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Highwall stability analysis under dragline loadings at a Saskatchewan coal mine

A.W.Clifton
Clifton Associates Ltd., Regina, Saskatchewan, Canada
D.G.Fredlund
Department of Civil Engineering, University of Saskatchewan, Saskatoon, Canada
O.Mickleborough
Saskatchewan Power Corporation, Regina, Canada

ABSTRACT: The design of a highwall in coal mining involves the consideration of many factors. The ongoing evaluation of a soft coal strip mine in southern Saskatchewan has provided an opportunity to combine both quantitative and qualitative factors in highwall design.

The mine is located near the southerly extent of continental glaciation in an area subjected to multiple advances of the ice front. One of the main factors affecting highwall design has been the structural disturbance of the sediments caused by glacial tectonics.

The original design was based on limit equilibrium analysis and shear strength parameters obtained from direct shear tests. Information obtained from full-scale test pits was also incorporated. The study was later up-dated using the results of highwall failure back-analysis. These studies were complemented with finite element stress analysis and further limit equilibrium analysis incorporating the effect of the dragline loadings.

The paper traces the history of stability analysis and modification of the recommended shear strength parameters from the pre-mining investigations through four years of operation. Particular emphasis is placed on the refinement of the shear strength parameters used for design. The importance of other factors (e.g., piezometric levels, climatic conditions, time effects, etc.) are briefly outlined.

INTRODUCTION

One of the primary questions which must be addressed in the opening of an open-pit mine in a new area is, "What are safe highwall slopes?" This was the case at the Poplar River Mine near Coronach, Saskatchewan. The mine was developed by Saskatchewan Power Corporation to supply coal to Poplar River Power Station near Coronach, Saskatchewan. On the basis of experience at Estevan and in North Dakota, it was initially perceived that highwall slopes of 45 to 60 degrees to the vertical could be used. A series of studies revealed that many factors must be taken into consideration in assessing highwall stability and the safety of the dragline. It was realized that these factors must also be evaluated on an ongoing basis. The studies at the Poplar River Mine have provided this type of information. This paper attempts to summarize the results of a number of studies and of ongoing surveillance over a period of four years. Emphasis will be placed on the role of analytical tools (finite element analysis and limit equilibrium analysis) in assisting the engineer to make important decisions.

HISTORICAL REVIEW OF STABILITY EVALUATION

A predevelopment evaluation of the mining area was conducted in 1974. The study involved stratigraphic drilling, soil sampling and the installation of piezometers to obtain information on the soil and groundwater conditions that would prevail in the mine area. The investigation revealed the following points:

i) the stratigraphy consisted of till overlying the Ravenscrag Formation bedrock,
ii) there were major differences in the consistency of the bedrock varying from softened to extremely hard,
iii) weak, softened and sheared zones were present in the overburden
iv) three major aquifers (the Ravenscrag sand, the Hart seam and the Empress Group sand and gravel) were present, and
v) the structure on the coal would control water levels in the coal and the water pressure in the overburden.

The study raised the possibility of potential highwall stability problems in the proposed mining area. A program of investigation was initiated to address the groundwater and stability problems.

Limited soil property information was available on the Ravenscrag Formation. Information on the Fort Union Formation in Montana suggested that highwall slope angles in the order of 45 to 60 degrees were feasible, depending upon groundwater conditions. Additional drilling was undertaken to establish the soil conditions in the mining area and to differentiate between the wet and dry coal areas. Small diameter test holes and large diameter test shafts were used in preliminary work to closely examine the subsurface conditions. Two full scale test pit locations were chosen on the basis of the testhole and test shaft information. Test Pit 1 was located in the dry coal area. Test Pit 2 was located in an area of saturated coal overlain with saturated bedrock sand.

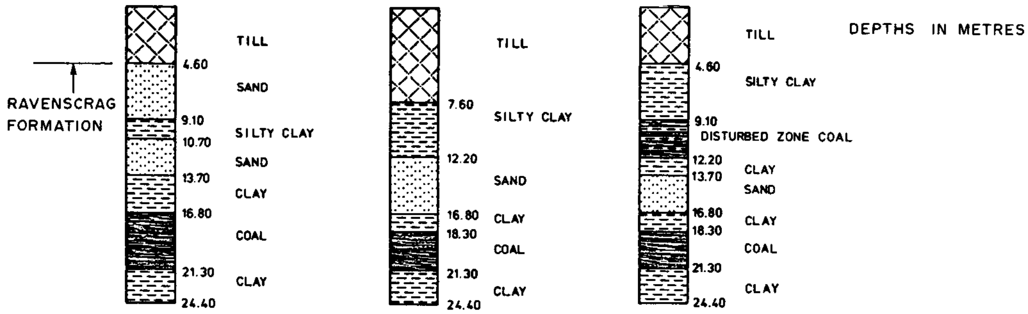


Figure 1. Typical stratigraphic columns at the mine site

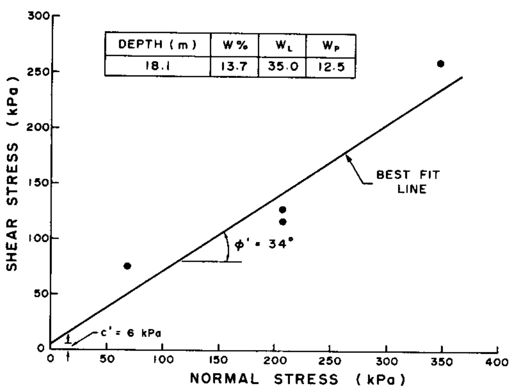


Figure 2. Direct shear test results on the till

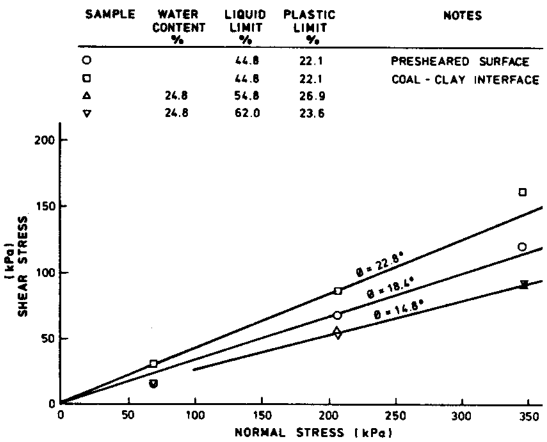


Figure 3. Direct shear test results on sample from location TC14 on the coal-clay interface

These pits were excavated with slopes varying from 45 to 60 degrees and were closely monitored. This program provided information on the soil properties and the water conditions. It was concluded that dewatering of the coal and overburden was necessary for highwall stability. A program for effective management and drainage of surface water was identified. The importance of softened shear zones in the bedrock was confirmed. Concerns were expressed regarding the potential liquefaction and trafficability of the upper saturated sand.

In 1976, additional field work was undertaken to identify areas where the upper sand in the bedrock was saturated and where the bedrock was disturbed and softened.

Dewatering tests were commenced in 1975 and 1976 to measure the rate at which the overburden might be dewatered in response to pumping from the Hart Seam. The results indicated that drainage was slow and several years of lead time in pumping were required in order to sufficiently improve highwall stability. A predevelopment dewatering program was initiated.

A study was undertaken in 1977 to address the question of trafficability of the saturated sand bedrock and assess the risk of liquefaction. The study concluded that liquefaction was not a serious concern and recommended that a 1.7 m pad of material be placed over the sand to improve trafficability.

Preliminary laboratory testing programs provided sufficient information for a preliminary design of the highwall and spoil slopes. The shear strength parameters varied widely. The results are summarized later in this paper. The preliminary design suggested that highwall slopes of 45 degrees could be excavated to a depth of 40 m provided the soils were drained. The design factor of safety was 1.3. It was also suggested that a highwall slope of 60 degrees could be excavated to a depth of 38 m under similar conditions.

Mining commenced in September, 1978. The box cut was successfully excavated, but one area of extensive instability was encountered. Further areas of instability were later observed in the turnover cuts. Each location of instability was studied and provided further information on the shear strength properties through the use of limit equilibrium back-analysis. These studies are summarized later in this paper.

With time the main factors affecting the stability of the highwall could be discerned:

i) the shear strength of the soil,
ii) the geologic conditions,
iii) the pore-water pressure conditions,
iv) climatic effects,
v) time effects, and
vi) the degree of support of the highwall.

The design of the highwall at the Poplar River Mine evolved through a combination of engineering analysis and operating experience between 1978 and 1984. The

Table 1. Summary of direct shear testing and back-analysis of failures

Location	Moisture Content	Liquid Limit	Plastic Limit	Residual Friction Angle	Cohesion	Comments
Test Pit 1-A	32	63	33	16	0	coal overlying carboneaous shale
Test Pit 1-B	29	63	24.5	11	0	base of slide coal over clay
Test Pit 2-A	-	48.8	18.2	18.5	0	-
Test Pit 2-B	25.2	54.2	29.7	19.5	0	below coal
Sample TC14A	-	44.8	22.1	22.8	0	coal-clay interface weak layer
Sample TC14B	-	44.8	22.1	18.4	0	lab pre-sheared surface, weak layer
Sample TC14c	24.8	54.8	26.9	14.8	0	near failure surface
Box Cut 2	28.4	62.0	23.6	14.0	0	till-shale interface
TC10	25.4	67.5	26.0	14.0	0	stability back-analysis
TC5	21.6	81.6	20.4	18.0	0	stability back-analysis
TC25	27.6	59.3	25.2	14.0	0	stability back-analysis
TC26	27.6	59.3	25.2	14.0	0	stability back-analysis

desire in the paper is to concentrate on two points:

i) the evolution in our understanding of the shear strength of the soils with time and
ii) the importance of engineering analysis in arriving at suitable design slopes.

GEOLOGY AND SOIL ENGINEERING PROPERTIES

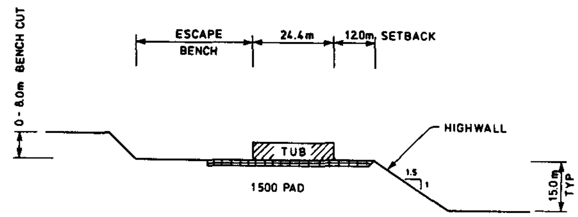
The stratigraphy at the mine site consists of till overlying bedrock of the Ravenscrag Formation (Figure 1). Glacial gravel is frequently encountered between the till and the Ravenscrag Formation bedrock. The till thickness ranges from approximately 0 to 15 m and the depth to the coal seam varies from 12 to 25 m below natural ground surface. The till has a silty clay matrix and possesses medium plasticity. The Ravenscrag Formation consists of stratified silty clay shale and fine-grained sandstone. The weak zones, upon which highwall movements have generally occurred, are the highly plastic, slickensided clay seams, near the bedrock surface. It has been postulated that glacial thrusting presheared these clay seams. Dewatering in the underlying coal seam maintained the piezometric surface in the coal stratum below the base of the pit.

The soil properties of primary interest are the shear strength of the till and the slickensided clay zones. Direct shear testing was performed on samples of the till taken from near the power plant (Figure 2). The effective cohesion intercept was 6 kPa and the effective friction angle was 34 degrees.

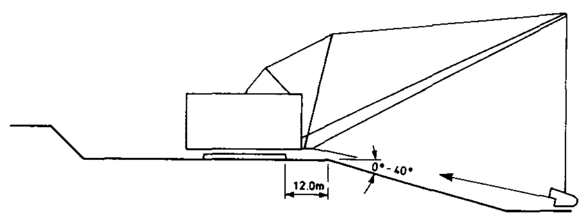
The shear strength properties of the shale were determined through direct shear testing carried out in conjunction with the test pit program. The testing was conducted on soil from a weak zone encountered at the full scale test pits. The observed failures in the test pit took place along slickensided weak zones along which previous shear displacement appears to have occurred. These weak zones are probably a result of glacial ice thrusting. The residual shear strength or the strength at large strains is of particular interest in this case.

The direct shear test results are summarized in Table 1. Table 1 also includes some shear strength parameters obtained from back-analysis of highwall failures. These are discussed later in the paper. The friction angle and plasticity characteristics vary substantially. Three direct shear tests were carried out on samples obtained from a weak layer of highly plastic, carbonaceous clay (sample TC14). The results of these tests are presented in Figure 3. Effective friction angles ranging from 14.8 degrees to 22.8 degrees were measured. The higher friction angles represent strengths along shear planes in the clay-coal interface. The specimens tested were from 100 mm diameter, undisturbed tube samples.

Another series of direct shear tests were conducted on a 100 mm diameter sample from the till-shale interface near Box Cut 2. Specimens tested from near the slickensided till-shale interface showed an effective friction angle of 14 degrees. The residual friction angles



TYPICAL SECTION
PERPENDICULAR TO HIGHWALL



TYPICAL SECTION
THROUGH FACE CUT

Figure 4. Sections illustrating the pit geometry

measured in the direct shear tests varied from 11.0 to 22.8 degrees. The range in residual friction angle reflects the variation in texture of the bedrock deposits. The lower friction angles correspond to soil with a higher liquid limit.

HIGHWALL AND DRAGLINE GEOMETRY

Sections illustrating the pit geometry are shown in Figure 4. Typical dimensions are given in Table 2.

Table 2 Typical pit dimensions

Location on Pit	Dimension
Tub to Highwall Crest	12 m
Typical Slope Height	15 m
Bench Cut Depth	0 to 8 m
Highwall Slope	1.5 horizontal to 1 vertical
Face Cut Slope	Typically 4 horizontal to 1 vertical
Desirable Set	24 m

The slope angle varied. The toe of the highwall was excavated to a constant horizontal distance from the highwall crest (toe offset). Therefore, where the overburden was deeper (depth from surface to top of coal seam), the slope was steeper. Typically, the slope angle ranged from 1.5:1 to 3:1.

The dragline was a Bucyrus-Erie 2570-W Walking Dragline. The pertinent dragline specifications are give in Table 3.

Table 3 Pertinent dragline specifications

Characteristic	Specification
Total Weight (with Ballast and Bucket)	12,823,000 pounds
Boom Length	110 m
Bucket Capacity	69 m^3
Shoe Dimensions	21.9 m x 4.3 m
Tub Dimensions	24.4 diameter
Load Distribution when Walking	70% on shoes, 30% on tub
Maximum Eccentricity on Tub	6 m shift in center of gravity
Maximum Drag Cable Pull	980,000 pounds

The dragline exerts different stresses on the soil for different working modes. The modes considered in the finite element analysis were:

i) No dragline load on high wall (Mode 1),
ii) Working with the dragline parallel to the highwall (Mode 2),
iii) Walking parallel to the highwall (Mode 3),
iv) Walking perpendicular to the highwall (Mode 4), and
v) High cam position walking perpendicular to highwall (Mode 5).

The stresses exerted on the ground for each mode are illustrated in Figure 5.

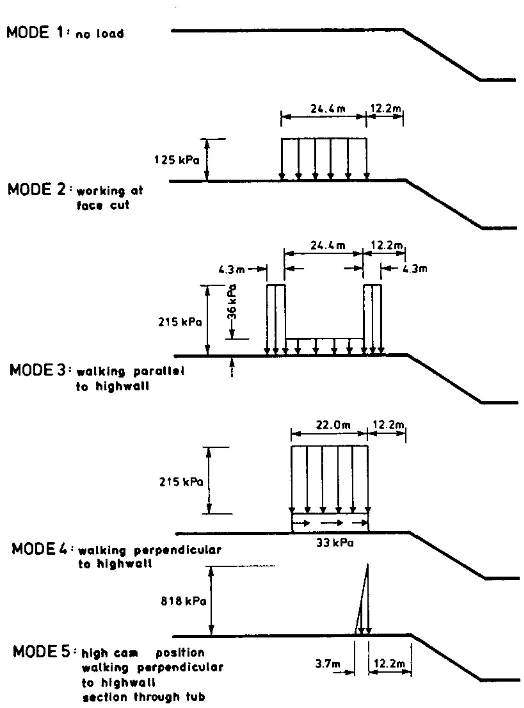


Figure 5. Stress distribution for various modes of dragline loading

HIGHWALL SLOPE FAILURES

Several highwall slope movements have been documented since 1981. Limit equilibrium back-analysis of failures, designated as TC5, TC10, TC25 and TC26, were performed to help assess the strength properties relevant to the time of failure.

Turnover Cut 5 (TC5)

A highwall failure occurred at location TC5 on 20 September 1981. The dragline was walking parallel to the highwall at the time of the failure (Mode 2). A cross-section showing the inferred position of the failure surface, as well as the stratigraphy obtained from a trench log adjacent to the failure is shown in Figure 6.

The failure occurred along a carbonaceous clay layer located approximately 4 m below the bench surface. The failure zone consists of interbedded coal and high plasticity clay . The failure appeared to be a result of the high stresses exerted by the shoe.

Back-analysis of the failure is presented in the form of various cohesion and friction angles which satisfy limit equilibrium conditions (Figure 6). For zero cohesion, the required friction angle was 18 degrees.

Turnover Cut 10 (TC10)

A highwall failure occurred at location TC10 on 22 September 1982. The TC10 failure is illustrated in Figure 7 which shows a plan view of the crack patterns. The inferred cross-section in the vicinity of the failure is shown in Figure 8. The slip plane was located in a slickensided, high plasticity clay at a depth of 7.6 m below the bench.

The results of the back-analysis are also summarized on Figure 8. The friction angle at failure, computed for a zero cohesion intercept, was 14 degrees.

Turnover Cut 25 (TC25)

Movement of the highwall occurred on 25 September 1984 at location TC25 as the dragline was moved parallel to the highwall. The slip plane was located 13 m below the bench. The highwall was 16.5 m in height and had a slope of 1.5:1. A ridge and valley buttress had been constructed in this area. The ridges were 5.8 m above the slip plane with a setback of 26.5m. The edge of the tub was located 9 m from the highwall, closer than the usual 12 m setback. Atterberg limits on samples collected from the slip surface showed a liquid limit of 59.3 percent and a plasticity index of 34.1 percent.

The factor of safety calculated, assuming the slip surface soil had a friction angle of 14 degrees with zero cohesion, was 0.93. The ridge and valley buttress is estimated to have increased the factor of safety to about 1.0. With the dragline located 12 m from the highwall, the factor of safety for a 24.4 m set was computed to be approximately 1.1.

Turnover Cut 26 (TC26)

A portion of the highwall at location TC26 failed on the weekend of the 27 and 28 October 1984. The dragline was not in the area. The highwall slope was 1.5:1 and the height was 13.4 m. The length of the failure was approximately 50 m with the scarp encroaching on the bench, 3.5 m from the crest of the highwall. The area was stripped approximately 31 days prior to the failure.

A back-analysis of the highwall slope, using zero cohesion and a friction angle of 14 degrees for the slip plane, indicated a factor of safety of 1.03.

On the basis of the above back-analysis (and the laboratory direct shear tests), it appears that the shear strength properties most representative of the slickensided clay are a friction angle of approximately 14 degrees for zero cohesion.

STRESS ANALYSIS OF THE HIGHWALL WITH THE DRAGLINE

The purpose of both the stress and limit equilibrium analyses is to assess the effect of weak soil layers occurring at various depths upon the highwall stability. Two questions were of interest:

i) Over what range of depths below the bench is a weak layer a concern?
ii) What berms or buttresses are required to increase the stability to an acceptable level?

This section presents the results of the stress analysis and explains their significance to the highwall stability. The stress analysis was performed to determine stress conditions within the soil under various dragline loading conditions (Figure 5). The stresses were then compared with stresses required to fail the soil.

The stress analysis was performed using a linear

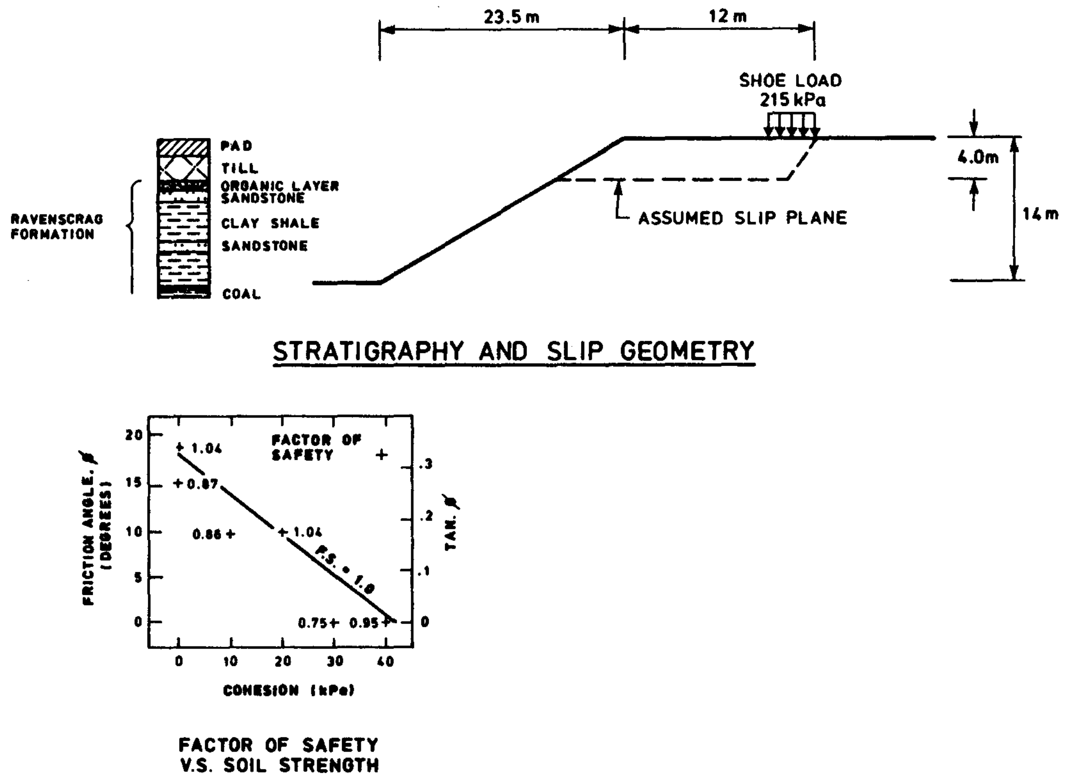


Figure 6. Summary of back-analysis of location TC5

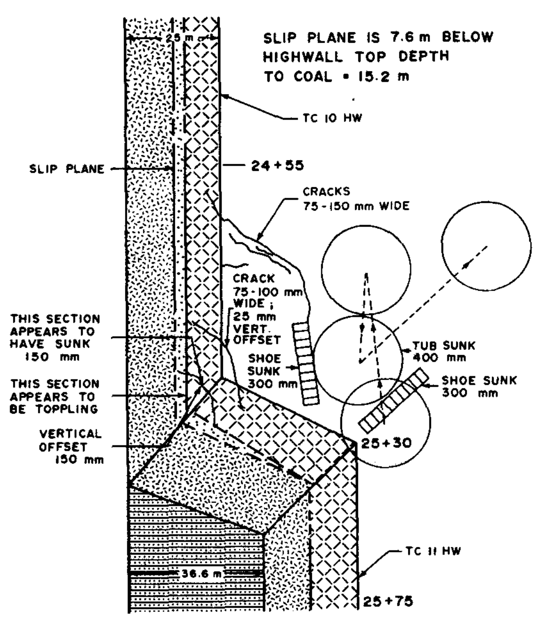


Figure 7. Plan of location TC10 showing the crack patterns

elastic, finite element computer model called FINEL at the University of Saskatchewan. The stress conditions along horizontal planes, corresponding to a potential slip plane, was of main concern. The ratio of the horizontal shear stress to the normal stress along a horizontal plane was computed and compared to the stress conditions which would result in soil failure. A critical friction angle of 14 degrees was used. This corresponds to a stress ratio of 0.25. Stress ratios within the soil mass greater than 0.25 are potential zones of failure when a weak horizontal layer is present. In reality, partial failure may occur. The resulting stress redistribution would cause the overstressed zones to be larger than that determined from the stress analysis.

The soil properties used in the stress analysis are shown in Figure 9. The elastic moduli were estimated. The calculated stress ratios are relatively insensitive to the magnitudes of the moduli selected. In all cases, the soil adjacent to the pit wall was overstressed for approximately 10 m from the highwall slope. This overstressing is a result of excavation and slope unloading.

Mode1 - No Load on Bench

Stress contours for this case of loading are shown in Figure 10. There are no loads acting on the bench. The slope is overstressed for approximately 2 m to 8 m from the highwall slope.

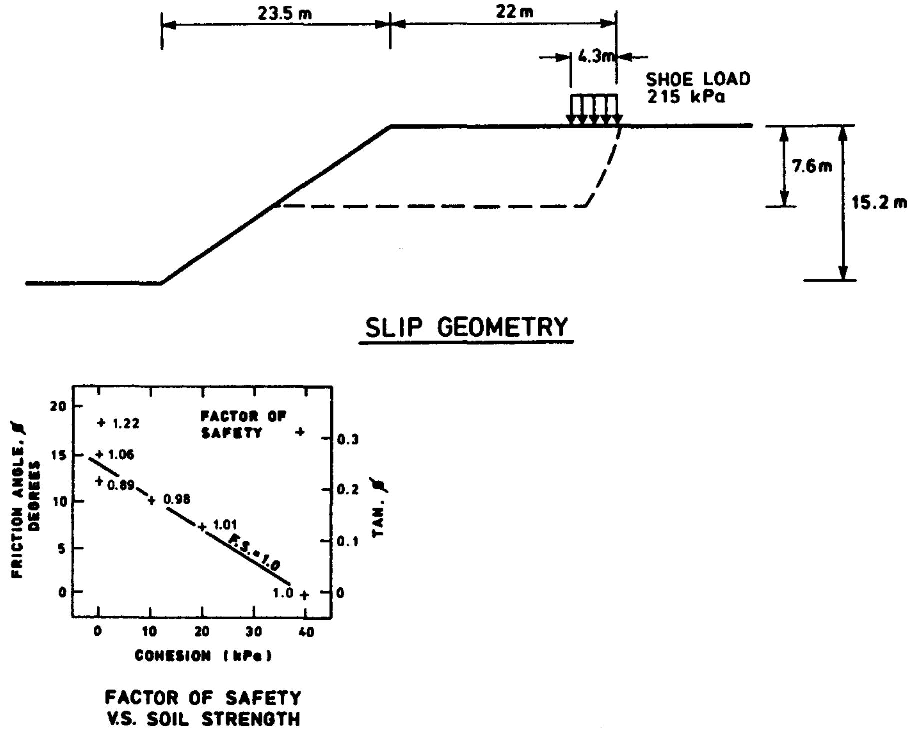


Figure 8. Summary of back-analysis of location TC10

LAYER	YOUNG'S MODULUS (kPa)	POISSON'S RATIO	UNIT WEIGHT (KN/m³)
1	124,000	0.3	19.3
2	249,000	0.3	19.3
3	689,000	0.3	19.3

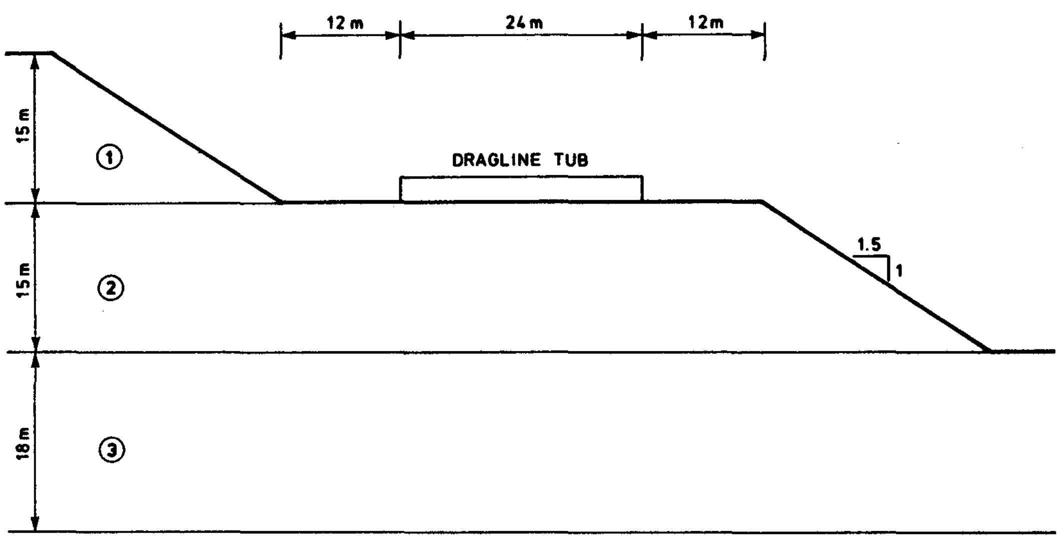


Figure 9. Summary of the geometry and the soil properties used in the finite element stress analysis

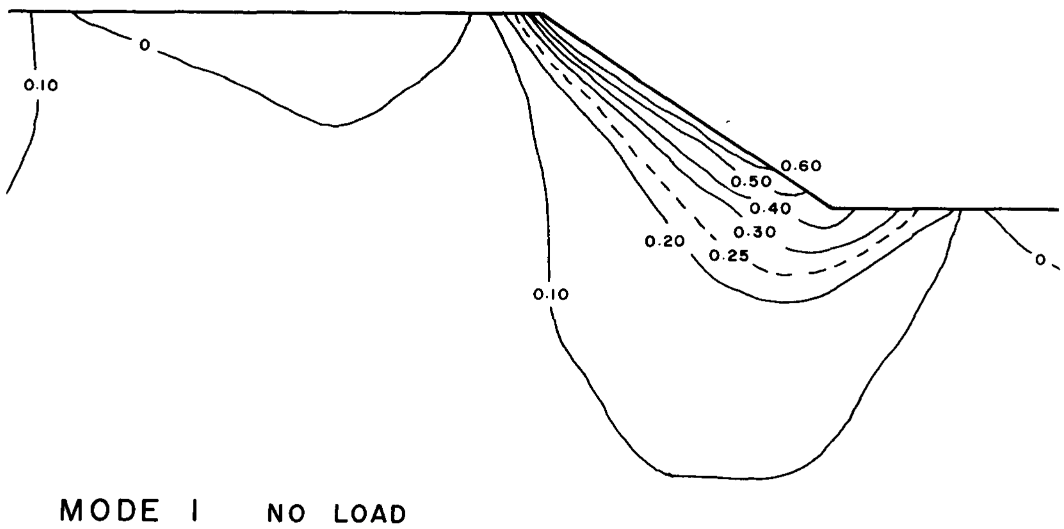


Figure 10. Contours of τ_{yx}/σ_y for loading Mode 1

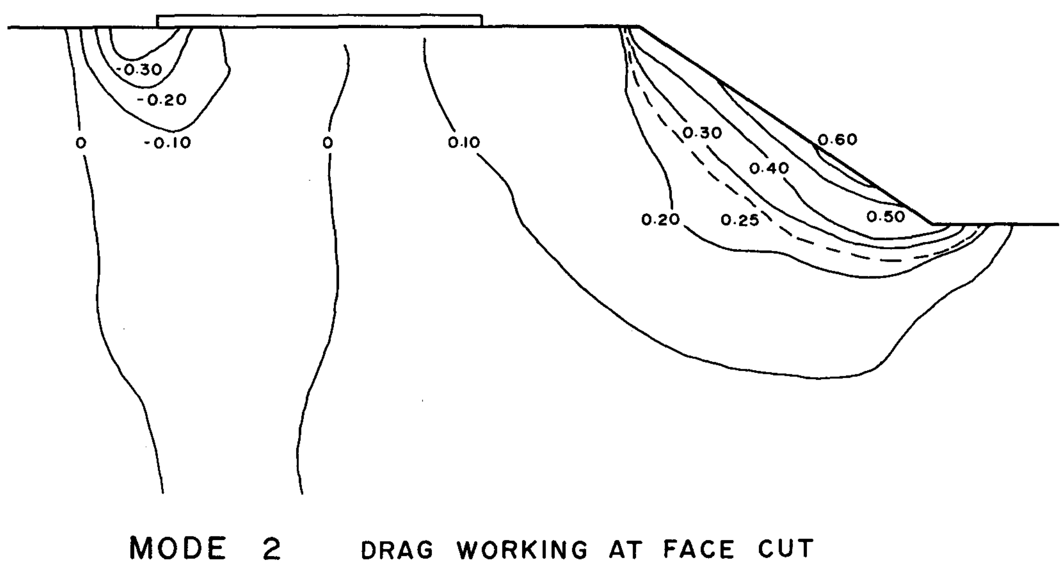


Figure 11. Contours of τ_{yx}/σ_y for loading Mode 2

Mode 2 - Dragline Working at the Face Cut

In this case the weight of the dragline is exerted uniformly across the tub. There are no shoe stresses. The stress contours are shown in Figure 11. The slope is overstressed for a distance extending about 10 m from the highwall crest as a result of excavation unloading. A small, local overstressed region is present near the tub edge.

Mode 3 - Walking Parallel to the Highwall

Contours of stress ratio, for the case of the tub being 12 m from the highwall, are shown in Figure 12. Similar contours, when the tub is 9 m from the highwall, are shown in Figure 13. An overstressed zone occurs, extending for a distance of approximately 10 m into the bench platform from the highwall crest. A larger and more heavily overstressed region exists under the exterior shoe. With the tub 12 m from the highwall, the overstressed zones coalesce and form a continuous zone which exists from beneath the shoe to the highwall, down to a depth of approximately 5 to 6 m. It would appear that failure would occur within this zone if a weak layer were present. With the tub 9 m from the highwall, the overstressed zone is larger, and exhibits higher stress ratios.

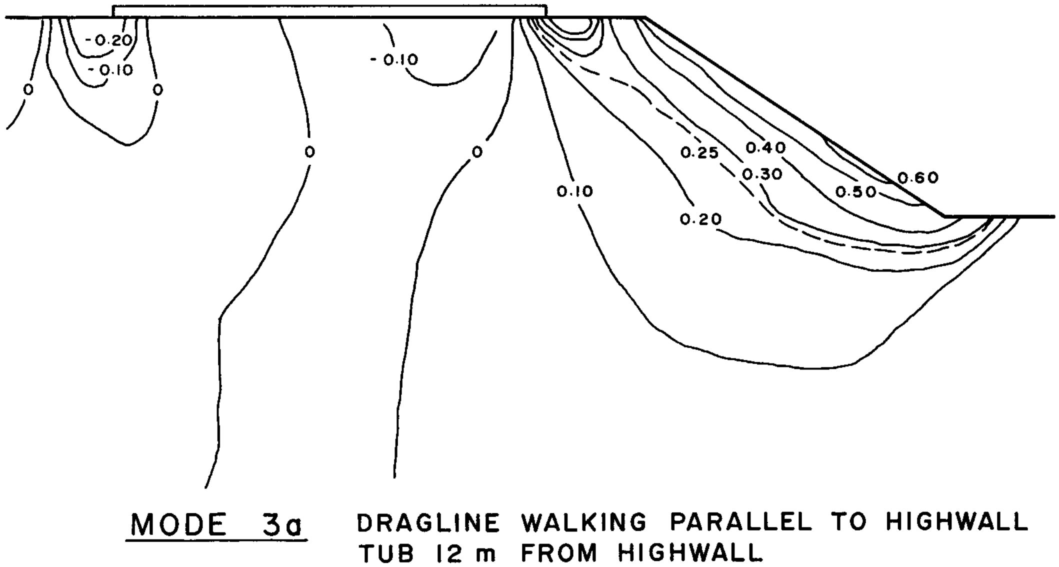


Figure 12. Contours of τ_{yx}/σ_y for loading Mode 3a

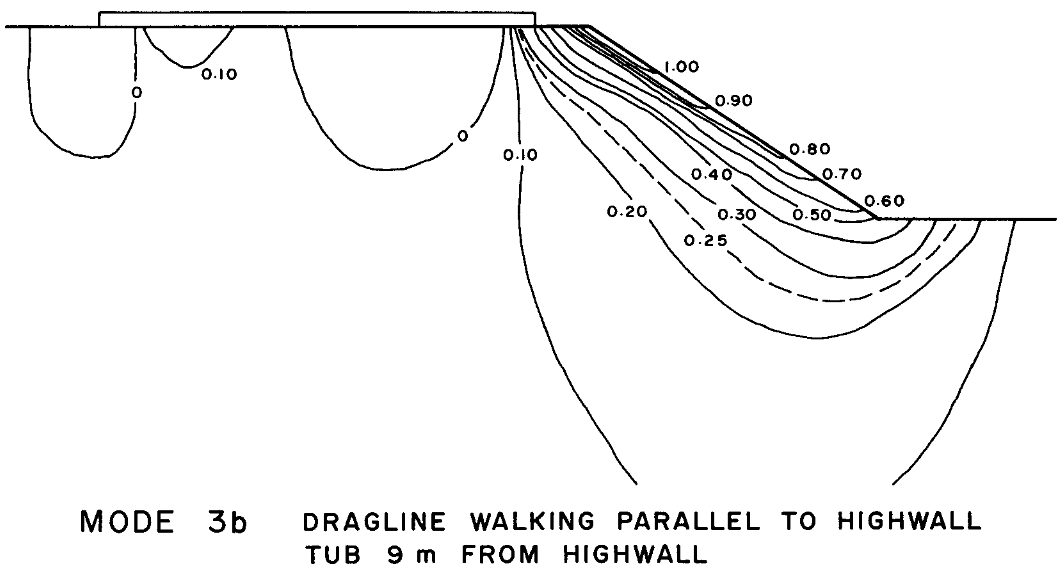


Figure 13. Contours of τ_{yx}/σ_y for loading Mode 3b

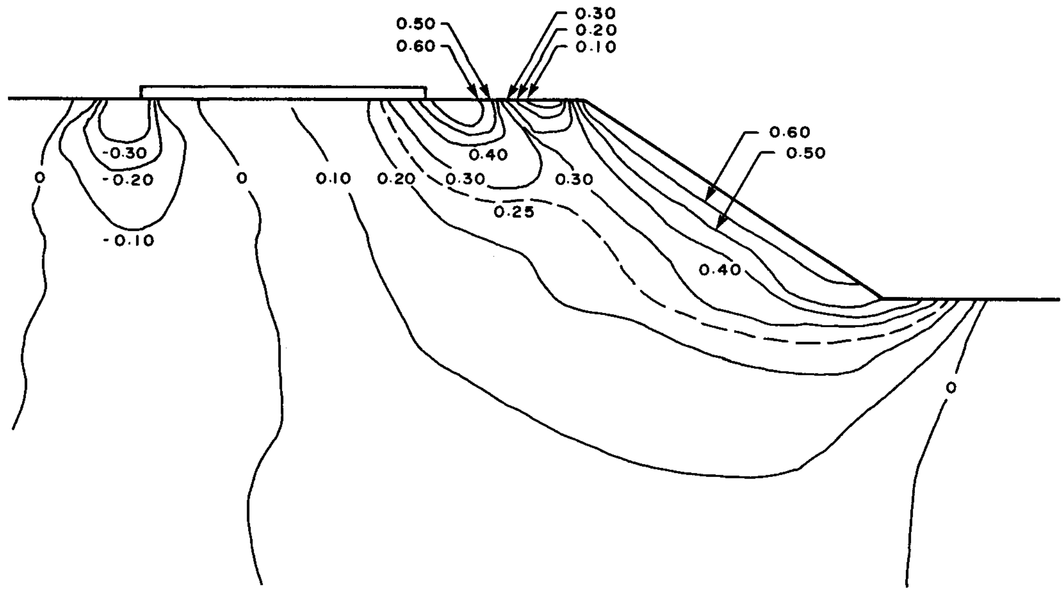
Mode 4 - Walking Perpendicular to the Highwall

Contours for this case are shown in Figure 14. Larger overstressed regions are indicated. These conditions are representative of the maximum stresses that occur through the shoe. However, the conditions are not completely representative of those that exist across a large section of the highwall. The stress analysis does show, however, that the outward thrust tends to increase the overstressed region and, therefore, reduces the overall stability of the slope.

Mode 5 - High Cam Position, Dragline Walking Perpendicular to the Highwall

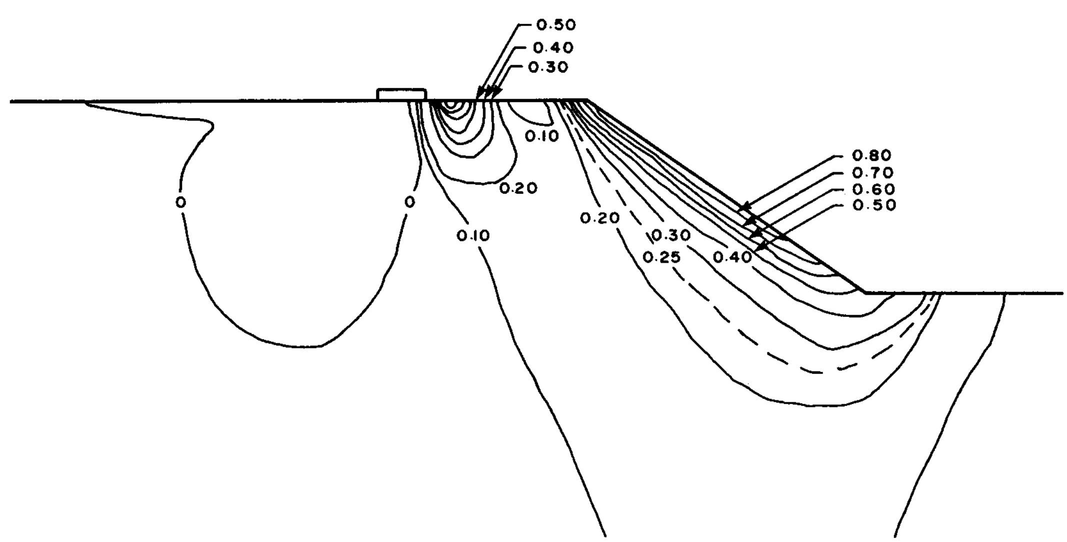
When the dragline walks perpendicular to the highwall at high cam position, only a portion of the tub is in contact with the bench. Assuming that 30 percent of the total load acts on this contact area, an average stress of 414 kPa is estimated. The shape of the stress distribution was assumed to be triangular, acting over a distance of 3.7 m perpendicular to the highwall.

An overstressed zone is indicated in Figure 15. The



MODE 4 DRAGLINE WALKING PERPENDICULAR TO THE HIGHWALL SECTION THROUGH SHOE

Figure 14. Contours of τ_{yx}/σ_y for loading Mode 4



MODE 5 DRAGLINE WALKING PERPENDICULAR TO THE HIGHWALL SECTION THROUGH TUB

Figure 15. Contours of τ_{yx}/σ_y for loading Mode 5

overstressed areas in the highwall due to excavation and to unloading, indicated that a localized failure might occur, being independent of a general slope failure.

In summary, approximately 10 m of soil next to the highwall may be overstressed along horizontal, weak layers. When the dragline is walking parallel to the highwall (Mode 3), the overstressed region is substantially expanded and a continuous, overstressed region extending to a depth of approximately 5 to 6 m is formed. When the shoe is closer to the highwall, the magnitude of the stresses is, also, increased. The stress analysis for the dragline walking perpendicular to the highwall (Mode 4) shows a largely expanded, overstressed region. However, the two-dimensional analysis is not completely appropriate, particularly in this instance.

LIMIT EQUILIBRIUM ANALYSIS

The limit equilibrium analysis was conducted to assess the stability of the highwall under three conditions:

i) various dragline loading conditions,
ii) various depths to the weak layer, and
iii) various buttress conditions.

The SLOPE-II limit equilibrium computer code was used for all factors of safety computations. Most of the slip surfaces analyzed were composite in shape. The soil parameters used are identified, where appropriate. The pore water pressures along the slip surface were assumed to be zero. This is quite reasonable since the highwall will have provided drainage for some time prior to being loaded by the dragline and the coal seams, generally associated with the weak layers, will have expedited drainage. No time effects were considered. All analyses assume a horizontally bedded, weak layer.

Comparison of Dragline Loading Conditions

The effects of various dragline loadings were assessed to determine which condition was most critical to the highwall stability. The loading conditions considered consisted of no dragline, the dragline working at the face cut, the dragline walking perpendicular to the highwall and the dragline walking parallel to the highwall. Pit geometry and the dragline loadings are the same as those illustrated in Figure 5. The highwall stability, for the dragline walking perpendicular to the highwall (Mode 4), is approximately 20 percent greater than when the dragline is walking parallel to the highwall (Mode 3). This is consistent with the observed failure mode at location TC10. These occurred with the dragline walking parallel to the highwall.

The intense loading, associated with Mode 5, results in a calculated factor of safety close to unity. In the high cam position with the dragline walking parallel to the highwall, the factor of safety is more than 10 percent higher.

The effect of the distance between the tub and the highwall was considered. With the dragline walking parallel to the highwall and with the tub 9 m (instead of 12 m) from the highwall, the factor of safety for a slip surface enclosing the shoe is lowered by approximately 5 to 10 percent.

The critical loading condition is with the dragline walking parallel to the highwall. Moving the dragline closer to the highwall decreases the stability significantly.

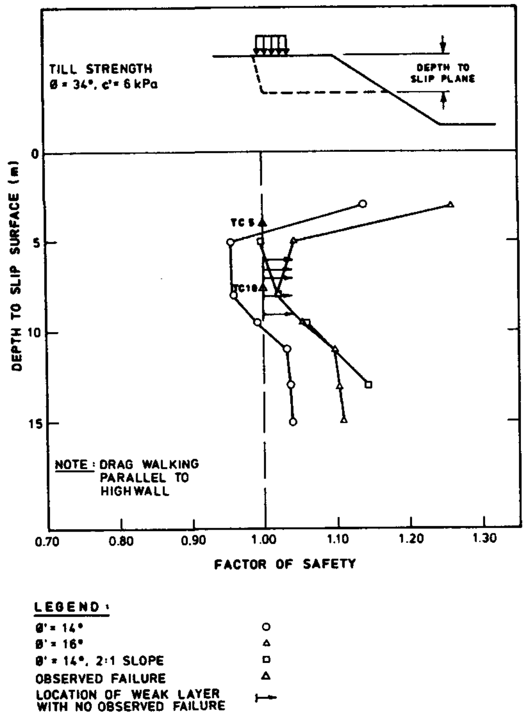


Figure 16. Factor of safety versus depths to the slip surface for differing soil strengths

Analysis of the Highwall

Another purpose of the stability analysis was to determine the effect of a weak clay layer at varying depths below the top of the highwall. The highwall was subjected to the critical dragline loading condition where it was walking parallel to the top of the highwall. Only those slip surfaces that enclosed the dragline shoe were considered. This failure mode is of greatest concern to the operational safety of the dragline. It should be noted that if the constraint that the slip surface must enclose the shoe is relaxed, the critical slip surface is generally found to be located between the highwall slope and the shoe. This becomes more apparent for slip surfaces located at increasing depths below the bench. Approximately 3.5 m of the bench was displaced when the slope failed with no dragline load present.

The results of the limit equilibrium stability analysis are summarized in Figure16. Stability analyses were carried out for weak layers at depths ranging from 2 to 15 m. Separate analyses were carried out for effective friction angles of 14 degrees and 16 degrees along the weak zone.

The minimum factor of safety occurs for a slip plane at a depth between approximately 4 to 11 m. For an effective friction angle of 16 degrees and for a slip surface at a depth of 5 m, the factor of safety was 1.04. For the same slip surface and with a friction angle of 14 degrees, the factor of safety was approximately 0.96. The factor of safety increased above unity at depths greater than 11 m for the range of soil strengths analyzed.

The results of a similar analysis for a 2:1 slope and a 14 degree friction angle are also shown in Figure 16. The 2:1 slope is only marginally more stable than the 1.5:1 slope. The small increase in stability results from a small relative increase in the length of the failure surface for the flatter slope. At greater depths, there is a greater relative increase in the length of the slip surface. Therefore, the increase in stability is greater at increased depths to the slip surface.

This relationship between the depth to the weak layer (or slip surface) and the factor of safety, illustrated in Figure 16, does not mean that stability of the highwall increases with an increase in height. The overall stability will, in general, decrease. However, the factor of safety of a slip surface that encloses the shoe is slightly greater.

The stability analysis results are consistent with the results of the stress analysis. The stress analysis for the Mode 3 loading conditions showed a major overstressed zone to a depth of approximately 5 to 6 m, and some overstressing of potential horizontal failure surfaces below that depth.

Further slope stability studies were performed to assess the effect of berming or buttressing the highwall with spoil material. The limit equilibrium analyses become complex due to the 3-dimensional nature of the problem. The results of the buttress study will not be presented in this paper.

The factors of safety presented have been expressed in absolute terms; however, they are of most value in a relative sense. The main difficulty arises in attempting to answer the question,"What factor of safety should be used in mine operation taking into consideration the conservatism of the analysis, and the risk to the dragline?"

SUMMARY OF FACTORS AFFECTING STABILITY

Many factors play an important role in assessing the stability of the highwall. A knowledge of some factors can be used in an analysis in a quantitative manner. Information on other factors must be used in a more qualitative manner. At the coal mine site considered in this paper, the stability of the highwall was mainly a function of the shear strength of the soils involved, geologic conditions, piezometric levels, climatic effects, time effects, the degree of support of the highwall and other factors.

Shear Strength

The principal factor in the stability of the slope is the shear strength of the soils, particularly the strength of the principal layer in which the failure surface develops. High, peak shear strength values may be used in hard undisturbed materials, but residual shear strength values may be required in the presheared zones. Figure 17 illustrates how the design values for the clays have changed since the initial study. The 1976 mine design was predicated on fully softened strength values (c' = 0) for the highwall, assuming the average strength in the highwall would be intermediate, between the peak and residual strength of the clays.

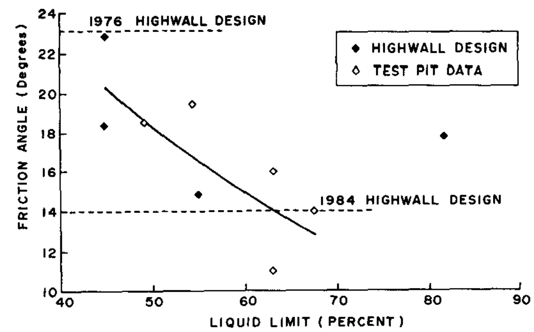


Figure 17. Summary of the friction angle versus the liquid limit for the badrock clay at the coal mine site

The design value for the weak, sheared layers has been reduced on subsequent re-analysis of 1984 highwall failures, until a value of 14 degrees was calculated. This value was utilized for the design of berms which would support the highwalls where the existence of shear planes was recognized.

Geology

The geologic environment controls whether the clay zones are present, whether or not they are presheared, and the depth at which they occur. Over substantial reaches of the highwall, the clay zone may be absent; or, it may be intact and not at its residual strength value; or, it may be at a depth where it does not influence stability. In these reaches, an unsupported highwall had adequate stability. Otherwise, excavation of the shear zones or buttressing was required to ensure adequate safety of the dragline.

Piezometric Levels

Piezometric levels are a function of recharge and the rate at which groundwater is being withdrawn from the overburden. Experience and analysis indicates that, where developmental dewatering is not adequately ahead of stripping, residual pore pressures will decrease highwall stability, leading to failures.

Climatic Conditions

Precipitation and runoff govern the amount of infiltration. Summer precipitation at Assiniboia averaged 300 mm between 1940 and 1971. Summer precipitation at Coronach averaged approximately 170 mm between 1980 and 1984. Hence, natural recharge has been substantially below expected levels and piezometric levels in the overburden are, likewise, relatively low. Thus, the degree of instability experienced was probably below the "average" frequency of occurrence.

Time Effects

The highwall will stand at a low factor of safety, only for a limited period of time, since the clays undergo substantial softening and grow weaker with time. This is the reason why more failures occur the longer the highwall is left exposed. It can be expected that if the cycle time between turnover cuts increases, the number of highwall and spoil failures can likewise be expected to increase.

Precision of Prediction

It must always be recognized that slope stability analyses do not allow an exact prediction of the factor of safety because of natural variations in soil conditions and the difficulty in predicting pore water pressures. It is normal that the design factor of safety reduces as more experience is gained. For instance, failures were experienced in the slope designed at a factor of safety of 1.3 for the new mine because relatively little was known about the shear strength of the soil. As more knowledge has been gained, the operating factor of safety has been reduced to 1.15. Failures related to unforeseen conditions can be expected at that design factor of safety.

Valuable experience has been obtained through the study of previous highwall failures. Monitoring and analysis of the type presented in this paper should be conducted on an ongoing basis. This will increase the engineers knowledge of the factors involved and of the reliability of the analyses that can be performed.

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