Tavis Mining Highway Stability Study - Methods Overview and Sensitivity Study

Matthew Jurgens

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Tavis Mining Highwall Stability Study – Methods Overview and Sensitivity Study

By

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A Senior Design
Submitted to the Undergraduate Faculty
of the
University of North Dakota
in partial fulfillment of the requirements
for the degree of
Bachelor of Science

Grand Forks, North Dakota
May 2005
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Introduction
The Falkirk Mining Company is considering mining the Tavis Creek coal seam in the Riverdale field southwest of Underwood, ND (Figure 1). The Hagel seams that lie directly above the Tavis Creek seam are currently being mined for power generation at the neighboring Coal Creek Station immediately south of the mining operation.

The Clean Air Act (1990) was set forth by the EPA "to protect and enhance the quality of the Nation's air resources" and "encourage and assist the development and operation of regional air pollution prevention and control programs" (Part A, Section 101 a. & b.). This has led to increasing regulations on stack emissions from power plants such as Coal Creek Station. Technological advances in power plant scrubbers, which remove NOx and sulfur dioxide from the plant emissions, as well as new breakthrough processes designed to remove methylmercury from coal are making progress in emission control. The most effective way to combat this issue; however, is by utilizing cleaner coal sources. The Tavis seam has potential to help meet this demand.

This paper is a sensitivity study of highwall stability at Falkirk Mine. Since mining of the Tavis Creek will require maximizing the highwall height, this paper explores factors that influence the strength of the wall. The first part consists of a previous works review composed primarily of geology and geotechnical studies at the mine. As well, a basic site characterization is provided highlighting the general geologic and hydrologic features of the Riverdale field. A compilation of the fundamental numerical techniques and theory behind a geotechnical study of this nature is provided, as well as a model comparison, using limit equilibrium methods, to one of the previous geotechnical studies performed at the mine. Finally, a sensitivity study of highwall stability is performed using finite difference methods.
Study Area
The exact location for the start of the Tavis mining is not known; however, it is believed that it will begin adjacent to the west-most Tavis crop line, roughly due west of the northwest "arm" of the Hagel crop line (Figure 1). This general area is approximately 8 miles west of the existing mine site, which can also be seen on the map. A crop line is a closed boundary that defines the edge of an un-weathered coal deposit. In this coal field, the Hagel and Tavis crop lines are predominantly defined by glacial erosion; however, other morphological processes, such as bedrock river valleys described by Klausing (1974), have "washed-out" the coal in some areas. This study concentrates on the area directly inside the Hagel Arm where the Hagel seam and Tavis seam co-exist.

Large Scale Geologic Setting
The sedimentary rocks, soils, and associated coals are part of sediments deposited in the Williston basin during the Paleocene Epoch (Lefever, 2002).

The Williston Basin (Figure 2) is a large, cratonic basin that began subsiding during the Ordovician period and is still believed to be subsiding today. The exact reason for subsidence is not well understood (Lefever, personal communication).

The Sentinel Butte Formation marks the youngest bedrock unit in the area with the Bullion Creek Formation conformably underlying the Sentinel Butte Formation (Blueme, 1971). These two formations form the upper portion of the Fort Union Group, which was deposited during the mid to late Zuni Sequence during the regression following the last significant sea
level rise (Lefever, et. al., 2002). The coals of these formations, most significantly the Hagel and Tavis seams, were deposited in lacustrine and swamp conditions where the sea rose and filled large depressions and subsequently subsided, eventually leading to stagnant conditions. The stagnant conditions allowed for plant material to be deposited and preserved, due to anoxic conditions, eventually forming lignite.

Unconformably overlying the Fort Union Group is a mix of glacial and interglacial sediments known as the Coleharbor Formation. The Coleharbor formation was deposited between several hundred to 9,000 years ago and overlies most of North Dakota (Bluemle, 1971). It has a highly variable lithology corresponding to multiple periods of glacial advance and retreat as well as changing glacial processes present during deposition (Bluemle, 1971). In the Hagel Arm, the “blanket-like, veneer of drift”, or till, was probably deposited as ground moraine or sheet moraine when the rate of glacial advance equaled melting (Bluemle, 1971). In effect, the glacial material held by the glacier was dropped as the glacier lost competency.

**Stratigraphy**

Stratigraphic cross-sections through the Hagel Arm are shown in Figures 3-7. The Tavis Creek coal seam belongs to the Bullion Creek formation, and is the lowest coal seam that is extensively mapped by Falkirk. There are two other economically important, regionally extensive seams, namely, the Hagel B and Hagel A. Two other seams, the C and Kinnemann Creek seam, are important for structural and water issues relating to pit stability. As well, the C seam, along with its associated sands, produces some measurable groundwater flow and is a local water source (James, 2003). The Kinnemann Creek and C seams are located above and below the Hagel seams and belong to the Sentinel Butte and Bullion Butte formations, respectively. The Sentinel Butte formation and Bullion Butte are part of the greater Fort Union Group, which is overlain by glacial deposits and Holocene age soils (James, 2003).
Figure 3. Generalized Stratigraphy (modified from James, 2003)
Figure 4. Cross Section Locations

Figure 5. Cross Section A-A'
Stratigraphic description in this paper will proceed starting from the base of the Tavis and move upward toward the surface. The well numbers used for mapping can be seen in Figure 8.

The Tavis is approximately 145ft deep on the edge of the Hagel cropline and gets progressively deeper moving radially inward until it reaches a maximum depth of approximately 193 ft. from ground surface. The Tavis is a relatively hard lignite coal with few cleats when compared to the Hagel A and B. Many of the cleats may be secondary fractures from the coring process. There is very limited data lower than the initial 15 ft. below the Tavis Creek seam. Multiple kicker seams and carby clay stringers
can be randomly found directly below the seam with parting thicknesses ranging from less than an inch to a few feet. No kicker seams are recorded in the drillholes sampled from the area of study; however, they have been in many other areas and are localized phenomena that occur sporadically. A carby clay stringer is also often found within the top portion of Tavis seam, similar to that observed in the coring description at 2.5 ft of hole RD03044C. Although relatively thin, a stringer at this footage was apparent in many of the cores pulled during the summer 2003 drilling and so may be relatively

Figure 8. Depth Contour Map of the Tavis Creek Bed
extensive. There is no strength data associated with these carbonaceous clay stringers and kicker seams, but from visual observation it appears the organic content in the clay greatly reduces shear strength. For this reason, these stringers and kicker seams may have played a role in past plane failures originating in the pit floor.

From the drilling data, the portion of the Bullion Creek formation above the Tavis is primarily composed of fat clay with occasional thin lenses of lean clay, shale, silty sand, and carbonaceous clay to coal zones. Lean clay is a soil that is predominantly clay, but has a noticeable amount of silt to sand particles that make it feel slightly gritty to touch. Conversely, fat clay is predominantly composed of clay size particles and feels greasy to touch. For engineering purposes, fat clay will generally have a slightly higher value of cohesion and lower angle of internal friction than lean clay, displaying slightly more plastic characteristics. Approximately 60-80 feet above the Tavis lies the C seam of the Sentinel Butte Formation, which ranges from carbonaceous clay interbedded with coal to a full coal seam averaging two feet in thickness. This seam is almost always associated with a sand unit that lies directly below it. Lean to fat clay persists until reaching the Hagel B seam usually located approximately 30 feet above the C seam. In this clayshale zone, a unique bluish color clayshale can be distinctly noted in some zones.

The depth of the Hagel A seam can be seen in Figure 9. The Hagel A and B beds contain approximately five to eight feet of coal separated by a parting averaging two to three feet thick, but ranging up to eight feet. While the Hagel B seam is consistently two to three feet thick, the Hagel A seam is more variable and in other areas of mining is known to part in the middle. The parted beds are classified as A1 and A2 for the upper and lower, respectively. No specific A1-A2 parting was noted in the area of study; however, small stringers of coal in the A-B parting may be remnants of the A2 bed which may just be thicker in other areas. The A-B parting also contains gray clayshale with carbonaceous zones. Here again these carbonaceous zones are believed to have extremely low shear strength.
Lean to fat gray clayshale overlies the Hagel A and B seams until being unconformably overlain by the Coleharbor formation (Bluemle, 1971). Bluemle described this unit as glacially derived stratigraphic unit directly over the Sentinel Butte formation that contains three distinct lithofacies: 1.) interlayered bouldery, cobbly, pebbly, sandy, silty clay 2.) sand and gravel 3.) silt and clay. It becomes apparent from studying Falkirk’s geology and geophysical logs that defining specific trends in the lithology of the Coleharbor formation and its boundary with Fort Union clays is problematic. The best way to identify this boundary is based on the geophysical curve by the irregular density “kick”, due to less consolidation and lithification and increased stratification of the Coleharbor, and slightly different gamma “kick” as compared to the underlying clays of the Sentinel Butte formation, or by watching for the change in downhole drilling pressure. The wide range in materials is the result of the many glacial processes.
occurring during the several episodes of advance and retreat. Much of the glacial load was derived from Fort Union clays and, to the untrained eye, is indistinguishable from the underlying bedrock in many places. In other locations, thick gravel deposits and large boulders characterize the formation. This poses difficulties for highwall stability analysis. The highly variable lithology of the Coleharbor makes it imprudent to evaluate this unit based entirely on the strength test data and a successful pit design will only be possible by actively monitoring for problem zones. Immediately above the Coleharbor, James (2003) noted the Oahe formation, which is the name given to the Holocene age soils that continue up to the surface.

Hydrogeology
As of the January 2004, the Riverdale field held 19 wells screened in the Hagel A seam, 15 in the Hagel B seam, 26 in the C sand (C sand plus C coal seam), and 20 in the Tavis Creek seam. There are two wells located inside the Hagel Arm. The data is broad, giving an excellent picture of the regional hydrogeology, but a limited view of the particular study area.

The coal seams can be considered confined aquifers due to the relatively impermeable nature of the surrounding clay and are the main sources of groundwater in the area. Locally, sand units are found amongst the clay bedrock and act as perched water tables. There are also aquifers associated with buried valleys and glacial deposits in the area (Klausing, 1974). One of these, Weller's Slough, is a buried valley and cuts between the northern and southern Hagel crops. Weller's Slough laterally recharges the Tavis seam. Other recharge of the coal seams primarily occurs in the potentiometric high areas through infiltration from depression storage (James, 2003). Potentiometric differences also cause flow from the upper aquifers to the lower aquifers such as the Tavis (James, 2003). Discharge in all aquifers is generally radial from the high depression storage areas, which are centrally located, and in the west area primarily occurs as baseflow into the Missouri River Valley. Figure 10 shows the potentiometric surface for the Hagel A.
Methods of Mining

Two different procedures for mining have been proposed through past studies, namely bench side and spoil side mining. Each of these methods is illustrated below.

Bench Side Mining (Figures 11 & 12)

In bench side mining, the dragline is located on the in-situ overburden labeled “bench”. The dragline walks into the page (+z direction), removing the overburden it was previously standing on. It turns to the right (-y direction), and throws the overburden into the previous pit. The instance of this diagram is immediately before the dragline has placed any spoil from the new pit. The last spoil pile shown is from the previous bench that has already been removed. As well, in this diagram the upper spoil lift is shown set back from the edge of the lower spoil lift to help increase stability of the spoil pile. This upper spoil lift could also illustrate an interburden strip of spoil that could be located between two coal seams that are located very close together. In this instance, the lower spoil lift would be overburden above the first seam. After the first coal seam is removed, the interburden spoil (upper spoil lift) is removed and placed on top of the lower spoil lift.
**Spoil Side Mining**

In this method, the dragline sits on top of the spoil (Figure 13). The blue colored in-situ overburden must be taken down and leveled. This can be done with explosives; however,
this is difficult and expensive in clayey soils as they don’t brittle fracture and the clay absorbs much of the shot. At this particular instance of the diagram, the dragline is sitting upon a bench that has been created by taking down the unmined overburden and leveling it with the dozer. It will now take the portion of the bench directly below it and move it further back near the other peak. This method is complicated and it doesn’t seem to have much potential in soft soils such as clay, as soft pockets in the bench can develop and the dragline can get stuck while walking.

Figure 13. Spoil Side Mining

Past Stability Issues
Past failures at Falkirk have occurred in the spoil as well as highwall. They generally seem to be water related, but can also be due to localized structure such as faulting. No regional jointing or faulting patterns have been determined, however.

In the spoil side mining study by Calder & Workman, Inc., an example of a large spoil failure from 1994-1995 is noted. The failure was termed a translational failure (Figure 14); a spoil peak down-dropped in place and acted as a wedge, separating the spoil piles on the weak contact plane and forcing the outmost pile into the active pit.

Calder & Workman, Inc. and the staff geologist believed the main failure occurred because the spoil and pit floor were saturated and facilitated movement of the main spoil pile. In these conditions, the spoil bases may be able to slide horizontally with the active wedge acting as the driving force. Recently, the staff geologist said he believed that the
base sliding could have occurred on the carbonaceous clay stringers that are often found below the main coal seams or between the Hagel seams.

In Figures 15 through 17, the results of a translational failure can be seen that occurred in October 2003. This failure is believed to have occurred because of structural weaknesses induced by the Kinneman Creek seam, which is located immediately below the brown, weathered soils at the top of the highwall in the picture. It can be seen that immediately to the west of the failure the KC seam dips rather sharply away from the failure and is split into a lower and upper seam. At the location of failure, the parting almost disappears only to reappear slightly thicker on the east side of the failure. This behavior is probably the result of differential compaction of materials, clay and sand, below the coal seam after it was deposited. Differential compaction is when a unit of soil loses more volume upon compaction than the soils adjacent to it. A sand body located immediately below the failure could cause this type of compaction.

This difference in compaction is well documented by the abrupt incline in the coal seam, which is assumed to be relatively flat upon deposition. Small shear planes or joints may have developed in the “dipping” region, which significantly lowered the strength of the
Figure 15. Highwall Failure in Center Pit (photo 1, view from pit ramp)

Figure 16. Highwall Failure (photo 2, view from bench)
overburden and subsequently facilitated failure. As well, water produced by the KC seam could have helped instigate failure.

Future stability issues will be similar to these, but will be further aggravated by the increased height of the highwall and more complex groundwater conditions. Pre-stripping down to the maximum ninety foot digging depth of the dragline would eliminate some in-situ stresses due to decreased overburden as well as eliminate the water issues related to the KC and Hagel seams. In this case, pit stratigraphy would be similar to present conditions, with the C seam in a comparable position as the KC seam. Highwall strength would most likely be lower; however, due to the increased head of the C seam and assumed lower strength of the associated sand unit.

**Previous Work**
Calder and Workman, Inc. has performed several geotechnical studies in various areas of the mine site. Two of these studies were reviewed for this paper.
First Study
Mining geometry for the Hagel seams in the Riverdale field was analyzed. Three holes were drilled to determine geotechnical properties of the material. As well, monitoring wells were installed to estimate water levels in the overburden. It was determined that there were five different types of materials that could be classified in a highwall in this area. Properties of these materials obtained from this drilling program can be found in the “Soil Strength” portion of this report. Till thicknesses ranged from 10 to 15 feet. Below this, fat clay predominated with pockets of silty sand and clayey sand. Parting between the Hagel A and B seams was primarily fat clay, except one hole had lenses of sand and silt.

Analysis was performed with the “PitBoss” program of a 60 foot highwall with a repose angle of 55 degrees and a 44 ft. depth to the water table. The method of mining is bench side with an upper spoil lift setback.

Variations in lithology were considered to adequately represent maximum, intermediate, and baseline strength conditions. The generalized lithology used is shown in Table 1.

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<thead>
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<th>Table 1. Calder &amp; Workman's Highwall Lithology with Tested Strength Parameters</th>
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<td>Soil Type</td>
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<td></td>
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<td>Fat Clay</td>
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<tr>
<td>Silty Sand</td>
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<tr>
<td>Fat Clay</td>
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<tr>
<td>Clayey Sand</td>
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<tr>
<td>Coal</td>
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<td>Parting</td>
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<td>Coal</td>
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Various dragline positions were considered to determine the factor of safety in all strength conditions. They are summarized in Table 2.
It was conclusive from this study that the highwall should be stable under all conditions, except for a 20 foot dragline setback under baseline conditions. Water conditions greatly influenced stability as did dragline positioning.

The spoil piles generated from this lithology were also analyzed for stability. Floor stability was a main concern in this study as wedge failures had occurred in the spoil. Calder & Workman compiled two samples from the drilling to represent the overburden and parting lifts that would be present in spoil piles. The first lift is generated when the coal is initially uncovered. The second lift is generated after the Hagel A seam is uncovered and a smaller dragline is working in the pit to uncover the Hagel B seam. The strength parameters of these spoil lifts, as well as the strength of the floor lithology below the coal, are shown in Table 3. A 20 foot water table in the spoil was assumed.

### Table 2. Calder & Workman's Analysis Results

<table>
<thead>
<tr>
<th>Highwall Strength</th>
<th>No Dragline</th>
<th>Dragline</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Setback-----</td>
<td>20 Ft.</td>
</tr>
<tr>
<td>Baseline</td>
<td>1.054</td>
<td>0.927</td>
</tr>
<tr>
<td>Intermediate</td>
<td></td>
<td>1.051</td>
</tr>
<tr>
<td>Maximum</td>
<td>1.429</td>
<td>1.233</td>
</tr>
</tbody>
</table>

### Table 3. Calder & Workman's Spoil Strength Parameters

<table>
<thead>
<tr>
<th>Spoil Unit</th>
<th>Cohesion</th>
<th>Angle of Internal Friction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lower (overburden)</td>
<td>400</td>
<td>20</td>
</tr>
<tr>
<td>Upper (parting)</td>
<td>400</td>
<td>15</td>
</tr>
<tr>
<td>Floor Samples</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fat Clay</td>
<td>1000</td>
<td>10</td>
</tr>
<tr>
<td>Clayey Sand</td>
<td>1000</td>
<td>20</td>
</tr>
<tr>
<td>Fat Clay</td>
<td>1000</td>
<td>10</td>
</tr>
</tbody>
</table>
Calder & Workman varied spoil height based on thickness of parting between the Hagel A and B seams. Parting thickness was taken at 5 feet and 20 feet. These two extremes for parting thickness gave spoil heights ranging from 114 feet to 97 feet high. They also analyzed the effect of setting back the upper lift of spoil between 20 and 40 feet from the lower lift. This terracing method of spoil placement helps increase spoil stability. The overall spoil angle was calculated by drawing a straight line from the base to the peak. The lower lift maintained an angle of 37 degrees for all calculations.

The results of the spoil stability analysis is summarized in Table 4. Generally, the spoil was considered stable when the overall angle was below 30 degrees for both cases.

<table>
<thead>
<tr>
<th>Overburden Thk</th>
<th>Parting Thk</th>
<th>Height</th>
<th>Overall Angle</th>
<th>Lower Lift Angle</th>
<th>Upper Lift Angle</th>
<th>Upper Lift Setback</th>
<th>UL Factor of Safety</th>
<th>LL Factor of Safety</th>
<th>Overall Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>60</td>
<td>20</td>
<td>114</td>
<td>30</td>
<td>37</td>
<td>37</td>
<td>NA</td>
<td>0.984</td>
<td>1.14</td>
<td>1.026</td>
</tr>
<tr>
<td>60</td>
<td>20</td>
<td>114</td>
<td>28</td>
<td>37</td>
<td>37</td>
<td>NA</td>
<td>0.946</td>
<td>1.136</td>
<td>1.134</td>
</tr>
<tr>
<td>60</td>
<td>5</td>
<td>97</td>
<td>37</td>
<td>37</td>
<td>37</td>
<td>0</td>
<td>NA</td>
<td>NA</td>
<td>0.819</td>
</tr>
<tr>
<td>60</td>
<td>5</td>
<td>97</td>
<td>34</td>
<td>37</td>
<td>37</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>0.945</td>
</tr>
<tr>
<td>60</td>
<td>5</td>
<td>97</td>
<td>31</td>
<td>37</td>
<td>37</td>
<td>20</td>
<td>NA</td>
<td>NA</td>
<td>1.077</td>
</tr>
<tr>
<td>60</td>
<td>5</td>
<td>97</td>
<td>28</td>
<td>37</td>
<td>37</td>
<td>40</td>
<td>NA</td>
<td>NA</td>
<td>1.206</td>
</tr>
</tbody>
</table>

Finally, Calder & Workman analyzed the stability when the dragline is on the spoil side removing the parting between coal seams, as well as any rehandle present. In general, the lower spoil lift was stable under the dragline load when parting thickness was 20 feet. The lower spoil lift was approximately 52 feet high in this situation and the rehandle strip at the beginning of parting removal was effective at stabilizing the toe of the spoil. The 5 foot parting situation generated a lower lift of 60 feet rather than the 52 feet as in the 20 foot parting situation. For these conditions, it was determined that the dragline be setback 40 feet from the edge. The results are summarized in Table 5.
Table 5. Calder & Workman Parting Removal Stability Analysis Results

<table>
<thead>
<tr>
<th>Parting Thickness</th>
<th>Lower Lift Height (feet)</th>
<th>Lower Lift Angle</th>
<th>Dragline Position (feet)</th>
<th>Parting Removed (yes/no)</th>
<th>Factor of Safety</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>52</td>
<td>37</td>
<td>20</td>
<td>no</td>
<td>1.956</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>52</td>
<td>37</td>
<td>20</td>
<td>yes</td>
<td>1.314</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>60</td>
<td>37</td>
<td>20</td>
<td>no</td>
<td>1.065</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>60</td>
<td>37</td>
<td>20</td>
<td>no</td>
<td>1.098</td>
<td>Only considered slip surfaces under dragline</td>
</tr>
<tr>
<td>5</td>
<td>60</td>
<td>37</td>
<td>40</td>
<td>no</td>
<td>1.067</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>60</td>
<td>37</td>
<td>40</td>
<td>no</td>
<td>1.251</td>
<td>Only considered slip surfaces under dragline</td>
</tr>
</tbody>
</table>

Finally, this study considered the possibility of wedge failures occurring in the spoil peaks. It was concluded that the crossover point of instability occurs when the angle of internal friction of the possible failure plane in the base of the spoil is less than 3.6 degrees. They emphasized the importance of keeping the pit floor dry in order to maintain stability.

Second Study
The main focus in the second study was to analyze the suitability of spoil side mining at Falkirk. The schematic was outlined earlier in mining methods. Bench heights of 95 and 75 feet were analyzed with variations in water table heights. It was found that the most stable position was when the spoil rises to the top of the coal. It was then further stabilized by compacting the toe.

Soil parameters were determined from backcalculations as well from previous studies. They are summarized in Table 6.
As with the bench side mining method, spoil side mining stability is predominantly governed by the face angle. Calder & Workman varied the face angle of the spoil to determine the effect on stability. Stability calculations were performed for a 35 degree face and the water table was hypothesized at 25 feet above the pit floor. A second scenario with the water table 45 feet above the pit floor was also analyzed. The results are summarized in Table 7. It was found that in these situations, a rotational failure of the dragline bench had a factor of safety very close to one, with added support from compacting the toe. Failure was likely with the increased water table height. Rotational failures of the entire spoil, which would include the peaks further back from initial bench, were unlikely in these conditions.

These conditions and geometry were also analyzed for wedge failure incorporating the base. The water table was set at 25 feet for all calculations. To try and model the effect of saturated conditions, Calder & Workman lowered the friction angle in the floor of the spoil to initiate failure. Their results are shown in Table 8. This doesn’t seem like the best methodology to model saturated conditions. According to accepted soil mechanics texts, such as Das (2001), water content does not theoretically affect the angle of internal friction. Rather effective stress is decreased by increase of pore pressure that acts to balance overlying stresses. This relationship is governed by the Mohr-Coulomb failure

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Density</th>
<th>Cohesion</th>
<th>Angle of Internal Friction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Glacial Till</td>
<td>110</td>
<td>350</td>
<td>23</td>
</tr>
<tr>
<td>Mixed Spoil</td>
<td>102.4</td>
<td>432</td>
<td>25</td>
</tr>
<tr>
<td>Floor Materials</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fat Clay</td>
<td>115</td>
<td>500</td>
<td>10</td>
</tr>
<tr>
<td>Silty Sand</td>
<td>105.5</td>
<td>175</td>
<td>30</td>
</tr>
<tr>
<td>Sandy Fat Clay</td>
<td>110</td>
<td>450</td>
<td>15</td>
</tr>
<tr>
<td>Silty Clay with Sand</td>
<td>105</td>
<td>300</td>
<td>23</td>
</tr>
</tbody>
</table>
criterion, which will be discussed later. In this study, setting the angle of internal friction to zero degrees would essentially be saying that pore pressure is equal to overlying stress from overburden and dragline loading. This may be the case; however, it should be incorporated in the correct manner and the current approach makes this study difficult to understand.

Analysis was subsequently performed for a 25 degree face angle in order to determine it’s effect on stability. Circular arc failures and wedge failures were analyzed as in the previous geometry. As would be inspected, stability improved; however, for the zero angle of friction scenario, wedge failure was still likely beneath the dragline bench. These results are shown in Table 9.

<table>
<thead>
<tr>
<th>Failure Type and Location</th>
<th>Bench Height</th>
<th>Water Table Height</th>
<th>Dragline</th>
<th>FS</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Circular Arc - Bench</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.057</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Circular Arc - Bench</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.086</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Circular Arc - Bench</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>0.992</td>
<td>Cohesion decreased to 250 psf in Mixed Spoil</td>
</tr>
<tr>
<td>Circular Arc - Bench</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.008</td>
<td>Cohesion decreased to 250 psf in Mixed Spoil</td>
</tr>
<tr>
<td>Circular Arc - Total Spoil</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.737</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Circular Arc - Bench</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.098</td>
<td>Compacted Toe until C = 1440 psf</td>
</tr>
<tr>
<td>Circular Arc - Bench</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.099</td>
<td>Compacted Toe until C = 1440 psf</td>
</tr>
<tr>
<td>Circular Arc - Total Spoil</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.754</td>
<td>Compacted Toe until C = 1440 psf</td>
</tr>
<tr>
<td>Circular Arc - Bench</td>
<td>95</td>
<td>45</td>
<td>Yes</td>
<td>0.945</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Circular Arc - Bench</td>
<td>95</td>
<td>45</td>
<td>Yes</td>
<td>0.977</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Circular Arc - Total Spoil</td>
<td>95</td>
<td>45</td>
<td>Yes</td>
<td>1.61</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Circular Arc - Total Spoil</td>
<td>95</td>
<td>45</td>
<td>Yes</td>
<td>1.551</td>
<td>Cohesion decreased to 250 psf in Mixed Spoil</td>
</tr>
<tr>
<td>Circular Arc - Bench</td>
<td>95</td>
<td>45</td>
<td>Yes</td>
<td>0.986</td>
<td>Compacted Toe until C = 1440 psf</td>
</tr>
<tr>
<td>Circular Arc - Bench</td>
<td>95</td>
<td>45</td>
<td>Yes</td>
<td>0.994</td>
<td>Compacted Toe until C = 1440 psf</td>
</tr>
</tbody>
</table>
Table 8. Spoil Side Mining - 35 Degree Face Angle, 25 Foot Water Table

<table>
<thead>
<tr>
<th>Failure Type and Location</th>
<th>Bench Height</th>
<th>Water Table Height</th>
<th>Dragline</th>
<th>FS</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wedge Failure - Bench</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.536</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Wedge Failure - Bench</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.158</td>
<td>Base Angle of Friction = 6 degrees</td>
</tr>
<tr>
<td>Wedge Failure - Bench</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>0.949</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
<tr>
<td>Wedge Failure - Spoil Peak</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.515</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Wedge Failure - Spoil Peak</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.045</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
<tr>
<td>Wedge Failure - Beyond Spoil Peak</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.603</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Wedge Failure - Beyond Spoil Peak</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.083</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
</tbody>
</table>

From these analyses, it can be seen that this configuration is not particularly stable under all conditions, especially when wet and for steeper face angles. Calder & Workman performed two additional analyses to analyze possibilities of buttressing the spoil bench. The first method was to extend the spoil bench out to the unmined overburden, which would provide toe support. This would also effectively lower the height of the spoil bench.

Table 9. Spoil Side Mining - 25 Degree Face Angle, 25 - 45 Foot Water Table

<table>
<thead>
<tr>
<th>Failure Type and Location</th>
<th>Bench Height</th>
<th>Water Table Height</th>
<th>Dragline</th>
<th>FS</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Circular Arc - Bench</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.118</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Circular Arc - Bench</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.13</td>
<td>Cohesion decreased to 250 psf in Mixed Spoil</td>
</tr>
<tr>
<td>Circular Arc - Total Spoil</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.636</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Circular Arc - Bench</td>
<td>95</td>
<td>45</td>
<td>Yes</td>
<td>1.068</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Circular Arc - Total Spoil</td>
<td>95</td>
<td>45</td>
<td>Yes</td>
<td>1.513</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Wedge Failure - Bench</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.577</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Wedge Failure - Bench</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>0.928</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
<tr>
<td>Wedge Failure - Before Spoil Peak</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.892</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
<tr>
<td>Wedge Failure - Beyond Spoil Peak</td>
<td>95</td>
<td>25</td>
<td>Yes</td>
<td>1.114</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
</tbody>
</table>

The second method calls for the dragline to dump overburden on the toe of the spoil bench. This overburden acts to further stabilize the spoil bench. For analysis, they chose a 36 foot high buttress that would fill the gap between the overburden and spoil bench. The results for both buttressing methods are shown in Table 10.
Table 10. Spoil Side Mining - 25 Degree Face Angle, 25 - 45 Foot Water Table

<table>
<thead>
<tr>
<th>Failure Type and Location</th>
<th>Bench Height</th>
<th>Water Table Height</th>
<th>Dragline</th>
<th>FS</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buttressing Method 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Circular Arc - Bench</td>
<td>80</td>
<td>40</td>
<td>Yes</td>
<td>1.234</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Circular Arc - Extended Bench</td>
<td>80</td>
<td>40</td>
<td>Yes</td>
<td>1.262</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Circular Arc - Near Spoil Peak</td>
<td>80</td>
<td>40</td>
<td>Yes</td>
<td>2.107</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Circular Arc - Bench</td>
<td>80</td>
<td>60</td>
<td>Yes</td>
<td>1.053</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Circular Arc - Near Spoil Peak</td>
<td>80</td>
<td>60</td>
<td>Yes</td>
<td>2.078</td>
<td>Initial Soil Parameters</td>
</tr>
<tr>
<td>Wedge Failure - Bench</td>
<td>80</td>
<td>40</td>
<td>Yes</td>
<td>1.577</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
<tr>
<td>Wedge Failure - Offset Bench</td>
<td>80</td>
<td>40</td>
<td>Yes</td>
<td>0.928</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
<tr>
<td>Wedge Failure - 1 Peak Back</td>
<td>80</td>
<td>40</td>
<td>Yes</td>
<td>1.892</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
<tr>
<td>Wedge Failure - 2 Peaks Back</td>
<td>80</td>
<td>40</td>
<td>Yes</td>
<td>1.114</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
<tr>
<td>Wedge Failure - 2 Peaks Back</td>
<td>80</td>
<td>60</td>
<td>Yes</td>
<td>0.977</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
<tr>
<td>Wedge Failure - Bench</td>
<td>80</td>
<td>60</td>
<td>Yes</td>
<td>1.533</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
<tr>
<td>Buttressing Method 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Circular Arc - Bench</td>
<td>95</td>
<td>45</td>
<td>Yes</td>
<td>1.397</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
<tr>
<td>Circular Arc - Past Spoil Peak</td>
<td>95</td>
<td>45</td>
<td>Yes</td>
<td>1.272</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
<tr>
<td>Wedge Failure - Bench</td>
<td>95</td>
<td>45</td>
<td>Yes</td>
<td>1.185</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
<tr>
<td>Wedge Failure - Offset Bench</td>
<td>95</td>
<td>45</td>
<td>Yes</td>
<td>1.416</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
<tr>
<td>Wedge Failure - Near Spoil Peak</td>
<td>95</td>
<td>45</td>
<td>Yes</td>
<td>1.033</td>
<td>Base Angle of Friction = 0 degrees</td>
</tr>
</tbody>
</table>

Both buttressing techniques gave satisfactory results. Calder & Workman found the second method to provide additional support and therefore recommended this method be adopted. The only downfall to this second method is that it requires additional rehandle from dumping the overburden to brace the bench.

Conclusions from Past Studies
Calder & Workman effectively provided structurally sound methodologies for mining the Hagel beds with bench and spoil side mining. They analyzed different heights and repose angles to determine the relationship to stability. Concern was given to saturated conditions in the pit floor and changes in strength parameters. The method for incorporating water and pore pressure into the design was to lower the angle of internal
friction to zero for saturated conditions. I do not have the actual calculations from these studies, but in the literature I have found that this is not the most reliable way to model these conditions. Instead a pore pressure and effective stress approach should be taken.

**Partial Validation of Previous Works**

Parts of Calder and Workman’s first study (bench side mining) were reproduced here to determine the mechanics involved in a highwall stability study.

**Drawdown of the Water Table**

The shape of the water table prior to mining can be considered flat close to the highwall, as the variations that occur over small lengths cannot be accurately determined and have little effect on stability. The parabolic shape of the water table after the pit is cut should be determined, however, as the large drawdown will have a positive effect on stability.

The steady state parabolic shape of the water table does not depend on aquifer characteristics, and therefore can be determined entirely from geometry. Adaptations from Miller(1981) outline the procedure of determining the parabolic drawdown shape, and will be summarized below. The analysis for this project was carried out in Excel and is in the file “Parabolic Drawdown Iteration”. For a specific highwall angle, there is only one parabolic curve that will satisfy steady state drawdown and it can be determined iteratively. The dimensions relating the highwall and water table (marked with a delta) are shown in Figure 14.

The shape of the water table can be modeled by the general equation of a parabola:

\[(y')^2 = Cx',\] where \(C\) is a constant (Eq. 1)

The coordinate system is defined by the wall of the excavation \((y')\) and the perpendicular axis \((x')\). Taking the derivative of this equation defines the slope at any point relative to the \(x'\) and \(y'\) axes as:

\[
\frac{dy'}{dx'} = \frac{1}{2} \sqrt{\frac{C}{x'}} \quad (Eq. 2)
\]
The goal is to determine the two dimensions, \( r \) and \( a \), which define the horizontal distance from the seepage face to point of null drawdown and the length of the seepage face, respectively.

The closest position to the wall where the water table height, \( h \), is unaffected is marked by the coordinates \((x_r', y_r')\) where:

\[
x_{r}' = (r + a \cos \beta) \sin \beta - h \cos \beta \quad \text{(Eq. 3)}
\]

![Figure 18. Dimensions for Parabolic Drawdown Iteration](image)

At the point of null drawdown, the slope of the water table is horizontal. In the \( x' \) and \( y' \) coordinate system this slope is easily seen to be:
\[ \frac{dy'}{dx'} = \frac{1}{\tan \beta} \quad \text{(Eq. 4)} \]

The constant, \( C \), can now be determined by inserting \( x_r' \) into equation 2 and combining it with equation 4. By doing this we obtain:

\[ C = \frac{4x_r'}{\tan^2 \beta} \quad \text{(Eq. 5)} \]

where \( x_r' = (r + a \cos \beta) \sin \beta - h \cos \beta \) (from Eq. 3)

Putting our new value for \( C \) into the original equation (eq. 1) and knowing the relationship for \( x_r' \), we can solve for \( y_r' \):

\[ y_r' = \frac{2}{\tan \beta} (r \sin \beta - h \cos \beta + a \cos \beta \sin \beta) \quad \text{(Eq. 6)} \]

We can relate the point of zero drawdown to the length of the seepage face, as our coordinate system origin is the top of the face and the base of the aquifer is the bottom. This length, \( a \), is found to be equal to:

\[ a = \frac{1}{1 + \cos^2 \beta} \left( \frac{h}{\sin \beta} - \frac{r \sin \beta - h \cos \beta}{\tan \beta} \right) \quad \text{(Eq. 7)} \]

The length of a parabolic curve defined by the equation above relative to our coordinate system and the top of the seepage face can be expressed as:

\[ L = \frac{y_r'}{2} \sqrt{\frac{(2y_r')^2}{C} + 1} + \frac{C}{4} \ln \left( \frac{2y_r'}{C} + \sqrt{\frac{(2y_r')^2}{C} + 1} \right) \quad \text{(Eq. 8)} \]

One more condition allows us to interpret the above geometry iteratively. The length of the seepage face, \( a \), is related to the length of the parabolic curve and the height of the original water table by:

\[ a = \sqrt{L^2 + \left( \frac{h}{\sin \beta} \right)^2} - L \quad \text{(Eq. 9)} \]
The routine is as follows:

1. Guess a value for \( r \) and calculate \( a \) with equation 7.
2. Calculate \( C \) from equations 5 and 3.
3. Calculate \( (y_r') \) with equation 6
4. Calculate \( L \) from equation 8
5. Check the first calculated seepage face length \( a \) from equation 7 with the verifying length determined from equation 9 with our value of \( L \). Repeat if necessary until they are equal.

Iterating on this routine defines the height of the water table at the highwall face and determines the distance to the point of zero drawdown.

It should be noted here that using this method for computing drawdown gave very good results. The iteration is shown below in Table 11 for a 44 foot depth to the water table. Notice how the values for \( a_1 \) and \( a_2 \) are very close at \( r \) equal to 34. This means that the calculated seepage face is 4.85 feet and the distance to zero drawdown is 34 feet. This value is remarkably close to the values measured from Calder & Workman’s diagrams.

The height of this parabolic curve relative to the base of the aquifer can be determined mathematically at any point, \( x \). The relationship is:

\[
h_x = \left[ h^2 - \frac{(h^2 - (a \cos \beta)^2)}{r} \right]^{\frac{1}{2}} \quad (Adapted \ from \ Fetter, \ 2001)
\]

It is often times more convenient; however, to graph the parabolic curve in a drafting package such as AutoCAD once the values of “\( a \)” and “\( r \)” are known and then measure the distance from the base of the aquifer to the water table to determine \( h_x \).

The height of the water table is called “head”. Head is the height that water rises above a fixed datum, and gives the value of potential energy and pressure in the aquifer at that
particular point. By using the above formula to find head at two points very close together and dividing by the length, the hydraulic gradient, which is the rate of change of hydraulic head in a given direction, can be approximated at the seepage face.

Table 11. Parabolic Drawdown Iteration
(44 foot depth to water)

<table>
<thead>
<tr>
<th>Known</th>
<th>Equations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free Surface Height</td>
<td>a1 C yr L a2</td>
</tr>
<tr>
<td>Angle of Repose 0.95931</td>
<td>6.584746 36.26416 25.89529 32.79381 5.376183</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Guess r</th>
</tr>
</thead>
<tbody>
<tr>
<td>a1 C yr L a2</td>
</tr>
<tr>
<td>1st iteration 30 6.584746 36.26416 25.89529 32.79381 5.376183</td>
</tr>
<tr>
<td>2 31 6.153158 37.47297 26.75847 33.84846 5.231364</td>
</tr>
<tr>
<td>3 32 5.721569 38.68177 27.62165 34.90311 5.093658</td>
</tr>
<tr>
<td>4 33 5.289981 39.89058 28.48482 35.95777 4.962587</td>
</tr>
<tr>
<td>5 34 4.858393 41.09938 29.348 37.01242 4.837713</td>
</tr>
<tr>
<td>6 35 4.426805 42.30819 30.21118 38.06707 4.718631</td>
</tr>
<tr>
<td>7 36 3.995216 43.51699 31.07435 39.12172 4.604969</td>
</tr>
<tr>
<td>8 37 3.563628 44.7258 31.93753 40.17638 4.496385</td>
</tr>
<tr>
<td>9 38 3.13204 45.9346 32.80071 41.23039 4.392563</td>
</tr>
<tr>
<td>10 39 2.700452 47.14341 33.66388 42.28568 4.293211</td>
</tr>
<tr>
<td>40 2.268863 48.35222 34.52706 43.34033 4.19806</td>
</tr>
<tr>
<td>41 1.837275 49.56102 35.39024 44.39499 4.106861</td>
</tr>
<tr>
<td>42 1.405867 50.76983 36.25341 45.44964 4.019382</td>
</tr>
<tr>
<td>43 0.974099 51.97863 37.11659 46.50429 3.93541</td>
</tr>
</tbody>
</table>

Note: Obviously this drawdown determination doesn’t take into account the interaction between the confined coal aquifers and the water table. This iteration assumes an impermeable base, which is the exact opposite of what is present. The water table and the confined aquifers will undoubtedly interact. I have been looking in the literature to try and find a solution to this problem and a method to quantify this interaction. Most seepage problems I have seen are defined with flownets or by knowing the point of zero drawdown or length of the seepage face. In this problem, both are unknown and it successfully complicates the issue. One possible solution to this problem may be using a finite difference program, such as FLAC.
Discharge and Pore Pressure

Two groundwater issues must be considered in order to have a successful pit design:

1. Discharge
2. Pore Pressure

1. Discharge

Although not as detrimental to stability issues as pore pressure, discharge, which is the volume of water seeping through the highwall face per unit time, can cause problems with operations. It can be dealt with by draining the aquifers, pumping, or channeling.

In the past, Falkirk has used a boxcut technique and pumping to deal with groundwater discharge issues. The boxcut is an initial cut in which dewatering begins. Trenches are made along the highwall to channel water discharging out of the wall toward a sump usually located on one end; however, multiple sumps could be possible depending on where water accumulates. As mining progresses to future cuts up gradient, the potentiometric water surface continues to be drawn down from drainage and the water is pumped from the holding sump to a holding pond away from mining. The discharge can be estimated with the application of Darcy’s law (Fetter, 2001). Darcy’s law is defined as:

\[
Q = -KA \frac{dh}{dl}
\]

where \( Q \) = discharge, \( K \) = hydraulic conductivity, \( A \) = x-sectional area of discharge face,

\[
\frac{dh}{dl} = \text{hydraulic gradient}
\]

Seepage velocity (darcy velocity, \( q \)) can be thought of as the speed that a flux of water moves through a soil. It can be related to discharge (\( Q \)) by:

\[
q = \frac{Q}{A}
\]

Discharge will be the greatest initially and decline until it finally reaches an approximated steady state drawdown. The greatest amount of discharge will occur from
the coal and sand aquifers. Some discharge will also occur from the overlying clay and till, but this will be negligible compared to the coal seams because of the much lower value of hydraulic conductivity of the clay and till. Hydraulic conductivity is the ability of water to flow through a specific porous medium. It depends on the properties of water, porosity, and the interconnectedness of aquifer pores. Average hydraulic conductivities of the most typical water bearing units are summarized below in Table 12. Also listed are values for specific yield and storativity for selected aquifers. Specific yield and storativity are properties that define the ability of an aquifer to be dewatered. Particularly, specific yield is the amount of porosity that dewatered due to the effects of gravity. Specific storage is a property that defines the compressibility of the aquifer skeleton and compressibility of water. Both are applicable toward all aquifers; however, unconfined aquifer dewatering is dependant on specific yield while conversely storativity governs confined aquifer dewatering. These properties were obtained from overdamped response slug tests performed by Falkirk Mine (James, 2003). This procedure is summarized in Fetter (2001). It consists of adding water, removing water, or dropping an object to displace water in a well in order to observe the hydraulic response of the surrounding formation. In this case, a pipe (slug) was used.

Dewatering is possible for the highwall; however, for tills and clay the specific retention can be upward of 90% of the effective porosity. The specific retention is the porosity of a soil that will retain water from gravity drainage due to the effects of cohesion and adhesion. For this reason, after dewatering there will still be the majority of the water present in the clays and till. This water held in tension will reduce the effective stress as the dragline loads the wall. As well, the water table in this clayey overburden material will fluctuate dramatically with relatively little precipitation. With a specific yield of around 0.02, one inch of water seeping to the water table would cause approximately a fifty inch rise. This shows the highly variable nature of the groundwater table and the detrimental impact that excessive rainfall could have on slope stability.
Table 12. Water Bearing Unit Properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Hydraulic Conductivity (ft/day)</th>
<th>Specific Yield (Sy)</th>
<th>Storativity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Glacial Till</td>
<td>0.3 to $10^3$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Glacial Channel Fill</td>
<td>8.6</td>
<td>0.14</td>
<td>0.0001 to 0.00045</td>
</tr>
<tr>
<td>Glacial Channel Fill Gravels</td>
<td>104 to 6,739</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hagel A Seam</td>
<td>0.16 to 15.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hagel B Seam</td>
<td>0.01 to 15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C Sand</td>
<td>0.01 to 10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tavis Seam</td>
<td>0.01 to 20</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

2. Pore Pressure

Pore pressure must be considered in stability calculations. It is a stress that acts to counteract overlying weight of soils and decrease the frictional resistance to sliding. It is equal in all directions. Under static conditions, pore pressure can be found for a point, A, in an unconfined aquifer by:

$$
\mu = (\gamma_w)h
$$

where $\gamma_w = \text{unit weight of water}$, $h = \text{point's distance below groundwater table}$

![Diagram of pore pressure at point A](image-url)
The pressure in a coal seam can be calculated by similar means by using the height, \( h \), that the water rises above the bottom of the well. This pressure is approximately equal everywhere in a confined aquifer. For lack of piezometers in the bedrock, pressure at a point between two confining aquifers can be estimated by linearly interpolating between the heads of the two confined aquifers.

A stress will also occur because of seepage, or movement of water. According to Das (2001) this stress is equal to:

\[
\sigma_S = \frac{dh}{dz} H \gamma_w
\]

where \( \frac{dh}{dz} \) = hydraulic gradient, \( H = \) distance between two points, \( \gamma_w = \) unit weight of water

If this seepage stress occurs because of flow downward, such as between coal layers, it effectively reduces pore pressure. If it occurs because of upward flow, it increases effective pore pressure. This stress is hard to calculate accurately and is insignificant compared to other stresses in the highwall. It can be ignored in this case.

**Soil Stresses**

The strength of the highwall is governed by the Mohr-Coulomb failure criterion:

\[
\tau = C + \sigma' \tan \phi
\]

where \( \tau = \) shear stress on plane, \( C = \) cohesion of soil particles, \( \sigma' = \) effective stress, and \( \phi = \) angle of internal friction

It is a measure of the driving forces (shear stress) relative to the resisting forces on a plane. Cohesion, \( C \), is the measure of the attraction between particles in a soil. The angle of internal friction is a measure of the frictional resistance to sliding and is a function of effective stress.

Effective and shear stress are the sums of all the normal and parallel components of stresses acting on a plane, respectively. In this study, these combined stresses can be
broken down into normal overburden stress, pore pressure, seepage stress, and normal surcharge loading:

\[ \sigma' = \sigma_n + \sigma_d - \mu \pm \sigma_{ss} \]

where \( \sigma' \) = effective stress, \( \sigma_n \) = overburden stress (normal), \( \sigma_d \) = dragline surcharge loading (normal), \( \mu \) = pore pressure, and \( \sigma_{ss} \) = seepage stress

\[ \tau = \sigma_s + \sigma_{ds} \]

where \( \sigma_s \) = overburden stress (shear), \( \sigma_{ds} \) = dragline surcharge loading (shear)

Pore pressure and seepage stress were discussed earlier. They do not contribute to shear stress, as they only act normal to planes.

The overburden stress, is the stress due to weight of the soil and water overlying the plane. It acts vertically downward. This can be calculated by:

\[ \sigma_n = (\gamma_{\text{insitu}})h \]

where \( \gamma_{\text{insitu}} \) = in place unit weight of soil, and \( h \) = height of soil mass above plane

The dragline surcharge loading is the stress that is transmitted through the subsurface from the dragline weight onto the subsurface. When a saturated sample of soil in equilibrium is stressed by an external load, the pore water will initially support the entire normal load due to its incompressible nature (Das, 2001). The excess pore pressure at this time, \( t = 0 \), can be expressed as:

\[ \Delta \mu_{t=0} = \frac{P}{A} \]

where \( P \) = weight of load applied to soil, \( A \) = area of application

Over time, the material will consolidate until enough water has dissipated and excess pressure becomes zero. At this time, the soil will now carry the entire load. The problem is that pore pressure does not help to carry the shear stress that is applied to the plane. This will momentarily affect the stability of the soil according to the Mohr-Coulomb
failure criterion, as the shear stress (driving force) is increased while normal stress (resisting force) is unaffected. Material properties control the time that it takes to alleviate this excess pressure. In sand, excess pore pressure will dissipate almost instantaneously. Conversely, clay’s low permeability traps water and pore pressure decreases slowly with applied loads. Clays do have relatively high cohesion values, which in most engineering situations tend to govern their strength. For this reason, the strength of clay is often considered to be equivalent to its undrained shear strength or value of cohesion. Undrained triaxial tests, as were performed by Midwest Testing, most accurately model this strength.

In this study, the transient state of pore pressure increase and alleviation will not be considered. The dragline moves relatively slow, so we will assume that all pore pressure increases will immediately be followed by pressure alleviation. Based on this assumption, surcharge loading at a point (Figure 19) located at depth, z, in the subsurface and r units away from the center of a circularly loaded area of radius, R (dragline), can be calculated by (Das, 2001):

$$\sigma_d = q(A' + B')$$

and $$q = \frac{W_d}{A_t}$$

where $$q = \text{stress imposed by dragline at surface}, W_d = \text{weight of dragline}, A_t = \text{area of dragline contact with ground}, I = \text{influence of dragline stress}$$

![Figure 19. Relationship Between Dragline Location and Stress at Point A](image-url)
The values $A'$ and $B'$ are functions of $z/R$ and $r/R$, respectively. They are very tedious to calculate and are graphed below in Figure 20 (Das, 2001).

Failure Mechanisms of Highwall or Spoil
Once the stresses acting in the highwall are known, a failure mechanism must be chosen to begin analyzing the strength. For most practical applications, soil slopes can be considered homogeneous. In a homogeneous mass, the plane of least resistance usually takes the shape of a circular arc (Hoek and Bray, 1974) and most soil analyses are carried out with this failure method. Other failure methods, such as wedge and translational failures noted earlier, usually occur because there is a discontinuity, such as a fault or

Figure 20. Influence Factor Chart for Dragline Surcharge Loading (Das, 2001)
joint, that is markedly weaker than an intact circular arc plane. These are the types of
failure Falkirk has most often seen in the past. In these instances, there is a discontinuity,
such as the sharply dipping bed shown earlier or the boundary between two spoil peaks,
that governs the failure of the highwall or spoil. These failure planes are difficult to
detect and even more difficult to strength test, therefore trying to design the highwall
around these discontinuous failure mechanisms is problematic. The following limit
equilibrium analysis assumes circular arc failure.

Circular Arc Failure
Bishop’s Method of Slices is the most widely used numerical method of evaluating
circular arc failure in soils. It allows for variations in lithology and pore pressure as well
as takes into account stresses induced on the sides of each slice. Carrying out a
calculation with this method is tedious for two reasons. First, the location of the plane of
weakness is indeterminate; that is, it must be found by guessing and subsequently honing
in on the weak plane based on past calculations. Second, the process of finding the factor
of safety, or resisting forces vs. sliding forces, must be found iteratively.

In this method, it is assumed that the failure can be modeled in 2-D. This assumption
implies that the highwall is infinitely long and changes in stresses induced in the third
dimension can be ignored.

An example failure plane is shown below in Figure 21. The highwall profile was drawn
in AutoCAD. The failure plane is defined by an arc that has a center defined by the circle
with the cross through it. The failure arc toes in immediately above the Hagel A coal
seam and emerges 20 feet to the left of the center of the dragline. The dragline is
positioned 20 feet from the edge of the highwall. The lithologic units are shown with
their relative thicknesses as defined by the highwall in Calder & Workman’s first study.
The free surface of the water table is shown at steady state as determined by the
drawdown equation evaluated above. Tested strength data is that from Midwest Testing
and is shown in Table 13. The failure wedge is broken into slices of arbitrary dimension.
In each slice, the volume of each soil type can be calculated based upon geometry and the
The sum of all volumes multiplied by associated unit weights gives the weight of soil bearing down on the bottom of the slice.

The forces acting on a reference slice are shown below to the upper left of the highwall. Note that this analysis is performed with force vector algebra, so all stresses will be converted to forces by multiplying by the area on the base of the failure plane. For small angles, \( \alpha \), this area can be approximated by the width, \( b_n \), of the slice.

The normal reaction force, \( N_r \), is shown perpendicular to the plane that is oriented at an angle, \( \alpha \), to the vertical. It is sum of the overburden and dragline forces acting on the plane. The pore pressure force, is termed \( p \). The other forces shown on the sides are due to the interaction with the other slices.

According to the Mohr – Coulomb failure criterion we know:

\[
\tau_f = C' + \sigma' \tan \phi \quad (\text{Eq. 1})
\]

which represents the shear strength of the surface at failure

Prior to failure, a shear stress will develop along the plane that is less than or equal to the shear stress represented above:

\[
\tau_d = C_d + \sigma' \tan \phi_d \quad (\text{Eq. 2})
\]

The factor of safety, is a relationship that provides us with a relative value for strength, it is defined by combining equations 1 and 2:

\[
F_s = \frac{\tau_f}{\tau_d} \quad (\text{Eq. 3})
\]

If \( F_s \) is equal to or greater than 1, the slope is stable.
Using this relationship for factor of safety, the shear force, \( T_r \), that develops on the plane, can be written as:

\[
T_r = (N_r - p\Delta L_n) \tan \phi'd + C_d \Delta L_n = (N_r - p\Delta L_n) \left( \frac{\tan \phi'}{F_s} \right) + \frac{C'L}{F_s} \quad (\text{Eq. 4})
\]

where \( \Delta L_n = \text{length of the failure surface which is approximately equal to width of slice} \)

The forces on the sides of the slice can be simplified by allowing \( P_n - P_{n+1} = \Delta P = 0 \), and \( T_n - T_{n+1} = \Delta T = 0 \), as these forces are negligible compared to the dragline and overburden forces.

Summing forces in the \( y \) direction gives:

\[
O + D = N_r \cos \alpha + \left[ \left( \frac{(N_r - p\Delta L_n) \tan \phi'}{F_s} + \frac{c' \Delta L_n}{F_s} \right) \right] \sin \alpha \quad (\text{Eq. 5})
\]

The moment about the point defining the circular arc center for all slices (1 thru m) on the failure plane yields:

\[
\sum_{n=1}^{n=m} (O + D) r \sin \alpha = \sum_{n=1}^{n=m} T_r r \quad (\text{Eq. 6})
\]

where \( r = \text{radius of the circle defining the failure arc} \), \( O = \text{overburden weight} \), \( D = \text{dragline surcharge force} \)

\[
T_r = \frac{1}{F_s} (c' \Delta L_n + (N_r - p\Delta L_n) \tan \phi') \quad (\text{Eq. 7})
\]

We can now substituting equations 5 and 7 into equation 6 and allowing \( \Delta L_n = b_n \), :

\[
\sum_{n=1}^{n=m} \left[ c' b_n + (O + D - pb_n) \tan \phi' \right] \frac{1}{m_\alpha} = \frac{\sum_{n=1}^{n=m} (O + D) \sin \alpha}{\sum_{n=1}^{n=m} \sin \alpha}
\]

where \( m_\alpha = \cos \alpha + \frac{\tan \phi' \sin \alpha}{F_s} \)
This equation can now be solved iteratively. The method is as follows:

1. Guess a factor of safety. Use this value in the equation for $m_{cz}$ to calculate the new factor of safety.

2. If the guessed value and calculated value are not equal, use the new factor of safety to recalculate. Repeat until the factor of safety converges.

Figure 21. Example Failure Plane for Bishop’s Method of Slices Calculation
Table 13. Highwall Lithology with Tested Strength Parameters

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Thickness</th>
<th>Depth</th>
<th>Density</th>
<th>Cohesion</th>
<th>Angle of Internal Friction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Baseline</td>
<td>Inter.</td>
</tr>
<tr>
<td>Glacial Till</td>
<td>12</td>
<td>0-12</td>
<td>124</td>
<td>1008</td>
<td></td>
</tr>
<tr>
<td>Fat Clay</td>
<td>8</td>
<td>12-20</td>
<td>119</td>
<td>800</td>
<td>1000</td>
</tr>
<tr>
<td>Silty Sand</td>
<td>10</td>
<td>20-30</td>
<td>120</td>
<td>200</td>
<td></td>
</tr>
<tr>
<td>Fat Clay</td>
<td>14</td>
<td>30-44</td>
<td>120</td>
<td>800</td>
<td>1000</td>
</tr>
<tr>
<td>Clayey Sand</td>
<td>16</td>
<td>44-60</td>
<td>120</td>
<td>1000</td>
<td></td>
</tr>
<tr>
<td>Coal</td>
<td>9</td>
<td>60-69</td>
<td>77</td>
<td>3744</td>
<td></td>
</tr>
<tr>
<td>Parting</td>
<td>20</td>
<td>69-89</td>
<td>128</td>
<td>1000</td>
<td></td>
</tr>
<tr>
<td>Coal</td>
<td>3</td>
<td>89-92</td>
<td>77</td>
<td>3744</td>
<td></td>
</tr>
</tbody>
</table>

Results of Calder & Workman Verification

The results obtained from using these methods are in good agreement with Calder & Workman's initial study. The initial geometry used was a 55 degree highwall with the dragline loading. The first trial was for a 20 foot setback of the dragline. The failure surfaces analyzed are shown in Figure 22. Calder & Workman found this situation to have a minimum factor of safety of around 0.93. This is what was found in this study. The distance to the weakest failure plane from the edge of the pit matches Calder & Workman's results. The other planes also prove that this surface is in the weakest location with the relative geometry.

Limit equilibrium analysis tends to produce a conservative factor of safety (Hoek and Bray, 1971). In Bishop's Method of Slices, it was assumed that the normal stress was concentrated at a single point on the bottom of each slice. Froehlich (1955) performed a study showing that this actually corresponds to a lower bound on the factor of safety. An alternate upper bound on the solution could be obtained by assuming the normal stress is located at the ends of each slice. In a corresponding study by Lambe and Whitman (1969), the lower bound and upper bound were 1.27 and 1.62, respectively. This shows that Bishop's Method of Slices is a safe way of determining geometry in a highwall slope; however, it may not be the most accurate. Therefore, it is prudent to examine other methods for determining the factor of safety, especially in cases such as mining the Tavis, where it will be necessary to maximize the height of the highwall.
Finite Difference Sensitivity Study

Finite difference methods are an alternative way to analyze the strength of a slope. In finite difference methods, engineering analysis of highwall stability will be performed using FLAC. FLAC uses a finite difference method to model the behavior of engineering materials that react to induced and field stresses according to some prescribed stress-strain law and failure criterion. The uniqueness of FLAC is that it allows the material being modeled to yield and flow throughout time so that the dynamics behind failure can be visualized. This is beneficial when compared to the traditional limit equilibrium analysis techniques, such as Bishop's Method of Slices, that only allow a factor of safety to be determined for a chosen failure surface. Modeling with FLAC allows the user to determine if failure of a surface or plastic flow within a material is excessive. Since all...
failures of material do not always constitute redesign, modeling with FLAC can be a very valuable tool in engineering analysis.

A FLAC model was built and compared to the analysis in the first part of the report, which used Bishop's Method of Slices. FLAC was then used to analyze the stability of the Tavis Creek highwall. All codes used in this study can be found in the appendix at the end of the report.

**FLAC Model Comparison**
The input code for this model can be found in Appendix 3, called "Hagel Pit Code". This model was designed to analyze the same highwall geometry that resulted in a factor of safety of 0.93 by Calder & Workman, Inc. and in the first part of this paper using Bishop's Method of Slices.

**Problem**
The goal was to analyze the strength of a 60 foot highwall. This problem is represented by Figure 22. It should be noted that the coal and parting layers were present during this model. This is consistent with the analysis performed using Bishop's Method of Slices, since the weakest slice did not penetrate the coal layers. The goal was to determine the difference, if any, in the factor of safety produced by the two analysis methods. It is again assumed that a 2-D analysis will accurately reflect the highwall strength. This assumption must be made in order to use FLAC, and it implies that stresses in the third dimension balance out and can be ignored.

**Initial Material Conditions**
A 100 x 40 grid was constructed representative of the highwall lithology analyzed by Calder and Workman. This grid size was chosen in order to best preserve a 1:1 length to height ratio of the grid nodes. This is important for preserving the validity of the solution. The stress-strain law and failure criterion used was elastic and Mohr-Coulomb, respectively. These constitutive models have been historically used to model the movement and failure in geotechnical materials, such as soils. In essence, they assumed that the movement of the soil can be modeled in the same manner as a spring. That is, the soil compresses and develops stresses according to a constant coefficient known as
the bulk modulus. The Mohr-Coulomb failure criterion determines the point at which shearing forces will overcome frictional and cohesive forces and the soil fails. Other models, such as Cam-Clay, have also been used to constitutively model soils. The main problem in using these other models in this study is that the input data required for the soil strength and elastic parameters is unavailable.

The lithology and properties are listed in Table 14. These were incorporated into the grid and dimensioned according to depth and thickness. The entire grid block dimension was explicitly defined at 400ft wide by 100ft tall. This was done in order to assure that the failure surface of the highwall and induced stresses of the dragline do not interact closely with the right and left boundaries of the grid and again preserve the integrity of the solution. It should be noted that units cannot be explicitly defined in FLAC; however, consistent sets of units must be used in order for the program to model properly. For this project, imperial units were used to maintain consistency with Calder and Workman's study. These units can be seen in Table 14.

Table 14. System of Units

<table>
<thead>
<tr>
<th>Length</th>
<th>Density</th>
<th>Force</th>
<th>Stress</th>
<th>Gravity</th>
<th>Bulk Modulus</th>
<th>Permeability</th>
</tr>
</thead>
<tbody>
<tr>
<td>ft</td>
<td>slugs/ft³</td>
<td>lbf</td>
<td>lbf/ft²</td>
<td>ft/sec²</td>
<td>lb/ft²</td>
<td>ft²/sec/slug</td>
</tr>
</tbody>
</table>

Table 15. Baseline Elastic and Strength Properties for the Hagel Highwall

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Depth (ft)</th>
<th>Density (slugs/ft³)</th>
<th>Bulk Modulus (lb/ft²)</th>
<th>Shear Modulus (lb/ft²)</th>
<th>Cohesion (lb/ft²)</th>
<th>Angle of Internal Friction (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Glacial Till</td>
<td>0-12</td>
<td>3.57</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>1008</td>
<td>20</td>
</tr>
<tr>
<td>Fat Clay</td>
<td>20-Dec</td>
<td>3.57</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>800</td>
<td>10</td>
</tr>
<tr>
<td>Silty Sand</td>
<td>20-30</td>
<td>3.76</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>200</td>
<td>30</td>
</tr>
<tr>
<td>Fat Clay</td>
<td>30-44</td>
<td>3.57</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>800</td>
<td>10</td>
</tr>
<tr>
<td>Clayey Sand</td>
<td>44-60</td>
<td>3.57</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>1000</td>
<td>18</td>
</tr>
<tr>
<td>Coal</td>
<td>60-69</td>
<td>2.39</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>3744</td>
<td>48</td>
</tr>
<tr>
<td>Parling</td>
<td>69-89</td>
<td>3.57</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>1000</td>
<td>10</td>
</tr>
<tr>
<td>Coal</td>
<td>89-92</td>
<td>2.39</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>3744</td>
<td>48</td>
</tr>
<tr>
<td>Fat Clay</td>
<td>92-100</td>
<td>3.57</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>1000</td>
<td>10</td>
</tr>
</tbody>
</table>
**In-Situ Stress**

Once the initial lithology and properties were specified, gravity was set and the material was allowed to step to equilibrium under the field. By fixing the base and setting roller boundaries on the left and right sides of the grid (Figure 23), FLAC is able to develop field stresses, $\sigma_{yy}$ and $\sigma_{xx}$, according to lithostatic pressure and elastic equilibrium based on the equations:

$$\sigma_{yy} = \gamma h$$
$$\sigma_{xx} = \sigma_{yy} \frac{\nu}{1-\nu}$$

where $\gamma = \text{in-situ unit weight of soil and water}$, $\nu = \text{poisson's ratio}$

This is an assumption that must be made to simplify this problem as the true initial state of stress is unknown; however, very rarely do horizontal stresses obey this law. Most often it is an assumption made when data is limited that would show that the state of stress varies due to other parameters such as tectonic activity, thermal gradients, or overpressuring. The state of stress could also be quite different in this case due to glaciation and lithostatic rebound. More testing, such as a hydrofracture test, would need to be done to determine the true initial stress state and this is beyond the scope of this paper.

![Figure 23. Boundary Conditions for Establishing In-situ Stress State](image-url)
Figure 24. Vertical Stress Field Induced by Gravity

Figure 25. Horizontal Stress Field Induced by Gravity
Equilibrium is reached when the maximum unbalanced force at any node falls below 10 lbf. The stress field developed can be seen in Figure 24 and Figure 25. The slight roughness in boundaries between colors designating change in stress can be attributed to slight node irregularities due to shaping of the grid to match geometry for later excavations.

*Groundwater in FLAC*

As in the Bishop's Method of Slices calculations, it is important to include the effects of groundwater in a stability analysis. This is because the pore pressure will act to counteract overlying stresses and decrease the effective stress between particles. Decreasing the effective stress decreases the frictional resistance between soil particles and increases the likelihood of failure.

FLAC has the ability to determine groundwater flow regimes based upon initial conditions given as pressure distributions, and boundary conditions applied to the grid. This flow regime can be determined separately from any mechanical behavior, or coupled with mechanical behavior. The difficulty underlies in the fact that the time taken to reach mechanical equilibrium does not correlate to a true time that can be observed in the field. Contrarily, groundwater flow does change in FLAC with respect to true time. Therefore, analysis of coupled behavior poses difficulties and can give misleading results. Often times, it is better to decouple the mechanical effects, reach a desirable groundwater flow regime, and then analyze for mechanical response under these conditions. That is what was done for this FLAC verification.

*Material Flow Properties*

It is necessary to specify the groundwater properties for the materials as well as fluid. Some permeability data specific to the coal units, clayshale, and till was obtained from James (2003). Other values for porosity and permeability were taken from Das (2001) based upon general lithology. Water properties are general properties relative to pure water at atmospheric temperature and pressure. All of these values are listed below in Table 16.
Table 16. Baseline Groundwater Properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Depth (ft)</th>
<th>Porosity</th>
<th>Permeability (ft²/slug)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Glacial Till</td>
<td>0-12</td>
<td>0.3</td>
<td>5.56E-08</td>
</tr>
<tr>
<td>Fat Clay</td>
<td>12-20</td>
<td>0.3</td>
<td>5.56E-08</td>
</tr>
<tr>
<td>Silty Sand</td>
<td>20-30</td>
<td>0.3</td>
<td>2.78E-06</td>
</tr>
<tr>
<td>Fat Clay</td>
<td>30-44</td>
<td>0.3</td>
<td>5.56E-08</td>
</tr>
<tr>
<td>Clayey Sand</td>
<td>44-60</td>
<td>0.3</td>
<td>5.56E-08</td>
</tr>
<tr>
<td>Coal</td>
<td>60-69</td>
<td>0.3</td>
<td>1.85E-06</td>
</tr>
<tr>
<td>Parting</td>
<td>69-89</td>
<td>0.3</td>
<td>5.56E-08</td>
</tr>
<tr>
<td>Coal</td>
<td>89-92</td>
<td>0.3</td>
<td>1.85E-06</td>
</tr>
<tr>
<td>Fat Clay</td>
<td>92-100</td>
<td>0.3</td>
<td>5.56E-08</td>
</tr>
</tbody>
</table>

Groundwater Properties

<table>
<thead>
<tr>
<th>Bulk Modulus (lb/ft²)</th>
<th>Density (slug/ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.20E+07</td>
<td>1.94</td>
</tr>
</tbody>
</table>

Initial Groundwater Conditions

Pore pressures were initialized based on water table data levels from Calder and Workman's study. The water table extension above the Hagel A seam into the clayey sand was assumed to be in equilibrium and therefore unconfined. The pressure induced by this water table and the water in the coal seams was assumed to vary linearly to the base of the grid (Figure 26). This assumption overlooks lateral recharge of the coal seams from surrounding areas. When comparing water table heads to average heads found by James (2003), these differences were small and will be neglected in order to simplify the pressure distribution.

Modeling Removal of Material

Shapes for various geometries can be modeled in FLAC by assigning coordinates to the grid in order to conform it. Alternatively, regions that need to be removed or excavated can be specified through the use of lines or “marks” and then the material properties of these regions can be subsequently “nulled”. These nulled grid nodes are unaffected by boundary stresses, pressures, or movements and cannot act upon adjacent grid nodes.
Through the use of the null property command an excavation was then made and boundary conditions set (Figure 27). Zero pore pressure boundary conditions were specified around the excavation and at the top of the grid. The far right and far left sides of the grid acted as sources of groundwater from the surrounding areas. The base of the grid was denoted impermeable with all other boundaries permeable. The pressures at the top of the grid and sides of the excavation were set to zero. A command was used to disable mechanical interaction with the groundwater flow. This effectively decoupled the mechanical and groundwater interaction. Then by commanding FLAC to "solve" or step through the calculations, the groundwater regime was allowed to equilibrate based upon governing pressure diffusion equations.

In order to determine when the pore pressure distribution has quit fluctuating, or steady state has been reached, the fish function qsolve.fis was utilized. This function calculates the ratio of the unbalanced flow to the average flow in the model. It does this by taking the difference between the inflow and outflow at various nodes and dividing it by the average flow in the grid. This is a dimensionless number and can help approximate

![Figure 26. Linear Pressure Distribution Before Excavation](image-url)
steady state. In general, when this number approaches a limit, it can be assumed that the pressure distribution will not change rapidly anymore. This limit can be seen by plotting the graph of \( \text{qratio} \) vs. step time (Figure 28). In this situation, steady state was reached in less than 10 minutes. Since this is a relatively small amount of time relative to the time required to excavate the pit, it can be assumed that steady state conditions will probably accurately model highwall strength.

The steady state pressure distribution is shown in Figures 29-30. The top line represents a pressure that would be due to an overlying water table of 8 feet. The shape and magnitude of the pressure distribution visually seems physically reasonable.
Figure 28. Qratio vs. Step Time

Figure 29. Steady State Pressure Distribution
Mechanical Response
Mechanical response of the highwall at various pore pressure regimes was analyzed varying from undrained to steady state was analyzed. In order to decouple groundwater flow for analysis, the FLAC groundwater flow was turned off and the bulk modulus of water was set to zero. The response of the highwall was the analyzed for a factor of safety. The factor of safety was incorporated by reducing the strength parameters of the highwall in the same way as Bishop’s Method of Slices:

$$FS = \frac{C_t}{C_a} = \frac{\tan \phi_t}{\tan \phi_a}$$

where $C_t = true\ value\ of\ cohesion$, $C_a = analyzed\ value\ of\ cohesion$

$\phi_t = true\ value\ of\ friction\ angle$, $\phi_a = analyzed\ value\ of\ friction\ angle$

By varying the strength parameters according to a factor of safety value, the limiting strength point can be determined. The factor of safety used at this limiting point is the factor of safety for the entire wall.
**Analyzed conditions**
The most conservative estimate for the factor of safety will be when the pore pressure distribution has not been allowed a long time to equilibrate. This is considered an undrained analysis of the highwall. FLAC was allowed a few seconds to reset the initial pressure conditions to the boundary conditions at the wall. Then the wall was analyzed for strength.

Steady state is the most stable configuration for the highwall. This was determined by the **qratio** function.

**Determining Failure**
Failure has to be determined by the user using various criterion available in FLAC. There are two general ways to determine this.

Plastic failure is based upon Mohr-Coulomb failure criterion. This can be plotted in FLAC to determine which grid nodes have failed in shear and which in tension. It also tells which nodes are behaving elastically.

Failure can also be determined by looking at velocities and displacements at nodes. If displacements are unacceptable for the project, failure has occurred. If slight displacements are acceptable and velocities are slow, then small movements may not result in failure. This is one of the benefits to analysis with FLAC. If the grid fails (grid node areas become negative) due to large displacements, the displacement value recorded by FLAC will not be the final displacement and should be noted when analyzing the results. In this study, any failure will almost certainly cause the highwall material to slide into the pit.

For this project, both analyses were used. Failure was considered to occur when a defined failure arc based on plastic indicators is linking the base of the dragline to highwall face. Any large displacements (>3 inches) below the dragline were also deemed failure depending on the velocity profile. For a steep wall, such as a highwall, failure is relatively easy to spot since it is usually catastrophic. Most graphs in this report are...
showing displacements since this plot most visibly defines the failure surface. Plastic indicators were also analyzed; however, since these indicators can usually pinpoint a failure sooner than displacement and velocity vectors.

**Analysis Results**

This particular highwall had a factor of safety that varied from 0.95 to 1.0 depending on drawdown conditions. The 0.95 factor of safety is for a pore pressure distribution close to initial conditions. The highwall at failure is shown in Figures 31-32 for various analysis techniques. This analysis showed that the highwall may be stronger than the Bishop’s Method of Slices first predicted. This would coincide with Hoek and Bray’s (1974) remarks about the conservative nature of Bishop’s Method of Slices.

A unique phenomenon should be noted. Decreasing the permeability of the clayshale results in increased pit floor movement at similar times in drawdown. The pit floor usually rises an inch or two due to excavation; however, in some situations displacements up to 4 inches were seen (Figure 33). This could be a possible mechanism for pit floor failures.

![Figure 31. Plastic Failure of the Highwall as Shown by Pink Nodes](image-url)
Figure 32. Failure Shown by Displacement Vectors

Figure 33. Pit Floor Displacement Example
Tavis Highwall Stability

*Initial Lithology*

The generalized lithology for the Tavis highwall is given in Table 5. For this analysis, it is assumed that Falkirk will pre-bench down to the maximum 90 foot digging depth. That would place the dragline immediately on top of the C sand after pre-benching. Pre-benching will be achieved by truck and shovel and should occur at a distance far in front of the dragline operation (Figure 34). This will leave a maximum 90 foot highwall before the Tavis Seam is removed. Additional pre-benching could bring the wall height down further.

![Figure 34. Truck and Shovel Operation Moving Ahead of Dragline](image)

*Initial Stress Distribution*

The initial stress distribution was set in the same manner as the Hagel highwall analyzed earlier.

*Initial and Boundary Groundwater Conditions*

The groundwater table will be lowered in account of the pre-benching operation moving ahead of the dragline. The exact amount of lowering is unknown, but it is known to be a function of the removal of soil, loss of vegetation, and varying boundary conditions due
to mining. The groundwater table will be the highest initially after the pre-bench by truck and shovel. This will be the worst case scenario for pore pressure distribution in the highwall. The pore pressure distribution at given points in time after the cut to the Tavis has been made will be a function of permeability.

Boundary conditions were set in a similar fashion to the previous analysis (Figure 35). The only conditions that changed were the applied pressure on the right hand side of the grid.

Figure 35. Initial and Boundary Conditions for Tavis Highwall
Table 17. Baseline Elastic and Strength Properties for the Tavis Highwall

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Depth(ft)</th>
<th>Density (slug/ft³)</th>
<th>Bulk Modulus (lb/ft²)</th>
<th>Shear Modulus (lb/ft²)</th>
<th>Cohesion (lb/ft²)</th>
<th>Angle of Internal Friction (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Poisson’s Ratio = 0.35</td>
</tr>
<tr>
<td>Glacial Till</td>
<td>0-20</td>
<td>3.57</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>1008</td>
<td>20</td>
</tr>
<tr>
<td>Fat Clay</td>
<td>20-40</td>
<td>3.57</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>1000</td>
<td>10</td>
</tr>
<tr>
<td>A Seam</td>
<td>40-45</td>
<td>2.39</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>3744</td>
<td>48</td>
</tr>
<tr>
<td>Parting</td>
<td>45-47</td>
<td>3.57</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>1000</td>
<td>10</td>
</tr>
<tr>
<td>B Seam</td>
<td>47-50</td>
<td>2.39</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>3744</td>
<td>48</td>
</tr>
<tr>
<td>Fat Clay</td>
<td>50-80</td>
<td>3.57</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>1000</td>
<td>10</td>
</tr>
<tr>
<td>C Seam</td>
<td>80-83</td>
<td>2.39</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>3744</td>
<td>48</td>
</tr>
<tr>
<td>C Sand</td>
<td>83-91</td>
<td>3.76</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>200</td>
<td>30</td>
</tr>
<tr>
<td>Fat Clay</td>
<td>91-170</td>
<td>3.57</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>1000</td>
<td>10</td>
</tr>
<tr>
<td>Tavis</td>
<td>170-180</td>
<td>2.39</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>3744</td>
<td>48</td>
</tr>
<tr>
<td>Fat Clay</td>
<td>180-200</td>
<td>3.57</td>
<td>5.40E+06</td>
<td>1.80E+06</td>
<td>1000</td>
<td>10</td>
</tr>
</tbody>
</table>

Groundwater Drawdown

It was necessary to determine the time it would take to reach steady state drawdown for the highwall. If steady state is reached rather quickly (order of minutes) then it could be assumed that this state adequately describes the pore pressure distribution for mechanical analysis. If large amounts of time are required to reach this state, then a transient analysis would be necessary in order to interpret strength results.

The qsolve and qratio fish functions were employed to help estimate steady state groundwater conditions. Figures 37-41 show the groundwater pore pressure distribution and flow vectors at various points in time. It can be seen that the pressure distribution tended to bulge behind the highwall, where the lower permeability clayshale was present. This pressure distribution would be difficult to detect using only piezometers. The C sand and Tavis Creek act as pipelines for water flow, and very little flow is seen initially in the highwall.
Figure 36. Initial Groundwater Conditions for Tavis Pit After Pre-Benching

Figure 37. Fully Saturated Highwall Drawdown - 1 minute
Figure 38. Drawdown At 2 Minutes

Figure 39. Drawdown At 5 Minutes
Figure 40. Drawdown at 7.5 Minutes

Figure 41. Steady-State At 9 - 60 Minutes
After 9 minutes, the pore pressure has stabilized (Figure 41). Further steps were taken and at 60 minutes the pressure regime had not changed. Since 9 minutes of flow time is much lower than the amount of time expected to excavate the pit below the water table, a steady state groundwater flow solution should accurately represent the highwall and pit strength.

**Analysis of a 90 Foot Highwall**
Analysis of a 90 foot, 55 degree highwall was performed with various dragline setbacks and initial groundwater levels. All situations are unsatisfactory for mining of the Tavis Seam.

There is a defining point where the dragline stops directly impacting the failure surface location. This point is somewhere between 40 and 60 foot dragline setbacks. It is assumed that the increase of pressure due to the dragline has reached some limiting value.

![Chart 1. Stability of 90 Foot Highwall](image-url)
at this point. In these situations, the failure surface does not encompass the dragline. All failures toe out approximately at the base of the highwall. Since the soil properties in the wall are relatively similar, this is reasonable since this would represent the most driving force for failure due to the increased failure surface weight.

Factors of safety are low enough and failures large enough to render this situation unacceptable. The results are graphed in Chart 1.

90 Feet, 55 Degree Highwall, Fully Saturated

Figure 42. 20 Foot Dragline Setback (FS - 0.71)
Figure 43. 40 Foot Dragline Setback (FS - 0.77)

Figure 44. 60 Foot Dragline Setback (FS - 0.76)
Figure 45. 80 Foot Dragline Setback (FS - 0.77)

Figure 46. No Dragline Present (FS - 0.77)
90 Feet, 55 Degree Highwall, 20 Foot Initial Depth To Water Table

Figure 47. Steady-State Drawdown, 20 Foot Initial Depth To Water Table

Figure 48. 20 Foot Dragline Setback (FS – 0.78)
Figure 49. 40 Foot Dragline Setback (FS − 0.85)

Figure 50. 60 Foot Dragline Setback (FS − 0.87)
90 Feet, 55 Degree Highwall, 40 Foot Initial Depth To Water Table

Figure 51. Steady-State Drawdown, 40 Foot Initial Depth To Water

Figure 52. 20 Foot Dragline Setback (FS – 0.85)
Figure 53. 40 Foot Dragline Setback (FS - 0.92)

Figure 54. 60 Foot Dragline Setback (FS - 0.94)
70 Foot Highwall Stability
Analysis of a 70 foot highwall with various depths to the water table was performed in order to determine the stability increase due to deeper pre-benching. In this situation, the mine would use truck and shovel to bench down to 70 feet above the Tavis Creek seam. The results can be seen in Chart 2.

Chart 2. Stability of a 70 Foot Highwall
70 Foot, 55 Degree Highwall, Fully Saturated

Figure 55. Steady-State Drawdown, Initially Fully Saturated

Figure 56. 20 Foot Dragline Setback (FS – 0.89)
Figure 57. 40 Foot Dragline Setback (FS – 0.98)

Figure 58. 60 Foot Dragline Setback (FS – 0.99)
Figure 59. Steady State Drawdown, 20 Foot Initial Depth Water

Figure 60. 20 Foot Dragline Setback (FS – 0.95)
Figure 61. 40 Foot Dragline Setback (FS – 1.06)

Figure 62. 60 Foot Dragline Setback (FS – 1.07)
70 Foot, 55 Degree Highwall, 40 Foot Initial Depth To Water Table

Figure 63. Steady State Drawdown, 40 Foot Initial Depth To Water Table

Figure 64. 20 Foot Dragline Setback (FS – 1.01)
Figure 65. 40 Foot Dragline Setback (FS – 1.13)

Figure 66. 60 Foot Dragline Setback (FS – 1.18)
50 Foot Highwall Stability
For a 50 foot highwall, only two drawdown situations were analyzed. The first is when the highwall is fully saturated. The second is when the depth to the water table is initially 20 feet. A 40 foot depth to water was not analyzed because the wall is only 50 feet in height and stable under the previous two conditions.

Chart 3. Factor of Safety vs. Dragline Setback For 50 Foot Highwall
Figure 67. Steady State Drawdown, Fully Saturated

Figure 68. 20 Foot Dragline Setback (FS - 1.10)
Figure 69. 40 Foot Dragline Setback (FS - 1.29)

Figure 70. 60 Foot Dragline Setback (FS - 1.29)
Figure 71. No Dragline Present (FS - 1.29)

50 Foot, 55 Degree Highwall, 20 Foot Initial Depth to Water Table

Figure 72. Steady State Drawdown, 20 Foot Initial Depth To Water
Figure 73. 20 Foot Dragline Setback (FS - 1.19)

Figure 74. 30 Foot Dragline Setback (FS - 1.29)
Figure 75. 40 Foot Dragline Setback (FS - 1.42)

Figure 76. 60 Foot Dragline Setback (FS - 1.44)
Conclusions

It is apparent that the highwall begins to become stable below 70 feet in height. The water table should be at least 20 feet below the bench surface before beginning to dig a pit of this depth. This could be measured by drilling a pilot hole with a hand auger or running some rotary holes with the rock drilling crew. At this borderline height, it is recommended that the dragline be set back approximately 40 feet from the edge of the wall; however, further setbacks over 40 feet will do little to stabilize the wall. If the groundwater table is abnormally high after pre-benching or if there is compromising lithology, it is recommended to pre-bench down to 60 or 50 feet in height. At 50 feet height, under the given strength conditions, the highwall should be stable in all dragline and groundwater conditions; however caution should be used during a 20 foot dragline setback with high water conditions.

There were several assumptions made in this study that must be weighed in order to analyze the results:

First, there is no data available showing how water effects the strength properties of the soils. Under Mohr-Coulomb failure criterion, the shearing resistance due to the angle of internal friction is the only affected strength parameter. In all actuality, the cohesion may also be affected for reasons that could be chemically induced and therefore ignored by the Mohr-Coulomb equation. More testing would need to be performed to quantify these affects. Second, the highwall was assumed to be composed entirely of silty clay after pre-benching. Since this is the dominant lithology below the C sand and seam, it was a reasonable assumption. Large amounts of sandy type lithologies will most likely negatively impact the strength of the wall, especially under high water table conditions, and if these are encountered adjustments may need to be made. In general, if any parameter is different than what was used in this model, the stability situation could change drastically.
Economic Considerations
The economic considerations that are usually present in pit design are of lesser importance at Falkirk than other mines.

Most other open pit mining operations, such as with open pit copper mining, have extremely large, deep pits that maintain the same walls over long periods of time. Maximizing the highwall angle is of extreme importance here since it is equivalent to many equipment hours removing extra overburden. At Falkirk, the area being mined out is relatively large and shallow. In this situation, maximizing the highwall angle will only save a very small amount of overburden on the very first and very last pit. As well, portions of the pit are used for things such as ramps, and therefore must have gradual slopes.

Instead, it is most important to maximize the height of the highwall, which was what was done in this study. By doing this it may be possible to eliminate some pre-stripping, which is the most expensive overburden removal technique. Exact costs per hour for equipment were not available for this study, as the mine holds that information as proprietary data and does not wish to disclose it. However, in general, the most yards moved by the dragline results in a cheaper mining operation.

Strip ratios are a good way to estimate if a coal seam is economically feasible to mine. According to James (personal communication), the maximum strip ratio that is acceptable for their current operation is 10 feet of overburden to 1 foot of coal. Therefore, if the mining operation wants to mine the Tavis Creek Seam that is 100 feet below the surface, it must be able to recover 10 feet of coal after uncovering the pit. Accurate mapping data is necessary to delineate economically feasible zones, and smaller coal seams, such as the C seam, can help to improve strip ratios if the coal is thick enough to mine and high enough quality. One technique that can be used to determine suitable mining areas is to overlay contour maps of coal thickness, overburden thickness, and coal quality. Intersecting zones can be analyzed closer to determine if mining would be economical.
The most important reason for determining stable highwall configurations is for safety. Many people work and drive in the pits on a daily basis, and failures can be detrimental if someone or equipment is buried. Also, because of the large size of the dragline, it is extremely important to ensure the stability to avoid the catastrophic problem of the dragline falling into the pit.

**Future Work**
There are many considerations that were ignored due to the lack of time for this project.

First and foremost, once the location of Tavis mining is known, accurate stratigraphic, groundwater, strength, and elastic data will need to be obtained. A model is only as good as the data and this will be extremely important since the magnitude of the mining operation will be large and the stability an extremely important part of this operation. Second, the transient effects of drawdown on stability need to be more accurately quantified. Determining the amount of drawdown over time and the extent that anisotropic and nonhomogeneous permeabilities affect this drawdown will be vital to accurate pit modeling. Third, it will be important to determine the maximum spoil pile that is stable. Fourth, and lastly, it will be important to more accurately delineate the mining method. If there are other possible ways to mine the Tavis, such as making a double bench with two draglines, then these methods need to be explored. Geometry is extremely important in developing a good stability model.
References


Appendix

**Hagel Pit Code**
**Defines a 60 foot highwall with various lithology based on Calder & Workman**
**Highwall gave a FS of 0.95-1.0**

grid 100,40
model mohr

**SET UP GENERAL GRID BOUNDARY**
gen 0,-100 0,0 400,0 400,-100 i=1,101 j=1,41

**DEFINE COORDINATES FOR SPECIFIC LITHOLOGY**
gen same 0,-92 400,-92 same i=1,101 j=1,5
gen same 0,-89 400,-89 same i=1,101 j=5,6
gen same 0,-69 400,-69 same i=1,101 j=6,14
gen same 0,-60 400,-60 same i=1,101 j=14,17
gen same 0,-44 400,-44 same i=1,101 j=17,24
gen same 0,-30 400,-30 same i=1,101 j=24,29
gen same 0,-20 400,-20 same i=1,101 j=29,32
gen same 0,-12 400,-12 same i=1,101 j=32,36
gen same 0,-0 400,-0 same i=1,101 j=36,41

**Define Factor Of Safety To Relate To Strength Parameters**
def FS
FS=1.0
end
*clay phi and coh*
def clfric
fric1=tan((10.0*(pi/180.0)))
clfric=(atan(fric1/FS))*(180.0/pi)
end
def clcoh
cloah=800.0/FS
end
*silty sand phi and coh*
def sdfric
fric2=tan((30.0*(pi/180.0)))
sdfric=atan(fric2/FS)*(180.0/pi)
end
def sdcoh
sdcoh=200.0/FS
end
*clayey sand phi and coh*
def clsfric
fric5=tan((18.0*(pi/180.0)))
clsfric=atan(fric5/FS)*(180.0/pi)
end
def clscoh
clscoh=1000.0/FS
end
*coal phi and coh*
def cofric
    fric3=tan((48.0*(pi/180.0)))
    cofric=atan(fric3/FS)*(180.0/pi)
end
def cohoh
    cohoh=3744.0/FS
end

till phi and coh*
def tfric
    fric4=tan((20.0*(pi/180.0)))
    tfric=atan(fric4/FS)*(180.0/pi)
end
def tcoh
    tcoh=1008.0/FS
end

**ASSIGN PROPS TO LITHOLOGY**

**Clay shale E varies from 11200-56000lb/ln^2**
**Army Corp for Ft Union Clay), poisson ratio~0.35**
model mohr j=1.51
*clay underburden*
prop bulk=5.4e6 shear=1.8e6 fric=clfric dens=3.57 j=1,5
prop coh=clcoh por=0.3 perm=5.56e-8 j=1,5
*B*
prop bulk=5.4e6 shear=1.8e6 fric=cofric dens=2.39 j=5,6
prop coh=cohoh por=0.3 perm=1.85e-6 j=5,6
*Parting*
prop bulk=5.4e6 shear=1.8e6 fric=clsfric dens=3.57 j=6,14
prop coh=clcoh por=0.3 perm=5.56e-8 j=6,14
*A*
prop bulk=5.4e6 shear=1.8e6 fric=cofric dens=2.39 j=14,17
prop coh=cohoh por=0.3 perm=1.85e-6 j=14,17
*Clayey Sand*
prop bulk=5.4e6 shear=1.8e6 fric=clsfric dens=3.57 j=17,24
prop coh=clcoh por=0.3 perm=5.56e-8 j=17,24
*Fat Clay*
prop bulk=5.4e6 shear=1.8e6 fric=clfric dens=3.57 j=24,29
prop coh=clcoh por=0.3 perm=5.56e-8 j=24,29
*Silty Sand*
prop bulk=5.4e6 shear=1.8e6 fric=sdfric dens=3.76 j=29,32
prop coh=sdcoh por=0.3 perm=2.78e-6 j=29,32
*Fat Clay*
prop bulk=5.4e6 shear=1.8e6 fric=clfric dens=3.57 j=32,36
prop coh=clcoh por=0.3 perm=5.56e-8 j=32,36
*Glacial Till*
prop bulk=5.4e6 shear=1.8e6 fric=tfric dens=3.57 j=36,41
prop coh=tcoh por=0.3 perm=5.56e-8 j=36,41

**SET MECHANICAL BOUNDARY CONDITIONS AND GRAVITY**
fix x i=1
fix x i=101
fix x y j=1
set grav=32.17

**SHOULD PAUSE HERE AND SOLVE FOR IN-SITU STATE OF STRESS**

**DEFINE PIT**

def outcrop
  slope=55.0*((pi)/180.0)
  ptwdth=90.0
  ptdpth=60.0
  outcrop=ptwdth+(ptdpth/(tan(slope)))
end

*relocate gridpoint if necessary*
*init x=outcrop y=-80 i=40 j=32*
gen line 0,-92 90,-92
gen line 90,-92 90,-60
gen line 90,-60 outcrop,0

**DEFINE EXCAVATIONS**

*Cut to Coal*
gen line 0,-60 90,-60
*Remove A*
gen line 0,-69 90,-69
*Remove Parting*
gen line 0,-89 90,-89
*Remove B*
gen line 0,-92 90,-92

**GROUNDWATER**

*initial AND boundary conditions*
ini sat=1 i=1,101 j=1,24
ini sat=0 i=1,101 j=24,41
ini sat=0 regi i=1 j=30
apply pp 3495 var 0 -3495 i=101 j=1,24
apply pp 2496.39 var 0 -2496.39 i=1 j=1,17
ini pp 3495 var 0 -3495 i=1,101 j=1,24
ini pp 2496.39 var 0 -2496.39 i=1 j=1,17
ini pp 0 i=1,101 j=24,41
ini pp 0 regi i=1 j=30
ini pp 0 mark i=1,36 j=17,41
fix pp mark i=1,36 j=17,41
fix pp i=1 j=1,17
fix pp j=41
water bulk=4.2e7 den=1.94

**PAUSE AND SOLVE FOR GROUNDWATER STATE**

**DEFINE DRAGLINE PLATFORM**
def setbck
  setbck=20.0
end
def dragl
drag1=outcrop+setbck
def drag2
    drag2=outcrop+setbck+80.0
end
gen line drag1,0 drag1,-100
gen line drag2,0 drag2,-100

***LOAD DRAGLINE 20 FEET FROM EDGE***

SET LARGE
app pressure=2685 from 39,41 to 59,41

**SOLVE PROBLEM FOR FS SPECIFIED AT TOP OF CODE**

NOTE: ONE CAN SOLVE FOR A GROUNDWATER STATE, SAVE THIS STATE, THEN WRITE A MOD FILE TO CHANGE MATERIAL PROPERTIES BASED ON FS. DRAGLINE SETBACK CAN ALSO BE CHANGED IN THIS MOD FILE. THIS SPEEDS CALCULATIONS SINCE THE FIRST FS GUESSED WILL MOST LIKELY NOT BE THE CORRECT CHOICE. AN EXAMPLE MOD FILE IS BELOW. THE WAY THIS IS IMPLEMENTED IS STOP THE CODE USING "PAUSE" AFTER THE GROUNDWATER STATE. THEN CALL THE MOD FILE AND SOLVE.

**Example Mod File**

**Define Factor Of Safety To Relate To Strength Parameters**

def FS
    FS=1.03
end
*clay phi and coh*
def clfric
    fric1=tan((10.0*(pi/180.0)))
    clfric=(atan(fric1/FS))*(180.0/pi)
end
def clcoh
    clcoh=800.0/FS
end
*silty sand phi and coh*
def sdfric
    fric2=tan((30.0*(pi/180.0)))
    sdfric=(atan(fric2/FS))*(180.0/pi)
end
def sdcoh
    sdcoh=200.0/FS
end
*clayey sand phi and coh*
def clsfric
    fric5=tan((18.0*(pi/180.0)))
    clsfric=(atan(fric5/FS))*(180.0/pi)
end
def clscoh
    clscoh=1000.0/FS
end
*coal phi and coh*
def cofric
    fric3=tan((48.0*(pi/180.0)))
    cofric=atan(fric3/FS)*(180.0/pi)
end
def cocoh
    cocoh=3744.0/FS
end
*till phi and coh*
def tfri c
    fric4=tan((20.0*(pi/180.0)))
    tfric = atan(fric4/FS)*(180.0/pi)
end
def tcoh
    tcoh=1008.0/FS
end

**Change the properties of the existing saved grid**
*clay underburden*
prop bulk=5.4e6 shear=1.8e6 fric=clf ric dens=3.57 j=1,5
prop coh=clcoh por=0.3 perm=1.85e-10 j=1,5
*B*
prop bulk=5.4e6 shear=1.8e6 fric=cofric dens=2.39 j=5,6
prop coh=cocoh por=0.3 perm=1.85e-6 j=5,6
*Parting*
prop bulk=5.4e6 shear=1.8e6 fric=clf ric dens=3.57 j=6,14
prop coh=clcoh por=0.3 perm=1.85e-10 j=6,14
*A*
prop bulk=5.4e6 shear=1.8e6 fric=cofric dens=2.39 j=14,17
prop coh=cocoh por=0.3 perm=1.85e-6 j=14,17
*Clayey Sand*
prop bulk=5.4e6 shear=1.8e6 fric=clf ric dens=3.57 j=17,24
prop coh=clcoh por=0.3 perm=1.85e-9 j=17,24
*Fat Clay*
prop bulk=5.4e6 shear=1.8e6 fric=clf ric dens=3.57 j=24,29
prop coh=clcoh por=0.3 perm=1.85e-10 j=24,29
*Silty Sand*
prop bulk=5.4e6 shear=1.8e6 fric=clf ric dens=3.76 j=29,32
prop coh=sdcoh por=0.3 perm=2.78e-6 j=29,32
*Fat Clay*
prop bulk=5.4e6 shear=1.8e6 fric=clf ric dens=3.57 j=32,36
prop coh=clcoh por=0.3 perm=1.85e-10 j=32,36
*Glacial Till*
prop bulk=5.4e6 shear=1.8e6 fric=tf ric dens=3.57 j=36,41
prop coh=tcoh por=0.3 perm=1.85e-10 j=36,41

**Define Pit Dimensions**
def outcrop
    slope=55.0*((pi)/180.0)
    ptwidth=90.0
    ptdpth=60.0
    outcrop=ptwidth+((ptdepth/(tan(slope))))
end
*relocate gridpoint*
*init x=outcrop y=-80 i=40 j=32*
gen line 0,-92 90,-92
gen line 90,-92 90,-60
gen line 90,-60 outcrop,0

*Cut to Coal*
gen line 0,-60 91,-60
*Remove A*
gen line 0,-69 90,-69
*Remove Parting*
gen line 0,-89 90,-89
*Remove B*
gen line 0,-92 90,-92*

**Stop PP from Changing**
set flow off
set mech on
water bulk=0

**DEFINE DRAGLINE PLATFORM**
def setbck
  setbck=20.0
end
def drag1
  drag1=outcrop+setbck
end
def drag2
  drag2=outcrop+setbck+80.0
end
gen line drag1,0 drag1,-100
gen line drag2,0 drag2,-100

**20 foot dragline setback=**
***LOAD DRAGLINE***
app pressure=2685 from 39,41 to 59,41
**Solve Problem for FS Defined Above**

**Qsolve and Qratio Functions for Steady State Determination (Taken from the FLAC Handbook)**

**This function should be called immediately before solving for the groundwater flow regime. Then instead of typing “solve”, stepping is instigated through the command “qsolve”.

**this subroutine determines when steady state has been reached**
ca qratio.fis
def qsolve
;default nstep=100
if nstep=0 then
    nstep=100
end_if

;default qrat_lim=l%
if qrat_lim=0.0 then
    qrat_lim=.01
end_if

loop k (0,1)
    command
    print qratio
    step nstep
    end_command
    if qratio > qrat_lim then
        k=k-1
    end_if
end_loop
end

**keeps track of the unbalanced flow**
def qratio
    inflow = 0.0
    outflow = 0.0
    loop i (1,igp)
        loop j (1,jgp)
            if and(flags(i,j),512) # 0 then
                if gflow(i,j) > 0.0 then
                    inflow = inflow + gflow(i,j)
                else
                    outflow = outflow - gflow(i,j)
                end_if
            end_if
        end_loop
    end_loop
    qbalance = inflow - outflow
    if inflow + outflow # 0.0 then
        qratio = 2.0 * abs(qbalance) / (inflow + outflow)
    else
        qratio = 0.0
    end_if
end

NOTE: This file calculates the stability for a 90 foot high wall into the TAVIS. Variations of this file were used for the subsequent analyses in the latter half of this paper.

************TAVIS PIT************
grid 100,30
model mohr

**SET UP GENERAL BOUNDARY**
**DEFINE COORDINATES FOR LITH ABOVE TAVIS**
gen 0,-120 0,0 400,0 400,-120 i=1,101 j=1,31

gen same 0,-100 400,-100 same i=1,101 j=1,6
gen same 0,-90 400,-90 same i=1,101 j=6,8
gen same 0,-11 400,-11 same i=1,101 j=8,28
gen same 0,-3 400,-3 same i=1,101 j=28,30
gen same 0,-400,-0 same i=1,101 j=30,31

**Define Factor Of Safety To Relate To Strength Parameters**
def FS
  FS=1.00
end

**Adjust Strength Properties for FS**
*clay phi and coh*
def clfric
  fric1=tan((18.0*(pi/180.0)))
  clfric=(atan(fric1/FS))*(180.0/pi)
end
def clcoh
  clcoh=1000.0/FS
end

*sand phi and coh*
def sdfric
  fric2=tan((30.0*(pi/180.0)))
  sdfric=atan(fric2/FS)*(180.0/pi)
end
def sdcoh
  sdcoh=200.0/FS
end

*coal phi and coh*
def cofric
  fric3=tan((48.0*(pi/180.0)))
  cofric=atan(fric3/FS)*(180.0/pi)
end
def cocoeh
  cocoeh=3744.0/FS
end

**ASSIGN PROPS**
model mohr j=1,31
*clay underburden*
prop bulk=5.4e6 shear=1.8e6 fric=clfric dens=3.57 j=1,6
prop coh=clcoh por=0.3 perm=1.85e-10 j=1,6
*Tavis*
prop bulk=5.4e6 shear=1.8e6 fric=cofric dens=2.39 j=6,8
prop coh=cocoeh por=0.3 perm=1.85e-6 j=6,8
*Clay*
prop bulk=5.4e6 shear=1.8e6 fric=clfric dens=3.57 j=8,28
prop coh=clcoh por=0.3 perm=1.85e-10 j=8,28
*Silty Sand*
prop bulk=5.4e6 shear=1.8e6 fric=sdfric dens=3.76 j=28,30

95
prop coh=sdcoh por=0.3 perm=2.78e-6 j=28.30
*C Seam*
prop bulk=5.4e6 shear=1.8e6 fric=cofric dens=2.39 j=30,31
prop coh=cocoh por=0.3 perm=1.85e-6 j=30,31

**SET MECHANICAL BOUNDARY CONDITIONS AND GRAVITY**

fix x i=1
fix x i=101
fix x y j=1
set grav=32.17

**solve for equilibrium conditions**

**DEFINE PIT**

def outcrop
   slope=55.0*(pi)/180.0
   ptwidth=90.0
   ptdpth=90.0
   outcrop=ptwidth+(ptdpth/(tan(slope)))
end
gen line 0,100 90,-100
ngen line 90,-100 90,90
ngen line 90,90 outcrop,0
ngen line 0,90,90
model null reg i=1 j=29

**Initial and boundary conditions for GW**

ini sat=1 i=1,101 j=1,31
ini sat=0 reg i=1 j=29
apply pp 6865.1 var 0-6865.1 i=101 j=1,28
apply pp 1872.29 var 0-1872.29 i=1 j=1,8
ini pp 6865.1 var 0-6865.1 i=1,101 j=1,28
ini pp 0 reg i=1 j=29
ini pp 0 mark i=1,40 j=8,31
fix pp mark i=1,40 j=8,31
fix pp j=31

water bulk=1e3 den=1.94
set flow on mech off
caqsolve.fis
hist nstep 1000
hist qratio
hist inflow
his outflow
set qrat_lim 0.01 nstep 1000
pause

set large

**DEFINE DRAGLINE PLATFORM**

def setbck
   setbck=20.0
end
def drag1
drag1=outcrop+setbck
end
def drag2
drag2=outcrop+setbck+80.0
end
gen line drag1,0 drag1,-120
gen line drag2,0 drag2,-120
**check for dragline location using "print mark" command"**
pause

**20 foot dragline setback=grid 44,64**
***LOAD DRAGLINE***

app pressure=2685 from 44,31 to 64,31