rule is commonly considered to be the optimum. This rule may be used by consulting engineers who have concerns regarding professional liability and governmental employees lacking direction regarding acceptable risk. Compounding conservatisms, an example of overuse of this rule, may result in too great of a proportion of the nation's resources being expended on hazard reduction.

The administration rule directs decision makers to take a fixed specified action for each problem situation and not consider alternate actions or forecast future values. This rule simplifies decisionmaking and creates codes and rules of practice. It is a useful and popular rule in government.

The foggy rule encourages decision makers to make a decision based on a recognized incorrect assessment of values. The key part of the decision situation is a recognition of the true source of value and refusal to employ these values in favor of a more attractive or less controversial justification. In some instances, end dumping may be prohibited because of uncertainties regarding embankment performance rather than the apparent advantages resulting from layer placement, increased storage capacity and concurrent reclamation.

The committee rule applies to group decisions and conflicting viewpoints. The decision involves a series of hedges and the resulting action is neither the best nor the poorest. This rule could result in acceptance of a combination of end dumping and layer placement construction methods. Such a combination would compromise short term stability for reduced cost and reduce risks regarding long term stability.

The strategy rule results in a sequence of decisions. The receipt of information is needed to implement the subsequent action. The observational approach in geotechnical engineering is based on this rule. The abandonment of waste embankments may limit the usefulness of this rule in mining.

The final rule being discussed is the expected value rule. The use of this rule will result in the selection of the alternative with the largest expected value. An expected value is the product of a probability times the consequences of the action. This rule is almost universally applied in economic decisionmaking but has no known history of application for waste embankments. Probabilistic based decisionmaking is not expected to eliminate the polarization of government and industry with respect to regulation. The apparent source of benefit from this rule is the resulting analysis of values of competing resources. The rule requires that the consequences of the alternatives being considered are comparable. Since human injury generally is not a hazard associated with slope failure of non-water impounding waste embankments, the consequences of safety requirements and landslides may be compared in monetary terms.

Conclusions

1. Mining regulation involves a multiplicity of agencies functioning at different levels and operating with different perspectives and standards within local, state and the Federal government. Health-safety and environmental legislation both directly and indirectly control waste embankment design.

2. The detail and scope of regulation is the major point of disagreement between the mining industry and government administrators.

3. Slope failures common to non-water impounding waste embankments may be classified as slumps, shallow flow slides, or foundation slides.

4. For non-water impounding earth embankments, the potential for slope failures appears to be primarily attributable to human error rather than inadequate technology.

5. The primary purpose of engineering regulation is the avoidance of human error and public assurance of a minimum design safety. The means of accomplishment are the use of safety factors, the control of design assumptions regarding adverse conditions, and peer review.

6. The major technical slope stability issues are the shear strength determination of embankment materials, the long term performance capabilities of end-dump embankments, and the achievement of a normalized perspective of safety.

7. The Tan$\emptyset$-Cohesion stability chart is useful for the review of waste embankment slope design.

8. The decision criteria used by the mining operators, government administrators, and engineers is the same. The selection of decision rules will vary with the degree of uncertainty regarding the outcome and the responsibilities of the decision maker.

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THE NECESSITY FOR SCRUTINIZING GOVERNMENT MINING REGULATIONS

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Governmental regulations which relate to engineering design are often ill-advised and not consistent with good engineering practice, current up-to-date technology; or the extension of new technology. These regulations are often written in a bureaucratic vacuum apart from input from engineers from the private sector. Ostensively, these regulators review the current literature and other data related to the subject. Unfortunately, the writers of the regulations often assume that all information in the literature is correct or is the best method, simply because it has been published. Both of these conclusions are unwarranted and are quite often incorrect.

Another common error the regulators often make is the misapplication of principles found in the literature. A case in point (as will later be described) is the Office of Surface Mining's misapplication of technology related to dam construction and construction of fills to support structures, to valley fills; and the misapplication of technology related to the construction of super highways to haul roads.

Quite often the regulator does not understand the industry nor the procedure he is trying to regulate. The regulator is also often subject to political pressures from various areas which cause the inclusion of wrong regulations and the exclusion of right ones. All of these inadequacies lead to incorrect regulations which often are expensive and unwarranted. This is particularly true in the case of the Office of Surface Mining's regulations governing surface mining. Engineers must closely scrutinize all regulations governing engineering projects and bring pressures to bear to see that the regulations permit safe, innovative, creative, and economical engineering.

The regulations promulgated by OSM pursuant to the Surface Mining Reclamation Act of 1977 are a prescriptive or "cookbook" type regulation. That is, these regulations specify not only the end product to be achieved, but also each minute detail or step that must be taken to achieve that end product. This approach to the regulations is fundamentally wrong and extremely inflationary. The act itself speaks of performance specifications and not method regulations.

The purpose of engineering is not only to design a safe and environmentally sound structure, but also to design a structure at the most economical cost. The Office of Surface Mining has prevented (by regulation) this latter part of the engineers' responsibility. A balanced design is necessary and society must demand objective and cost-effective solutions to its problems. To remove, by regulation, the opportunity to make an economical design (as OSM has done) is unthinkable. If the engineer is not allowed to use creative and innovative designs to achieve a final product, he cannot use the best currently available technology and, equally absurd, he cannot improve on the basic techniques for mining coal. Our economy demands that we develop more economical methods for producing energy in the future. It cannot be the objective of environmental or health and safety regulations to freeze technological innovation and thus prevent the development of methods to more effectively meet the objectives of the legislation; yet these regulations do effectively freeze technological innovation by their prescriptive nature.

Government data attempts to place the cost increase of the regulations at somewhere between \$.50 and \$2.00 per ton of coal. We contend that the cost increase over and above what would be required to meet the intent of the act will vary between an average of \$3.00 and \$5.00 per ton of coal in gentle terrain and \$8.00 to \$12.00 in mountainous terrain. I would like to emphasize that these costs are not the total costs of meeting the regulations, but are the differences in cost between what good engineering could accomplish to meet the intent of the act and the costs which will be required by the extraneous regulations which accomplish nothing. In other words, a safe and environmentally sound design using the best currently available technology would meet the mandate of the act and cost from \$3.00 to \$12.00 per ton less than the costs incurred because of the regulations.

To promulgate regulations which require stringent and extreme measures and which completely ignore the technology of the engineering community (which is mandated to design these structures) is indefensible. Certainly this procedure is not in keeping with President Carter's policy of eliminating unneeded and inflationary regulations.

We, as engineers, have been active in the comment periods previous to the final promulgation of the regulations. We submitted testimony and written comment relative to the OSM proposed interim regulations on August 10, 1977. At various times after the promulgation of the interim regulations, we met with OSM officials to outline our objections to the "cookbook" nature of the regulations. On October 24, 1978, we presented verbal testimony at the hearings in Washington related to the final regulations and submitted on November 22, 1978, written comments to the regulations. All of these comments were documented and we deplored the fact that the OSM regulations are primarily prescriptive and not performance-oriented. Since that comment period OSM has made minor changes in the regulations; however, the Office of Surface Mining has chosen largely to ignore the comments from the engineering community and has made only minor changes in a few areas. The final regulations are still prescriptive (method) regulations. We, the engineering community, must therefore continue to oppose these regulations which will unnecessarily cost the American taxpayer billions of dollars in an already inflationary economy.

Some changes were made in the regulations as a result of the comments from the engineering community. Some specific suggestions for language change in the reulations made by the engineering community; and changes that followed are documented below.

1. <u>Valley Fills</u>

"The following is suggested language for a new section to OSM regulations for valley fills.

816.74; Professional Engineer Designed Fills: Head-of-Hollow and Valley Fills.

If an operator elects to construct a fill in a manner other than that set forth in Section 816.71, 816.72 and 816.73, a detailed engineering analysis shall be performed to assure the stability of the fill. This analysis must meet the requirements of Section 816.71(a) and (b) and the additional requirements of this section.

a. Fills in remote areas where there is no danger to loss of human life shall be designed to attain a long-term static factor of safety of 1.25. Fills in areas where there is danger to human life shall be designed for a long-term static factor of safety of 1.5. This factor of safety shall be based on data obtained from subsurface exploration, geotechnical testing, foundation design, and accepted engineering analyses.

b. This analysis shall address and provide for all of the following site parameters:

1. Geology.
2. Topography.
3. Test of overburden and foundation materials to ascertain insitu strength parameters and predict as-placed strength parameters.
4. Location of all springs and wet-weather seeps.
5. Adequacy of internal drainage commensurate with site specific conditions.
6. Adequacy of external drainage commensurate with site specific conditions.
7. Factor of safety based on strength parameters using an engineering acceptable method of slope stability analyses.
8. Design of diversion ditches, including design for both volume and velocity of diverted water.
9. Detailed plan of spoil placement.
10. Ultimate land-use considerations
11. Sediment control measures.
12. Measures for preventing erosion.

c. The engineer shall submit supportive data to show the following:

1. The long-term factor of safety of the fill meets the requirements of Section (a) above.
2. The internal-drainage system will meet the site-specific requirements of both soundness and percolation of all internal water.
3. The external drainage system will divert all surface waters in such a manner that the following is accomplished:
   i. The stability of the fill is maintained.
   ii. Excessive erosion is prevented
   iii. No ponding of water on the fill is permitted.
   iv. The diversion is designed for both the volumes and velocities of a 100-year, 24-hour precipitation event.
   v. Diversion design shall comply with the requirements of Section 816.43(f)."[1]

OSM responded by including in the regulations Section 816.74, Sound Rock Disposal. This section permits the dumping of rock in valley fills if at least 80% sound rock is encountered. Part of the documentation to OSM showed how economical designs could be made by dumping rock even if there were not 80% sound rock. They completely ignored these comments and allowed dumping only if 80% sound rock is encountered. This allows an engineered design for sound rock where sound rock is encountered, but still is a prescriptive method. The regulations should more properly specify that a factor of safety of 1.5 be achieved.

2. <u>Coal Waste and Dams</u>

"The following are the suggested language changes for processing waste, dams and embankments.

816.94; Professional Engineered Designed Fill: Processing Waste, Dams, and Embankments.

If an operator elects to construct a dam or embankment with coal processing waste in a manner other than that set forth in Section 816.81, 816.82, 816.83, 816.85, 816.92, and 816.93, a detailed engineering analysis shall be performed to insure the stability and hydrologic and hydraulic adequacy of the structure. This analysis must also meet the requirements of this section.

a. Embankments and dams shall be designed to attain a long-term static factor of safety commensurate with its site specific location. Embankments in remote areas where there is no danger to loss of human life shall be designed to attain a long-term static factor of safety of 1.25. Embankments in areas where there is danger to human life shall be designed for a long-term static factor of safety of 1.5. Dams shall be designed with an adequate factor of safety based on their location, size, hazard classification, and impounding capability. In all cases this factor of safety shall be based on data obtained from subsurface explorations through geotechnical testing, foundation design and accepted engineering analyses.

b. Dams shall be designed to safely pass or store runoff from a design flood commensurate with its size, impounding, capability, hazard classification, and period of use. In no case shall this storm be less than the 100-year frequency 6-hour duration storm or greater than the Probable Maximum Flood. A spillway or spillways shall be provided for the dam to pass that portion of the design storm that cannot be safely stored. Spillways and outlet works shall be designed to provide adequate protection against erosion, and inlets shall be protected against blockage. The design freeboard distance between the lowest point of the embankment crest and the maximum water elevation during passage of the design storm shall be at least 3 feet, unless analysis indicates that a lower freeboard is adequate.

c. The engineering analysis shall address and provide for all of the following site parameters:

1. Geology.
2. Topography.
3. Test of overburden and foundation materials to determine insitu strength parameters and predict as placed strength parameters.
4. Location of all springs and wet water seeps.
5. Adequacy of internal drainage commensurate with site specific conditions.
6. Adequacy of external drainage commensurate with site specific conditions.
7. Factor of safety based on strength parameters using an engineering acceptable method of stability analyses.
8. Design of diversion ditches, including design for both volume and velocity of diverted water.
9. Detailed plan of waste disposal.
10. Abandonment considerations.
11. Sediment control measures.
12. Erosion prevention measures.

d. The engineer shall submit supportive data to show the following:

1. The long-term factor of safety of the fill or dams meets the requirements of paragraph "a" of this section.
2. The internal drainage system will meet the site specific requirements for both percolation of internal water and soundness of material.
3. The external drainage system will divert all surface waters in such a manner that the following is accomplished.
   i. Stability of the fill is maintained.
   ii. Excessive erosion is prevented.
   iii. No ponding of water on the fill is permitted.
   iv. The diversion is designed to control volume and velocity from a design storm commensurate with its use. The minimum design storm shall be a 1-year frequency, 24-hour duration storm while the maximum storm will be a 100-

year, 6-hour storm.
v. Diversion shall comply with the requirements of Section 816.43(f)."[1]

OSM largely changed this section to permit the design in accordance with factors of safety and floods established by previous legislative activity.

3. Soundness of Underdrains

"Many states have a specification for slaking tests for shales. We submit that this specification would be more appropriate for a specification relating to the underdrain as required by Section 816.72 (b)(5) and 817.72 (b)(5)."[2]

OSM responded by permitting underdrains to be designed in accordance with a slake test suggested by the engineering community.

Other changes were made in other areas and the comment period was effective in effecting some changes, however, the overall nature of the "cookbook"" regulations is still prescriptive or "cookbook" and that should be rectified.

OSM in their justification of many of the "cookbook" provisions of their regulations cited a total of 51 references. A team of engineers, including engineers from the mining industry and from private consulting practice, reviewed all 51 of those references. In that review several things were found. It was found that the vast majority of the references stated by OSM did not refer to or support OSM's position at all. A summary of this review may be found in "Consol Comments on the OSM Regulatory Program for Surface Mining," dated December 15, 1978. One chapter of that submission to OSM details and outlines the specific review of each of the references outlined by OSM. Only five (5) references were found which seemed to be supportive of the OSM point of view. These references, however, had been misinterpreted by OSM. OSM had cited references based on limited data and in some cases based on soil only, and applied them to rock. In other instances OSM had not understood the author's main point. The authors of those various references were contacted and stated to OSM that the conclusions drawn on the basis of their work were incorrect.

The following errors were found to have been committed by OSM in their analysis of the various references upon which they based their conclusions.

1. The assumption that all information contained in the literature is correct and is the best way to achieve the end product. OSM failed to note that much of what is contained in the literature is the author's opinion based on his experiences. In many cases the author does not present his way as the best way, but he is merely reporting a documentation of various projects. For a regulatory agency to write "cookbook" regulations based on data in the literature without recognizing the limitations of this data is frightening. The action of OSM in this case represents a very dangerous precedent.

2. OSM referenced articles related to limited studies and even to different types of projects from those being governed by the regulations; and misapplied the information contained therein regulations. A specific example is some of the data which OSM had cited as being supportive of their regulation for compacting spoil in either 18-inch or 4-foot lifts. Much of the data "supporting" this point of view was based on triaxial tests on soil. OSM tried to apply this data to large diameter rocks. It is obvious that a triaxial test performed on recompacted samples of soil bears no resemblance to the strength of in-place large diameter angular rock. Yet, OSM tried to apply that data to the technology of valley fills composed of rock. OSM also erroneously tried to apply the procedures for construction of dams to valley fills. They apparently did not recognize that a valley fill is not a dam; it does not back water; and the strength requirements for valley fills are not as stringent as those for a dam or a compacted earthfill to support structures.

3. In many cases, the regulations were written on the assumption that principles extrapolated from one type of structure or installation should govern a different type of structure. In many cases in the OSM regulations the two are not compatible.

4. From the review of the references used by OSM it was obvious that the regulators in many cases had a total lack of knowledge of the structure they were attempting to regulate.

In other instances in the promulgation of the regulations OSM bowed to pressure in spite of information to the contrary posed by the engineers. A case in point is the inclusion of a chimney drain design for valley fills. In answering the engineers' comments to this procedure, OSM said the following.

"On the one hand, several professional engineers stated that long-term clogging of the rock core by fine-grained sediment in the drainage and in some cases piping (internal erosion) caused by the flow of water within the fill could lead to instability and potential failure of the fill (Loy and others, 1978, p. 106; Robins and others, 1977, pp. 1-4; Report of Committee on Interior and Insular Affairs H.R. 95-218, April 1977, pp. 121-123). One commenter said the rock-core method should be prohibited because rock drains should only be used for passage of seepage or groundwater flows, not surface flow."

"To date, the Office is not convinced that rock core fills are potentially less stable than the rock underdrain fills. Some engineers have expressed doubt that the rigorous West Virginia construction requirements could be adequately monitored in a State that was just beginning a strict inspection program and that inadequate engineering practices would be more likely to result in failure of the rock core system. The Office emphasize that it is critical that the rock core maintain its permeability throughout. If one impermeable section of the core is constructed or if a section subsequently becomes impermeable, failure could result."

"In summary, the rock-core method has been the subject of debate, but it reflects currently acceptable technology based upon the performance record of 250 fills (Green, 1978, p. 2). On the basis of the investigation, the Office is providing a permanent program revision to the regulations permitting the rock core system of head-of-hollow fills to be used at the discretion of the regulatory authority with adequate inspection and supervision. At the same time, the Office is instituting a formal study to investigate various types of fills."[3]

It is indeed odd that OSM in many of the other regulations states that they are protecting the public by including stringent regulations in spite of information from the engineering community proving that a method is safe. However, in the case of the valley fills with chimney drains, OSM states that the engineering community, although protesting that these fills may be unsafe, has not presented sufficient data to prove them unsafe. They, therefore, permit their inclusion in the regulation in spite of the fact the engineering community, in general, states that they may be unsafe. This is certainly a reversal of the OSM statement dealing with their mandate to protect the public by writing "cookbook" regulations. It is obvious that an inconsistency exists at this point.

All of these items were brought to the attention of OSM by the engineering community and others. Volumes of data were submitted to OSM to prove that the "cookbook" approach is fundamentally wrong. It is extremely inflationary, unwarranted, and in some cases may be unsafe. In spite of all of this documentation and warning by the engineering community, we are still saddled with "cookbook" or prescriptive regulations. The engineer is still hamstrung by the refusal of the OSM regulators to permit economical and innovative engineering. On numerous occasions, we the engineering community, have stated that the proper approach to the regulations would be to simply outline the desired end product and to leave the design of that end product up to the engineer. The act itself mandates the engineer as responsible for designing these structures. OSM mandates the engineer as being the responsible party and he must seal the work. Yet, the engineers' long and loud protests to the nature of the regulations have remained essentially unheeded.

The so-called public review period in which a governmental agency (in this case OSM) seeks public comment has, at least in this case, been largely unproductive. Very little change has been effected by the public review period. One must therefore assume that the channel for making comments through a public review period is not the proper way to cause our voice to be heard by the regulators. Certainly we should use these channels and we should make the comments to acquaint the regulators with the errors they may be making. This procedure, however, would appear to not be effective.

It is obvious that any time the bureaucracy begins to set itself up as the only expert or repository of knowledge relating to engineering, a dangerous situation exists. This type of attitude will lead not only to continued inflation and uneconomical designs, but also to the stifling of the creation of new technology. Engineering is best performed by professional engineers and not the bureaucracy. OSM has set a dangerous precedent in writing this "cookbook" regulation. This occurrence may lead other agencies to do the same thing. The question becomes (since we, the engineering community, were largely unsuccessful in effecting substantial change in this fundamentally wrong approach to the regulations during the public comment period) what recourse is there in dealing with the surface mining regulations? More importantly, what role can we play in the future in assuring that this sort of bureaucratic boondoggle does not again happen?

As previously stated, from past experience, it would appear that the public comment period is largely a formality and will not result in any substantial changes. I believe we as engineers, therefore, have the obligation to use more direct approaches. The following may be effective in relation to the current regulations.

1. Engineers involved in the design of mining projects should scrutinize the regulations. They should document the areas that are incorrect and petition our elected representatives at the federal level to change the nature of these regulations. Certainly, the engineers should mount a concerted drive to comment to our elected officials that the "cookbook" approach is fundamentally wrong; and that since OSM has chosen to largely ignore the comments of the engineers, it is incumbent upon the Congress to act.

2. Engineers working in the coal fields to design and supervise construction

of projects should document the implementation of these regulations by OSM. The "cookbook" regulations have placed a very powerful tool in the hands of OSM and in many cases that tool is already being used unwisely and unjustly. We as engineers should document those actions of OSM that are unnecessary and that lead to uneconomical procedures. These documented cases should be brought to the attention of not only the OSM authorities but also to our elected representatives.

In the case of new legislation related to engineering projects, engineers should educate themselves relative to impending legislation and make comment before the legislation becomes law. We should utilize our expertise and knowledge in the engineering field to educate the law makers and try to insure that the legislation itself mandates a method approach to engineering which permits creative and innovative engineering.

Above all we should not simply acquiesce and accept regulations assuming that the bureaucratic machine can not be changed. Perhaps it can not, but we owe it to the American public to make that effort. That effort should be a concerted effort over the entire engineering community and not just a few individuals. Unless we are effective in changing the current "cookbook" nature of the regulations on surface mining back into regulations that permit good, economical, creative, innovative engineering we can expect more of the "cookbook" regulations in the future and ultimately we, the engineers, will simply become technicians who enforce bureaucratic dictated "cookbooks" rather than creative designers.

References

1. Letter to Department of Interior, Office of Surface Mining, Reclamation and Enforcement, dated November 22, 1978 by David C. Cowherd, M.S., P.E., Chairman of American Consulting Engineers Council Committee on Mining.

2. Letter to Office of Surface Mining, U. S. Department of the Interior, dated December 11, 1978 by David C. Cowherd, M.S., P.E., Chairman of American Consulting Engineers Council Committee on Mining.

3. Federal Register, Volume 44-No. 50, dated March 13, 1979, Book 2: Pages 14901-15309.

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