Abstract

A desire for increased recovery in highwall mining has fueled an interest into the possibility of applying backfill technology to highwall mining. Backfill technology offers the potential for additional advantages, including increased stability and profit. This technique is currently applied in underground mines, but has not yet been implemented in highwall mining. There are current plants to start backfilling highwall mines at West Bokaro mine in Jharkhand, India. The purpose of this thesis is to investigate the possibility of applying backfill technology to highwall mining and to determine whether doing so will lead to an increase in recovery, both in a general sense and in the particular case of the West Bokaro mine.

In order to investigate this, a numerical model was created using the finite difference modeling program FLAC3D. Input data was acquired from UCS tests that were performed on samples of rock material as well as backfill material obtained from the West Bokaro mine site. The model ran various simulations focusing on the effects of pillar width, mine and backfill sequencing, backfill material, partial backfilling, and mining in multiple lifts.

For the situation at West Bokaro, an optimum pillar width of 2.9 m was found. However, this does not take into account existing regulations that limit the minimum dimensions of the pillar. The mining sequence with the highest stability was a 1-by-1 sequence, where adjacent drives are excavated and backfilled one after the other, ensuring that excavation of the next drive doesn’t start until the previous drive has been backfilled. However, a more practical solution would be mining and backfilling simultaneously, while maximizing the distance between the drives in the process of being excavated and backfilled.

The efficiency of backfilling increases with increased cohesion and stiffness of the backfill material. For this reason, loose, dry material is not recommended as backfill. Fly ash composite materials are suitable as a backfill material, provided that cohesion and strength are sufficiently high.

Partial backfilling imparts some strength to the surrounding pillars, but not enough to be significant. However, a small gap left open at the top should not excessively affect the process. Multi-lift mining increases overall stability, but comes with an increased risk of roof instability.

According to the simulations done in this thesis, recovery as a result of backfilling can theoretically be increased up to 36% when compared to conventional methods. However, the data available for this thesis was limited and therefore more research is required to make any conclusive predictions. More research should also be done on the economic feasibility of implementing backfill technology in highwall mining.
Completing this thesis was no easy task and I have many people to thank for helping me along the way. First of all I would like to thank my supervisors Mike Buxton and Dominique Ngan-Tillard for their help during the course of this project, both with their input and their encouragement.

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Nomenclature

\begin{itemize}
  \item \( a \) \hspace{1cm} \text{Empirical constant dependent on rock mass characteristics}
  \item \( A_0 \) \hspace{1cm} \text{m}^2 \hspace{1cm} \text{Cross-sectional area}
  \item \( B \) \hspace{1cm} \text{Body force}
  \item \( b \) \hspace{1cm} \text{Pa} \hspace{1cm} \text{Body force per unit mass}
  \item \( c \) \hspace{1cm} \text{Pa} \hspace{1cm} \text{Cohesion}
  \item \( c_i \) \hspace{1cm} \text{Pa} \hspace{1cm} \text{Cohesion of the fill}
  \item \( d \) \hspace{1cm} \text{m} \hspace{1cm} \text{Diameter}
  \item \( D_c \) \hspace{1cm} \text{kg/m}^3 \hspace{1cm} \text{Density of coal}
  \item \( D_{\text{design}} \) \hspace{1cm} \text{m} \hspace{1cm} \text{Design overburden depth}
  \item \( D_{\text{MAX}} \) \hspace{1cm} \text{m} \hspace{1cm} \text{Maximum overburden depth}
  \item \( D_{\text{MIN}} \) \hspace{1cm} \text{m} \hspace{1cm} \text{Minimum overburden depth}
  \item \( D_{\text{DB}} \) \hspace{1cm} \text{kg/m}^3 \hspace{1cm} \text{Average density of overburden}
  \item \( D_r \) \hspace{1cm} \text{kg/m}^3 \hspace{1cm} \text{Average density of roof rock material}
  \item \( E_{\text{W}} \) \hspace{1cm} \text{External work rate}
  \item \( E \) \hspace{1cm} \text{Pa} \hspace{1cm} \text{Young's modulus}
  \item \( E_h \) \hspace{1cm} \text{GPa} \hspace{1cm} \text{Average deformation modulus of the upper part of the earth's crust}
  \item \( e_{ijk} \) \hspace{1cm} \text{Permutation symbol}
  \item \( F \) \hspace{1cm} \text{N} \hspace{1cm} \text{Force}
  \item \( F_1 \) \hspace{1cm} \text{Force per unit mass}
  \item \( F_{\text{max}} \) \hspace{1cm} \text{N} \hspace{1cm} \text{Failure load}
  \item \( G \) \hspace{1cm} \text{Shear modulus}
  \item \( G_{p} \) \hspace{1cm} \text{Plastic potential function}
  \item \( g_s \) \hspace{1cm} \text{Shear flow function}
  \item \( g_t \) \hspace{1cm} \text{Plastic flow function}
  \item \( h \) \hspace{1cm} \text{m} \hspace{1cm} \text{Pillar height}
  \item \( H_c \) \hspace{1cm} \text{m} \hspace{1cm} \text{Cumulative height of coal seams}
  \item \( h_f \) \hspace{1cm} \text{m} \hspace{1cm} \text{Height of the fill}
  \item \( H_r \) \hspace{1cm} \text{m} \hspace{1cm} \text{Cumulative height of roof rock material}
  \item \( H_{\text{T}} \) \hspace{1cm} \text{m} \hspace{1cm} \text{Total height of overburden}
  \item \( I_{\text{W}} \) \hspace{1cm} \text{Internal work rate}
  \item \( J_a \) \hspace{1cm} \text{Joint alteration number}
  \item \( J_n \) \hspace{1cm} \text{Joint set number}
  \item \( J_r \) \hspace{1cm} \text{Joint roughness number}
  \item \( J_{w} \) \hspace{1cm} \text{Joint water reduction factor}
  \item \( K \) \hspace{1cm} \text{Bulk modulus}
  \item \( k \) \hspace{1cm} \text{Horizontal to vertical stress ratio}
  \item \( K_{p} \) \hspace{1cm} \text{Coefficient of passive earth pressure}
  \item \( K_{pp} \) \hspace{1cm} \text{Coefficient dependent on characteristics of coal pillar}
  \item \( L \) \hspace{1cm} \text{m} \hspace{1cm} \text{Original length}
  \item \( l \) \hspace{1cm} \text{m} \hspace{1cm} \text{Pillar length}
  \item \( L_{\text{p}} \) \hspace{1cm} \text{Pa} \hspace{1cm} \text{Average vertical load on the pillar}
  \item \( \text{mb} \) \hspace{1cm} \text{Hoek-Brown constant}
  \item \( n \) \hspace{1cm} \text{Unit normal vector}
  \item \( N_{\text{e}} \) \hspace{1cm} \text{Flow value of the backfill material}
  \item \( p \) \hspace{1cm} \text{Pa} \hspace{1cm} \text{Confining stress}
  \item \( q \) \hspace{1cm} \text{Pa} \hspace{1cm} \text{Plastic multiplier}
  \item \( Q \) \hspace{1cm} \text{Rock Tunneling Quality Index}
  \item \( q_{s} \) \hspace{1cm} \text{Pa} \hspace{1cm} \text{Surcharge load}
  \item \( q_u \) \hspace{1cm} \text{Pa} \hspace{1cm} \text{Unconfined compressive strength}
  \item \( s \) \hspace{1cm} \text{Empirical constant dependent on rock mass characteristics}
  \item \( S \) \hspace{1cm} \text{Slake durability}
\end{itemize}
\( S_c \) Pa In situ coal strength
\( S_i \) Pa Cohesion
\( S_p \) Pa Pillar strength
\( S_v \) Pa In situ vertical stress
\( t \) m Thickness
\( \mathbf{t} \) Traction vector
\( V \) m\(^3\) Volume
\( W \) m Pillar width
\( W_e \) m Entry width
\( Y \) Yield function
\( z_s \) m Depth below surface
\( \gamma_f \) N/m\(^3\) Unit weight of the fill
\( \gamma_r \) N/m\(^3\) Unit weight of the overlying rock
\( \delta l \) m Change in length
\( \varepsilon \) Strain
\( \varepsilon_{\text{axial}} \) Axial strain
\( \varepsilon_i \) Strain rate
\( \varepsilon_{\text{trans}} \) Transverse strain
\( \Theta_p \) Rotation angle required to achieve the principal stress state
\( \nu \) Poisson's ratio
\( \xi \) Strain-rate tensor
\( \sigma \) Pa Stress
\( \sigma_1 \) Pa Minor principal stress
\( \sigma_1' \) Pa Major principal stress at failure
\( \sigma_2 \) Pa Intermediate principal stress
\( \sigma_3 \) Pa Major principal stress
\( \sigma_3' \) Pa Minor principal stress or confining pressure
\( \sigma_c \) Pa Uniaxial compressive strength
\( \sigma_{ci} \) Pa Uniaxial compressive strength of the intact rock
\( \sigma_h \) Pa Horizontal stress
\( \sigma_i \) Pa Stress corresponding to the strain rate
\( \sigma_{\Omega} \) Pa Symmetric stress tensor
\( \sigma_N \) Pa Peak normal stress
\( \sigma_p \) Pa Original pillar strength
\( \sigma_p' \) Pa Backfilled pillar strength
\( \sigma_t \) Pa Tensile strength
\( \sigma_v \) Pa Vertical stress
\( \sigma_x \) Pa Initial normal stress in the x-direction
\( \sigma_y \) Pa Initial normal stress in the y-direction
\( \tau_{\text{max}} \) Pa Maximum shear stress
\( \tau_p \) Pa Peak shear stress
\( \tau_{xy} \) Pa Shear stress
\( \mathbf{v} \) Velocity vector
\( \Phi \) Internal friction angle
\( \Phi_f \) Friction angle of the fill
\( \psi \) Dilatancy angle
\( \Omega \) Angular velocity
\( \omega \) Rate-of-rotation tensor
### Abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>ARMPS</td>
<td>Analysis of Retreat Mining Pillar Stability</td>
</tr>
<tr>
<td>BF</td>
<td>Backfill</td>
</tr>
<tr>
<td>EOD</td>
<td>End of drive</td>
</tr>
<tr>
<td>FCM</td>
<td>Fly ash composite material</td>
</tr>
<tr>
<td>FGD</td>
<td>Flue gas desulfurization</td>
</tr>
<tr>
<td>FISH</td>
<td>FLAC-ish</td>
</tr>
<tr>
<td>FLAC3D</td>
<td>Fast Lagrangian Analysis of Continua in 3 Dimensions</td>
</tr>
<tr>
<td>GSI</td>
<td>Geological Strength Index</td>
</tr>
<tr>
<td>HW</td>
<td>Highwall</td>
</tr>
<tr>
<td>LHD</td>
<td>Load-haul-dump loader</td>
</tr>
<tr>
<td>RMR</td>
<td>Rock Mass Rating</td>
</tr>
<tr>
<td>RQD</td>
<td>Rock Quality Designation</td>
</tr>
<tr>
<td>SAM</td>
<td>Self-advancing Miner</td>
</tr>
<tr>
<td>SF</td>
<td>Factor of Safety</td>
</tr>
<tr>
<td>SRF</td>
<td>Stress reduction factor</td>
</tr>
<tr>
<td>UCS</td>
<td>Uniaxial Compressive Strength</td>
</tr>
<tr>
<td>XRF</td>
<td>X-Ray Fluorescence</td>
</tr>
</tbody>
</table>
1. Introduction

Caterpillar Inc. is an American corporation which is the world’s leading manufacturer of construction and mining equipment, diesel and natural gas engines, industrial gas turbines and diesel-electric locomotives. With the acquisition of Bucyrus International, Inc. in 2011, Caterpillar added that company’s vast range of highwall mining equipment to their portfolio. Highwall mining is a method for mining coal from outcropping horizontal seams. In this method of mining, an unmanned continuous miner is driven underground and operated in front of the highwall.

Although the vast majority of highwall mining operations are currently located in the USA, particularly in the coalfields of the Appalachian region, Caterpillar is looking to expand the market for its highwall mining equipment internationally, with particular focus on Asia and Australia. With this in mind, research into the improvement and optimization of the highwall mining process has become desirable. One of the areas of interest is the application of backfill in the highwall mining process, a technique which is currently already applied in underground mining. Backfilling has the potential to increase recovery and stability, as well as reduce overall mining costs, and could therefore be a valuable optimization tool for the highwall mining industry.

Caterpillar’s interest in investigating the potential of incorporating backfill into highwall mining technology coincides with plans to implement such a system for the first time at West Bokaro Mine in Jharkhand, India. For this reason, West Bokaro was chosen as a case study to study the specific implications at this site, but also as a baseline to draw more general conclusions about the possibility of incorporating backfill into highwall mining.

1.1 Motivation

There is much information available about backfill technology as it is applied in underground mines. This data cannot be directly extrapolated to a highwall mining scenario due to the difference in mining techniques as well as mining depth, and, consequently, the in situ stress situation. Although the application of backfill technology in highwall mining has not yet been put into practice, several authors have investigated the possibility in the past (Clark and Boyd (1998), Hume and Searle (1998), Swiegard and Wang (1996)). These studies were limited in scope but they all suggested that there was significant potential for the application of techniques.

The interest in the possible application of backfill in highwall mining is based on three major potential benefits: increased recovery, increased stability, and increased profit. Increased recovery is an important benefit, because it will result in both a greater output and a longer possible life of the mine. However in some cases increased stability alone could be reason to use backfill. Increased stability results in increased safety, but it is also advantageous in situations where ground control is particularly important, such as when buildings or other important structures are housed on top of the highwall. Most importantly, if the increased recovery is high enough to offset the costs of backfilling, it is possible to decrease the overall cost of mining.

As it is the most important potential advantage in the West Bokaro case, this thesis will focus primarily on increased recovery as a goal.

1.2 Research questions

The purpose of this thesis is to give a recommendation on whether or not to implement backfill technology in highwall mining, both in a general sense and for West Bokaro in particular. Therefore, the main research question of this thesis is defined as follows:
“Can backfilling be an effective tool to increase recovery in highwall mining?”

In order to answer this question, the following sub-questions are defined with regards to a highwall mining scenario:

1. What are the most important factors affecting the efficiency of backfilling?
2. What effect do the various factors have on the efficiency of backfilling?
3. What method should be used to implement backfill technology?
4. Is West Bokaro a suitable location for the implementation of backfill technology?

1.3 Aim and objectives
As stated above, the aim of this project is to be able to give a conclusive and comprehensive recommendation on whether or not to implement backfill into existing highwall mining techniques. To this end, the following objectives are defined:

1. Create a numerical model to investigate the influence of various factors on the efficiency of the backfilling process.
2. Investigate the geomechanical properties of the West Bokaro mine site to determine the in situ stress situation.
3. Assess the potential for increased recovery as a result of backfilling.

1.4 Hypotheses
Due to the history of successful implementation of backfill technology in underground mining, as well as the comparatively simple geometry of a highwall mine, it is hypothesised that the use of backfill technology will be an effective tool to increase the overall recovery. A significant reduction in in situ stress compared to an underground scenario, as a result of a much lower mining depth, should make the placement of backfill comparatively simple.

1.5 Thesis outline
A basic introduction to the project is given in Chapter 2. It starts with a general outline of the case study area, including regional and local geology. Subsequently, it gives explanations of some basic concepts relevant to highwall mining and backfill technology. Chapter 3 provides some theoretical background on basic concepts of rock mechanics and numerical modeling relevant to this thesis. These concepts are further expanded on in Chapter 4, which offers an in-depth examination of the various factors involved in designing a highwall mine with backfilling. This chapter includes an explanation of the theoretical concepts behind pillar stability and selection of backfill methods and materials.

Chapter 5 focuses on experimental work and how the data derived from UCS tests was used to obtain the input parameters required for the numerical model. It gives a detailed description of the geomechanical testing of the various samples. This is followed by an overview and explanation of the numerical model in Chapter 6. Finally, Chapter 7 elaborates on the various simulations and their results, followed by a discussion of the research findings in Chapter 8, and the conclusion and recommendations in Chapters 9 and 10.
1.6 Research scope and limitations

The purpose of this thesis is to give an indication of the potential for increased recovery offered by the application of backfill techniques to highwall mining. The possible scope of this topic is endless, and therefore only the main topics that are relevant to the technical aspect of this question have been selected to be discussed. Table 1 gives an overview of what is included and what is excluded in the scope of the research.

<table>
<thead>
<tr>
<th></th>
<th>Included</th>
<th>Excluded</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Mine design</strong></td>
<td>• Web pillar width</td>
<td>• Barrier pillar width</td>
</tr>
<tr>
<td></td>
<td>• Mine and backfill sequencing</td>
<td>• Number of drives in a panel</td>
</tr>
<tr>
<td></td>
<td>• Backfill material:</td>
<td>• Varying pillar size in a panel</td>
</tr>
<tr>
<td></td>
<td>- Fly ash composite materials</td>
<td>• Feasibility</td>
</tr>
<tr>
<td></td>
<td>- Loose sand</td>
<td>• Environmental factors</td>
</tr>
<tr>
<td></td>
<td>- Cohesion</td>
<td>• Availability of backfill materials</td>
</tr>
<tr>
<td></td>
<td>- Stiffness</td>
<td>• Backfill material:</td>
</tr>
<tr>
<td></td>
<td>• Partial backfill</td>
<td>- Paste material</td>
</tr>
<tr>
<td></td>
<td>• Multi-lift mining</td>
<td>- Aggregate mixes</td>
</tr>
<tr>
<td><strong>Case study</strong></td>
<td>• Local rock mass properties</td>
<td>• Stand-up time</td>
</tr>
<tr>
<td></td>
<td>• General local geology</td>
<td>• Roof coal</td>
</tr>
<tr>
<td><strong>Laboratory experiments</strong></td>
<td>• Uniaxial compressive strength</td>
<td>• Upward-oriented seams</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Effect of mining multiple seams</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Removing all pillars</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Stacking pillars and septums</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Faulting</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Jointing</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Detailed rock mass characterization</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Triaxial tests</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Composition analysis of rock and backfill material</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Failure envelopes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• In-situ stress measurements</td>
</tr>
</tbody>
</table>

*Table 1: Research scope*

The original outline for this thesis also included a general review of the economic feasibility of implementing backfill at West Bokaro. However, due to a lack of available information, it was not possible to do so. Most other excluded factors would be useful to examine in the context of this thesis; however, time restraints prevented them from being included.
2. Project background

In this chapter, an overview is provided of the project background as well as basic theoretical concepts related to the topic of this thesis. Section 2.1 gives a brief description of the West Bokaro mine site. Section 2.2 describes the regional geology, followed by sections 2.3 and 2.4 which concern the basics of highwall mining and backfill technology, respectively.

2.1 Case study area

The West Bokaro mine is an open cast coal mine operated by Tata Steel in the Ramgarh and Hazaribagh districts of the state of Jharkhand in Northeast India, approximately 400 kilometers northwest of Kolkata and approximately 70 kilometers to the northeast of Jharkhand state capital Ranchi. This location was chosen because the application of backfill in highwall mining is currently being considered by the operators. Figure 1 shows the location of the West Bokaro mine site.

![Figure 1: Location of West Bokaro Mine in Jharkhand state, India (Mapbox.com).](image)

The West Bokaro coillery began operations in 1948 as an independent coal company. In 1976, it was made a division of Tata Steel. As one of the few mines in India producing captive coal intended for use in steel production, it was exempted from the nationalisation of Indian coal mines in 1974. West Bokaro’s coal is notable for the low ash content, being between 13-15% after processing in the coal washeries. Current research is being done to lower the ash content even further to 8% (Thomas 2012). The beneficiation plant at West Bokaro is the only one in the world to achieve such a reduction in ash content. These numbers are significantly lower than the typical ash content values for Indian coal (40-45%) and demonstrate why West Bokaro is one of the few Indian coal mines to produce metallurgical grade coal. India’s first coal washery was built on this location in 1951. The coal is transported by rail to Tata’s steel plant at Jamshedpur, approximately 200 kilometers to the south of the
mine site, to be used as coking coal. Total coal reserves at West Bokaro have been estimated at 4,246.30 million tonnes (Singh et al. 2007).

The West Bokaro Coalfield houses several quarries, two of which are currently operational as open-cut mines: Quarry AB, which opened in 1976, and Quarry SEB, which opened in 2003. Annual production for each of these quarries amounts to around 3 million tonnes, with a total production of 2.3 MT of post-processing clean coal for the West Bokaro division as a whole in 2013. Figure 2 shows an overview of the mine site. The quarries are excavated as a truck-and-shovel operation, utilizing 90-tonne haul trucks to transport the coal.

![Figure 2: Overview of the West Bokaro mine site (Google Maps).](image)

### 2.1.1 Regional geology

In order to describe the geology associated with the coalfields in West Bokaro, a short examination of the geology of the entire Indian Subcontinent follows below, as described by Roy (2005).

With the exception of the Himalayas and associated mountain ranges, the Indian Sub-Continent is traditionally considered a shield of Precambrian rocks with a younger cover. The geological evolution of the Indian Sub-Continent took place in two stages. The first stage covered the entire Precambrian, during which period growth and cratonisation took place. During the second stage of its evolution, the cratonised Indian Shield underwent considerable changes that ultimately produced the present-day geomorphology as well as the tectonic character of the region. Figure 3 shows a generalised geological map of the Indian Sub-Continent. On this map, the West Bokaro region is located in the Precambrian Cratonic belt, just north of the Singhbhum Craton (number 3 on the map).
Peninsular India south of the Indo-Gangetic Alluvial Plain is an old landscape, a considerable part of which is covered by Phanerozoic rocks, the Gondwanas, Deccan traps, marine Mesozoic-Tertiary formations, and recent alluvium. The peninsula is a shield area that has remained free of any orogenic deformation since the Cambrian. The Indian Shield is a coherent unit, comprising a number of Precambrian crustal blocks. The crustal blocks include four cratons (Dharwar, Bastar, Singhbhum, and the Aravalli-Budelkhand) and two granulite terrains (Eastern Ghats and Southern Granulite Terrains). The most ancient rocks constituting the basement in each craton are between 3.3 and 3.5 billion years old (Roy 2005).

After a break in sedimentation for over 200 million years between the Ordovician and the Early Permian, the deposition of sediments in the Indian Subcontinent started with the formation of tillites and glacial boulder beds in close association with Permian marine beds. This was accompanied by the deposition of fluvial and fluvio-lacustrine sediments in linear intracontinental rift basins. These sediments, along with intercalated plant remains that ultimately turned into coal seams, constitute the Gondwana Supergroup (Roy 2005).

### 2.1.1.1 The Gondwana Supergroup

India, along with the continents of the southern hemisphere, was part of the supercontinent called Gondwanaland, which existed as a single landmass from the Cambrian until its eventual break-up in phases during the Jurassic to Lower Cretaceous periods. India, Madagascar, Western and Northern Australia and East Antarctica formed East Gondwana while Africa and South America were part of West Gondwana. These two were sutured along the Neo-Proterozoic mobile belt of Arabia - Nubia - Ethiopia - Kenya - Mozambique (Hoffman 1991).

Gondwanaland and the other major supercontinent to the north, Laurasia, were separated by an extensive sea, the Tethys, and into this sea a number of large rivers, draining in a general northerly direction, carried much of the sediment derived from the Indian portion of Gondwanaland.
Following a frigid period during the late Carboniferous, in which the basal strata of the Gondwana system were deposited in the form of the glacial and fluvi-glacial conglomerates of the Talchir formation, a warmer and more humid climate, as well as a change of depositional conditions to shallow freshwater enabled vegetation to flourish (Roy 2005). This included the *Glossopteris* flora, which played a large role in the development of the vast Permian coal deposits of the Southern Hemisphere continents. As a result, during the early Permian, immense volumes of decaying vegetation were incorporated in the sand, gravel and clay debris that was eroded from higher levels, and these sediments were deposited overlying the Talchir strata in a number of the valley areas of Gondwanaland. These lower Permian sediments including the accumulations of vegetation are represented in the Barakar coal measures. They comprise some 2,000 feet of feldspathic sandstones, conglomerates, clays including fireclays, occasional ironstones and the majority of India’s most important coal seams. Evidence suggests that the coal has been formed from accumulations of drift vegetation, rather than in situ (Gee 1940).

Gondwana sedimentation during the initial stage took place in eroded topographic depressions. Various explanations have been suggested for the isolation of the basins as they currently are, including the presence of a master basin followed by faulting and topographic relief, as well as a rift origin of the Gondwana basins, which at a later stage developed into half or full grabens. The present-day basin geometry is a combined effect of faulting in three stages: at the initiation, during sedimentation and after sedimentation. (Roy 2005).

The Gondwana Basins of Peninsular India occur along several major linear belts, as shown in Figure 4. A detailed geological map of the Jharkhand state in Appendix A shows the location of the Damodar-Koel Valley Basins, in which the West Bokaro coalfields are located.

*Figure 4: Map of India showing distribution of Gondwana belts in Pensinular and extra-pensinular India (Bhattacharya et al. 2012).*
The Gondwana basins account for nearly 99% of the coal resource of India. Strata of up to five kilometers thick were deposited over a period of 200 million years, from the Upper Carboniferous to the Lower Cretaceous, and preserved in these basins. Together they form the Gondwana Supergroup. The Gondwana Supergroup is subdivided into the Permo-carboniferous Lower Gondwana Group and the Mesozoic Upper Gondwana Group. Coal seams are found only in the lower group (Mukhopadhyay et al. 2010).

At the end of the Permian, during the major Permian-Triassic extinction event, the sea level regressed to a large extent and an enormous amount of marine species were wiped out. Nearly 80% of marine genera perished, and *Glossopteris* and other peat-forming plants faded out. Coal formation ceased completely. Gondwana sedimentation continued until the Lower Jurassic. The breakup of Gondwana around 165 million years ago and India’s separation from Antarctica and Australia marked the end of Gondwana sedimentation at the end of the Albian period 105 million years ago (Mukhopadhyay et al. 2010).

2.1.2 Local geology
The Bokaro coalfield lies between 23° 45’ and 23° 50’ N. latitude and 85° 30’ and 86° 03’ E. longitude in the district of Hazaribagh, Bihar. It covers an area of 572 square kilometers and includes a narrow belt of Gondwanas extending 65 kilometers from east to west and 10 to 16 kilometers from north to south. (Casshyap 1964). The Bokaro coalfield, which is named for the river that flows through it, is one of several coalfields located in the Damodar Valley. The Lugu Hills divide the Bokaro coalfield into West Bokaro and East Bokaro, as shown in Figure 5.

![Figure 5: Coal fields of the Damodar Valley (Kelafant & Stern 1998).](image)

The West Bokaro field stretches over 64 km in an east-west direction with a maximum width of about 11 km in the north-south direction and covers a total area of 207 km². On the western side, it is flanked by the North Karapuna coalfield, separated only by a narrow belt of metamorphic basement rocks. Structurally, the West Bokaro field consists of two E-W trending synclines separated by a central antiform. These three features converge into a single syncline towards the eastern end of the coalfield. The coalfield is cut by numerous faults and in some places, coal seams have been affected by igneous dikes and sills. The Karharbari and Barakar formations are the main coal-bearing formations. There are 13 major seams in the Barakar (29 seams in all), and one seam in the Karharbari (Holloway et al. 2008).
The coal seams at West Bokaro occur in sediments consisting of beds and interbeds of sandstone, shale, and carbonaceous shale. The coal seams are hard and competent. Agapito Associates, Inc (AAI) compiled an overview of the lithologies and their geomechanical properties in 2005. The results are shown in Table 2.

<table>
<thead>
<tr>
<th>Strata Location</th>
<th>Rock Type</th>
<th>(\sigma_c) (MPa)</th>
<th>(E) (GPa)</th>
<th>(n)</th>
<th>(\sigma_t) (MPa)</th>
<th>S (%)</th>
<th>C (MPa)</th>
<th>(\phi) (deg)</th>
</tr>
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<td>Overburden</td>
<td>Sandstone</td>
<td>35.4</td>
<td>5.4</td>
<td>0.11</td>
<td>5.9</td>
<td>96/95</td>
<td>13.7</td>
<td></td>
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<tr>
<td>10 Immediate</td>
<td>Carbonaceous shale</td>
<td>55.8</td>
<td>7.3</td>
<td>0.21</td>
<td>4.6</td>
<td>98/97</td>
<td></td>
<td></td>
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<tr>
<td>Roof Seam 10</td>
<td>Coal</td>
<td>11.2</td>
<td>2.1</td>
<td>0.05</td>
<td>0.6</td>
<td>96/95</td>
<td></td>
<td></td>
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<td>10 Floor</td>
<td>Shale</td>
<td>49.3</td>
<td>8.2</td>
<td>0.11</td>
<td>10.4</td>
<td>96/95</td>
<td>13.2</td>
<td>39.9</td>
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<tr>
<td>Seam 9</td>
<td>Coal</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td></td>
</tr>
<tr>
<td>8/9 Interburden</td>
<td>Carbonaceous shale</td>
<td>23.1</td>
<td>3.5</td>
<td>0.28</td>
<td>3.6</td>
<td>99/99</td>
<td>10.8</td>
<td>44.6</td>
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<td>4.5</td>
<td>0.18</td>
<td>4.7</td>
<td>97/96</td>
<td>13.8</td>
<td>38.3</td>
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<td>Shale</td>
<td>42.6</td>
<td>9.7</td>
<td>0.13</td>
<td>9.9</td>
<td>98/98</td>
<td>14.9</td>
<td>37.6</td>
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<td>4.0</td>
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<td>3.0</td>
<td>96/94</td>
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<td>2.5</td>
<td>0.21</td>
<td>1.2</td>
<td>98/98</td>
<td>4.0</td>
<td>50.1</td>
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<td>Shale</td>
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<td>6.6</td>
<td>0.13</td>
<td>6.4</td>
<td>100/-</td>
<td>5.2</td>
<td>44.9</td>
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<td>6/7 Interburden</td>
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<td>25.7</td>
<td>4.2</td>
<td>0.21</td>
<td>2.9</td>
<td>97/96</td>
<td>7.5</td>
<td>41.4</td>
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<td>—</td>
<td>—</td>
<td>4.6</td>
<td>98/98</td>
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<td>30.1</td>
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<td>4.1</td>
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<td>97/96</td>
<td>12.4</td>
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<td>Shale</td>
<td>18.3</td>
<td>2.7</td>
<td>0.19</td>
<td>1.1</td>
<td>98/98</td>
<td>—</td>
<td></td>
</tr>
<tr>
<td>5/6 Interburden</td>
<td>Sandstone</td>
<td>36.1</td>
<td>6.6</td>
<td>0.19</td>
<td>5.0</td>
<td>98/98</td>
<td>7.9</td>
<td>44.4</td>
</tr>
<tr>
<td>5 Upper Roof</td>
<td>Shale</td>
<td>22.9</td>
<td>5.3</td>
<td>0.24</td>
<td>8.7</td>
<td>—</td>
<td>8.2</td>
<td>40.4</td>
</tr>
<tr>
<td>5 Immediate Roof</td>
<td>Sandstone</td>
<td>32.5</td>
<td>5.4</td>
<td>0.24</td>
<td>2.6</td>
<td>96/95</td>
<td>9.0</td>
<td>35.4</td>
</tr>
<tr>
<td>Seam 5</td>
<td>Coal</td>
<td>30.7</td>
<td>4.9</td>
<td>0.16</td>
<td>3.1</td>
<td>94/92</td>
<td>9.8</td>
<td>33.6</td>
</tr>
<tr>
<td>All</td>
<td>Sandstone</td>
<td>11.1</td>
<td>2.1</td>
<td>0.07</td>
<td>2.1</td>
<td>98/97</td>
<td>4.7</td>
<td>43.1</td>
</tr>
<tr>
<td>All</td>
<td>Coal</td>
<td>32.1</td>
<td>4.8</td>
<td>0.17</td>
<td>5.0</td>
<td>96/95</td>
<td>11.1</td>
<td>37.0</td>
</tr>
<tr>
<td>All</td>
<td>Shale</td>
<td>45.0</td>
<td>7.0</td>
<td>0.16</td>
<td>7.3</td>
<td>98/98</td>
<td>11.9</td>
<td>39.7</td>
</tr>
<tr>
<td>All</td>
<td>Carbonaceous shale</td>
<td>39.4</td>
<td>5.4</td>
<td>0.25</td>
<td>4.1</td>
<td>98/98</td>
<td>—</td>
<td></td>
</tr>
<tr>
<td>All</td>
<td>Coal</td>
<td>13.7</td>
<td>2.4</td>
<td>0.14</td>
<td>1.3</td>
<td>98/97</td>
<td>4.1</td>
<td>48.4</td>
</tr>
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</table>

Table 2: Summary of Physical Property Test Results (Agapito Associates 2005).

Rock Quality Designations (RQD) values calculated by Tata indicate that the sandstone is the most intact rock type in the strata column, with an average RQD of about 80, followed by shale (70), carbonaceous shale (40) and coal (25). The coal seams themselves are hard and competent. Seam dips are variable across the property, although dips within individual pit areas are consistent from seam to seam, at angles varying from 3° to 7°. Faulting impacts seam continuity at West Bokaro, and Tata Steel has mapped many faults across the property. Jointing studies have not been made; however, AAI observed during a site visit that no preferred jointing direction was evident - an indication that jointing does not have a major impact on stability. Tata Steel has indicated that the groundwater table is somewhere below Seam 7, the lowest seam that has been previously mined; therefore, the possibility exists that groundwater will be encountered during highwall mining of Seams 5
and/or 6. Surface runoff is a major issue during the monsoon season. Flooding of drive entries that are mined downdip is a potential hazard. For this reason, the bottom seams will not be mined during the monsoon season (Agapito Associates 2005).

Agapito Associates (2005) concluded that highwall mining is geotechnically feasible for most of the candidate seams in Banji Village, Quarry D, and Quarry SEB (refer to Figure 2 for locations), and operations are scheduled to start as soon as all required permits have been acquired and the highwall miner has arrived on site. As of August 2014, the highwall miner has not yet arrived and consequently operations have not yet commenced. An overview of the seam thicknesses for Banji Village, Quarry D, and Quarry SEB, as well as a generalized lithology log for Quarry SEB, can be found in Appendices B and C.
2.2 Highwall mining

‘Highwall’ is a term used for the steep face of exposed overburden rock that remains after open pit, open cast, or contour strip mining has taken place. Contour mining is a traditional truck and shovel mining method that consists of removing overburden from the seam in a pattern following the contours along a ridge or around a hillside. Figure 6 shows a schematic overview of such an operation. However, after the stripping limit has been reached, mining this coal by traditional contour mining methods is no longer economically feasible.

![Figure 6: Contour mining (Brikowski 2008).](image)

In the 1940s, a method for extracting the remaining coal seams was developed in the form of auger mining. Until that time, auger drills were used primarily for drilling blast patterns vertically in open cut coal; however, coal mining contractors discovered that those auger drills could be turned horizontal to access the coal remaining in the highwall (Seib 1992), and used this to their advantage in order to maximize the recovery from contour mining sites after the contour mining process itself had outlived its feasibility.

Auger mining, shown in Figure 7, has many advantages, mainly due to the fact that it is a very low-cost method with high recovery rates and flexibility. Moreover, the circular shape of the drill holes ensures stability within the highwall. However, the depth limits of auger mining - initially no more than 30 m - meant that there was still a lot of potential for increased recovery, as the coal seams often stretch back for hundreds of meters. Even after the auger mining system eventually improved its reach to a depth of 120m, large amounts of coal were still left behind and there came an increased demand for a new and better system.

In the 1980s, the first conceptual self-advancing highwall mining machine was developed by the Dutch company RSV. This machine had an initial (horizontal) mining depth capacity of 67 meters, and had several major advantages over the auger mining system. These included increased recovery due to rectangular drives, and the ability to vertically follow variations in the orientation and thickness of the seams (Kleiterp 2010). Since then, the machine has been improved and is now sold by Caterpillar worldwide as the Cat HW300. The vast majority of existing machines are currently in operation in Central Appalachia in the United States, although there are also machines in operation in Russia, India and Indonesia. Non-US sales are expected to expand significantly in the coming years.
2.2.1 Technical aspects

Highwall mining is essentially a hybrid between underground and surface mining. Although it might technically be classified as an underground method, the fact that it is usually applied in former surface mining areas, as well as the fact that it requires no workers to go underground, generates a strong association with surface mining. Figure 8 shows a schematic overview of a highwall mine in operation.

Highwall mining typically generates drives of 3.5 meters in width and, depending on the seam thickness, anywhere from 0.8 - 3 m in height (Caterpillar 2011). Pillar design incorporates regular web pillars and thicker barrier pillars placed at intervals between the web pillars. Barrier pillars serve several important safety functions including stabilization of the highwall and prevention of possible cascading pillar failure. This provides economic benefits through prevention of lost production revenues caused by roof falls or trapped mining equipment (Zipf 2005). Pillar width is determined by various factors including in situ rock strength and seam height. Figure 9 shows a highwall miner being operated at Fola Mine in West Virginia, USA, with the drives clearly visible on the right hand side, and the conveyor belt transporting the coal away from the machine on the left.
Figure 9: Highwall miner in operation at Fola Mine, WV, USA.

The Caterpillar Highwall Mining System, the Cat HW300, consists of three main components: the cutter head, the push beam string, and the base unit which also houses the control system and the reel and chain. Figure 10 shows a schematic overview of the Cat HW300.

Figure 10: Schematic overview of the Cat HW300 (Caterpillar 2011).
The cutter head is pushed into the coal seam and cuts its way through, with a gamma-ray sensor installed on the cutter head which can detect rock boundaries and ensure that the excavated coal is not contaminated with waste rock. As the cutter head cuts through the coal, new push beams are loaded onto the base machine and towed through the drive until maximum mining depth is reached (around 300 m or when the cutter head encounters an impenetrable boundary). Each of the push beams houses a double auger system which transports the coal to the drive entrance. After the drive is completed, the chain is retracted and the push beams are removed. Each of the push beams has a length of only 6 meters; consequently, a large number of pushbeams are required to achieve the desired mining depth. The Cat HW300 also allows for the extraction of thin seams using the Low Seam Cutter Module, which creates drives 2.9 meters in width rather than the standard 3.5 meters.

The current configuration of the highwall mining system allows for the mining of seams with various complications such as varying thickness, undulations and dips (maximum 12°, although steeper seams have been mined), that would have rendered them un-minable by conventional methods. The Cat Highwall Mining system is suitable for open cast mining, contour mining and trench mining.
2.3 Backfill technology

Backfilling is a technique whereby voids generated by the mining process are refilled, usually utilizing some type of mining refuse. Any type of underground mining alters the stress state of the rock mass below the surface and is therefore at risk of causing subsidence of the surface material. This is understandably an undesirable phenomenon, both underground, where subsidence can cause injury to miners and damage to equipment, and on the surface, where structures, natural or man-made, or even whole villages are at risk. Backfill is often applied as a means of improving the underground stability.

Early mining methods either left open voids after the ore had been removed or permitted the caving of the surrounding waste rock. Caving methods often resulted in surface subsidence. Temporary and permanent timber supports enabled larger sized workings and there is a rich history of mining in the 18th and 19th centuries using timber alone (Grice 1998).

In the more recent past, backfill has been used in underground mines, particularly in cut-and-stope operations. Backfill material is introduced underground into previously mined stopes to provide a working platform and localized support, reducing the volume of open space which could potentially be filled by a collapse of the surrounding pillars (Barret et al. 1978). Backfill can be placed through a variety of methods depending on the type of backfill material, including hydraulic transportation for fluid materials and pneumatic or mechanical transportation for solid materials.

Developments in the North American mining industry in the 1980s and 90s led to an emphasis on backfill material that could accommodate high stress conditions and had a rapid curing time. The use of cement in backfill material became increasingly common during this time (Masniyom 2009). Backfill can provide support merely by occupying voids left by mining, thus preventing large-scale movement of the surrounding rock mass, increasing the effective load-bearing capacity of the pillars and minimizing chances of subsidence and collapse. Implementing backfilling technology can be desirable for a number of reasons:

- **Increase in stability.** In some cases where the risk or implications of subsidence of the rock mass are high, backfill can be a viable option to increase stability.

- **Disposal of waste material.** At many mine sites, both surface and underground, there is a significant amount of overburden and processing waste, which requires disposal. Backfill negates the need for large tailings dams at the surface, because it provides an opportunity for large amounts of waste material to be stored underground.

- **Increase in recovery.** The increased stability of the surrounding rock mass generated by backfill allows pillars to be smaller and recovery to be higher.

There are several important factors that must be taken into account before backfill can be considered viable, including material availability, material properties, placement method, mining sequence, as well as cost-efficiency. These will be discussed in detail in Chapter 3. The mechanics of backfilling a highwall drive bear many similarities to backfilling in an underground thin-seam coal mine. In reviewing existing literature on the subject, the focus will therefore be on backfilling thin-seam coal mines, and the mechanics related to backfilling in other situations such as cut-and-fill mines will not be discussed.

For the West Bokaro mine site, a fly ash composite material (FCM) has already been selected as the backfill material to be used. When discussing the selection of backfill materials in this thesis, FCM will therefore be of particular interest.
3. Theoretical background

This chapter will describe some theory surrounding some of the aspects of rock mechanics and numerical modeling related to this thesis. Section 3.1 introduces some basic rock mechanical concepts. Section 3.2 gives an outline on numerical modeling and also gives a basic description of the modeling program used in this thesis, FLAC3D.

3.1 Rock mechanics

Before delving into the specifics of the geomechanical parameters discussed in this thesis, some basic concepts of rock mechanics and rock mass properties relevant to this thesis will be discussed in this section.

3.1.1 Stress

The term stress is used to express the loading in terms of a force applied over a cross-sectional area, typically described using the symbol \( \sigma \) and quantified in Pa, as illustrated in Figure 11.

\[
\sigma = \frac{\text{Force}}{\text{Cross-sectional area}} = \frac{F}{A_0}
\]

Any point within an undisturbed rock mass is always subject to various forms of stress as a result of tectonic activity, the weight of overlying rock strata, confinement, and past stress history (Goodman 1989). In rock mechanics, stress can be divided into three major categories:

Compressive stress is stress acting on a rock mass that is directed towards the center of that mass. At high depths, rock masses experience high vertical compressive stresses as a result of the weight of the overburden (lithostatic pressure). Horizontal compressive stress can be caused by tectonic movements. On a geological scale, horizontal compressive stress can result in thrust faults, while vertical compressive strength can cause normal faulting.

Tensile stress occurs when a rock mass is stretched in two opposite directions. Tensile stress is much less common than compressive stress in rock masses, but can occur, for instance, as a result of tectonic movement or fluid pressure within the rock body. Tensile stress is a common cause of jointing in a rock mass.
Shear stress differs from compressive and tensile strength in that it operates parallel to the surface of the material, rather than perpendicular to it. Shear stress can cause rocks on either side of a plane to slide alongside one another, for instance along a fault. While normal stresses are described by the symbol $\sigma$, shear stresses are typically described by the symbol $\tau$.

Figure 12 shows these three different types of failure.

![Figure 12: Tensile, compressive and shear stress (United States Geological Survey).](image)

There are two main types of stress that a rock mass can be subjected to: in situ stress and induced stresses. In situ stresses are the stresses that are already present in the rock mass and are mainly caused by gravitational sources (weight of the overburden), tectonic stresses (displacement of lithospheric plates), and residual stresses (such as through unloading by erosion or de-glaciation). Induced stresses are caused by external forces acting upon the rock mass, such as through applying loads or removing rock material.

3.1.2 Strain
The term strain is used to indicate the deformation that occurs in a body as a result of stress, for instance when a rod is stretched out as a result of applied tensile stress. It is defined as the change in length per unit length, and is therefore a dimensionless quantity. Deformation in rocks in response to stress often takes place in two stages: the first stage is elastic deformation, in which the deformation is reversible, followed by plastic deformation, in which the deformation is permanent, followed, finally, by failure. Figure 13 shows a stress-strain curve exhibiting the typical elastic-plastic deformation pattern.

![Figure 13: Stress-strain curve showing elastic and plastic deformation (Wikipedia).](image)
The stress at which a rock fails determines its maximum strength. In rocks and soils, the relative displacement is usually quite low and is therefore often measured in microstrains. The strain rate is the change in strain per unit in time, and is usually given in microstrains per second.

Strain can be defined as follows:

\[ \varepsilon = \frac{\text{change in length}}{\text{original length}} = \frac{\delta l}{L} \]  \hspace{1cm} \text{(3.2)}

3.1.3 Stress and strain ratios

There are two important ratios relating to stress and strain that are commonly used as rock mass properties, which are described below.

3.1.3.1 Young’s modulus

Young’s modulus (\( E \)) is a measure of elasticity, equal to the ratio of the stress acting on a substance to the strain produced. The Young’s modulus essentially describes the stiffness of a material, or how susceptible it is to deformation as a result of stress. It is only applicable below a certain maximum amount of stress (the yielding point of the material), in the region where the rock exhibits elastic deformation. The Young’s modulus can be determined by measuring the slope of the stress-strain curve of a material that has been subjected to geomechanical testing. It is defined as follows:

\[ E = \frac{\sigma}{\varepsilon} \]  \hspace{1cm} \text{(3.3)}

where

- \( E \) = Young’s modulus (Pa)
- \( \sigma \) = stress (Pa)
- \( \varepsilon \) = strain

3.1.3.2 Poisson’s ratio

Poisson’s ratio (\( \nu \)) is the negative ratio of transverse to axial strain. Axial strain refers to the deformation in the direction of the stress, while transverse strain refers to the deformation that occurs perpendicular to that direction. When a material is subjected to a stress in the axial direction, it tends to also deform in the transverse directions - transverse expansion in the case of axial compression, or transverse compression in the case of axial expansion. Poisson’s ratio describes this effect (known as the Poisson effect). It is essentially the fraction of expansion divided by the fraction of compression. In most cases, a material will contract when stretched, and expand when contracted, and therefore Poisson’s ratio is usually a positive number. It is defined as follows:

\[ \nu = -\frac{\varepsilon_{\text{trans}}}{\varepsilon_{\text{axial}}} \]  \hspace{1cm} \text{(3.4)}

where

- \( \nu \) = Poisson’s ratio
- \( \varepsilon_{\text{trans}} \) = transverse strain
- \( \varepsilon_{\text{axial}} \) = axial strain
3.1.4 Principal stress

The total stress on a solid body can always be resolved into three vectors in a cartesian coordinate system. A representation of this concept is given in Figure 14, with the body being, in this case, a small, hypothetical cube within the rock mass. For a given orientation of the body, three normal stress components, $\sigma_x$, $\sigma_y$, and $\sigma_z$, either compressive or tensile, act on each of the planes. Additionally, shear stresses ($\tau$) act parallel to the planes.

![Figure 14: Representation of the stress state on a solid body (Calabrò 2013).](image)

For engineering purposes, it is often useful to know the maximum stress acting on a body at any given time, since this stress will determine the strength required for the rock mass to remain intact. However, since the stresses vary with respect to the rotation angle, the maximum normal stress may not necessarily act in the directions illustrated in Figure 14. The direction in which the maximum and minimum stresses act on the body is known as the principal stress direction, with the three normal stresses in that direction known as the principal stresses $\sigma_1$, $\sigma_2$, and $\sigma_3$. Convention dictates that $\sigma_1 \leq \sigma_2 \leq \sigma_3$, with compressive stresses being negative and tensile stresses being positive. Thus, the major compressive stress (in mining, usually the lithostatic pressure) will always be $\sigma_1$, and the major tensile stress will always be $\sigma_3$. Shear stresses are considered positive when they act either in a positive direction on a positive face, or in a negative direction on a negative face. Otherwise, they are negative. By definition, in the principal press state, the shear stress component is equal to zero. Figure 15 shows, in 2D, the rotation of the body from the given coordinate system to the principal stress state.

![Figure 15: Rotation of the body to the principal stress orientation (Efunda).](image)
The principal stresses and the direction in which they act can be calculated from the initial stresses using the following equations:

\[ \tan 2\theta_p = \frac{2\tau_{xy}}{\sigma_x - \sigma_y} \]  
3.5

\[ \sigma_{1,2} = \frac{\sigma_x + \sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2} \]  
3.6

where
- \( \theta_p \) = the rotation angle required to achieve the principal stress state
- \( \tau_{xy} \) = shear stress
- \( \sigma_x \) = initial normal stress in the x-direction
- \( \sigma_y \) = initial normal stress in the y-direction
- \( \sigma_{1,2} \) = maximum and minimum principal stresses

In practice, it is often assumed that the principal stresses, even at depth, act in the horizontal and vertical directions. This assumption is made for various reasons:

- It has been shown to be generally true, through research done into the fault angles of normal and reverse faults at depth (Goodman 1989).
- It reduces the amount of variables and required calculations.
- Excavated faces, which are often horizontal and vertical, are always principal stress faces, with the principal stress acting directly on that face being equal to zero.

The maximum shear stress can be found at an angle, \( \theta_s \), 45° from the rotation angle \( \theta_p \) for the principal stress state. It can be calculated as follows:

\[ \tau_{\text{max}} = \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2} = \frac{\sigma_1 - \sigma_2}{2} \]  
3.7

3.1.4.1 Vertical stresses
Because most other vertical stress components are negligible compared to the lithostatic pressure, the weight of the overlying rock is often the only factor taken into account when calculating the vertical stress component:

\[ \sigma_v = \gamma_r z_s \]  
3.8

where
- \( \sigma_v \) = vertical stress (MPa)
- \( \gamma_r \) = unit weight of the overlying rock (N/m³)
- \( z_s \) = depth below surface (m)

If the overburden is not uniform it can be useful to calculate average density of the overlying rock to calculate the unit weight.
3.1.4.2 Horizontal stresses

Horizontal stresses are far less straightforward to calculate than vertical stresses. They often come about as a result of tectonic movement and are difficult to measure. The ratio, $k$, between horizontal and vertical stress is defined as follows:

$$\sigma_h = k\sigma_v$$  \hspace{1cm} (3.9)

with $\sigma_h$ the horizontal stress and $\sigma_v$ the vertical stress.

Sheorey (1994) developed an elasto-static thermal stress model of the earth, which considers curvature of the crust and variation of elastic constants, density and thermal expansion coefficients through the crust and mantle, summarized in the following equation:

$$k_1 = 0.25 + 7E_h\left(0.001 + \frac{1}{z_s}\right)$$  \hspace{1cm} (3.10)

where $z_s$ = depth below surface (m)
$k_1$ = horizontal to vertical stress ratio
$E_h$ = average deformation modulus of the upper part of the earth's crust measured in a horizontal direction (GPa)

This equation is generally considered to be a reasonable basis for estimating the value of $k$. Additionally, the World Stress Map (Heidbach et al. 2008), which is a worldwide open-access online database of present-day tectonic stresses compiled by a collaboration of academia, industry and governments over the past 30 years, provides reliable data for horizontal stresses for a limited number of locations worldwide.

3.1.5 Failure mechanisms

Failure is a fracture that occurs in rock material when the rock has been stressed beyond its ultimate strength. The possible modes of such failure are numerous: the rock can flow, yield, crush, crack, buckle or fail in a number of other ways. How and when a material fails is largely dependent on the magnitude and the type of the stress imposed upon it, and the strength of the rock mass. Goodman (1989) classified the following types of failure:

**Flexure** refers to failure by bending, with development and propagation of tensile cracks. This may tend to occur in the layers above a mine roof.

**Shear failure** refers to formation of a surface of rupture where the shear stresses have become critical, followed by release of the shear stress as the rock suffers a displacement along the rupture surface. This is common in slopes cut in weak, soil-like rocks such as weathered clay shales and crushed rock of fault zones. Shear zones are characterized by a slick surface with much powder from crushing and comminution of rock. Failure in mine pillars is usually related to shear failure.

**Direct tension** is failure as a result of excessive tensile stress, and can occur, for instance, in rock layers resting on convex upward slope surfaces and in sedimentary rocks on the flank of an anticline, where the lower part of the layer exercises a tensile pull on the upper part, eventually resulting in failure. Tensile failure zones are usually rough and free from crushed rock particles and fragments.
Crushing or compressive failure occurs in volumes of rock that have undergone severe compression. Examination of processes of crushing shows it to be a highly complex mode, including formation of tensile cracks and their growth and interaction through flexure and shear.

3.1.6 Failure criteria

In many engineering disciplines, it is useful to make predictions about the behavior of a rock mass, particularly with regards to failure. In order to predict under what circumstances a particular material may fail, various failure criteria have been established.

3.1.6.1 Mohr-Coulomb

The most well-known failure criterion is the Mohr-Coulomb failure criterion, which relates normal stress and shear stress to failure, and suggests that failure occurs not as a result of either the maximum normal or shear stress, but rather as a critical combination of both. This criterion can be illustrated empirically by plotting Mohr circles for various experimentally obtained principal stresses on the normal vs shear stress plane - in the case of a triaxial compression test, \( \sigma_3 \) being represented by the confining pressure and \( \sigma_1 \) being represented by the corresponding loading pressure. The tangent connecting the circles represents the Mohr-Coulomb failure envelope, shown in Figure 16.

![Mohr-Coulomb criterion](image)

**Figure 16:** The Mohr-Coulomb failure criterion (Goodman 1989).

The Mohr-Coulomb failure criterion dictates that a material will only be stable as long as the combination of principal stresses lies somewhere within its failure envelope; outside of the envelope, failure will occur. The Mohr-Coulomb failure criterion as illustrated in Figure 16 can be described by the following equation (Goodman 1989):

\[
\tau_p = S_i + \sigma_N \tan \phi
\]

where

- \( \tau_p \) = peak shear stress (Pa)
- \( S_i \) = cohesion (Pa)
- \( \sigma_N \) = peak normal stress (Pa)
- \( \phi \) = friction angle
Note that the cohesion, commonly referred to by the letter $c$, is noted here as $S_i$.

In terms of the principal stresses at peak load conditions, the Mohr-Coulomb criterion can be written as follows (Goodman 1989):

$$
\sigma_1 = q_u + \sigma_3 \tan^2 \left( 45 + \frac{\phi}{2} \right)
$$

where

- $\sigma_1$ = major principal stress (Pa)
- $q_u$ = unconfined compressive strength
- $\sigma_3$ = minor principal stress (Pa)
- $\phi$ = friction angle

The Mohr-Coulomb criterion as seen in Figure 16 is limited on the left side of the axis by the maximum tensile strength. The minor principal stress, $\sigma_3$, cannot exceed the maximum tensile stress $-T_0$, because tensile failure will occur. This limit is called the “tensile cutoff”, also referred to as the Rankine cutoff. In effect, the Mohr-Coulomb criterion shows the limits of the stability of a material inside the envelope. Outside of the envelope, failure will occur, either in the form of shear failure (to the right of tension cut-off) or in the form of tensile failure (to the left of tension cut-off). The Mohr-Coulomb criterion with tension cut-off superimposed upon it is generally referred to as the modified Mohr-Coulomb criterion.

The Mohr Coulomb criterion is a relatively simple method of predicting failure and is therefore the most widely-used criterion in the industry. However, it is not surprising that such a relatively simple method has many limitations. Some of the drawbacks of the Mohr-Coulomb criterion include:

- It assumes that the major mode of failure is shear failure. This is not the case for all rock types, and in those cases Mohr-Coulomb is likely to give incorrect interpretations.
- It is linear, while most experimentally determined failure envelopes are non-linear (parabolic).
- It assumes that friction and cohesion are related, which is not necessarily the case.
- It considers only of the major and minor principal stresses, and neglects $\sigma_2$.

### 3.1.6.2 Hoek-Brown

Other failure criteria have been developed. Hoek and Brown (1980) used empirical data to formulate a criterion particularly suitable for brittle, jointed rocks. Their criterion was based on a parabolic version of the Mohr-Coulomb failure envelope, and was further developed through trial and error. The generalized Hoek and Brown failure criterion is described as follows (Hoek 1990):

$$
\sigma_1' = \sigma_3' + \sigma_{ci} \left( m_h \frac{\sigma_1'}{\sigma_{ci}} + S \right)^a
$$

where

- $\sigma_1'$ = major principal stress at failure (Pa)
- $\sigma_3'$ = minor principal stress or confining pressure (Pa)
- $\sigma_{ci}$ = uniaxial compressive strength of the intact rock (Pa)
\[ m_b = \text{Hoek-Brown constant} \]
\[ s = \text{empirical constant dependent on rock mass characteristics} \]
\[ a = \text{empirical constant dependent on rock mass characteristics} \]

There are many other, less common failure criteria which include various modifications of Mohr-Coulomb, Drucker-Prager and Griffith, among others, but they will not be discussed here.
3.2 Numerical modeling

Numerical modeling is a branch of mathematics that is concerned with the approximation of solutions by means of numerical analysis, rather than with absolute solutions. Although there are many physical and mathematical equations that describe various processes in areas such as natural sciences, business and engineering, these are generally impossible to solve analytically for many real-life situations. For this reason, numerical analysis has become an integral part of the research done into these fields. Statistical analysis shows that non-trivial mathematical models and methods are used in 70% of the papers appearing in the professional journals of engineering sciences (Vuik et al. 2007).

Numerical modeling is not only useful in situations where an analytical solution is impossible to obtain. It is almost always a cheaper and more practical alternative to experimental work. Moreover, most systems that require modeling nowadays are so complex or so extensive in scope that obtaining an experimental solution would be impractical. The significant progress made in computer technology over the past decades has made it possible to generate detailed and reliable models of all types of processes at increasingly high speeds.

Various fields of numerical analysis exist, including those focusing on optimization, integrals and differential equations. Differential equations can be used to describe most systems that are experiencing change, which is appropriate for this thesis, as the main hypothesis concerns itself with the change in stress states that the rock mass experiences during the mining process. Using numerical analysis, differential equations can be examined by three methods: finite element, finite difference and finite volume. In this thesis, the finite difference method was used due to its relative simplicity and ease of use. The method is particularly suitable for the relatively uncomplicated geometry that is associated with highwall mining.

3.2.1 Finite difference modeling

Finite difference modeling is based on interpolating unknown values from known function values at nearby points. The difference between such points gives an indication for the required differential value, incorporating a truncation error which must be sufficiently small for the approximation to be reliable. Three different mechanisms exist for finite difference modeling:

1. Forward differentiation
2. Backward differentiation
3. Central differentiation

The use of these three methods for first-order differentials are described below and illustrated in Figure 17.

![Figure 17: Different mechanisms for finite difference modeling.](image-url)
**Forward differentiation** approximates the value by interpolating from a value for the function at point \( x + h \), with \( h \) being the step size:

\[
f'(x) \approx \frac{f(x + h) - f(x)}{h}
\]  

The truncation error \( \tau_n \) is found using a Taylor expansion of \( f(x+h) \) around point \( x \):

\[
\tau_n(h) = f'(x) - \frac{f(x) + hf'(x) + \frac{h^2}{2} f''(\xi) - f(x)}{h} = -\frac{h}{2} f''(\xi)
\]

**Backward differentiation** approximates the value by interpolating from a value for the function at point \( x - h \):

\[
f'(x) \approx \frac{f(x) - f(x - h)}{h}
\]

Similarly, the truncation error for backward differentiation is found as follows:

\[
\tau_n(h) = \frac{h}{2} f''(\xi)
\]

**Central differentiation** combines the two previous methods, approximating the value by interpolating from both sides:

\[
f'(x) \approx \frac{f(x + h) - f(x - h)}{2h}
\]

Using the same method of Taylor expansion, the truncation error is as follows:

\[
\tau_n(h) = -\frac{h^2}{6} f'''(\xi)
\]

It is clear that the truncation order of the central differentiation is much lower than that of the forward or backward differentiation. This demonstrates that the central differentiation is the most accurate method.

The numerical approximation will clearly be more accurate as step size \( h \) becomes smaller. The truncation error can never be completely eliminated. Determining which truncation error size is acceptable will depend on several variables, including required accuracy, available time and computational capacity.

Numerical solutions can be implicit or explicit. Explicit methods can be directly deduced from available data (estimate a value later by using a value from the current time), whereas implicit methods require additional calculations (estimate a value later by using both the current value and the later value). It is clear that explicit methods will be less time-consuming because they require less calculations, although implicit methods may have benefits in increased accuracy.
3.2.2 FLAC3D

*FLAC3D* (Fast Lagrangian Analysis of Continua in 3 Dimensions) is an explicit finite difference program to study, numerically, the mechanical behavior of a continuous three-dimensional medium as it reaches equilibrium or steady plastic flow. The following description of the program is provided in the user manual by Itasca:

“*FLAC3D* is a three-dimensional explicit finite-difference program for engineering mechanics computation. The basis for this program is the well-established numerical formulation used by our two-dimensional program, *FLAC*. *FLAC3D* extends the analysis capability of *FLAC* into three dimensions, simulating the behavior of three-dimensional structures built of soil, rock or other materials that undergo plastic flow when their yield limits are reached. Materials are represented by polyhedral elements within a three-dimensional grid that is adjusted by the user to fit the shape of the object to be modeled. Each element behaves according to a prescribed linear or nonlinear stress/strain law in response to applied forces or boundary restraints. The material can yield and flow, and the grid can deform (in large-strain mode) and move with the material that is represented. The explicit, Lagrangian calculation scheme and the mixed-discretization zoning technique used in *FLAC3D* ensure that plastic collapse and flow are modeled very accurately. Because no matrices are formed, large three-dimensional calculations can be made without excessive memory requirements. The drawbacks of the explicit formulation (i.e., small timestep limitation and the question of required damping) are overcome by automatic inertia scaling and automatic damping that does not influence the mode of failure. *FLAC3D* offers an ideal analysis tool for solution of three-dimensional problems in geotechnical engineering.”

In short, *FLAC3D* is a tool that can be used to study the behavior of a rock mass and the stress within as various forces act upon it. Using *FLAC3D*, it is possible to generate a 3D model of the mine site in which the extraction and backfilling process can be simulated using a large amount of varying parameters. Based on the input parameters, the model can then calculate the resulting stress state as well as the displacement. The basic premise of the model is as follows:

1. A three-dimensional representation of the mine site is created. The code for this geometrical framework is saved under the FISH extension.

2. Depending on the simulation scenario, a list of commands is defined which describe, in this case, the extraction and backfilling processes. This code is saved as a DAT file.

3. The required values with regards to dimensions and rock mass properties are determined and entered into the DAT file.

4. The DAT file is run and the simulation takes place. The calculation time required for this step depends on the resolution of model as well as desired accuracy. The higher the resolution and the accuracy, the longer the calculation time.

5. After the simulation has been completed, the results can be studied. The model can give information about the development of variables such as stress, displacement, and failure modes throughout the simulation.

Additional theory on *FLAC3D* is provided in Chapter 6. The results of the simulations carried out for this thesis are described in Chapter 7.
4. Mine design

The most important factor affecting the efficiency of the backfilling technique is mine design. Further on in this thesis, various aspects related to mine design will be tested to determine their influence on the backfilling process. This chapter introduces several relevant concepts. Section 4.1 gives an overview of ground control issues that must be countered through mine design. Section 4.2 focuses on highwall mine design, with an emphasis on pillar design. Section 4.3 concerns the design of backfilled drives as well as the various factors involved: pillar strength, delivery systems, and material selection, with a particular emphasis on backfill containing fly ash composite material, the type of material that will be used to backfill the drives at West Bokaro. Section 4.4 combines highwall and backfill design and discusses various variables that can influence the efficiency of the mine design, and which are the focus of this thesis. Section 4.5 contains a brief discussion on the cost analysis aspect of mine design.

4.1 Ground control

In underground mines, there is usually an abundance of rock mechanical data available for mine planning purposes, as the implications of failure can be very severe at such great depths. In surface mining, implications of failure are much less severe, and usually do not justify the cost of the equipment required to obtain exact in situ data. Many calculations in planning a surface mine are therefore done using estimates, rather than the exact data that is used in underground mining. However, highwall mining brings with it new risks that are not present in surface mining, and the use of exact data becomes much more important in order to prevent the ground control hazards associated with it. The two most significant such ground control hazards in a highwall mining setting are rock falls from the highwall and equipment entrapment underground (Zipf and Mark 2007). Shen and Duncan Fama (2000) identified three major geomechanical issues associated with highwall mining:

- **Highwall stability.** The stability of the highwall is particularly important because mining operations take place beneath the highwall, and equipment as well as personnel are at risk. Highwall stability issues can be subdivided into mass instability (deep seated movement of rock mass into the pit, caused by subsidence or major faulting); face instability (failure at the highwall face due to incomplete pre-split blasting or joints parallel to the highwall face); and block instability (localised falls of rock blocks or wedges from the highwall face at various sizes).

- **Pillar/panel stability.** Pillar failure can result in caving of the overburden and a resulting loss of equipment. Pillar design is therefore of paramount importance.

- **Span stability.** Drives must be designed with a stand-up time that allows for safe mining without the danger of roof collapse. Stand-up time is usually estimated using Bieniawski’s Rock Mass Rating (RMR), which assigns the rock a quality number ranging from 0 to 100. Stand-up time is then estimated using a table constructed from empirical data, which is shown in Figure 18.
An additional problem in ground control for highwall mining, particularly in the central Appalachians, where the majority of highwall mining occurs in the U.S., are hillseams. Hillseams are near vertical fractures in the rock that are formed in response to natural weathering and erosion of hillsides (Medhurst 1999). These fractures may cause vertical wedges or long rectangular slabs to separate from the highwall (Zipf and Bhatt 2004). In order to avoid problems with hillseams, Zipf and Mark (2007) recommend good practices with regards to planning, inspection and monitoring and blasting, as well as decreasing the highwall slope angle and avoiding mining near ridge points.

4.1.2 Implications of failure
One of the most significant ground control issues in highwall mining is the risk of subsidence. Other mining techniques that produce associated surface subsidence include longwall mining, sub-level caving, room-and-pillar mining, block caving and stope mining (Whittaker and Reddish 1989). The surface impact that occurs as a result of the extraction of underground material is a large shallow depression or basin in the ground, which is usually circular or elliptical in shape, depending primarily on the geometry of the mine workings and the geological conditions. There are four ways in which the risk of subsidence as a result of mining activities can be reduced (Masniyom 2009):

1. Alteration in mining techniques
2. Post-mining stabilization
3. Architectural and structural design
4. Comprehensive planning

Alteration in mining techniques can be accomplished through a variety of methods including partial mining, backfilling, mine layout or configuration, and extraction rate. In addition to improving stability, backfilling may also have a beneficial effect on the environment by addressing water quality impacts, reducing waste rock disposal requirements, reducing ground fissuring, and increasing long-term strata stability and providing roof
support. Backfilling does not eliminate subsidence entirely, but only reduces the amount of subsidence (Masniyom 2009).

In case of improper design or monitoring, failure of the highwall can occur, with disastrous consequences. Unexpected movement of ground causes the potential to endanger lives, demolish equipment, or destroy property. Between 1995 and 2001, approximately 15% of all surface mine fatalities were caused by ground instability (Girard 2001). An example of a fatal highwall collapse occurred on May 24, 2000 in the Coalburg highwall miner pit at the in Martin County, KY, shown in Figure 19. The accident occurred due to the highwall mining activity taking place in an area where an unsafe ground condition existed. The highwall collapsed on top of the highwall mining machine, and resulted in a fatality (Newberry and Bellamy 2000).

![Figure 19: Highwall collapse in Martin County, KY (Newberry and Bellamy 2000).](image)

An additional ground control-related safety concern with highwall mining is a “stuck” or trapped miner and the ensuing retrieval or recovery operation. Anecdotal evidence suggests that many trapped miners result from ground control problems such as roof falls, web pillar failures, or floor failure in multiple lift mining. When a highwall miner gets trapped it may be retrieved by means of either surface retrieval, surface excavation, or underground recovery. However, each of these methods brings with it additional costs and hazards. In spite of this, data collected in 2003 regarding trapped miners suggests that the odds are that about 1 in 4 highwall miners will become trapped during any given year and require a major recovery/retrieval effort (Zipf and Bhatt 2004).

In addition to safety concerns, the protection of the environment has a high priority in modern mining. While highwall mining is generally a particularly non-invasive mining method, highwall collapses can have disastrous consequences on the local environment, particularly if any structures are situated on the surface above the drives. Highwall failures can release an enormous amount of energy: one catastrophic highwall collapse at a mine in New South Wales, Australia, released sufficient energy to register as a magnitude 4 earthquake and propelled rock and coal horizontally from web entries for over 90m, embedding them in the neighbouring highwall (NSW Department of Primary Industries 2008). In large mines, the damage can be even more severe, as evidenced by a massive highwall failure in Bingham Canyon Mine in April 2013, as shown in Figure 20 (Romero and Adams 2013). When the extent of failure is so significant, it can impact ground stability for many miles beyond the mine. Therefore it is imperative to not only minimize the risk of highwall failure, but also to constantly monitor its presence and take appropriate precautions when failure appears inevitable.
4.2 Pillar design

In order to ensure stability, pillar design is the most important aspect in highwall mine design. Two types of pillar are used in highwall mining: web pillars and barrier pillars. Figure 21 shows an example of web and barrier pillars in an underground room-and-pillar mine design. The pillars should be designed in such a way that the web pillars are able to support the entirety of the overburden. However, in the event of a web pillar collapse, the barrier pillars must be capable of containing the failure. Vandergrift and Garcia (2005) identified the three basic steps involved in web and barrier pillar design:

1. Application of empirical pillar design formulas
2. Back-analysis of available information from past augering operations
3. Numerical modeling analysis to confirm design performance and test its robustness.

Although numerous empirical equations have been developed over the years with regards to pillar design, the most widely-accepted formula in the United States is the Mark-Bieniawski pillar design formula (Mark et al. 1995):
\[ S_p = S_c \left[ 0.64 + \left( 0.54 \frac{W}{h} \right) - \left( 0.18 \frac{W^2}{hl} \right) \right] \]

where  
\( S_p \) = pillar strength  
\( S_c \) = in situ coal strength  
\( h \) = pillar height  
\( W \) = pillar width  
\( l \) = pillar length

In highwall mining, the length of the drive (around 300m) is much greater than pillar height or width, and therefore, the final term becomes negligible, resulting in the following equation (Vandergrift and Garcia 2005):

\[ S_p = S_c \left[ 0.64 + \left( 0.54 \frac{W}{h} \right) \right] \]

In situ strength can be difficult to determine, particularly in the case of coal mining. Mark and Barton (1997) report that various discontinuities common in coal bodies, including microfractures, cleats, bedding planes, partings, shears and small faults, make the quantification of in situ coal strength difficult and often unreliable. In practice, the assumption of an in situ strength of 6.2 MPa (900 psi) for all seams tends to yield more reliable results than using laboratory UCS results for each individual seam. The reason for this is likely that the parameter tested in the laboratory - intact coal strength - bears little relevance to the in situ coal strength, which is influenced by the many discontinuities and cleats in the coal structure. It should be noted that, although similar results have been found in Australia and South Africa, these findings are based on coal seams in the United States, for the purpose of being used with the software program ARMPS (Analysis of Retreat Mining Pillar Stability), and may not be representative for other locations or different types of software.

Because of the large factor of uncertainty, pillars must be designed with a safety factor against failure. The selection of a safety factor is usually based on mining experience or back-analysis of pillar failures. It is also dependent on specific knowledge of the underground environment such as roof and floor conditions (Donovan and Karfakis 2004). It is important that the selection of an appropriate safety factor is done with care, as an overly optimistic design poses too much risk for mining personnel and equipment, whereas an overly conservative design causes unnecessary loss of resource (Shen and Duncan Fama 2000). Regulations usually stipulate a minimum factor of safety of 1. To determine the safety factor, an estimate of pillar loading is required, which is calculated as follows (Vandergrift and Garcia 2005):

\[ L_p = S_V \left( W + W_E \right) / W \]

where  
\( L_p \) = average vertical load on the pillar  
\( S_V \) = in situ vertical stress  
\( W \) = pillar width  
\( W_E \) = entry width

Entry width (the width of the drive) is determined by the width of the cutter head and is 3.5 or 2.9 meters in the case of the Cat HW3000. In situ vertical stress is determined by the depth and density of the overburden material. The vertical stress gradient is typically 0.025 MPa/m. Overburden depth may be taken as the maximum
overburden depth on a highwall mining web pillar, which is very conservative, or alternatively as a high average value computed as (Zipf 2005):

\[
D_{\text{Design}} = 0.75 \times D_{\text{MAX}} + 0.25 \times D_{\text{MIN}}
\]

where \( D_{\text{MAX}} \) = maximum overburden depth
\( D_{\text{MIN}} \) = minimum overburden depth.

The safety factor is then calculated as follows:

\[
SF = \frac{S_p}{L_p}
\]

Using a predetermined safety factor - common numbers are around 1.5 - the pillar width can then be determined. If a high safety factor has been taken into account for designing the web pillars, the likeliness of failure will be relatively low and a lower safety factor can be used for the barrier pillars (Vandergrift and Garcia 2005).

Additional caution is required when designing pillars in unusual situations, such as highwall mining through old auger holes and close-proximity multiple seam highwall mining. Zipf (2005) found that the strength of a highwall mining web pillar containing auger holes is 25% to as little as 15% of the solid web pillar strength and that the strength of such a pillar is independent of its \( W/H \) ratio, although the detrimental effect of the auger holes diminishes as they are spaced further apart. For close-proximity multiple seam highwall mining, conventional pillar design methods do not apply well to closely spaced seams less than about one highwall miner hole width apart. It is important in such a situation to carefully stack the pillars. This is relatively straightforward when mining downwards, but becomes more complicated when mining bottom-up, after backfill of the pit has obscured the location of the lower-level pillars.
4.3 Backfill design

Highwall mining is particularly suited as an area of application for backfill, as recovery rates are relatively low (<40%) and there is much room for improved recovery. There are various reasons to apply backfill in highwall mining in different parts of the world; depending on the mine there may be a demand for increased production, improved stability, or both. An important advantage of backfilling in highwall mining is an improvement in safety conditions. This is particularly relevant in Australia, where safety regulations tend to be more stringent than elsewhere. Backfill applied to highwall mining is envisioned as a means to increase recovery of mineable coal to levels comparable with underground longwall mining, i.e. approximately 70% in plan area (Clark and Boyd 2008). A small amount of research into the possibility of backfilling highwall mine adits has already been performed.

In designing a highwall mine backfilling scenario, consideration needs to be given to the following parameters (Clark and Boyd 1998):

- the strength of coal pillars as a function of confinement;
- sequence of extraction and backfilling;
- in the case of multi-lift, thick-seam mining: whether a septum (horizontal pillar) be left between vertically aligned openings;
- quality of backfill.

The most important concept of backfill is that the fill itself does not support the overburden or bear any vertical load until deformation of the roof has occurred. In the case of backfilling a Self-Advancing Miner (SAM) drive, it is impossible to fill the drive one hundred per cent; therefore, the fill and roof will not be in contact immediately following fill placement. Research and in situ testing have shown that fill cannot rigidly support the total weight of overburden and acts only as a secondary support system (Cai 1983).

The additional strength that the fill provides to the pillars is imparted as a horizontal pressure along the sides of the pillars. The subsequent increase in pillar strength is due to the confinement provided by the backfill. The resistance is contingent upon deformation of the roof and pillars; thus, the fill acts as a passive support. In order to promote yielding the coal pillars will have to be ‘underdesigned’, or purposely left smaller so that the load bearing capacity of the pillars is significantly less than that required to support the overburden pressure (Donovan and Karfakis 2004). Figure 22 shows the progression of the pillar support.

Donovan and Karfakis (2001) note that in order to restrain the pillars, the backfill should have similar or greater initial stiffness than the coal pillar. However, they also indicate that even very compressible fills will eventually develop passive resistance to surrounding deformations based solely on volumetric constraints.

![Figure 22: Development of fill pressure and passive restraint (Donovan and Karfakis 2004).](image)

Sometimes backfill only acts as a void filler and needs only sufficient strength to prevent any form of remobilisation. Where backfill is used as an engineering material it requires sufficient strength. Lean cement addition is used to generate unconfined compressive strengths ranging from 0.5 to 4 MPa (Grice 1998).
4.3.1 Design of backfilled coal pillars

For cut-and-fill mining, mine design incorporating backfill has been developed mostly from experience. However, a method for determining the strength of backfilled pillars using earth pressure theory was developed by Donovan and Karfakis (2004). The concept utilizes the idea that the passive resistance that the pillars exhibit can be considered similar to passive earth pressure, which is defined as the state of maximum resistance mobilized when a force pushes against a soil mass and the mass exerts a resistance to the force.

Donovan and Karfakis developed the following equation to determine the horizontal earth pressure and combined it with an equation from Cai (1983) for strength of a pillar subjected to a confining pressure to create the following expression for determining the strength of a backfilled pillar:

$$\sigma_p' = \sigma_p + K_{pp} \times \left( \gamma_f \times h_f \times K_p + 2 \times c_f \times \sqrt{K_p} + q_s \times K_p \right)$$

4.6

where

- $\sigma_p'$ = backfilled pillar strength (MPa)
- $\sigma_p$ = original pillar strength (MPa)
- $K_{pp}$ = coefficient dependent on characteristics of coal pillar
- $\gamma_f$ = unit weight of the fill (N/m$^3$)
- $h_f$ = height of the fill
- $c_f$ = cohesion of the fill (Pa)
- $q_s$ = surcharge load (Pa)
- $K_p$ = coefficient of passive earth pressure. It is equal to the flow value $N_{\phi}$ of the backfill material where:

$$N_{\phi_f} = \tan^2 \left( 45 + \frac{\phi_f}{2} \right)$$

4.7

and $\phi_f$ = friction angle of the fill.

The “new” pillar strength can be readily substituted into the Mark-Bieniawski pillar strength formula (Equation 4.1).

4.3.2 Pillar stability

Sweigard and Wang (1996) used finite element modeling to develop a preliminary understanding of whether or not a highwall drive, having been filled with an FGD waste mix, could be stable. One of the main concerns in this case is the compatibility of the deformation of the rock roof to that of the backfilled FGD materials. Laboratory testing programs were used to obtain the optimum mix design parameters of the FGD material and the rock characteristics around the adits. For the FGD material used in this study, a 12% prehydrated FGD mix with 31% water provided highest strengths, lowest mixing temperatures and relatively high expansive property qualities, which would ensure complete backfilling of highwall openings. For this model, laboratory testing programs were used to obtain the optimum mix design parameters of the FGD material and the rock characteristics around the adits, including unconfined compressive strength, Young’s modulus, and Poisson’s ratio. The vertical stress component was calculated using the weight of the overlying rock mass, and a $k$ value of 0.4 was used to determine the horizontal stress component. The material properties used in this simulation, as well as the results, can be found in Appendix D.
Due to the backfilling with FGD materials, the size of the coal pillar could be reduced and an increase of up to 30 percent in coal recovery could be obtained. However, it was stressed that adequate roof stress and displacement data for the backfilled pillar are essential to the evaluation of the feasibility of enhanced recovery using FGD material for support. Therefore, it is essential in similar studies to have a sufficient amount of data related to stress and strain distribution in the rock mass.

4.3.3 Backfill classifications

In choosing a backfill material for a mine site, many different factors must be taken into consideration. Figure 23 shows a flowchart for the systematic selection process of a backfill material which takes into account all of these factors.

![Flowchart of systematic selection and application of backfill flowsheet](image-url)
Practical considerations such as cost and availability, as well as environmental restrictions, are the main deciding factors. The primary source of backfill material must be local to the mine, and the material must be strong enough to support mining equipment and/or subsequent backfill exposures with acceptable stability and minimal failures, to ensure complete extraction of ore reserves. The cost and benefit of adding binders, which can significantly increase the strength of the backfill material, must be considered, and preliminary testing must be done to determine the most cost-effective mix of materials. If possible, the use of commercial by-products such as fly ash can be advantageous. Preliminary testing to determine the effect on things such as the impact of the backfill material on ore recovery and the environment is essential. Finally, it is important to accept that the optimum system is the one that contributes to achieving the maximum value for the entire operation, not necessarily the lowest capital cost or the lowest cost per tonne of backfill placed. Therefore the backfill system selection cannot be made in isolation, but must be integrated into the overall design and optimisation process (Masniyom 2009).

The most common types of backfill material are waste rock, mill tailings, quarried rock, sand, and gravel. Of these materials, waste rock and mill tailings, generated during the mining process, are the most accessible and usually the most economic. Waste rock and tailings come from three major sources: (1) development work of the mine, (2) washer-concentrator reject, and (3) old refuse piles. Additives such as portland cement, lime, pulverized fly ash, and pastes are also added to the fill in order to alter its characteristics and improve its effectiveness (Donovan 2001). For the purposes of backfilling the highwall mine at West Bokaro, a specific backfill material incorporating fly ash has been developed. For this reason, the initial focus of this project will be on similar fly ash composite materials (FCMs). In order to determine the relative value of FCMs as backfill material, several other types of material will also be discussed. The following sections will discuss the various options available with regards to backfilling methods and materials.

4.3.3.1 Backfill methods
A wide variety of backfill methods has been developed over the years. Masniyom (2009) distinguishes between four major backfill techniques: mechanical backfill, pneumatic backfill, hydraulic backfill, and paste backfill.

4.3.3.1.1 Mechanical backfill
Mechanical backfill is a method in which the backfill material is mechanically slung into voids created by mining, using a variety of equipment. Such equipment includes LHDs, front-end loaders, slinger belts, mobile scoops and ram cars. Mechanical backfill methods also include gravity transport, which makes use of gravity to move material down vertical or inclined chutes, boreholes or pipes. Mechanical backfilling techniques are generally used for the backfilling of waste rock and for the construction of packwalls. Hume and Searle (1998) list the following mechanical backfill delivery methods:

- Conveying with slinger discharge: using the augers in reverse to place the backfill material. After the mining phase has been completed, the filling system is placed on retreat. Filling with this technique is a discontinuous operation as no filling will occur while the conveyor system is located to depth in each hole.

- Remote trucking with slingers. This method is similar to conveying with slinger discharge, but replacing the conveying segment of fill placement with remotely controlled trucks. Filling could be relatively continuous.

- Remote trucking with rammer: After fill placement by remotely controlled trucks, a rammer is used to compact the backfill material.
• Filling on retreat using the mining unit. This seems perhaps like the most logical option, since no additional machinery would be required, although the highwall mining machine itself would need an adjusted design. For this scenario, hydraulic fill would not be suitable as the miner would be flooded. Instead, a dry or damp aggregate could be used that is projected past the cutting head. However, this would require significant modification of the miner, as at present this is not yet possible with the CAT HW300.

4.3.3.1.2 Pneumatic backfill
Pneumatic backfill utilizes pressurized air to move the material along a pipeline. In order to be suitable for pneumatic backfill, the backfill material must be dry and free-flowing. Although backfill placement using this method is relatively simple, the use of dry backfill material can cause excessive wear on conveying pipelines. An effective dust control system is required when this method is applied underground (Spearing and Wilson 1988). Pneumatic systems must be run continually to prevent damage, so backfill material must be continually available. In recent years, systems have been developed to supply material at up to 150 m³/h on a continuous basis and the discharge nozzle can be directed to optimise fill placement, particularly to ensure tight filling to the roof (Hume and Searle 1998).

4.3.3.1.3 Hydraulic backfill
Hydraulic backfill incorporates the use of water as a transporting agent to create a slurry that is transported through pipes. It consists of sand- to silt-sized particles of aggregate, water, and usually some type of binder. It generally results in a backfill strength that is much higher than the strength achieved using pneumatic backfilling. Hydraulic backfilling generally makes use of a slurry containing solid waste material and binding agents such as cement, and requires expensive abrasion-resistant pipes and slurry lines. Because of the high cost of such materials, much attention must be paid to the economic feasibility of this method. Whether or not the implementation of a hydraulic system is possible is also dependent on the local availability of water. Hydraulic systems may be open (one-way flow) or circulating. For open systems, mine water is generally supplied by existing dewatering systems within the mine. Circulating systems recycle the water used to transport the fill material back to the surface to be used again. Hume and Searle (1998) list the following hydraulic backfill delivery methods:

• Borehole placement of fluid aggregates. Fill is placed via medium to large diameter boreholes (150 to 300 mm) drilled from the surface at regular intervals along the entry drive alignment. In order to get the fill to spread laterally from each borehole, it will have to have the consistency of a very fluid concrete.

• Hydraulic delivery. For backfill material with increased fluidity, pumping or gravitation through pipelines and boreholes becomes an option. Suitable materials could be tailings, ash or screened alluvials with or without coarse washery reject or crushed spoil pile material. For sufficiently down-dipping holes, hydraulic fill could be placed from the collar only. For up-dipping holes, borehole or sacrificial fill line placement could be used.

4.3.3.1.4 Paste backfill
Paste backfill is similar to hydraulic backfill, but contains a larger fraction in the silt size particle range and is therefore much denser. Paste backfill has a lower water content than hydraulic backfill and its transport mechanism is plug flow, rather than slurry flow. This type of flow is characterized by a slow moving annulus of fines that coat the sidewall and a central plug moving at a relatively greater velocity (Keshvary 2013). This provides a benefit over hydraulic backfill, as the paste does not settle in the pipe in case of blockage, in contrast
to hydraulic backfill. However, paste backfill requires gravity feeding, shown in Figure 24, which can be a disadvantage. Although hydraulic fill is usually preferred over paste fill, decreasing capital costs for paste fill have resulted in an increase in its use.

*Figure 24: Paste backfill pouring into a stope (Google Images, Photo © H. Rumbino).*

Comparing these strategies, Hume and Searle (1998) concluded that, for backfilling in a highwall mine, filling on retreat using the mining unit was the most cost-effective method, with borehole placement of fluid aggregates, hydraulic delivery and pneumatic delivery also within reasonable ranges. They found that other methods were disproportionately more costly.

### 4.3.3.2 Backfill materials

There are two basic types of backfill materials. The first, uncemented backfill, does not make use of binding agents such as cement. The second category, cemented backfill, makes use of a small percentage of binder such as Portland cement or a blend of Portland cement with another pozzolan. A pozzolan is a material capable of reacting with lime in the presence of water at ordinary temperature to produce cementitious compounds. Examples of pozzolanic materials are fly ash, gypsum or blast furnace slag (Sivakugan et al. 2006).

Although uncemented backfilling is less costly, cemented methods are desirable in many cases due to the superior strength of the material. Fly ash, a waste product of coal combustion, is a common additive to backfill material. Ash has a number of advantages for use in coal mining, such as favourable geomechanical properties (including cement-like or pozzolanic characteristics), a capacity for placement in flowable paste or slurry form, and availability in large quantities from power stations near many mine sites. It may also have chemical properties that can be used to ameliorate other mine-related problems, such as the generation and discharge of acid waters from particular mining operations (Ward et al. 2006). Sweigard and Wang (1996) researched the use of flue gas desulfurization (FGD) by-products, including fly ash, in backfill and found that a prehydrated FGD mixture offered the most promise for providing a suitable backfill material. This research also mentioned two additional advantages of FGD by-products, namely their expansive properties, which is considered a liability for most applications but can be an asset in roof support, and low permeability, which is a major factor in the
assessment of their leaching potential and their ability to resist attack from commonly acidic mine water, while maintaining long-term strength.

Donovan and Karfakis (2001) determined that the density of the fill, regardless of material type, should be greater than 1.3 ton/m³, as fills below this density do not meet economic or structural requirements. Their analysis is based on the fact that cost of fill per ton decreases with increasing fill density. Moreover, the water content of the fill mixture should be kept at the minimum required for proper mixing of the cementing agents.

4.3.3.2.1 Dry backfill materials
Although hydraulic backfill methods generally provide the highest strength, the use of wet backfill materials may not be possible due to factors such as local geology, equipment, and method of placement. This may be the case if the coal seam dips towards the drive entry, which makes it difficult to fill the drive without flooding the front of the drive, or in cases where it is necessary to fill on retreat, using a method such as slinger discharge. In these cases, it makes more sense to make use of a dry backfill material, such as crushed aggregate. Alternatively, it would be possible to dewater wet backfill material to such an extent that it may be suitable for use as a dry backfill material. In other cases, cemented wet backfill material may simply not be an economically feasible option.

Donovan and Karfakis (2001) found that, for thin-seam coal mining, a mixture of coal waste and crushed aggregate was the most feasible material option. It is not possible to use coal waste on its own, as its density is too low. Hume and Searle (1998) tested the suitability of various backfill materials for application in highwall mining, and found that crushed aggregate and coarse coal reject both have the potential to be used as backfill material and highwall mining when mixed with coal tailings, provided there is sufficient water to offset the cohesive nature of the tailing. Such a mixture would provide some support to pillars and to mechanised equipment; however, as the fill materials settle away from the roof, the support provided to the roof and overlying equipment would be limited.

4.3.3.2.2 The use of fly ash composite materials in backfill
Fly ash is a by-product of coal combustion that consists of very small particles that rise up with the flue gases and are captured at the top of the flue. Ash particles that do not rise up are referred to as bottom ash. Depending on the composition of the source coal material, the make-up of fly ash can vary widely, although typical elements present in bituminous coal include silica, alumina, iron oxide, and calcium, with varying amounts of carbon, as measured by the loss on ignition (LOI) (Ahmaruzaman 2010). Fly ash is a pozzolan, which makes it a desirable binding ingredient in cementitious materials. Fly ash has also been shown to have thixotropic properties, which prevents the slurry from settling down too fast as well as helping reduce friction on the pipeline wall (Masniyom 2009).

Coal is a major source of electricity in India: 54% of the total installed electricity generation capacity was coal based in 2013 (Chakrabarti et al. 2013). Indian coal also typically has a very high ash content (25-45%). These factors result in a large amount of fly ash being produced every year. Disposal of this fly ash is becoming increasingly problematic, both due to cost and various environmental concerns. Currently, most fly ash is disposed of in landfills, which takes up a lot of valuable agricultural land as well as being very expensive. To minimize the propagation of fine dust particles, fly ash is often stored wet in ponds. However, this brings with it an additional risk of leaching toxic elements into both ground and surface water. Until recently, it was commonly maintained that fly ash is a non-toxic material. However, this is now being disputed by various studies (Gottlieb et al. 2010; Chang et al. 2013). Fly ash can contain trace elements of toxic elements and heavy metals, which are
inherent in some coal-bearing strata and which become enriched in the fly ash during the combustion process. Ground water and surface water are both susceptible to contamination if proper precautions are not taken to ensure containment of the material. Failure to contain the disposed fly ash can have disastrous consequences, as evidenced by the 2008 Kingston Fossil Plant coal fly ash slurry spill in Roane County, Tennessee. That spill, caused by a failure in the earthen walls containing the slurry, released 4.1 million cubic meters of fly ash, damaging property and endangering local wildlife (Dewan 2008). Figure 25 shows an aerial view of this spill and its impact on the local environment.

![Figure 25: Aerial view of the Kingston Fossil Plant fly ash spill (Tennessee Valley Authority).](image)

Because of the problems associated with fly ash disposal, much research has been done in recent years into possible applications for fly ash. Bhattacharjee and Kandpal (2002) identified the major areas of fly ash utilisation as follows:

1. The making of bricks/blocks, cellular concrete products and lightweight aggregates
2. Manufacture of cement and asbestos
3. Road construction
4. Embankment, backfill, land development, etc.

The use of fly ash for such applications is not straightforward. The main problems arise from the variability in the composition of the fly ash, which is dependent on the equally variable composition of the coal received by the power stations. In some cases fly ash may not be locally available and transportation costs may inhibit its use in industrial applications. In India, many power plants use wet fly ash collection systems, which diminishes the pozzolanic properties of the material and thus makes it less suitable for cementitious applications (Bhattacharjee and Kandpal 2002).

Finding applications for fly ash is of particular importance in India, due to regulations issued by the Ministry of Environment and Forests, which stipulate that 100% of the fly ash produced by new coal thermal plants must be used within the first nine years of operation (Subrahmanyam 2009).
4.4 Parameters in mine design
Considering the various requirements for creating a design for a highwall mine as well as designing a backfill system, it is necessary to combine the two to determine the main relevant factors in designing a highwall mine which will be backfilled. Various geometrical and sequential factors can affect the efficiency of the mine design. This section will discuss these factors.

4.4.1 Pillar width
The long, slender pillars used in highwall mining behave in much the same way with regards to their stability as do such pillars in underground mines, that is to say: their stability increases along with the width to height ratio of the pillars. Figures 26 a-b illustrate this concept.

\[
\sigma_p = \gamma z \left( 1 + \frac{W_b}{W_p} \right)
\]

\(W_p\) = Pillar Width
\(H_p\) = Pillar Height
\(H_{app}\) = Apparent Pillar Height For Half Filled CHIM Drive
\(H_{app}\) = Apparent Pillar Height For Filled CHIM Drive

**Figure 26:** Effect of slenderness on the stability of the pillar (Clark and Boyd 1998).

Figure 26(a) shows the stabilizing effect of increasing the width of the pillar, while Figure 26(b) shows the stabilizing effect of increasing the height of the backfill.

Donovan and Karfakis (2004) examined the influence of pillar width on recovery in underground thin-seam coal mining using a Mathcad 2000 program to determine the critical pillar width (one that satisfies a safety factor of 1.5) based on input values of the cut length, cut width, seam height, and a range of overburden depths. They concluded that the rate of recovery from the thin-seam mine will be very low if pillars alone are the only source of ground support. Since 70% to 80% of the coal left within each panel will be in the form of web pillars (rather than barrier pillars), they suggest that backfilling each individual cut may provide the kind of decrease in pillar width that is needed for improved recovery (Donovan and Karfakis 2004).

How much the addition of backfill truly contributes to the stability of the web pillars is one of the main areas of interest in this thesis. The relation shown in Figure 26 works well in an abstract sense, but does not take into account any differences in geomechanical properties between the pillars and the backfill material, nor any alterations to the in situ stress state between excavation and backfill. For this reason, various pillar widths will be examined using the numerical model to determine:

1. How much support is offered by the backfill material
2. How the in situ stress state is influenced by the addition of backfill vs. the non-backfilled scenario.
Caterpillar uses a guideline which is commonly applied in the mining industry that stipulates that the pillar width-to-height ratio should be no less than 1. Reduction of this ratio should be done only after experiences have shown that a lower ratio would be acceptably stable.

4.4.2 Sequencing of mining and backfill and partial fill

The sequence of the mine-to-backfill cycle can affect the stress state of the rock mass in different ways. Clark and Boyd (1998) suggested that, during sequential mining and concurrent backfilling, the last formed pillar is effectively within a stress arch, as shown in Figure 27(a). This can have an advantageous effect on the pillar in the center of this abutment zone, and will give the freshly placed backfill a chance to set-up and stiffen before loading from the overburden is redistributed throughout the pillar (Donovan and Karfakis 2001). Another way to make use of the abutment effect would be to mine every second hole which is then backfilled before a second pass recovers the coal in the remaining drives, as shown in Figure 27(b).

![Figure 27: Loads due to backfilling and mining sequence (Clark and Boyd 1998).](image)

Hume and Searle (1998) also made an economical comparison of various extraction strategies and mine-to-backfill sequences to determine the best scenario. The three variables compared were 1) single-pass mining (backfilling occurring directly after mining) and two-pass mining (mining as a first stage, 2) filling as a separate stage well back from mining and not in exact synchronisation with it); and 3) complete fill vs partial fill. They also considered a scenario in which all of the coal was removed, leaving just the backfill to support the entire overburden. Results clearly showed significantly lower costs for partial filling vs complete filling, although there was nothing conclusive to be said about single-pass mining vs two-pass mining. Hume and Searle concluded that filling strategies to improve coal recovery should not necessarily be based on complete recovery and complete filling. Similar conclusions were drawn by Clark and Boyd (1998). Figure 28 a - c illustrates the various scenarios that Hume and Searle considered.
In determining the mine-to-backfill sequence, logistics will play an important part: certain mining sequences may not be desirable or possible in conjunction with the available mining equipment or specific layout of the mine. In order to determine the magnitude of the effect of abutment loading, various sequences will be simulated with the FDM model.

4.4.3 Multi-lift mining

In some cases, the coal seam may be thicker than the 3 m range of the highwall miner. In such cases, it may be desirable to excavate the seam in multiple lifts, so as to maximize recovery. In order to utilize backfill to facilitate such a scenario, mining would have to occur in an upwards direction. The backfill material would have to have a high load-bearing capacity, as the mining equipment would operate on top of the backfill once it has settled.

Figures 29(a) and 29(b) show two schematic overviews of a possible scenario where a 6 m-thick seam is mined in three lifts.
In the first scenario, the lifts are stacked directly on top of one another, and as a result the bearing capacity is dependent on the stiffness of the backfill. Such a situation can be avoided by leaving horizontal pillars in between the lifts, leaving bearing capacity and stiffness independent of one another. In both cases, it is imperative that no yield develop in the slender pillars at any level during the extraction.

Alternatively, if the mining situation allows for a downward mining sequence, it could be preferable to mine the drive in various lifts in a downward direction, followed by backfilling the entire drive. This would prevent the machine cutting backfill on the bottom of the drive, as that could lead to damage on the cutter head.
4.5 Cost analysis

It is imperative that both the backfill method and the backfill material be of relatively low cost. Typical costs of backfill range from $2 to $20 per cubic metre (Grice 1998), depending on its requirements. These costs can be a significant contribution to the operating costs of the mine. Where cemented backfills are used, these costs tend to be between 10 and 20% of the total operating cost of the mine, with cement representing up to 75% of that cost (Grice 1998).

The most important factor is the cost of the fill required to produce an additional ton of coal. The cost of fill required to produce an additional ton of coal is equal to the cost per ton of fill multiplied by the ratio of the recovery due to backfilling to the additional recovery. This results in the following equation (Donovan and Karfakis 2001):

\[ \text{Cost of filling} = \text{cost of fill per ton} \times \frac{\text{recovery with fill}}{\text{recovery with fill - recovery without fill}} \]  

Figure 30 shows the progression of these costs as a function of mining depth.

![Figure 30: Cost of backfilling per ton of extra coal recovered (Donovan and Karfakis 2001).](image)

The cost of fill decreases with increasing depth and pillar strength, because filling has a greater effect on recovery under these circumstances. With the cost of fill per ton of extra coal recovered displayed over a range of backfilled pillar strengths, a cut-off point for the cost can then be determined.

Factors that must be taken into consideration when determining the economical advantages of a given method include backfill preparation, maintenance costs, and durability of the backfill system. Slurry fill has high cleanup costs, but generally has lower capital costs than paste backfilling. Rock fill may be less costly than either method because of the reduced need for binding agents and elimination of water-based transportation systems.
Bulkheads are another important factor in backfilling, as shown in Figure 31. Bulkheads are structures used to seal off the drive entry after backfilling has occurred. Various types of bulkheads exist. Wooden bulkheads are the least expensive and easy to install. However, they are vulnerable to deterioration and offer relatively low resistance to the pressures they are subjected to - they can withstand pressures of up to 207 kPa, in contrast to concrete bulkheads which can resist pressures of up to 2.8 MPa (Hassani and Archibald 1998). Concrete bulkheads, however, are much more expensive than wooden bulkheads and require a curing period which makes them a less practical choice.

Bulkhead costs vary greatly, depending on the type and amount of bulkheads used. In a cut and fill operation, bulkheads can account for half the operating costs due to the large amount of bulkheads required. However, for methods such as long hole mining, where bulkheads are not used as frequently, their contribution to the total operating costs will be much lower (2% - 7%) (Hassani and Archibald 1998).

Figure 31: Wooden bulkhead (left) and concrete bulkhead (right). (Google Images, Photo © Tim Durham).
5. Determination of rock properties

A model is only as reliable as its input data. Therefore, the rock properties that are entered into the numerical model are of paramount importance. The Mohr-Coulomb model in FLAC requires five rock properties as input parameters: compressive strength, Young's modulus, Poisson's ratio, cohesion, and friction angle. This chapter explains how those rock properties were determined. In section 5.1, some theoretical background about the various relevant laboratory strength tests is discussed. Section 5.2 concerns the most commonly used rock mass classification systems. Section 5.3 discusses the procedure and results of the geomechanical experiments, and section 5.4 elaborates on how the data derived from these experiments was converted into parameters to be entered into the FLAC model.

5.1 Laboratory strength tests

Various laboratory tests are available to test the strength of a rock material. These tests commonly apply pressure to a test core at various orientations to determine the maximum stress the material can bear before failing. However accurate these tests may be for rock cores, which are commonly around 5cm in diameter, they do not generally give a good indication for the strength of an entire rock mass, which is subjected to an infinite amount of factors that do not apply to a rock core drilled through a small portion of the mass. However, they are still very useful as a general indication to the rock strength. An overview of various laboratory rock strength tests can be found in Figure 32.

Figure 32: Common laboratory tests for characterizing rock strength criteria. (a) Unconfined compression. (b) Triaxial compression. (c) Splitting tension (Brazilian). (d) Four-point flexure. (e) Ring shear (Goodman 1989).

5.1.1 UCS test

The most commonly used test is the UCS test, variably described as uniaxial compressive test or unconfined compressive test. In this test, a core with a minimum height to diameter ratio of 2 is placed on a bench and subjected to a vertical compression by means of a load that moves downward onto the core at either a constant
strain rate or a constant velocity. Axial strain is measured by means of two axial deformation measurement devices, while transverse strain can be measured using a circumferential chain around the core. The load is measured by a load transducer and is applied until the core fails (peak load), and slightly beyond. There is a large variation in load capacity between different types of UCS machines, and machines with a low load capacity may not be able to provide a load required to generate failure in stronger samples. As a general rule, however, it is preferable to use a machine with as low a range as possible, as this improves accuracy. The properties that can be obtained with a UCS test are uniaxial compressive strength (UCS), Young’s modulus and Poisson ratio. UCS strength is defined as the failure load divided by the initial cross sectional area of the core:

$$\sigma_t = \frac{F_{\text{max}}}{A_0}.$$  

(5.1)

Young’s modulus and Poisson ratio are both calculated using stress and strain rates at 50% of uniaxial compressive strength.

After the peak strength has been surpassed and the material has failed, the rock may still have some load-carrying capacity, which is referred to as the residual strength. UCS can be a useful indicator for the general strength of a rock material.

### 5.1.2 Triaxial test

Triaxial tests are similar in principle to UCS tests, but differ in that they incorporate axisymmetric confining pressure in addition to compression. Thus, there are two separate (principal) stresses contributing to the overall stress state of the core: the compressive stress $\sigma_1 (= F/A)$ and the confining stress $\sigma_3 (= p)$. As with the UCS tests, cores must have a height to diameter ratio of 2 or higher. The core is placed in a triaxial cell, inside an impervious jacket, and surrounded by the confining fluid, typically hydraulic oil. An initial confining stress is applied on all sides of the core using a hydraulic pump. Then, the compressive load is applied at a constant strain rate or speed while the confining stress remains constant, until failure occurs. Most rocks experience a significant strengthening as a result of confinement. Highly fissured rocks in particular can experience increased strength when confined, as movement along the fissures is inhibited by the confinement.

Triaxial tests yield triaxial compressive strength ($\sigma_1$), Young’s modulus and Poisson ratio. In addition, it is possible to plot Mohr’s stress circles using axial stress $\sigma_1$ and confining stress as $\sigma_3$ and determine failure envelopes and parameters of specified failure criterion. Triaxial tests often offer a more accurate indication of rock strength as in situ rock masses also experience confinement. However, they are also more time-consuming and costly than UCS tests, which is the main reason that UCS is still the more commonly used method.

### 5.1.3 Other tests

In addition to compression, there are also tests that measure tensile and shear strength. Brazilian tests require a cylindrical sample about 50mm in diameter with a thickness approximately equal to the radius. The sample is placed on its side into the machine and compressive loading is applied continuously at a constant rate. This yields a tensile strength, $\sigma_s$ which is determined by the following equation:

$$\sigma_s = \frac{2F_{\text{max}}}{\pi dt}.$$  

(5.2)

where $F_{\text{max}}$ is the failure load, $d$ is the diameter of the sample and $t$ is the thickness of the disk.
Flexural tests, like Brazilian tests, measure tensile strength, but do so by bending a rock beam. The resulting flexural strength is the maximum tensile stress on the bottom of the rock corresponding to the peak load. Ring shear tests, on the other hand, give an indication of shear strength by applying a compressive load as well as a confining pressure to a short core inserted into the machine.
5.2 Rock mass classification

Strength measurements obtained from laboratory strength tests can be useful as an indication of the overall characteristics of a site, but hardly ever represent the actual in situ situation. This is because cores generally represent only a very small fraction of the overall rock mass. Moreover, fractured cores are not suitable for testing and are therefore disregarded in the overall results. Strength measurements obtained from laboratory testing, therefore, can be biased and unreliable.

In order to compensate for such inaccuracies, several rock mass classification systems have been developed which attempt to characterize the rock based on the entire rock mass, rather than just specimens that are tested in a lab. The four most commonly used systems are described below.

5.2.1 Rock Quality Designation

The Rock Quality Designation (RQD) is a commonly used index for the description of rock mass in a fractured state, and is based on locally obtained cores rather than the entire rock mass. It was initially introduced for civil engineering applications, and has been adopted in mining, engineering geology and geotechnical engineering. It was initially proposed by Deere (1963) and is defined as the percentage of intact core pieces longer than 100 mm in the total length of the core. Figure 33 summarizes the use of the RQD.

\[ Q = \frac{RQD}{f_n} \times \frac{J_a}{J_n} \times \frac{J_w}{SRF} \]  

where \( RQD \) = the Rock Quality Designation

\( f_n \) = the joint set number

5.2.2 Rock Tunneling Quality Index

The Rock Tunneling Quality Index (Q) was proposed by Barton et al. (1974) for the determination of rock mass characteristics and tunnel support requirements. The RQD value itself is dependent on six separate values that can be determined using the tables listed in Appendix E. The Q value is determined by the following equation:

\[ Q = RQD \times \frac{J_a}{J_n} \times \frac{J_w}{SRF} \]  

Figure 33: Procedure for measurement and calculation of RQD (Hoek 2007).
\[ J_r = \text{the joint roughness number} \]
\[ J_a = \text{the joint alteration number} \]
\[ J_w = \text{the joint water reduction factor} \]
\[ SRF = \text{the stress reduction factor} \]

5.2.3 Rock Mass Rating

The Rock Mass Rating (RMR) was developed by Bieniawski (1976). It combines a large number of geological features to give an indication of the quality of the rock mass and a general idea of its suitability for excavation. The Rock Mass rating can be determined using Table 9 in Appendix F, which incorporates the following parameters to determine the RMR value:

1. Strength of intact rock material
2. Drill core quality (RQD)
3. Spacing of discontinuities
4. Condition of discontinuities
5. Groundwater conditions
6. Strike and dip orientations

From these values, an overall RMR rating can be calculated, culminating in the assignment of a rock class ranging from I (very good) to V (very poor). These rock classes give an indication for average stand-up time, cohesion of the rock mass, and friction angle of the rock mass.

5.2.4 Geological Strength Index

The Geological Strength Index (GSI) was introduced by Hoek (1994) in an attempt to meet the need for reliable input data, particularly those related to rock-mass properties required as inputs into numerical analysis or closed form solutions for designing tunnels, slopes or foundations in rocks (Marinos et al. 2005). To this end, two tables exist that can be used to classify the strength of a rock mass based on general outward characteristics that can be studied on-site. Both tables, one for jointed rocks, and one for heterogeneous rock masses such as flysch, can be found in Appendix G.
5.3 Experimental work
During a site visit in July 2012, several rock samples were obtained from the West Bokaro mine site for further testing in Delft. Unfortunately, due to safety hazards, it was not possible to obtain samples directly from the highwall face. Instead, fragmented pieces of the coal seam and the roof rock were collected on site. The samples consisted of two pieces of roof rock and one piece of coal, obtained from the bottom of seam 5 in the SEB quarry. Various types of backfill material were also tested, including a sample of the backfill material that is intended for use at the West Bokaro mine site. This material has been created specifically for this purpose. It is a fly ash composite material which will be pumped into the drive in slurry form. It then requires a drying time of approximately 28 days before reaching a strength of 10 MPa. These samples were each subjected to a UCS test in the geomechanical laboratory in Delft.

5.3.1 Experimental setup
The principles of the UCS tests performed in Delft are congruent with the description given in section 5.1.1. The geomechanical lab in Delft has two UCS testing machines available: one with a maximum capacity of 50 kN, and one with a maximum capacity of 500 kN. Whenever possible, it is preferable to use the 50 kN machine, because this machine gives more accurate results for the lower stress range. However, materials that did not or were not expected to fail under a load of approximately 25 kN were tested on the 500 kN machine. The cutoff point was chosen well below 50 kN so as to avoid the risk of the testing process being suddenly cut short. Figure 34 shows the 50 kN testing machine measuring one of the coal samples.

Figure 34: UCS machine (50 kN) measuring one of the coal samples.
As the UCS machine applies a measured amount of force on the sample, the pressure can be calculated by dividing the force over the total area of the sample. The machine also keeps track of the vertical displacement, and thus, the axial strain of the sample. In the case of a smooth, intact core, it is also possible to measure the radial strain using a chain which is placed around the circumference of the core. This information can then be plotted in a stress-strain curve which shows the development of both stress and strain over the duration of the test. From these stress-strain curves, the following three parameters can be derived:

- **UCS** - peak strength of the material
- **Poisson ratio** - indication to the deformability of the material
- **Young’s modulus** - indication to the stiffness of the material

When multiple tests can be carried out for a sample, a failure envelope can then be derived which yields cohesion and friction angle. However, in this case there were unfortunately not enough samples available to do so.

A total of fourteen samples were tested: eight rock samples and six backfill samples. The rock samples were all collected on-site at West Bokaro and consisted of slabs of rock not exceeding approximately 50cm in each direction. Additionally, three different types of backfill were tested: a grouting material used to backfill old auger holes at Hill Creek Mine, Virginia; a foam-type backfill material from Australia supplied by Caterpillar; and a sample of the fly ash composite backfill material, supplied by Cuprum Bagrodia Ltd., that is to be used at West Bokaro Mine. Where possible, cores were drilled from each of these samples to a diameter of approximately 50mm, with a length twice the diameter, 100mm. All cores that were tested dry were placed in an oven at 60°C for two days before testing in order to remove moisture from the rock. Wet cores were submerged in water for a minimum of 2 days each in order to saturate them before measurements. An complete overview of the stress-state plots and transverse-axial strain plots is listed in Appendix H.

### 5.3.2 Samples
The initial measurements of each of the samples can be found in Table 3, as well as the orientation of the drill direction in relation to the stratification, and whether the cores were measured wet or dry.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Orientation to stratification</th>
<th>Wet/Dry</th>
<th>m (g)</th>
<th>h (mm)</th>
<th>D (mm)</th>
<th>V (m³)</th>
<th>ρ (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coal 1</td>
<td>perpendicular</td>
<td>Dry</td>
<td>318.72</td>
<td>103.32</td>
<td>50.06</td>
<td>2.03·10⁻⁴</td>
<td>1567.30</td>
</tr>
<tr>
<td>Coal 2</td>
<td>perpendicular</td>
<td>Dry</td>
<td>311.73</td>
<td>101.53</td>
<td>50.13</td>
<td>2.00·10⁻⁴</td>
<td>1555.60</td>
</tr>
<tr>
<td>Coal 2 #2</td>
<td>perpendicular</td>
<td>Dry</td>
<td>311.73</td>
<td>101.43</td>
<td>50.13</td>
<td>2.00·10⁻⁴</td>
<td>1557.14</td>
</tr>
<tr>
<td>Coal 3</td>
<td>perpendicular</td>
<td>Dry</td>
<td>298.76</td>
<td>98.94</td>
<td>50.12</td>
<td>1.95·10⁻⁴</td>
<td>1530.52</td>
</tr>
<tr>
<td>Coal 4</td>
<td>parallel</td>
<td>Dry</td>
<td>196.38</td>
<td>68.73</td>
<td>50.05</td>
<td>1.35·10⁻⁴</td>
<td>1452.29</td>
</tr>
<tr>
<td>2W - Roof rock 2 wet</td>
<td>parallel</td>
<td>Wet</td>
<td>239.37</td>
<td>47.85</td>
<td>50.17</td>
<td>9.46·10⁻⁵</td>
<td>2530.52</td>
</tr>
<tr>
<td>2D - Roof rock 2 dry</td>
<td>parallel</td>
<td>Dry</td>
<td>269.41</td>
<td>54.13</td>
<td>50.12</td>
<td>1.07·10⁻⁴</td>
<td>2522.69</td>
</tr>
</tbody>
</table>
Possibly as a result of weathering, both the coal samples and the roof rock samples had a tendency to fracture upon coring and it was therefore rather difficult to obtain cores suitable for testing. Four cores were drilled out of the coal, three of which were drilled perpendicular to the stratification and one parallel to it. Two cores each were drilled from the two roof rock samples. Unfortunately, the shape of the two roof rock slabs made it impossible to drill cores perpendicular to the stratification. Therefore, all four of the roof rock samples have been drilled in the direction parallel to the stratification. Moreover, the shape of the rock slabs was a limiting factor in the dimensions of the cores, and the cores for roof rock sample #2 were necessarily shorter than ideal. The roof rock samples were tested in both dry and wet states, after being saturated with water over a period of two days. The cores were subjected to a uniaxial compressive strength test with a chain around the core to measure the radial (transverse) strain. Results were obtained for maximum compressive strength, and the Poisson ratio and Young’s modulus were derived from the obtained strain and stress rates. The coal samples were initially all measured on the 50 kN bench, and the roof rock samples were measured on the 500 kN bench. Because coal sample #2 had not yet failed at 25 kN on the 50 kN bench, the initial test was halted and continued on the 500 kN bench.

Most of the cores were relatively intact before measurement. However, coal sample #3 had a small chunk missing from the top, and coal sample #4 contained some obvious joints. The roof rock cores, in particular core 3, were fairly weathered.
During the testing, when possible, the cores were briefly unloaded and then re-loaded several times. This results in an obvious "loop" in the stress-strain curve. This line is a more accurate representation of the actual elastic behavior of the material, and is therefore used to determine the Young's modulus.

Both of the roof rock samples #2 were too short to fit into the UCS machine. They were placed on a round base of hardened steel with a diameter of 49.85 mm and a height of 24.07 mm. The strain caused by the load on the steel base thus had to be subtracted from the total strain to obtain the strain in the rock sample. Assuming a Young's modulus for steel of 210 GPa, the total displacement attributed to the steel under a maximum load of 74,75 kN (wet core) and a maximum load of 164,135 kN (dry core) was calculated to be 0.0044 mm and 0.0096 mm respectively. This accounts for 2% of the total displacement (0.21518 mm) for the wet core, and 4% of the total displacement (0.2362 mm) for the dry core. Therefore, the total strain on the core must be reduced by those amounts and the Young's modulus was adjusted accordingly.

The UCS machine measured the stress imposed upon them as well as the axial strain, which was measured through two displacement sensors whose values are averaged. The machine is also capable of measuring the transverse (radial) strain through use of a chain around the circumference of the core. This data was then fed to a computer and saved as a comma-separated values file which were viewed in Excel. The maximum compressive strength, the Young's modulus and, if the transverse strain was measured, the Poisson ratio, can then be determined. The machine can operate using either a constant displacement rate or a constant load rate. In these tests, a constant displacement rate was used. Before testing, all samples were measured and weighed. Measurements in each direction were done at least three times and then averaged.

A similar testing procedure was followed for the three types of backfill material. However, these samples proved very difficult to core. The Indian sample in particular was problematic, because the material was so loose that it practically fell apart before coring was even attempted. For this reason, it was decided not to core the Indian backfill material, but to test the full cube of backfill material as-is, even though this meant the results would be limited in accuracy as well as variability, since it was impossible to use a chain to measure radial strain for a cube shape. Moreover, the cubic shape meant that the regular displacement sensors could not be used, and axial strain could not be measured. Instead, only the downward penetration in mm was measured. Approximate strain values were calculated manually afterwards.

The material provided for the Australian backfill was, again, quite limited. The first sample yielded two cores, while the second sample yielded only one. Both of these samples had clearly hardened after using a foaming agent and were very light and porous. Because of this, sample #5 in particular proved difficult to core. The resulting core was somewhat irregular in shape, as well as very rough and porous, and it was therefore impossible to measure the transverse strain using a chain. The first part of the stress-strain curve for this sample looks somewhat unusual because, due to its low strength, the core was already being loaded when the machine was closed, before any additional force was applied.

The Indian backfill material was packaged in a wooden container consisting of five planks nailed together. In order to remove the cube of backfill material from this container, the four planks on the side had to be manually removed. However, because the material was so soft and loose, removing the plank on the bottom would likely have resulted in the cube falling apart altogether and would certainly have made it impossible to measure it on a UCS machine. Therefore it was decided to leave the plank. After the testing was finished, the plank was removed and its weight subtracted from the total weight of the cube and the plank.
In order to test the cube, a steel plate weighing 12.5 kg was placed on top of it. Thus, in determining the pressure on the cube, the additional pressure caused by the weight of the plate had to be added to the total amount. Figure 35 shows the Indian backfill sample and Australian backfill sample #5 on the UCS machine.

![Figure 35: Indian backfill material (left) and Australian backfill sample #5 (right) during testing.](image)

5.3.3 Results
The stress-strain curves for each of the samples are shown in Figures 36 - 40. As mentioned previously, the stress-strain curve for the Indian backfill material was created manually after dividing the height of the sample by the total displacement, as the regular strain sensors could not be used. The stress-strain curve for this material is plotted in Figure 40.

![Figure 36: Stress-strain curves for coal samples](image)
Coal samples 1 - 3 have a peak strength in the range from 18 - 30 MPa. Sample #4 has a significantly lower peak strength, at approximately 8 MPa. This is likely a result of the fact that this core was drilled parallel to the stratification, while the other samples were drilled perpendicular to it. While the rock strength among the samples varies, it consistently exceeds the strengths measured in Agapito Associates (2005), as listed in Table 2 in Section 2.1.2, which vary between 10 and 20 MPa for the coal seams. The strength listed there for seam 5, from which these samples were taken, is 11 MPa. Due to the small number of samples, it is difficult to say what could be the exact cause of this. It is possible that the difference in equipment is responsible for some of the variation. However, it is also possible that the cores measured by Agapito Associates were not representative of the rock mass as a whole. The Young's Modulus and Poisson ratio for samples #1 - #3 are fairly consistent across all three samples at approximately 5.1 and 0.2, respectively. While the Poisson ratios are comparable to those measured by Agapito Associates, the Young's Moduli are significantly higher. This suggests that the stiffness of the coal could also be greater than previously supposed.

As expected, the wet cores show a significant decrease in strength. Sample 2 has a relative decrease in strength of 55% while sample 3 has a decrease of 70%. This decrease in strength should be taken into account when considering the possibility of water entering the drives.

Figure 37: Stress-strain curves for roof rock samples

Figure 37 shows the UCS results for the roof rock samples. As described in the previous section, the samples were briefly unloaded and re-loaded to obtain a more accurate slope to calculate the modulus of elasticity. These ‘breaks’ are represented by the short deviations from the main stress lines.

As with the coal, these samples also show a significant increase in strength compared to the Agapito results. Table 2 lists the roof rock strength for seam 5 at approximately 30 MPa, while both of the dry samples tested for this thesis obtained a peak strength exceeding 60 MPa, with Sample 2 even exceeding 80 MPa. This, again, suggests that the estimates made by Agapito Associates may be overly conservative.
Figure 38: Stress-strain curves for American backfill samples

Figure 38 shows the UCS results for the American backfill material. The peak strength for this material is 13.4 MPa, with the wet strength being 28% lower at 9.7 MPa. This is slightly below the typical compressive strength range for normal concrete at 20 - 50 MPa (Monteiro 2002). However, in a backfill capacity where its main function is confinement rather than support, this type of material could be adequate.

Figure 39: Stress-strain curves for Australian backfill samples

Figure 39 shows the stress-strain curves for the Australian backfill samples. These samples, which were foam-based and very porous, clearly have much less strength than the American samples, at 1.2 MPa dry and 0.7 MPa wet for Sample 4. Moreover, both samples have a very low Young's Modulus, indicating very high elasticity. This would likely reduce their value as a confining material.
Figure 39 also shows that Sample 5 did not actually fail. However, even under a low load, there is a very significant amount of strain. With such degrees of deformation, it is unlikely that this material would be suitable as a backfill material.

Figure 39: Result UCS test - Indian Backfill

![Result UCS test - Indian Backfill](image)

\[ y = 0.0147x - 0.0232 \]

**Figure 40**: Stress-strain curve for Indian backfill sample.

Figure 40 shows the results for the Indian backfill sample. The peak strength measured here, at approximately 0.13 MPa, is vastly inferior to its purported strength of 10 MPa. It is not clear what caused this difference. It is possible that both samples, which were similar in texture and consistency, were faulty. Although the backfill material may offer some value with regards to confinement - although at 0.015 GPa, the low Young's modulus likely detracts from that, as well - it does not seem advisable to use it as a backfill material based on the results obtained here.

The results of the tests are summarized below in Table 4.

<table>
<thead>
<tr>
<th>Sample</th>
<th>UCS (MPa)</th>
<th>Poisson ratio</th>
<th>Young's modulus (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 - Coal</td>
<td>25.35</td>
<td>0.16</td>
<td>5.14</td>
</tr>
<tr>
<td>2 - Coal</td>
<td>29.18</td>
<td>0.18</td>
<td>5.11</td>
</tr>
<tr>
<td>3 - Coal</td>
<td>17.99</td>
<td>0.18</td>
<td>5.10</td>
</tr>
<tr>
<td>4 - Coal</td>
<td>7.87</td>
<td>0.50</td>
<td>2.02</td>
</tr>
<tr>
<td>2W - Roof rock 2 - wet</td>
<td>37.81</td>
<td>0.35</td>
<td>11.83</td>
</tr>
<tr>
<td>2D - Roof rock 2 - dry</td>
<td>83.19</td>
<td>0.11</td>
<td>31.01</td>
</tr>
<tr>
<td>3W - Roof rock 3 - wet</td>
<td>17.76</td>
<td>0.33</td>
<td>4.16</td>
</tr>
</tbody>
</table>
Table 4: Summary of UCS test results.

<table>
<thead>
<tr>
<th>Sample</th>
<th>UCS (MPa)</th>
<th>Poisson ratio</th>
<th>Young’s modulus (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3D - Roof rock 3 - dry</td>
<td>60.53</td>
<td>0.14</td>
<td>24.20</td>
</tr>
<tr>
<td>4W - Australian backfill 1, wet</td>
<td>0.71</td>
<td>0.10</td>
<td>0.86</td>
</tr>
<tr>
<td>4D - Australian backfill 1, dry</td>
<td>1.24</td>
<td>0.13</td>
<td>0.81</td>
</tr>
<tr>
<td>5 - Australian backfill 2, dry</td>
<td>0.48</td>
<td>N/A</td>
<td>0.49</td>
</tr>
<tr>
<td>American backfill, wet</td>
<td>9.7</td>
<td>0.12</td>
<td>7.16</td>
</tr>
<tr>
<td>American backfill, dry</td>
<td>13.4</td>
<td>0.15</td>
<td>6.27</td>
</tr>
<tr>
<td>Indian backfill</td>
<td>13</td>
<td>N/A</td>
<td>0.0147</td>
</tr>
</tbody>
</table>

The strength of the coal as well as that of the sandstone turned out to be much higher than expected. Previous tests found a UCS of 11.1 for the coal and 30.7 for the sandstone (Agapito Associates 2005). However, the test results obtained here suggest a coal strength between 20 and 30 MPa and a roof rock strength of more than 60 MPa. This might indicate that the projections using this previously obtained data could be overly conservative. However, due to the limited amount of material, it is difficult to state anything conclusively about the rock mass, and further testing should be done to ascertain whether these measurements are representative of the rock mass as a whole. It is clear, however, that moisture has a great weakening effect on the roof rock material. Therefore, the presence of water should be closely monitored.

Fracturing in the coal samples seemed to indicate a shear failure mechanism. Although fracturing did not occur entirely along the 60°/30° angles typical of shear failure, the irregularities are likely to be caused by damage to the cores, such as the chunk missing from the top of sample 3, as well as internal jointing. Figure 41 shows coal samples 3 and 4 post-failure.

Figure 41: Coal samples 3 (left) and 4 (right) post-failure.
The roof rock samples #2 fractured along the stratification, which was to be expected as they were drilled parallel to the stratification. The fracture planes for samples #3 were at a slight angle but this is likely due to weathering in the rock. Figure 42 shows the failure planes for the various roof rock cores.

![Figure 42: Left to right: roof rock sample #2 (dry), sample #2 (wet), sample #3 (dry), post-failure.](image)

For the Australian and American backfill samples, there is a clear weakening of the wet sample vs the dry sample. The Australian samples show very low strength but high deformability. It is therefore questionable whether either the Indian or Australian backfill materials would actually be suitable for that purpose. The American backfill material, however, shows both high strength and relatively low deformability, and is more typical of a cement-like material.

Failure in the Indian backfill sample occurred mostly in cracks along the corners of the cube. After testing, the Indian sample fell apart altogether, as shown in Figure 43.

![Figure 43: Indian backfill sample after testing](image)

Australian backfill sample #5 showed a large amount of deformation but no actual failure. Australian backfill samples #4, however, showed near-perfect shear failure, as illustrated in Figure 44.
5.3.4 Discussion
The results obtained from these tests vary widely and their significance is consequently limited. However, they serve well to give a general indication of the material. The results for the in situ coal and roof rock material indicate that their strength may be higher than originally assumed. It would be worthwhile to do additional UCS testing, which is relatively quick and cheap, to determine whether or not this is the case for the rock mass as a whole. The significant decrease in rock strength as a result of water saturation should be taken into account when planning the mine design.

The test results obtained from the various backfill samples give a good indication of the wide variety of possibilities available for backfill. Depending on the situation, each of these materials may have their use. However, it seems that the cost-reducing benefits of the foam material may be offset by sacrifices made in strength and stiffness. The low stress required to generate a very large amount of deformation indicates that this type of material may not be suitable for backfilling in highwall applications. Out of all the tested materials, the American grouting material seems like it would be the most suitable for the purpose of backfilling, due to its relatively high strength and stiffness.

It is not clear what the benefits are of the Indian backfill material samples which were supplied to Caterpillar. The strength measured in the lab is much lower than what it is purported to be. The lack of strength and cohesion indicate that it would be a poor backfill material for load-bearing applications. However, if the only requirement is confinement, then it may be suitable enough, although it doesn’t seem to have any added benefits over loose sand or other pulverized material, as it falls apart quickly and is practically cohesionless.

Based on the results obtained here, the input for the UCS and Young’s modulus for coal in the FLAC model were taken as 25 MPa and 5 GPa, respectively. However, for the roof rock as well as the backfill material the results were too variable. For these groups, data was used from available literature. The selection procedure for this data will be expanded upon in the next section.
5.4 Rock properties for FLAC input
For the rock properties, use was made, where possible, of data obtained from laboratory tests. In some cases this was not possible, and data had to be taken from literature instead. However, all of the data had to be modified to be representative for the rock mass in the FLAC model. The following sections will describe how the data obtained from the experimental work and literature were translated to data that is usable in the FLAC model.

5.4.1 Rock mass properties
In order to obtain data representative for the entire rock mass, rather than just a single core measured in a laboratory, the program RocLab was used to convert the laboratory data into usable rock mass data. RocLab is a software program developed by Evert Hoek for determining rock mass strength parameters, which converts lab-obtained data into reliable data representative for the rock mass, that can be used for applications such as numerical modeling.

RocLab requires the following parameters as input:

- unconfined compressive strength of the intact rock (UCS)
- geological strength index (GSI)
- intact rock parameter \( m_i \)
- disturbance factor \( D \)
- intact Young's modulus \( E_i \)

For tunneling applications, the program also requires the unit weight and tunneling depth.

RocLab then yields rock mass values for cohesion \( c \), friction angle \( \varphi \), and Young's modulus for the rock mass \( E_{rm} \). Poisson's ratio \( (\nu) \) can be determined from the Geological Strength Index through the following empirical relation formulated by Lorig and Pierce (2000):

\[
\nu = 0.32 - 0.0015 \cdot \text{GSI}
\]

For the coal, GSI and the disturbance factor were based on in situ observations at the West Bokaro mine site using visual comparisons between the in-situ rock face and the GSI chart (Appendix G), as well as the comparison values for the disturbance factor listed in RocLab. Figure 45 shows a close-up of the coal strata at West Bokaro, with a moderate amount of discontinuities and a fairly smooth surface. Based on these observations, the GSI was estimated to be in the overlapping region between a blocky structure, which is described as well interlocked undisturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets, and good surface quality, which is described as rough, slightly weathered, iron stained surfaces.
Thus, the initial GSI was set at 70. The disturbance factor was set at 0.5, which indicates modest rock mass damage. The list of values for the intact rock parameters in RocLab does not contain values for coal; for this reason, the range for claystones at $4 \pm 2$ was chosen instead, and the intact rock parameter was set at 5.

For the coal, UCS and Young’s modulus were derived from lab data. Considering the difference between the lab-obtained data and the data listed in Agapito Associates (2005), a conservative value was used for the coal strength at 25 MPa. For the Young’s modulus, the measured value of 5 GPa was used. For the roof rock, use was made of data obtained from Agapito Associates (2005), because the data obtained from the experimental work was too variable to be representative. The UCS was thus set at 32.1 MPa and Young’s Modulus at 4.8 GPa. The intact rock parameter was set at the typical value for sandstone listed in RocLab, at 17. The disturbance factor was estimated at 0, while the GSI was estimated, very roughly, at 80.

The application chosen in RocLab was ‘Tunneling’. Tunneling depth was set at 140 m. Average density of the coal, taken from the lab measurements as recorded in Table 3, was set at 1550 kg/m$^3$, and thus the corresponding unit weight was set at 0.0152 MN/m$^3$.

However, simulations using the initial data yielded by RocLab resulted in failure of the coal strata before excavation of the drives had even commenced. As this result does not reflect the actual situation, it was necessary to slightly alter the input into the RocLab program (specifically the disturbance factor, the intact rock parameter and the GSI) so as to result in a stable initial rock mass. The final input data and the resulting output can be found in Table 5.

**Figure 45:** Detailed view of coal strata at West Bokaro Mine.
<table>
<thead>
<tr>
<th>Input</th>
<th>Coal</th>
<th>Overburden</th>
<th>Output</th>
<th>Coal</th>
<th>Overburden</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCS (MPa)</td>
<td>25</td>
<td>32.1</td>
<td>$c$ (MPa)</td>
<td>1.21</td>
<td>3.097</td>
</tr>
<tr>
<td>GSI</td>
<td>75</td>
<td>80</td>
<td>$\phi$</td>
<td>40.19</td>
<td>43.83</td>
</tr>
<tr>
<td>$m_i$</td>
<td>4</td>
<td>17</td>
<td>$E_m$ (MPa)</td>
<td>3,468.49</td>
<td>4,225.7</td>
</tr>
<tr>
<td>$D$</td>
<td>0.2</td>
<td>0</td>
<td>$\nu$</td>
<td>0.2075</td>
<td>0.2</td>
</tr>
<tr>
<td>$E_i$ (MPa)</td>
<td>5000</td>
<td>4,800</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Table 5: Data input and output from RocLab program.*

### 5.4.2 Backfill properties

Because the structure of the Indian backfill material made it impossible to obtain any data other than a very rough indication of the uniaxial compressive strength, it was not possible to use the properties of this material in the FLAC simulations. Because no other samples of fly ash composite material were available, properties for similar material had to be obtained from literature. The properties used in the simulations were obtained from Mishra and Karanam (2006) for fly ash composite material obtained from Rourkela Steel Plant in India, containing 15% lime and 5% gypsum, after 28 days of curing. This is comparable to the curing time required for the Indian backfill material. The properties as used for the FCM backfill material in the simulations for this thesis were as follows:

- Compressive strength (MPa) 8.08
- Young's modulus (MPa) 316.9
- Poisson's ratio 0.35
- Cohesion (MPa) 1.17
- Friction angle 29.6°

Although these values may not be representative for the situation at West Bokaro, they serve more as a rough indication for backfilling using FCM as a backfill material.

In addition to a very limited UCS test, an XRF test was carried out on the Indian backfill material to gain better insight into its composition. The results of this test can be found in Appendix I.
6. Numerical modeling

The basic theory behind numerical modeling has been briefly discussed in section 3.2. This chapter will discuss more detailed concepts of numerical modeling as they relate to the field of rock mechanics, and in particular how they are applied in FLAC3D. Section 6.1 concerns the concept of elasto-plastic deformation, related concepts, and how they apply to the FLAC3D model. In section 6.2, various theoretical concepts are described that form the basis of the FLAC3D model. Finally, section 6.3 discusses the particular model that was created for this thesis, and explains its geometry and other parameters.

6.1 Elasto-plastic deformation

As stress is applied to a material, it experiences deformation that is either elastic or plastic. As stress increases, most materials demonstrate an elastic-plastic deformation pattern, where a period of elastic (reversible) deformation is followed by plastic (irreversible) deformation. The transition point between elastic and plastic deformation is referred to as the yield point.

Elasto-plastic deformation behavior can be illustrated in a stress-strain curve based on laboratory testing. Figure 46 shows idealized versions of various types of deformation behavior as a load is applied at a fixed strain rate.

In Figure 46(a), deformation is immediately plastic and therefore irreversible. Figures (b), (c) and (d) exhibit an initial elastic phase, followed by a plastic phase that is constant (b), hardening (c) or softening (d). In hardening material, the application of stress results in an increase in strength, while in softening material it has the opposite effect.

![Figure 46: Elasto-plastic deformation behavior: A. Fully plastic, B. elasto-plastic, C. strain hardening, D. strain softening (Karstunen 2012).](image)

6.1.1 Plastic potential theory

The plastic potential theory is used to predict the displacement of a medium at yield, and incorporates, in addition to the previously discussed failure criteria, the concepts of a plastic potential function (\( G_p \)) and a flow rule.

6.1.1.1 Yield surface

To graphically display the deformation state of a material, a yield surface can be defined. The yield surface is a graphical representation in the stress-space of the boundary of elastic deformation in a certain material, given a failure criterion. The yield surface is essentially an extrapolation of the failure envelope of that criterion into the 3D principal stress space. If the yield surface is defined by a function \( F = 0 \), then the conditions of the yield surface are such that for \( F < 0 \) (inside the yield surface), the material behaves elastically. On the yield surface
itself, where \( F = 0 \), plastic behavior occurs. By definition the stress state cannot move outside of the yield surface, but further deformations may alter the shape and size of the yield surface. Each of the situations illustrated in Figure 46 has a corresponding reaction in the yield surface: perfect plasticity results in a stationary yield surface; hardening behavior causes an expanding yield surface, while softening behavior causes a contracting yield surface.

By rewriting the cohesion \( c \) and the friction angle \( \varphi \) in terms of the major and minor principal stresses, the Mohr-Coulomb failure criterion on a 2D stress plane can be formulated as follows (Briaud 2013):

\[
f = \sigma_1 - \sigma_3 - 2c \cos \varphi - (\sigma_1 + \sigma_3) \sin \varphi = 0
\]

In order to plot this on a 3D stress plane, this equation can be broken down into six separate equations that incorporate the intermediate principal stress, \( \sigma_2 \):

\[
\begin{align*}
f_1 &= \sigma_1 - \sigma_3 + (\sigma_1 + \sigma_3) \sin \varphi - 2c \cos \varphi \\
f_2 &= \sigma_2 - \sigma_1 + (\sigma_2 + \sigma_3) \sin \varphi - 2c \cos \varphi \\
f_3 &= \sigma_2 - \sigma_1 + (\sigma_2 + \sigma_1) \sin \varphi - 2c \cos \varphi \\
f_4 &= \sigma_3 - \sigma_1 + (\sigma_3 + \sigma_1) \sin \varphi - 2c \cos \varphi \\
f_5 &= \sigma_3 - \sigma_2 + (\sigma_3 + \sigma_2) \sin \varphi - 2c \cos \varphi \\
f_6 &= \sigma_1 - \sigma_2 + (\sigma_1 + \sigma_2) \sin \varphi - 2c \cos \varphi
\end{align*}
\]

Together, these six equations form the 3D Mohr-Coulomb yield area as illustrated in Figure 47.

*Figure 47: The Mohr-Coulomb yield surface in three-dimensional stress space (Borst 1987).*

6.1.1.2 Plastic flow

As previously stated, although the stress state cannot move beyond the confines of the yield surface, it is possible for the yield surface itself to change in shape and size as a result of plastic strain. Determining the amount of plastic strain that has occurred requires a concept in plasticity known as plastic flow. In order to determine the behavior of this plastic flow, rules must be established. Similar to the failure surface in stress space, a plastic potential function, \( G_p \), is defined in strain space as related to the strain rate \( \varepsilon \) (Dartevelle 2003):

\[
\varepsilon_i = q \frac{\partial G_p}{\partial \sigma_i}
\]
where $\varepsilon_i$ is the strain rate, $q$ is a proportionality coefficient dependent on the flow conditions sometimes named the "plastic multiplier", and $\sigma_i$ is the stress corresponding to the strain rate.

Various approaches exist to describe the plastic potential function, $G_p$. Some of these approaches define the plastic potential function as being equal to the yield surface of a particular failure criterion. Such equations are known as associated flow rules, because they are associated with a particular failure criterion. For associated flow rules, equation 6.3 thus becomes:

$$\varepsilon_i = q \frac{\partial Y}{\partial \sigma_i}$$  \hspace{1cm} 6.4

where $Y$ is the yield function defined by the failure criterion. For the Mohr-Coulomb associated flow rule, $Y$ thus becomes:

$$Y = \tau_p - c - \sigma \tan \phi$$  \hspace{1cm} 6.5

6.1.1.3 Dilatancy

The Mohr-Coulomb associated flow rule results in a positive volumetric strain rate. This volumetric increase as a result of shear stress is a phenomenon that has been observed extensively in practice, and is known as dilatancy. Shear stress can cause a realignment of grains in a matrix, which increases the overall volume of the material. Dilatancy is defined using the dilatancy angle ($\psi$): the change in the orientation of grains on a shearing surface. By definition the dilatancy angle is always smaller than the friction angle. The associated Mohr-Coulomb flow rule tends to overestimate the degree of dilatancy. More commonly used than the associated flow-rule, therefore, is a non-associated Mohr-Coulomb flow rule which exchanges the friction angle in the yield equation for the dilation angle, to enable a more accurate representation of the actual dilation. This non-associated flow rule defines the plastic potential function as follows:

$$G_p = \tau_p - c - \sigma \tan \psi$$  \hspace{1cm} 6.6

6.2 FLAC3D concepts

The following sections discusses some basic theoretical concepts underlying the FLAC3D model.

6.2.1 Definition of variables

The three main properties of interest in the FLAC model are stress, strain and velocity. The state of stress is defined by a traction vector given by:

$$t_i = \sigma_{ij}n_j$$  \hspace{1cm} 6.7

where $\sigma_{ij}$ is the symmetric stress tensor and $n_j$ is the unit normal vector.

The next state to be examined is the spatial change of volume elements within the model. This deformation is dependent on three deformation mechanisms: strain, translation and rotation.

The rate of strain is described by the strain-rate tensor:
\[ \xi_{ij} = \frac{1}{2} (v_{i,j} + v_{j,i}) \]  

6.8

where \([v]\) is the velocity vector at which the particles in the medium move. This strain-rate tensor characterizes the deformation of an element, but in addition to this deformation, the volume element itself also experiences a change in position by mechanisms of translation and rotation. This displacement is dependent on the angular velocity \(\Omega\):

\[ \Omega_i = -\frac{1}{2} e_{ijk} \omega_{jk} \]  

6.9

where \(e_{ijk}\) is the permutation symbol, and \(\omega\) is the rate-of-rotation tensor which is described as follows:

\[ \omega_{jk} = \frac{1}{2} (v_{j,k} - v_{k,j}) \]  

6.10

The final property of interest is the velocity of the movement described above. This velocity is described using Cauchy’s equations of motion:

\[ \sigma_{ij,j} + \rho b_i = \rho \frac{dv_i}{dt} \]  

6.11

where \(\rho\) is the mass-per-unit volume of the medium, \([b]\) is the body force per unit mass, and \(d[v]/dt\) is the material derivative of the velocity.

6.2.2 Discretization

In order to model these properties and their change over time on a large rock mass, it is necessary to discretize the model into many small parts. Models in FLAC3D are all based on tetrahedral strain-rate elements, in which all the forces involved are concentrated at the nodes. Figure 48 shows a tetrahedron as it is used in FLAC3D:

\[ \text{Figure 48: Tetrahedron showing the positions of nodes and faces (Itasca 2009).} \]

As illustrated in Figure 48, each tetrahedron consists of four faces and the corresponding nodes situated opposite their respective faces.

For each tetrahedron, the stress-rate tensor can be described for these nodes 1-4 by the following equation:
\[ \xi_{ij} = -\frac{1}{6V} \sum_{l=1}^{4} (\nu_{j}^{l})^T (n_{j}^{l}) S^{(l)} \] 6.12

where \([\nu]\) is the velocity vector, \([n]\) is the unit normal vector, \(V\) is the volume of the tetrahedron and \(S\) is its surface. The superscript \(l\) is used to define the node (1-4).

The equivalent rate-of-rotation tensor is defined as follows:

\[ \omega_{ij} = -\frac{1}{6V} \sum_{l=1}^{4} (\nu_{j}^{l}) - (\nu_{j}^{l}) S^{(l)} \] 6.13

Finally, the velocity tensor is defined as follows:

\[ \nu_{i,j} = -\frac{1}{3V} \sum_{l=1}^{4} (\nu_{j}^{l}) S^{(l)} \] 6.14

During a simulation, node locations are continuously updated using the central finite difference approximation, until stability is achieved.

### 6.2.3 Out-of-balance force

The forces acting on the volume bodies within the model and the resulting work is modeled in FLAC3D by application of the theorem of virtual work. As such, the system will be in equilibrium when the internal work rate equals the external work rate.

Considering the equilibrium state defined by the following equation:

\[ \sigma_{ij,j} + \rho B_i = 0 \] 6.15

with body forces defined as:

\[ B_i = \rho \left( b_i - \frac{d
}{dt} \right) \] 6.16

it is possible to define equations for both the external and the internal work rate.

The external work rate can be expressed as

\[ E_w = \sum_{n=1}^{4} \delta v_i^T f_i + \int_V \delta \nu_i B_i dV \] 6.17

while the internal work rate is given by

\[ I_w = \int_V \delta \xi_{ij} \sigma_{ij} dV \] 6.18

The difference between these two forces is called the out-of-balance force and is given by

\[ F_i^{<i>} = \left[ \left( \frac{\sigma_{ij} n_i^{l} S^{(l)}}{3} + \frac{\partial b_i V}{4} \right) \right]^{<i>} + P_i^{<i>} \] 6.19
This force is equal to zero when the medium has reached equilibrium. In practice, it is not possible to reach an absolute zero; therefore, an acceptable limit for the maximum unbalanced force must be implemented. The default value for the maximum unbalanced force in FLAC3D is $1 \times 10^{-5}$.

6.2.4 Implementation

FLAC3D utilizes a technique called mixed discretization to subdivide groups of tetrahedra into hexahedral zones. This technique is designed to increase the flexibility of the element by calculating the first invariant for the zone as the volumetric average value of the first invariant over all tetrahedra in that zone.

When a model is generated, the hexahedral zones are generated first, later to be subdivided into tetrahedra. There can be multiple ways to incorporate the tetrahedra into the overlying zone. A hexahedron, which by definition has eight node points, can be discretized into two possible configurations of five tetrahedra, as illustrated in Figure 49. The default setting in FLAC3D is to implement both overlays and average the two obtained values, in order to ensure a symmetric zone response.

![Figure 49: An 8-node zone with 2 overlays of 5 tetrahedra (Itasca 2009).](image)

Once the geometry of the model and the required parameters are set up, the general calculation sequence utilized in FLAC3D is as follows, as described in Itasca (2009):

1. Nodal forces are calculated from stresses, applied loads and body forces (velocity and displacement vary linearly; stress and strain are constant within an element).
2. The equations of motion are invoked to derive new nodal velocities and displacements.
3. Element strain rates are derived from nodal velocities.
4. New stresses are derived from strain rates, using the material constitutive law.

This sequence is repeated for every timestep until the desired maximum unbalanced force is achieved, and the system can be considered to be in equilibrium.

6.2.5 Constitutive model

FLAC3D offers thirteen basic constitutive models that can be applied for various situations. In this thesis, the Mohr-Coulomb model was chosen for its simplicity and the relatively uncomplicated geology at West Bokaro, which make the Mohr-Coulomb model well-suited to it. Because it was the only model that was utilized, it will therefore be the only model to be discussed here.

All constitutive models in FLAC3D share the same incremental numerical algorithm. Given the stress state at time $t$, and the total strain increment for a timestep, $\Delta t$, the purpose is to determine the corresponding stress increment and the new stress state at time $t + \Delta t$. (Itasca 2009)
The Mohr-Coulomb criterion is expressed with the principal stresses $\sigma_1$, $\sigma_2$ and $\sigma_3$, as discussed in chapter 2, with $\sigma_1 \leq \sigma_2 \leq \sigma_3$. The corresponding strain vector is described by the principal strains $\varepsilon_1$, $\varepsilon_2$ and $\varepsilon_3$.

We recall the Mohr-Coulomb criterion as described in Equation 6.1:

$$f = \sigma_1 - \sigma_3 - 2c \cos \varphi - (\sigma_1 + \sigma_3) \sin \varphi = 0$$

where $\varphi$ is the angle of internal friction and $c$ is the cohesion. Rewriting this equation, one obtains (Clausen 2007):

$$f = \sigma_1 - \sigma_3 N_\varphi + 2c \sqrt{N_\varphi}$$

where:

$$N_\varphi = \frac{1 + \sin(\varphi)}{1 - \sin(\varphi)}$$

As discussed in chapter 2, the Mohr Coulomb failure criterion has a tendency to overestimate the tensile strength of a material, and is often used in combination with a tensile cutoff to compensate for this overestimation. This is also the case in FLAC3D, whose version of the Mohr Coulomb failure criterion is combined with the Rankine tensile cutoff criterion, described in Section 3.1.6.1, into the Modified Mohr-Coulomb model.

The tension failure criterion in FLAC is defined as follows:

$$f_t = \sigma_3 - \sigma'_t$$

The tensile strength of the material now cannot exceed the value of $\sigma_3$ corresponding to the intersection point of the straight lines $f' = 0$ and $\sigma_3 = \sigma_1$ in the $f(\sigma_3, \sigma_1)$ plane. This maximum value is given by:

$$\sigma'_{t,\text{max}} = \frac{c}{\tan \phi}$$

Figure 50 illustrates the Modified Mohr Coulomb criterion in the $f(\sigma_3, \sigma_1)$ plane.

![Figure 50: FLAC3D Mohr-Coulomb failure criterion (Itasca 2009).](image)
The plastic potential, as discussed in chapter 2, is described by means of two functions, \( g^s \) and \( g^t \), used to define shear plastic flow and tensile flow, respectively. The function \( g^s \) corresponds to a non-associated law and has the form

\[
g^s = \sigma_1 - \sigma_3 N_\psi \tag{6.25}
\]

where \( \psi \) is the dilation angle and

\[
N_\psi = \frac{1 + \sin(\psi)}{1 - \sin(\psi)} \tag{6.26}
\]

The function \( g^t \) corresponds to an associated flow rule and is written

\[
g^t = -\sigma_3 \tag{6.27}
\]

The failure criterion visualized in Fig. 4 is now divided into two areas: shear failure and tensile failure. In order to do this, the following function is defined:

\[
h = \sigma_3 - \sigma^i + a^\rho (\sigma_1 - \sigma^\rho) \tag{6.28}
\]

where \( a^\rho \) and \( \sigma^\rho \) are constants defined as

\[
a^\rho = \sqrt{1 + N_\phi^2 + N_\psi} \tag{6.29}
\]

\[
\sigma^\rho = \sigma^i N_\psi - 2c\sqrt{N_\psi} \tag{6.30}
\]

Figure 51 shows the final domain of the Mohr Coulomb model.

\[\text{Figure 51: Mohr-Coulomb model - domains used in the definition of the flow rule (Itasca 2009).}\]
When a step in the simulation is carried out, the program makes an elastic guess which violates the composite yield function. This guess is represented by a point in the $(\sigma_3, \sigma_1)$-plane, located in either domain 1 or 2, corresponding to negative or positive domains of $h = 0$, respectively. After making this guess, there are three possible options:

1. If the stress point falls within domain 1, shear failure is declared, and the stress point is placed on the curve $f^s = 0$ using a flow rule derived using the potential function $g^s$.

2. If the point falls within domain 2, tensile failure takes place, and the new stress point conforms to $f^t = 0$ using a flow rule derived using $g^t$.

3. If the stress point is located outside of the representation of the composite failure envelope in the plane $(\sigma_3, \sigma_1)$, no plastic flow takes place for this step.

The program then moves on to the next step and makes a new elastic guess.

Figure 52 shows a 3D representation of the Mohr-Coulomb criterion, the Modified Mohr-Coulomb criterion and several others as projected on the 3D principal stress plane.

\textbf{Figure 52:} Examples of linear yield criteria in the principal stress space: a) The Tresca criterion; b) the Mohr-Coulomb criterion; c) the Rankine criterion; d) the Modified Mohr-Coulomb criterion (Clausen 2007).
6.3 FLAC3D model
In order to examine the geomechanical implications of backfilling a highwall mining site, a model representing the highwall was generated in FLAC3D. A physical description of the West Bokaro mine site on which the model was based is given first, followed by a more detailed explanation of the numerical model itself.

6.3.1 Highwall mining at West Bokaro
Candidate seams for highwall mining in the Banji Village (Quarry AB) area are 5, 6, 7, 8, and 10. The current configuration of highwalls is suitable to highwall mining, with only minor modifications required to create the minimum required bench width of 30m. Underground mining has previously occurred in Seams 7 and 10, affecting the layout of highwall mining in the area. Bottom elevations suggest that water could be accumulated in the old workings; therefore, AAI recommended that no less than 30m be left between highwall miner openings and areas of underground works with pooled water potential. In Quarry D, four seams are highwall mining targets: Seams 5, 6, 7, and 9. Quarry D has not been surface mined; therefore a box cut will have to be constructed. Finally, Quarry SEB contains seven seams that are considered for highwall mining: 5, 6, 7, 8, 9, 10L, and 10U. Figure 53 shows the highwall at the SEB Quarry during a site visit the author made in July of 2012.

![Figure 53: Highwall at Quarry SEB.](image)

The interburden between Seams 10L and 10U is too thin to safely mine both seams from the same highwall position; however, both seams are minable at different positions along the highwall. This area underlies a village, making it uncertain whether it can be undermined. The increased need for ground control makes this a particularly suitable site for the use of backfill. At the time of the visit, the SEB quarry, which had already been mined down to seam 5, was intended to be the first area where backfill in highwall mining will be applied, mining upwards from seam 5. For this reason, that is the area that this thesis will focus on.
6.3.2 Geometry
The highwall at West Bokaro is several hundred meters long, with little significant curvature. For this reason, it was decided to make the model more or less rectangular in shape. The model, in essence, is a slice of the highwall with the trench at the front, and straight boundaries at all the other edges. From a mathematical point of view, these edges do not exist in the model, as FLAC mirrors the model on each edge for the purposes of the calculations. However, the image shown is that of a single “slice”. Panels at the West Bokaro mine site will be a maximum of 100 meters in length. The simulations carried out here were done with panels containing 10 drives each, resulting in a maximum panel length of 86 m in the case of 3.9 m pillars.

Figure 54 shows a simplified representation of a version of the model containing four drives, which is, in effect, one “slice” representing a single panel. The front and side views of the model used in this thesis, containing ten drives, are shown more clearly in Figures 55 and 56. Note that the front view in Figure 56 contains a break in the middle to enable it to fit better on the page. Full-page versions of images 55 and 56 can be found in Appendix J.

Figure 54: Simplified representation of the FLAC model.

The geometry of the highwall miner, which creates rectangular drives as a result of its own shape, fits well into such a rectangular model. Since the model is surrounded on both sides by barrier pillars, the model can be considered as a single panel within the mine. For the purposes of this thesis, the number of drives in one panel was chosen as 10. This number was arbitrarily chosen, but is a realistic value when compared to other highwall mines. Moreover, it is an easy number to calculate with and is adequate as an initial value to start the simulations with. However, the model allows for the input of any number of drives.

Before defining the geometry in FLAC, the conceptual layout and various groups incorporated in the model had to be determined. In order to make the model as versatile as possible, the coal seam is divided into three sections: a bottom layer, a top layer, and a layer of roof coal. Three such layers in the coal seam allow for the possibility of modeling the extraction process two lifts while also leaving a layer of coal in the roof in the case of poor roof conditions.
Figures 55 and 56 show a side view and a front view of the model, respectively. The side view shows that in the y-direction, the model is divided by five planes into four sections. The first and last sections represent the footwall and the virgin rock behind the drive, respectively. The drive itself is divided into two sections, to accommodate
the modeling of partial backfilling in the horizontal direction. The planes, as seen from left to right in Figure 55, are referred to in the model as follows:

1. Footwall
2. Highwall
3. Backfill
4. End of drive
5. Virgin rock

The coal seam is divided into three different layers: top, bottom and ‘roof coal’, which allows for the modeling of various types of highwall mining situations and makes it possible to model partial backfill from top to bottom (i.e., only backfilling the bottom half of the drive). The length of the drive is divided in two by plane 3: this makes it possible to model partial backfill sequences lengthwise (i.e., only backfilling the back part of the drive). At the end of the drive is a zone bounded by planes 4 and 5, which is modeled as ‘virgin’ rock. This part of the rock mass remains intact and serves to ensure that relevant parts of the model (the end of the drive) are not situated at the edge of the model, where results are more likely to be skewed.

The seam is modeled at an incline. Due to the restrictions of the FLAC geometry, this model can only incorporate downward dipping seams, which is representative of most highwall mining seams. The highwall itself can be either vertical or inclined.

Figure 56 shows a front view of the model with 10 drives. Each of the nodes is defined and numbered individually. In order to accommodate the modeling of varying numbers of pillars between barrier pillars, the node numbers are generated using a loop in the code that refers to the input number of pillars. In the 10-drive scenario, the resulting total number of nodes is 132, as illustrated in Figure 56. Note that the model on either sides ends at the halfway point on the barrier pillars. Thus, the width of the barrier pillars displayed in the model is half that of the actual barrier pillar width.

In order to define the geometry of the model, a FISH file is created. FISH (sort for FLACish) is the programming language built into FLAC. In this file, each of the planes following Plane 1 has a standard suffix added to the standard x,y,z coordinates: HW for plane 2, BF for plane 3, EOD for plane 4, and V for plane 5. Thus the coordinate corresponding to coordinate x(1) on plane 1 becomes xhw(1) on plane 2, xbf(1) on plane 3, xeod(1) on plane 4, and xv(1) on plane 5.

The definition of the nodes is followed by the definition of the various groups described above. The following groups are defined in the model, with the drives each numbered from 1 to the total number of drives (ndrives):

- Footwall
- Left barrier pillar
- Right barrier pillar
- Web pillars
- Bottom half drive front (#1 - ndrives)
- Bottom half drive back (#1 - ndrives)
- Top half drive front (#1 - ndrives)
- Top half drive back (#1 - ndrives)
- Roof coal front (#1 - ndrives)
• Roof coal back (#1 - ndrives)
• Virgin rock
• Overburden

In order to account for the inclined geometry, a trench group and a wedge-shaped group are also defined. The wedge group has the same properties as the footwall group, while the trench bottom half and trench top half groups are extensions of the coal strata and the trench overburden group is an extension of the overburden group. All of these groups are removed after the initial equilibrium is reached, modeling the excavation of the trench.

Figures 57 and 58 show overviews of the various model groups in FLAC3D before and after excavation of the trench, respectively. In both of these figures, the roof coal group has been left out, and the coal strata is modeled as two distinct layers (top and bottom).

![Figure 57: Model groups before trench excavation](image)

In Figure 58, the green and pink barrier pillars on either side are clearly distinguished. In between the red web pillars are the upper and lower layers of the seam, which will be extracted as drives. Most of the simulations will extract these upper and lower layers in a single pass. Although in this case the upper and layers are modeled as being of equal thickness, the model allows for both layers to have their own dimensions. Thus, it is possible for either one of the layers to be modeled as being thicker than the other.
**Figure 58:** Model after trench excavation: oblique view (a), front view (b) and side view (c).
6.3.3 Input

The model requires the input of various parameters. The first group of parameters, illustrated in Figures 55 and 56, relates to the geology and the mining plan, and defines the geometry of the model:

- Thickness of the footwall (thfootwall)
- Thickness of the overburden (thoverburden)
- Width of the bench (wbench)
- Length of the stretch of virgin rock (lvirgin)
- Thickness of the bottom half drive (thbottomhalfdrive)
- Thickness of the top half drive (thtophalfdrive)
- Thickness of roof coal (throofoal)
- Width of the drive (wdrive)
- Width of the web pillars (wpillar)
- Width of the barrier pillar (wbarrier)
- Length of the drive (ldrive)
- Number of drives (ndrives)
- Dip of the seam (α)
- Angle of the highwall (β)
- Ratio of horizontal backfill (fbackfill)

The ratio of horizontal backfill is defined as a fraction of the total backfill length, and indicates the location of Plane 3 as illustrated in Figure 55. It can be used to model partial backfilling of the drive. The roof coal group can be modeled as part of the overburden if the model does not require the presence of roof coal to be modeled.

In addition to the above parameters, the following rock properties are required in order to apply the Mohr-Coulomb criterion:

- Density of the overburden (ob_density)
- Density of the coal (coal_density)
- Density of the backfill material (backfill_density)
- Angle of friction (._phi)
- Cohesion (._coh)
- Young’s modulus (young)
- Poisson’s ratio (poisson)
- Bulk modulus (K)
- Shear modulus (G)
- Ratio of horizontal to vertical stress (K0)

The bulk modulus and shear modulus are calculated using the Young’s modulus and Poisson ratio:

\[
K = \frac{E}{3(1-2v)} \quad 6.31
\]

\[
G = \frac{E}{2(1+v)} \quad 6.32
\]

where

\[
K = \text{Bulk modulus}
\]

\[
G = \text{Shear modulus}
\]
Finally, the size of the mesh can be determined by the number of blocks in each direction. The size of these blocks can be increased or decreased in a given direction using block ratios. Two adjacent blocks to be as close in size to one another as possible, with the general rule being that no one block should be more than twice the size of an adjacent block in any direction. In determining the amount of blocks, a balance must be struck between the accuracy of the model and the required calculation time, which both increase with the number of blocks in the model.

6.3.4 Execution
The model is run from the DAT file. The steps in running a simulation (and their corresponding commands) are as follows:

1. Input geometry data. This includes all geological and mining plan-related data.
   ```
   @input_geom
   ```
2. Call the FISH file, which incorporates the definition of all the nodes and groups.
   ```
   call code.fis
   ```
3. Generate the mesh created by the nodes as defined in the FISH file.
   ```
   @mesh_geom
   ```
4. Generate the groups.
   ```
   @gen_groups
   ```
5. Assign rock mass property values to the various groups (overburden, coal) using the Mohr Coulomb model.
   ```
   model mohr
   ```
6. Set the remaining parameters such as gravity, K0, initial stresses.
   ```
   set gravity; set k0; @get_stresses
   ```
7. Solve the initial equilibrium.
   ```
   solve
   ```
8. Excavate the trench.
   ```
   @excavate_trench
   ```
9. Excavate the drives.
   ```
   @excavate_drive
   ```
10. Backfill the drives.
    ```
    @backfill_drive
    ```

Depending on the selected mining sequence, the excavation and backfilling of the drives may occur in overlapping stages. The standard mining sequence in this model is a three by three sequence, where two drives are skipped between each drive that is mined, and then backfilled before the next sequence of drives is mined, as illustrated in Figure 59.

The three-by-three sequence was chosen as thfore initial sequence because it leaves a lot of space between the drives, thus presumably offering increased stability and making it less likely to collapse. In order to model different mining and backfilling sequences (such as one-by-one or four-by-four), a separate FISH file must be used in which the sequencing code for the mining and backfilling process are slightly altered to reflect the new mining sequence.
The simulation continues until the maximum mechanical force ratio, as described in section 6.2.3, is reached. The full codes from the DAT file and the FISH file can be found in Appendices K and L respectively.

![Diagram of mining and backfilling sequence](image)

**Figure 59:** Three-by-three sequencing of mining and backfilling

### 6.3.5 Simulations

For this thesis, multiple simulations were carried out to determine the effect of the following factors on the backfill process:

- Pillar width
- Mine/backfill sequence
- Backfill material
- Partial backfill
- Multiple pass mining

To carry out the simulations, input data from the West Bokaro mine site was used. This data was collected both from existing reports about the mine site as well as from laboratory testing. Due to the limited availability of exact data, some assumptions had to be made. A description of the set-up of the simulations and the input data follows below.

### 6.3.7 General input data

The geological data for the model have been obtained from Agapito Associates (2005). Table 6 shows a summary of the Quarry SEB Seam Model (Agapito Associates 2005). This Table, as well as Table 2, are where most of the data was derived from.

---

84
where

\[ \overline{D_{OB}} = \frac{H_{r} \times D_{r}}{H_{r}} + \frac{H_{f} \times D_{f}}{H_{f}} = \frac{113.8 \times 2500}{140} + \frac{26.2 \times 1550}{140} = 2322.2 \text{ kg/m}^3 \]

Table 6: Summary of Quarry SEB Seam Model (Agapito Associates 2005).

<table>
<thead>
<tr>
<th>Unit</th>
<th>Thickness (m)</th>
<th>Average Thickness (m)</th>
<th>Approximate Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seam 10U</td>
<td>1–2</td>
<td>1.5</td>
<td>50</td>
</tr>
<tr>
<td>Seam 10L/10U IB</td>
<td>0–2</td>
<td>1.0</td>
<td>—</td>
</tr>
<tr>
<td>Seam 10L</td>
<td>0–5</td>
<td>1.2</td>
<td>50</td>
</tr>
<tr>
<td>Seam 9/10L IB</td>
<td>7–20</td>
<td>12.0</td>
<td>—</td>
</tr>
<tr>
<td>Seam 9</td>
<td>1–2</td>
<td>1.5</td>
<td>64</td>
</tr>
<tr>
<td>Seam 8/Seam 9 IB</td>
<td>2–4</td>
<td>3.0</td>
<td>—</td>
</tr>
<tr>
<td>Seam 8</td>
<td>1–6</td>
<td>4.5</td>
<td>69</td>
</tr>
<tr>
<td>Seam 7/Seam 8 IB</td>
<td>11–30</td>
<td>20.0</td>
<td>—</td>
</tr>
<tr>
<td>Seam 7</td>
<td>6–11</td>
<td>8.0</td>
<td>95</td>
</tr>
<tr>
<td>Seam 6/7 IB</td>
<td>10–20</td>
<td>15.0</td>
<td>—</td>
</tr>
<tr>
<td>Seam 6</td>
<td>1–5</td>
<td>4.0</td>
<td>120</td>
</tr>
<tr>
<td>Seam 5/6 IB</td>
<td>5–20</td>
<td>13.0</td>
<td>—</td>
</tr>
<tr>
<td>Seam 5</td>
<td>2–10</td>
<td>4.5</td>
<td>140</td>
</tr>
</tbody>
</table>

As stated in section 6.3.1, seam 5 is the first seam that will be mined, and is therefore the seam that will be used in this model. The total seam thickness (4.5 m) has been split into two parts (top and bottom) of half the total thickness each (2.25 m). The width of the drive, 3.5m, is the standard drive width for the Caterpillar HW300. Furthermore, the number of drives was set to 10, as this is a realistic number and is large enough to give detailed results of the stress distribution within a panel. Similarly, the thickness of the footwall and the width of the bench, as well as the length of the virgin rock, were set at numbers that are large enough to allow for the edges of the model to have a minimal influence on the inside of the model, without excessively affecting the calculation speed. The dimensions entered into the FLAC model as derived from Agapito Associates (2005) are as follows:

- Thickness of footwall: 80m
- Thickness of overburden: 140m
- Width of bench: 50m
- Length of virgin rock: 300m
- Seam dip (α): 5°
- Highwall angle (β): 60°
- Thickness bottom half drive: 2.25m
- Thickness top half drive: 2.25m
- Width of the drive: 3.5m
- Length of the drive: 250m

The density of the coal was based on measurements taken from on-site samples and was set at 1550 kg/m³. The density of the overburden was based on an average using density measured from on-site samples of the roof rock material as well as the coal strata, and averaging out the total density based on the thickness of the various strata as described in Table 2:
For the initial simulations, the backfill ratio, $f_{\text{backfill}}$, was set at 0.5. However, this only has any effect when the simulations are modeling partial backfill in the horizontal direction; in all other instances both front and back parts of the drives are modeled the same. The thickness of the roof coal was initially set at 1; however, the roof coal group is generally modeled as part of the overburden group unless simulation calls for roof coal to be modeled.
7. Simulations and results

This chapter will discuss the procedures and results of the various simulations that were done. The results discussed in this chapter are the main focal point of the thesis, and give a clear indication to the validity of the hypothesis that backfilling is beneficial to overall recovery in highwall mining. First, a short overview of all the simulations is given in section 7.1. This is followed by a description of the indications of failure in the model in section 7.2: what constitutes failure, and what does not. In order to determine this, some virtual triaxial tests were performed on the material in FLAC3D. Then, in sections 7.3 through 7.7, the simulations are discussed regarding pillar width, mine and backfill sequencing, backfill material, partial backfilling, and multi-lift mining, respectively.

7.1 Overview

Before running the simulations, it was necessary to determine the exact parameters of interest to this thesis. These parameters were chosen because previous literature, as discussed in Chapter 4, has determined they are the most consequential factors in mine design when considering the implementation of backfill in a highwall mining scenario. As described in Chapter 4, they are as follows:

- Pillar width
- Mine and backfill sequencing
- Backfill material
- Partial backfilling
- Multi-lift mining and backfilling

Pillar width was chosen as the initial variable, because in addition to determining the ideal pillar width, it would be worthwhile to investigate the overall effect of pillar width on the efficiency of the mining process. An initial, conservative estimate had to be made, which is elaborated on in section 7.3.1.

The choice for the mine and backfill sequencing was less straightforward. However, it was decided to take a three-by-three sequence as the base sequence, as depicted in Figure 59. This sequence allows a relatively large amount of space between drives, while still being able to extract three or four drives on each pass. Moreover, it is the sequence that is intended to be used at West Bokaro. Analogous to the three-by-three sequence, one-by-one, two-by-two, and four-by-four sequence simulations were also done. Next, a few simulations with variations on the three-by-three sequence and two-by-two sequence were done. Finally, a five-by-five sequence was simulated. Each of these sequences is explained as follows:

1. **One-by-one - 10 passes**
   Excavate drive 1 - backfill drive 1 - excavate drive 2 - backfill drive 2 - excavate drive 3 - backfill drive 3 - excavate drive 4 - backfill drive 4 - excavate drive 5 - backfill drive 5 - excavate drive 6 - backfill drive 6 - excavate drive 7 - backfill drive 7 - excavate drive 8 - backfill drive 8 - excavate drive 9 - backfill drive 9 - excavate drive 10 - backfill drive 10.

2. **Two-by-two - 2 passes**
   Excavate drives 1, 3, 5, 7, 9 - backfill drives 1, 3, 5, 9 - excavate drives 2, 4, 6, 8, 10 - backfill drives 2, 4, 6, 8, 10.

3. **Three-by-three - 3 passes**
Table 7 shows an overview of the FLAC3D simulations that were done, and the parameters that were varied for each simulation.

<table>
<thead>
<tr>
<th>Number</th>
<th>Simulation parameter</th>
<th>Pillar width (m)</th>
<th>Backfill material</th>
<th>Sequence</th>
<th>Full/partial backfill</th>
<th>Number of lifts</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Pillar width</td>
<td>3.9</td>
<td>FCM &amp; no BF</td>
<td>3 by 3</td>
<td>Full</td>
<td>One</td>
</tr>
<tr>
<td>2</td>
<td>Pillar width</td>
<td>1.4</td>
<td>FCM &amp; no BF</td>
<td>3 by 3</td>
<td>Full</td>
<td>One</td>
</tr>
<tr>
<td>3</td>
<td>Pillar width</td>
<td>2.9</td>
<td>FCM &amp; no BF</td>
<td>3 by 3</td>
<td>Full</td>
<td>One</td>
</tr>
<tr>
<td>4</td>
<td>Pillar width</td>
<td>3.4</td>
<td>FCM &amp; no BF</td>
<td>3 by 3</td>
<td>Full</td>
<td>One</td>
</tr>
<tr>
<td>5</td>
<td>Sequence</td>
<td>2.9</td>
<td>FCM</td>
<td>1 by 1</td>
<td>Full</td>
<td>One</td>
</tr>
<tr>
<td>6</td>
<td>Sequence</td>
<td>2.9</td>
<td>FCM</td>
<td>2 by 2</td>
<td>Full</td>
<td>One</td>
</tr>
</tbody>
</table>
Table 7: Overview of simulations and corresponding parameters.

<table>
<thead>
<tr>
<th>Number</th>
<th>Simulation parameter</th>
<th>Pillar width (m)</th>
<th>Backfill material</th>
<th>Sequence</th>
<th>Full/partial backfill</th>
<th>Number of lifts</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>Sequence</td>
<td>2.9</td>
<td>FCM</td>
<td>4 by 4</td>
<td>Full</td>
<td>One</td>
</tr>
<tr>
<td>8</td>
<td>Sequence</td>
<td>2.9</td>
<td>FCM</td>
<td>3 by 3 *</td>
<td>Full</td>
<td>One</td>
</tr>
<tr>
<td>9</td>
<td>Sequence</td>
<td>2.9</td>
<td>FCM</td>
<td>2 by 2 *</td>
<td>Full</td>
<td>One</td>
</tr>
<tr>
<td>10</td>
<td>Sequence</td>
<td>2.9</td>
<td>FCM</td>
<td>2 by 2 * with direct BF</td>
<td>Full</td>
<td>One</td>
</tr>
<tr>
<td>11</td>
<td>Sequence</td>
<td>2.9</td>
<td>FCM</td>
<td>5 by 5</td>
<td>Full</td>
<td>One</td>
</tr>
<tr>
<td>12</td>
<td>Material</td>
<td>2.9</td>
<td>Loose sand</td>
<td>3 by 3</td>
<td>Full</td>
<td>One</td>
</tr>
<tr>
<td>13</td>
<td>Material</td>
<td>2.9</td>
<td>FCM with increased stiffness</td>
<td>3 by 3</td>
<td>Full</td>
<td>One</td>
</tr>
<tr>
<td>14</td>
<td>Partial</td>
<td>2.9</td>
<td>FCM</td>
<td>3 by 3</td>
<td>Halfway (vertical)</td>
<td>One</td>
</tr>
<tr>
<td>15</td>
<td>Partial</td>
<td>2.9</td>
<td>FCM</td>
<td>3 by 3</td>
<td>Halfway (horizontal)</td>
<td>One</td>
</tr>
<tr>
<td>16</td>
<td>Multi lift (up)</td>
<td>2.9</td>
<td>FCM</td>
<td>3 by 3</td>
<td>Full</td>
<td>Two</td>
</tr>
<tr>
<td>17</td>
<td>Multi lift (down)</td>
<td>2.9</td>
<td>FCM</td>
<td>3 by 3</td>
<td>Full</td>
<td>Two</td>
</tr>
</tbody>
</table>

This chapter will discuss the results of these simulations with regards to stress, displacement, and failure. Section 7.1 first discusses the mechanisms of failure in the FLAC model and what indications the model gives that failure has taken place. After that, each chapter is devoted to one of the five parameters listed above and the results of the related simulations.

7.1.1 Chart legends

In the following sections, many charts generated by FLAC3D will be shown. In these charts, there are three parameters that will be studied: failure, displacement, and stress.

FLAC3D can display these values for the entire model in a single plot, where each color is indicative for a particular value range. For the displacement and stress plots, the range of values is given in the legend to the left of the plot, with a color gradient where blue and red represent the edges of the spectrum. Such gradients give valuable insight into the location of the zones where stress and displacement are concentrated.

Failure is indicated by the plasticity state plots. For these plots, there is also a legend displayed to the left of the plot, consisting of individual colors rather than a gradient. Two types of failure mechanisms are indicated by the plasticity state plot: shear failure and tensile failure. Each type is designated by a different color on the plot. The plot also indicates whether stresses within a zone are currently on the yield surface (i.e., the zone is at active failure now, -n), or the zone has failed earlier in the model run, but now the stresses fall below the yield surface (the zone has failed in the past, -p). Initial plastic flow can occur at the beginning of a simulation, but subsequent stress redistribution unloads the yielding elements so that their stresses no longer satisfy the yield criterion, indicated by shear-p or tension-p (on the plasticity state plot).
Generally speaking, in the plasticity state plots, inactive zones are designated as dark blue, tensile failure zones are designated as bright green, and shear failure zones are designated as various shades of red and purple. However, many different combinations of shear failure and tensile failure are possible, so a variety of colors can be visible in the plots. The exact legend is always listed to the left of the plot.

7.2 Indications of failure
As a general indication of failure, the plasticity state plot is a useful tool in FLAC. This plot displays those zones within a model in which the stresses satisfy the yield criterion. Such an indication usually denotes that plastic flow is occurring, but it is possible for an element simply to “sit” on the yield surface without any significant flow taking place.

Initial simulations showed tensile zones being generated much sooner than failure would be expected. To examine how the failure mechanisms in the model work exactly, a triaxial test on the coal was simulated in FLAC to determine the exact correlation between the tensile zones in the model and actual failure.

Using practical examples from the FLAC manual, a DAT file was created containing the code for the triaxial test, which can be found in Appendix M. The test was carried out both without confinement and at a confinement of 20 MPa. The properties used were the same properties used for the coal in the highwall model. The results of the unconfined and confined tests are summarized in the stress-strain plots in Figure 60 and 61, respectively.

![Stress-strain curve and plasticity state plots of the core at various moments during virtual triaxial test (no confinement)](image)
The highwall model is representative of the in situ conditions, so the virtual triaxial test was carried out using the adjusted data for the rock mass obtained using RocLab, rather than the results obtained from the lab. For this reason, it is unsurprising that the coal fails more quickly than in the lab test using cores obtained from on-site samples.

In both figures, the plasticity state plots have been placed at the approximate moment that they occur during the triaxial test. There are two notable differences between the confined vs the unconfined test: the peak strength of the material is much higher in the confined test, and tensile zones appear only in the unconfined test. However, in both tests, failure occurs shortly after the core displays shear zones.

Both phenomena can be explained by increased strength of the material - both shear strength and tensile strength - as a result of confinement. In the unconfined test, tension zones appear soon after the start of the test; however it is clear that failure does not yet take place. Shear failure zones appear shortly before peak strength is achieved and failure takes place. At the point of failure, in both scenarios, the entire core is experiencing shear failure. This suggests that the tensile failure zones act as a precursor to the shear failure zones, and that the presence of shear failure zones, not tensile failure zones, are an indication for overall failure of the material. For this reason, shear failure zones will be used as a main indication of failure in the analysis of the simulations.
7.3 Pillar width

Initial simulations were done with a variety of pillar widths, to determine at which width the pillars fail and what is the ideal pillar width for the given situation at West Bokaro. This ideal pillar width is also the pillar width that will be used for all subsequent simulations. Simulations were carried out both with and without backfill, to determine its effectiveness.

7.3.1 Initial pillar width

The purpose of the first set of simulations is to determine the ideal pillar width for the given scenario. However, an initial width must first be defined to start off the simulations with. Because the point of failure is the main focus of interest here, the initial factor of safety used was conservative.

Initial calculations were done using the Mark-Bieniawski equation (equation 4.2), as described in section 4.2, to obtain a general idea of the magnitude of pillar width:

\[ S_p = S_c \left[ 0.64 + \left( 0.54 \frac{W}{h} \right) \right] \]

where

- \( S_p \) = pillar strength (MPa)
- \( S_c \) = in situ coal strength (MPa)
- \( h \) = pillar height (m)
- \( W \) = pillar width (m)
- \( l \) = pillar length (m)

Additionally, the factor of safety is determined as follows:

\[ SF = \frac{S_p}{L_p} \]

Where \( L_p = S_v \frac{W + W_E}{W} \)

and

- \( L_p \) = average vertical load on the pillar
- \( S_v \) = in situ vertical stress (MPa)
- \( W_E \) = width of the entry (drive) (m)

The assumption for coal strength is usually taken around 6.2 MPa. In situ vertical stress depends on the overlying rock density and overburden depth, and the vertical stress gradient is typically 0.025 MPa/m (Zipf 2005). The drive height is the total thickness of the seam, 4.5m. Thus what remains is the following:

\[ SF = \frac{S_v \times \left( 0.64 + 0.54 \frac{W}{h} \right)}{S_v \times \left( \frac{W + W_E}{W} \right)} = \frac{6.2 \times \left( 0.64 + 0.54 \frac{W}{4.5} \right)}{(0.025 \cdot 140) \times \left( \frac{W + 3.5}{W} \right)} \]

Using this equation, a pillar width of 3.9 results in a factor of safety of slightly over 1. Thus, the starting point for the pillar width is set at 3.9.
In India, where coal strengths are higher than in many other places, the Sheorey equation is often used in lieu of the Mark-Bieniawski equation (Sheorey 1992):

\[ S_p = 0.25\sigma_{c25}h^{-0.36} + \left( \frac{H}{250} + 1 \right) \left( \frac{w}{h} - 1 \right) \]  \hspace{1cm} 7.5

where

- \( S_p \) = pillar strength (MPa)
- \( \sigma_{c25} \) = compressive coal strength of one 25 mm cube (MPa)
- \( h \) = pillar height (m)
- \( w \) = pillar width (m)
- \( H \) = coal seam depth (m)

Substituting equation 7.5 into equation 7.2, and using 25 MPa as the coal strength, 4.5 m as the pillar height, and 140 m as the coal seam depth, one obtains a pillar width of 4.5 m for a factor of safety of 1. However, the estimate for the coal strength is likely conservative. The Mark-Bieniawski estimate of 3.9 m is maintained as the initial value for the simulations.

### 7.3.2 Evaluation of input data

As the 3.9 m simulation was the first simulation, it was also the first opportunity to see if there are any issues with the input parameters. The initial simulations indicated that there was a problem with the input data, as the model showed failure of the coal strata before excavation of the drives had even taken place as shown in Figure 62. For this reason, the parameters entered into RocLab to obtain the input data for the FLAC model were adjusted to their final values as described in Chapter 5, in order to obtain a stable initial highwall in the model. Other problems included various errors in block numbers and ratios, which were all amended before continuing with the simulations.

![FLAC3D 4.00](image)

**Figure 62: Failure of the coal strata before excavation of the drives.**

In addition to the failing coal strata, Figure 62 also shows an irregularity in the plasticity plot at the barrier pillars. These sections show blue sections (no tension) where most of the rest of the model is green (tension-p). This cannot be correct, as the state of the model before extraction of the drives has taken place should be horizontally homogenous. This problem was amended by increasing the number of blocks in the barrier pillars, thereby making them the same size as the neighboring blocks in drives 1 and 10. However, this resulted in a
disproportionately large increase in calculation time. Since the barrier pillars are of little relevance in these simulations, it was decided to leave the irregularity as-is.

Many of the results of early simulations turned out to be useless because of bugs in the code that were subsequently discovered, or data that was entered improperly. Several of the simulations turned out to be invalid as a result of the drive width accidentally being entered as the pillar width, or an accidental reverse of the block ratios. Such errors were not immediately apparent, but became obvious once the results of the simulations were examined. Although the code itself is fairly straightforward, the many parameters make it easy to overlook a mistake. For this reason, much attention should be paid when altering the model parameters, so as to ensure that the model will function as intended. For this thesis, an estimated 25 or so simulations had to be discarded as the result of some error. With the average simulation taking about a little over a day, that means that about a month was lost due to unusable simulations.

7.3.3 Simulation results
The following sections describe the simulation results for the various pillar width simulations. The plasticity state plots are taken at a cutting plane 100 m into the drive - this is about halfway into the drive and should give a good representation of the stress state throughout the drive. Simulations with the very narrow pillar width of 1.4m, although unrealistic, are used to illustrate an exaggerated version of effects that occur in a less obvious manner at larger pillar sizes.

7.3.3.1 Failure
Initial simulations were carried out at a pillar width of 3.9m, both without backfill and with backfill. Figures 63 and 64 show the plasticity state plots of these simulations. Neither of these plots showed any failure. Subsequent simulations therefore focused on smaller pillar widths.

![Figure 63: Plasticity state for pillar width 3.9m, no backfill. Inset is close-up of drives 1 and 2.](image-url)
In the next simulation, the pillar width was chosen to be significantly smaller, at 1.4 m, in order to determine the sensitivity of the model - whether the model would indeed show failure at a pillar width where there is no reason to expect stability. If this is not the case, there is likely to be a problem with the configuration of the model. Figures 65 and 66 show the plasticity state plots for these simulations.
As expected, these simulations show obvious failure of the pillars, including some failure at the edges of the barrier pillars. These results show that the optimum pillar width must lie somewhere between 1.4 and 3.9m. Subsequent simulations were done at 0.5 m intervals, moving upwards from 1.4m.

The simulation of the most interest was that of 2.9m. The results of this simulation, shown in Figures 67 and 68, show the most obvious contrast between backfilled drives vs non-backfilled drives. The non-backfilled drives show failure all the way through the pillar. By contrast, the backfilled drives show areas of failure, but at no point has any pillar failed completely.

**Figure 66:** Plasticity state for pillar width 1.4m, after backfill. Inset is close-up of drives 1 and 2.

**Figure 67:** Plasticity state for pillar width 2.9m, no backfill. Inset is close-up of drives 1 and 2.
In the simulations for pillar width 3.4m, shown in Figures 69 and 70, the difference between backfilled vs non-backfilled is much less obvious, and in both cases the pillars are relatively stable.

**Figure 68:** Plasticity state for pillar width 2.9m, after backfill. Inset is close-up of drives 1 and 2.

**Figure 69:** Plasticity state for pillar width 3.4m, no backfill. Inset is close-up of drives 1-3.
In all of the simulations so far a pattern is visible in the drives after they have been backfilled, showing stress zones throughout the drive with the exception of the stress-free zones along the edges at the top and bottom and the lower sides. These stress-free patterns encapsulated within stress zones represent the edges of the backfill material where the backfill meets the coal walls. These zones likely occur as a result of the lack of cohesion between the coal and the backfill material, which results in a small zone that is not able to bear any stress. Towards the top corners, there is likely more compression and cohesion between the two layers, resulting in an area that is more susceptible to stress and consequently displays failure in the plot.

In the plasticity state plots for pillar width 3.4 m, shear failure is concentrated at the edges of the pillars. However, in neither scenario complete failure of the pillar has taken place. Therefore there seems to be no obvious advantage to backfilling at this pillar width.

Failure in the pillars is concentrated mostly around those drives that were backfilled the last (drives 3, 6 and 9). This suggests that the backfilled drives do not compensate fully for the removed coal, as the stress on the coal pillars increases as the total amount of coal in the panel decreases and is replaced by backfill material. This theory is supported by the fact that the backfilled drives themselves show no failure at all, indicating that they may not be load-bearing. The contrast between the non-backfilled simulation and the backfilled simulation shows that the backfill does have some added benefit. However, the fact that the backfilled simulations for a pillar width of 2.4m show full failure of the pillars suggest that the advantage gained is no more than 0.5m. Figure 71 shows that failure does not occur until the final pass of mining.
Figure 71: Plasticity state plot for pillar width 2.9m, during second pass (A), having excavated and backfilled drives 1, 4, 7, 10, and 2, and having excavated drives 5 and 8, and during third pass (B), having excavated and backfilled drives 1, 4, 7, 10, 2, 5, and 8, and having excavated drives 3 and 6.

7.3.3.2 Displacement

In addition to failure, the risk of displacement is very important in pillar design. In stiff materials, pillars may still remain upright even after failure has taken place. Areas of interest are movement in the x-direction (horizontally) for the pillars, and movement in the z-direction (vertically) for the drive roof and floor.

Figure 72 shows the general vertical movement over the model as a whole, after excavation and backfilling of the drives. This image shows downward movement in the back of the model and upward movement in the front of the model. This effect can be explained by the additional weight of the overburden towards the back of the model pushing down and causing an upwards lift towards the front. However, the scale of the movement is minimal - a maximum of 6.5 cm of lift at the very front of the model.
Displacement within the drives shows a similar picture: displacement occurs, but on a small scale. Even in an extreme situation, such as the 1.4m drive, the movement occurs as a matter of centimeters, as shown in Figures 73 and 74.

Figure 72: Z-displacement in the entirety of the model.

Figure 73: Z-displacement in the pillars, no backfill (pillar width 1.4 m)
The lack of significant displacement is surprising, as it would be expected that with such significant failure, a large amount of displacement would occur. Considering the fact that the virtual unconfined triaxial test showed a similar lack of displacement, it can be reasonably assumed that the lack of displacement is due to the inherent material properties. The Young’s Modulus used in these simulations was obtained by entering the measured Young’s Modulus of 5 GPa into RocLab, as described in section 5.4.1. This resulted in an adjusted Young’s Modulus of approximately 3.5 GPa. However, Agapito Associates (2005) reported a Young’s Modulus of 2.1 GPa, as noted in Table 1. It is possible that the stiffness of the material is overestimated. Regardless, even though the FLAC model displays no significant displacement, it should not be assumed that such narrow pillars will be stable, particularly when they display such a large amount of failure.

From Figure 74, it can be seen that displacement occurs outwards from the center of the panel: in the leftmost web pillar displacement occurs only to the left, while in the rightmost web pillar it occurs to the right. This is congruous with the fact that the displacement occurs in the direction away from the largest amount of stress; that is to say, away from the center of the panel, where the ratio of extraction is the largest. The pillars are, in essence, being crushed and are consequently undergoing some swelling at the center.

Simulations at a pillar width of 2.9 m show a similar pattern, as shown in Figures 75 and 76.
The maximum horizontal displacement at this pillar width for the non-backfilled state is about 2 mm, while the maximum vertical displacement is about 15 mm. However, the overall reduction in vertical strain is significant: the 1.4 m simulations show a total vertical compression of approximately 4.3 mm, while for the 2.9 m simulations this value is approximately 0.76 mm - an 82% reduction. In contrast to the 1.4 m simulations, the barrier pillars in the 2.9 m simulations show some displacement towards the center of the panel. This is likely due to the fact that the increase in pillar width has caused an overall increase of the panel size by 13.5 m, resulting in increased stress on the barrier pillars.
For the non-backfilled state, the maximum displacement in the negative and positive x-direction are 2.02 mm and 2.12 mm, respectively. The corresponding maximum displacement values for the backfilled state are 1.65 mm and 1.67 mm, as shown in Figure 77.

![FLAC3D 4.00 Contour Of X-Displacement](image)

**Figure 77:** X-displacement in the pillars, after backfill (pillar width 2.9 m)

This means that there is an overall reduction in horizontal displacement of approximately 20% for backfilled drives vs non-backfilled drives. However, the absolute values are still very small when compared to the pillar width and drive width of 2.9 m and 3.5 m, respectively.

As shown in Figures 76 and 78, backfilling does not cause much difference in vertical displacement: both plots show the same upwards displacement trend, as well as a maximum upwards displacement of approximately 16 mm. However, Figure 78 shows that the backfill material itself, likely as a result of gravitational forces, experiences downwards displacement. This displacement is slightly smaller in magnitude for the drives that were backfilled last. However, the net movement of the drive floor is upwards.
Figure 78: Z-displacement in the pillars, after backfill (pillar width 2.9 m)

Figure 79 shows the net y-movement within the model.

Figure 79: Y-displacement in the entirety of the model.

It should be noted here that in the design of the model, the positive y-direction is towards the back of the model. Y-displacement therefore occurs towards the front, increasing in magnitude around the drive entries. Most of this movement takes place as a result of trench excavation, however, a small amount of additional movement occurs as a result of extraction of the drives. Figure 80 shows y-movement at a cutting plane 100 m into the drive.
This shows that even though the surrounding rock experiences a slight forward displacement, the backfill material itself remains more or less stationary. There is no difference in displacement in the y-direction between the various drives as a result of the mining sequence. This suggests that the cohesion between the backfill material and the surrounding rock is negligible.

### 7.3.3.3 Stress

To examine the stress state in the panel, contour plots were made of the minor principal stress, which represents the major compressive stress. Stress state plots after completion of the simulations show a concentration of the stress in the web pillars. This is the case both for the backfilled state and for the non-backfilled state, as shown in Figures 81 and 82. The roof and floor of the drives experience a very low amount of stress. Comparing these plots with the corresponding plasticity state plots, shown in Figures 67 and 68, suggests that shear failure takes place at stress levels of around 7.5 MPa. The plasticity state plots show that failure starts at the edges of the pillars before moving inwards. Therefore the lower stress zones at the edges of the pillars indicate failure zones: failure has already taken place, and the lower stress levels now are at the residual strength of the material.
The stress levels in the pillars increase towards the center of the panel, while the outer web pillars experience much lower levels of stress. This is likely due to the load on the center pillars being higher as they are at the furthest distance from the additional support of the barrier pillars.

A different image emerges for the simulations at 1.4 m pillar width, shown in Figures 83 and 84, where the pillars display shear failure both with and without backfill as seen in Figures 65 and 66.
Figure 83: Contour of minor principal stress, no backfill (pillar width 1.4 m).

In the backfilled simulation, the highest levels of stress are concentrated in pillars 4 and 7 and somewhat in pillar 1, as seen in Figure 84. This is likely correlated to the mining sequence. Pillars 1, 4 and 7 have each undergone excavation on both sides and surrounded completely by backfill by the time the second excavation/backfill pass has been completed. The other pillars were still supported on one side by the remaining coal layers until they were subsequently extracted in the third pass. Pillar 1’s proximity to the barrier pillar likely relieved some of its load.

Figure 84: Contour of minor principal stress after backfilling (pillar width 1.4 m).
However, such a distribution of the stress levels is absent from the non-backfilled simulation (Figure 83). Not only are the stress levels much more evenly distributed over the web pillars, but they are also lower than in the backfilled simulation. In the non-backfilled simulation the barrier pillars experience a large amount of stress of up to 10.5 MPa, much higher than any of the stress levels in the backfilled simulation, where the maximum stress level was 8.8 MPa.

With the exception of pillars 1, 4 and 7 in the backfilled simulation, stress levels in the pillars in both simulations are similar - around 6 MPa. Such results correspond with a scenario in which all the pillars have failed and the barrier pillars are now containing the failure. This is supported by the fact that concentrated areas in the web pillars experience higher amounts of stress of up to 8.8 MPa before regressing to the uniform stress level of around 6 MPa. The decrease in stress between the non-backfilled pillars at 1.4 m (Figure 83) vs. the non-backfilled pillars at 2.9 m (Figure 81) is likely due to the fact that at 2.9 m, the pillars are still in the process of failing, and are still partially at peak strength, whereas at 1.4 m, failure is complete, and the pillar strength has regressed fully to a residual strength of about 6 MPa.

For the non-backfilled simulation, failure of the pillars is preceded by a peak strength of about 8.8 MPa. Figures 85 and 86 compare the plasticity state and stress contour of the non-backfilled simulation for pillar width 1.4 m, during the last pass of mining, having just extracted drive 3, and shows clearly the high stress zones that precede failure.

Comparison between the 1.4 m simulation and the 2.9 m simulation suggests that the pillars in the non-backfilled simulation at 2.9 m pillar width have not yet failed completely, but rather are still in the process of failing. As the virtual triaxial tests showed, shear failure occurs just before failure takes place. Therefore it is possible that failure has not yet occurred in certain areas in the pillars even though the model appears to show shear failure. These are likely the high-stress zones in the center of the pillars. This corresponds to the plasticity state plots, which indicate that the center of the pillar is the last area to succumb to shear failure.

**Figure 85: Plasticity state for pillar width 1.4m, no backfill, during third pass of extraction after extracting drive 3.**
The plasticity state plot and the stress contour plots for pillar width 1.4 indicate unequivocally that the non-backfilled pillars have failed. However, as with the non-backfilled state at 2.9 m, it seems that for the backfilled state, even though the plasticity state plot indicates shear failure, actual failure of the pillars may not yet have taken place.

It can be concluded that the backfill clearly does have an effect on the strength of the pillars. This additional strength cannot be derived from any significant load-sharing with the backfill material, as the stress levels in the backfill material are very low, particularly compared to the stress levels in the pillars. However, the confinement likely has a positive effect on the strength of the pillars, similar to the effect demonstrated in the virtual triaxial test, where compressive strength almost doubled at a confinement of 20 MPa.

Although the load-bearing function of the backfill material is minimal, Figure 84 shows that there is some difference in the amount of stress on the drives that were backfilled first vs those that were backfilled last. The stress on drives 4 and 7 is approximately 1 MPa, while the stress on pillars 2, 5 and 8, which were backfilled during the second pass, is about half that, at approximately 0.5 MPa. The stress levels in drives 3, 6 and 9, which were backfilled last, are much lower at about 0.015 MPa. Moreover, the zones directly adjacent to the backfill material in the pillars show a slight decrease in stress levels that is not present in the non-backfilled situation, shown in Figure 83. This alleviation of the stress within the pillars seems to occur only in pillars adjacent to drives 3, 6, and 9 - the drives that were backfilled the last. By contrast, higher stress zones appear on the sides of the backfill material next to which a drive was last excavated. Figure 87 shows such an increased stress zone on the right side of drive 5, adjacent to drive 6 which was excavated during the last excavation pass.
Figure 87: Close-up of drives 5 and 6 in the minor principal stress contour after backfilling (pillar width 1.4 m), showing the green zones in the areas adjacent to drive 6 in pillars 5 and 6.

7.3.3.3.1 Principal stress behavior

The visual contour plots discussed so far have one major drawback: they only show the stress state at a single moment in time during the course of the simulation. In order to better understand the stress behavior in the model during the various stages of extraction and backfilling, it is more useful to examine the behavior of the various principal stress components throughout the entire simulation rather than just at one instant. To this end, the progress of the principal stresses values was tracked and plotted during the course of the entire simulation for a few select zones in the model. Because it is impossible to do this for every single block within the model, it was necessary to identify a few locations within the model that would be of particular interest. The stress changes within the pillars, the backfilled drives, and the roof rock are of the most interest. Three such zones were identified within the model in which the stress values were tracked throughout the simulation.

The zones were all chosen in the vicinity of drive #2. The selection of this drive was mostly arbitrary, but influenced by the fact that it is a drive that is excavated and backfilled during the second pass of mining - neither the first nor the last - and so should be a good indicator of the influence of extraction of neighboring drives both before the drive itself is exactracted, and after, without being excessively influenced by proximity to the barrier pillars. A schematic visualization of the approximate location of the zones is shown in Figure 88. However, due to the way in which FLAC defines its zones there may be minor discrepancies between the zone locations shown here and the actual locations that were used.

Tracking of the principal stress behavior was done for two simulations: pillar width 1.4 m and pillar width 2.9 m. Table 8 shows the exact coordinates of the zones for these simulations within the model.
Figure 88: Red dots indicate the locations of the zones in which principal stress progression is tracked.

<table>
<thead>
<tr>
<th>Pillar width</th>
<th>1.4 m</th>
<th></th>
<th>2.9 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coordinate</td>
<td>x</td>
<td>y</td>
<td>z</td>
</tr>
<tr>
<td></td>
<td>x</td>
<td>y</td>
<td>z</td>
</tr>
<tr>
<td>Roof zone</td>
<td>13.93</td>
<td>147.83</td>
<td>76.46</td>
</tr>
<tr>
<td></td>
<td>15.43</td>
<td>147.83</td>
<td>76.46</td>
</tr>
<tr>
<td>Pillar zone</td>
<td>16.88</td>
<td>147.27</td>
<td>73.56</td>
</tr>
<tr>
<td></td>
<td>19.76</td>
<td>147.27</td>
<td>73.56</td>
</tr>
<tr>
<td>Drive zone</td>
<td>13.93</td>
<td>147.27</td>
<td>73.56</td>
</tr>
<tr>
<td></td>
<td>15.43</td>
<td>147.27</td>
<td>73.56</td>
</tr>
</tbody>
</table>

Table 8: Cartesian coordinates for the zones in which principal stresses were tracked.

FLAC3D keeps track of the principal stress values throughout the entire simulation. These values can be plotted on the y-axis in a graph against the calculation steps on the x-axis. For the purposes of this study the calculation steps can be considered to be equivalent to a measure of time.

In order to understand the meaning of the variations in stress levels in these plots, it is necessary to know the exact number of steps at which certain operations occur. These are shown in Table 9. Each of the step numbers refers to the moment at which that operation starts. The subsequent steps are used to achieve numerical equilibrium, after which the next operation begins.

<table>
<thead>
<tr>
<th></th>
<th>Pillar width 1.4 m</th>
<th></th>
<th>2.9 m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No backfill</td>
<td>Backfill</td>
<td>No backfill</td>
</tr>
<tr>
<td>Initial equilibrium</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Excavation of trench</td>
<td>0</td>
<td>0</td>
<td>592</td>
</tr>
<tr>
<td>Excavation Drive 1</td>
<td>43183</td>
<td>43183</td>
<td>34168</td>
</tr>
<tr>
<td>Excavation Drive 4</td>
<td>46820</td>
<td>46820</td>
<td>37152</td>
</tr>
<tr>
<td>Excavation Drive 7</td>
<td>50171</td>
<td>50171</td>
<td>38866</td>
</tr>
<tr>
<td>Excavation Drive 10</td>
<td>53308</td>
<td>53308</td>
<td>40334</td>
</tr>
</tbody>
</table>

111
<table>
<thead>
<tr>
<th>Operation</th>
<th>Pillar width 1.4 m</th>
<th>Pillar width 2.9 m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No backfill</td>
<td>Backfill</td>
</tr>
<tr>
<td>Backfill Drive 1</td>
<td>N/A</td>
<td>57132</td>
</tr>
<tr>
<td>Backfill Drive 4</td>
<td>N/A</td>
<td>57313</td>
</tr>
<tr>
<td>Backfill Drive 7</td>
<td>N/A</td>
<td>57422</td>
</tr>
<tr>
<td>Backfill Drive 10</td>
<td>N/A</td>
<td>57488</td>
</tr>
<tr>
<td>Excavation Drive 2</td>
<td>57132</td>
<td>57560</td>
</tr>
<tr>
<td>Excavation Drive 5</td>
<td>62606</td>
<td>62517</td>
</tr>
<tr>
<td>Excavation Drive 8</td>
<td>69884</td>
<td>69011</td>
</tr>
<tr>
<td>Backfill Drive 2</td>
<td>N/A</td>
<td>76684</td>
</tr>
<tr>
<td>Backfill Drive 5</td>
<td>N/A</td>
<td>77265</td>
</tr>
<tr>
<td>Backfill Drive 8</td>
<td>N/A</td>
<td>78024</td>
</tr>
<tr>
<td>Excavation Drive 3</td>
<td>78815</td>
<td>78765</td>
</tr>
<tr>
<td>Excavation Drive 6</td>
<td>103606</td>
<td>93012</td>
</tr>
<tr>
<td>Excavation Drive 9</td>
<td>139332</td>
<td>109964</td>
</tr>
<tr>
<td>Backfill Drive 3</td>
<td>N/A</td>
<td>126498</td>
</tr>
<tr>
<td>Backfill Drive 6</td>
<td>N/A</td>
<td>127520</td>
</tr>
<tr>
<td>Backfill Drive 9</td>
<td>N/A</td>
<td>128467</td>
</tr>
</tbody>
</table>

Table 9: Number of steps corresponding to each operation in the FLAC simulation.

The fact that certain calculations take longer than others has no meaning and only indicates the complexity of the calculation, rather than any practical application. Initial equilibrium and excavation of the trench take up a large portion of each graph.

Although major and intermediate principal stresses were tracked in addition to the minor principal stress, they will not be discussed here as the minor principal stress is the main pertinent stress component. However, the relevant graphs can be found in Appendix N.

Figure 89 shows the complete minor principal stress plot for the pillar zone at pillar width 2.9 m, with backfill. Operations that cause a noticeable change in the stress situation have been marked on the chart. The excavation of drives 1 and 4 causes small increases in the pillar stress. However, the excavation of drives 2 and 3 impact the stress state more significantly. This is expected, because drives 2 and 3 are the drives that are in direct contact with pillar 2. This simulation also includes backfilling of the drives, but there is no evidence of this on this plot.

All data points before step $3.4 \times 10^4$ are irrelevant, as they represent the process establishing the initial equilibrium and equilibrium after trench excavation. In all the following graphs, therefore, this section of the chart has been left out. All the stress history files were exported to Excel and combined both the backfilled simulation and the non-backfilled simulation into one plot. These plots are shown in Figures 90 - 95.
Figure 89: Minor principal stress history for backfilled simulation at pillar width 2.9 m.

With some minor exceptions, the plots for the backfilled vs non-backfilled simulations look remarkably similar. This suggests that backfilling has very little effect on the stress state within the rock. In some plots, such as the pillar zone at 2.9 m pillar width, shown in Figure 90, various operations occur at different moments in the simulation, which separates the two graphs.

However, since this difference in simulation timing is meaningless in a real-world sense, many of the graphs are even closer together in reality than portrayed. At 2.9 m pillar width, all the plots are virtually the same. The only relevant deviation occurs in the drive zone plot, shown in Figure 91 - the stress in the non-backfilled simulation
remains at 0 towards the end, while the stress in in the backfilled simulation increases slightly at that same point. This can be presumed to be a representation of an empty drive vs. the backfill taking on a small amount of stress.

![Minor principal stress plot for the drive zone, pillar width 2.9 m.](image1)

**Figure 91:** Minor principal stress plot for the drive zone, pillar width 2.9 m.

The roof zone at 2.9 m pillar width, shown in Figure 92, displays an increase in stress as a result of the excavation of drive 1, and a smaller increase as a result of the excavation of drive 4. This is likely due to the fact that drive 4 is located further away from pillar 2 and therefore has a smaller effect on it. The excavation of drives 7 and 10 does not show up on the chart at all.

![Minor principal stress plot for the roof zone, pillar width 2.9 m.](image2)

**Figure 92:** Minor principal stress plot for the roof zone, pillar width 2.9 m.
A sharp drop in stress in Figure 92 is visible at the excavation of drive 2. Again, this is to be expected as the stress is redistributed from the roof towards the pillars as a result of the excavation. Another, much smaller, drop occurs as a result of the excavation of drive 3.

The pillar zone at 2.9 m pillar width, shown in Figure 90, similarly shows small increases in stress for the excavation of drives 1 and 2, and then large increases in stress for the excavation of drives 2 and 3, the drives neighboring pillar 2.

The drive zone at pillar width 2.9 m, shown in Figure 91, shows a very similar scenario to the roof zone, up until the excavation of drive 2. At this point, naturally, the stress in the drive drops to 0 as a result of being excavated. After backfilling the backfill material takes on a small amount of stress.

At pillar width 1.4 m, some larger differences can be detected, particularly in the roof zone, shown in Figure 93. There, after the excavation of drive three, the stress for the backfilled state remains the same while the non-backfilled state results in a drop in stress. For both simulations, the subsequent excavations of drives 6 and 9 result in an increase in stress. Clearly the backfill has caused a minimization of the redistribution of stress as a result of the excavation of drive 3.

![Minor principal stress for roof zone (PW 1.4 m)](image)

**Figure 93:** Minor principal stress plot for the roof zone, pillar width 1.4 m.

This same effect can also be seen in the stress plot for the pillar zone at 1.4 m pillar width, shown in Figure 94: a slight reduction in the stress increase in the backfilled simulation can be observed as a result of the excavation of drive 3.
Figure 95: Minor principal stress plot for the pillar zone, pillar width 1.4 m.

The stress plot for the drive zone at pillar width 1.4 m, shown in Figure 95, is mostly similar to the same plot at pillar width 2.9 m, with the exception that the amount of stress taken on by the backfill material is more than twice as high for the narrower pillar width. This corresponds with the higher stress load in the narrow pillar simulation vs. the wider pillar simulation.

Figure 95: Minor principal stress plot for the drive zone, pillar width 1.4 m.

At no point in any of these stress plots does the stress exceed values of approximately 5.5 MPa. However, in the simulations it was evident that the pillars had failed. Therefore, it seems that the peak unconfined strength of the pillars in this simulation must be about 5.5 MPa. This corresponds with the results from the unconfined virtual triaxial test in section 7.2, which found a similar peak strength. The confined version of that test showed a significant increase in strength as a result of confinement. This explains the high stress zones of 8 MPa and up in the plasticity state plots, all of which occur in confined areas.
7.4 Mining-backfill sequence

In order to examine the effect of mining sequence on the backfilling process, a number of simulations were done in which the sequences varied from one-by-one to four-by-four. All of these simulations were done at a pillar width of 2.9 m, the “ideal” pillar width determined in the previous section. In order to execute these simulations, the FISH file for each of the different sequences had to be slightly altered to adjust the loop in the code for extraction and backfilling.

7.4.1 Failure

The initial plasticity state plot for backfilling at a three-by-three sequence is shown in Figure 96. Shear failure is displayed along the edges of some of the pillars, with failure concentrated around drives 3, 6 and 9. This suggests that failure occurs mostly in the pillars surrounding the drives that were sequentially the last to be excavated. Figures 97 - 99 show the plasticity state plots for one-by-one, two-by-two, and four-by-four sequences, respectively.

![FLAC3D 4.00](image)

**Figure 96: Plasticity state for three-by-three sequence.**

Figure 96 shows the plasticity state plot for the three-by-three sequence. Shear failure zones occur in all the pillars, but are concentrated in the pillars surrounding drives 3, 6, and 9, which are the last drives to be excavated and backfilled. Failure zones are initially concentrated in the corners of the pillars, spreading to the rest of the outer edges as they increase.
Figure 97: Plasticity state for one-by-one sequence.

Failure zones in the one-by-one sequence, as shown in Figure 97, follow the same pattern as the three-by-three sequence, but are significantly diminished in size.

Figure 98: Plasticity state for two-by-two sequence.

The two-by-two sequence shows similar failure zones to the three-by-three sequence, but the zones are distributed more equally among the drives, with larger failure zones concentrated in the pillars surrounding drives that were backfilled during the last pass - drives 2, 4, 6, and 8.
Figure 99: Plasticity state for four-by-four sequence.

The four-by-four sequence shows similar failure zones, but the zones here are concentrated in the pillars surrounding drives 4 and 8, which were the last drives to be excavated and backfilled.

The same trend appears to hold for all the sequences: it seems that in every simulation, the drives that were excavated in the last pass generate a similar amount of failure in the surrounding pillars, regardless of the total amount of passes. Therefore the two-by-two sequence, which has only two passes but excavates 5 drives in each of them, exhibits the largest amount of failure, with shear failure concentrated in the pillars surrounding drives 2, 4, 6, and 8. Proximity to the barrier pillar is the likely cause for the smaller amount of failure in the pillar adjacent to drive 10.

The one-by-one sequence appears to be the most effective at minimizing failure. Shear failure still occurs, increasingly towards the right side of the panel, but the amount of failure is noticeably lower than for the other sequences. The two-by-two sequence is clearly the least desirable scenario. It seems advisable to minimize the amount of extracted drives in each pass - it is best to operate multiple passes with one or two extracted drives in each pass. The improved performance for the one-by-one sequence is likely related to the theory suggested by Clark and Boyd (1998) of such a sequence of mining causing the drive that is being excavated to be positioned inside an abutment zone where it is able to endure higher levels of stress, as described in section 4.4.2.

7.4.2 Displacement
Displacement for all sequences is roughly the same - each with a maximum x-displacement in both directions of about 1.7 mm, and a maximum z-displacement of about 2.3 mm in the downwards direction and about 16 mm in the upwards direction, following the same pattern displayed in the three-by-three sequence in Figures 77 and 78. The displacement plots will not be discussed further here but can be found in Appendix 0.

7.4.3 Stress
The stress plots are virtually the same for all sequences, following the same pattern displayed for the three-by-three sequence in Figure 84, with a maximum minor principal stress of about 6.9 MPa in the pillars and a
minimum minor principal stress of about 1.68 MPa in the drives. There are very minute differences for the one-
by-one sequence and the two-by-two sequence: the one-by-one sequence has a maximum minor principal stress
of 6.8 MPa in the pillars and a minimum minor principal stress of 1.75 MPa in the drives; while the two-by-two
sequence has a minimum minor principal stress of 1.56 MPa. However, these differences are too small to be of
any significance. The principal stress plots can also be found in Appendix 0.

7.4.4 Modified sequences
In addition to the simulations listed above, two more simulations were carried out to determine whether other
factors in sequencing affect the stability. The first factor is whether it makes a difference if each drive is backfilled
directly after mining, or mining and backfilling is done in separate stages, with several drives being mined as a
group before being backfilled. The second factor is whether it is possible to mine two drives directly next to each
other without suffering significant loss of stability.

The first of these simulations was a modified three-by-three sequence, following the same sequence as in the
initial three-by-three setup, but backfilling each drive directly after extraction and before the start of the
extraction of the next drive. This resulted in the plasticity state plot as displayed in Figure 100.

Figure 100: Plasticity state for modified three-by-three sequence.

Comparing this plot to the plasticity state plot for the initial three-by-three sequence (Figure 96), it is clear that
the differences between the two are negligible. Failure zones occur in the same areas and to the same extent in
both scenarios. This would suggest that it makes little difference to the stability whether or not the mines are
backfilled directly after extraction.

The second of these sequences is a modified two-by-two sequence. In this sequence, drives were extracted and
backfilled in pairs, leaving a gap of two drives between each extraction pass. Thus, the sequence was to excavate
drives 1 and 2, backfill drives 1 and 2, excavate drives 5 and 6, backfill drives 5 and 6, excavate drives 9 and 10,
backfill drives 9 and 10, excavate drives 3 and 4, backfill drives 3 and 4, excavate drives 7 and 8, backfill drives 7
and 8. The plasticity state plot resulting from this simulation is shown in Figure 101. It is clear from this plot that
mining two drives next to one another does not necessarily have a destabilizing effect on the pillar separating them. However, the pillar in between two pairs of drives that are excavated last - that is, the pillars in between drives 3 and 4, and 7 and 8, are clearly overloaded to the point where the amount of failure is much more widespread than in any of the other sequence simulations. Thus, when determining mining and backfill sequencing, it is advisable to always leave at least one drive intact or backfilled between two that are being excavated.

![FLAC3D 4.00](image)

**Figure 101:** Plasticity state for modified two-by-two sequence.

In order to illustrate the effect of direct backfilling vs backfilling in stages, this two-by-two sequence was repeated with each drive being backfilled directly after extraction. So the sequence in this new simulation becomes: Extract and backfill drive 1, extract and backfill drive 2, extract and backfill drive 5, extract and backfill drive 6, extract and backfill drive 9, extract and backfill drive 10, extract and backfill drive 3, extract and backfill drive 4, extract and backfill drive 7, extract and backfill drive 8. All other factors are exactly the same. The plasticity state plot for this simulation can be seen in Figure 102.
It is noteworthy that even such a minor difference can have a positive effect on the stability. It can be seen in Figure 102 that there is a clear reduction in shear failure zones. This seems contrary to the results achieved in the three-by-three sequence with direct backfill. Even though the direct backfilling had no beneficial effect there, the opposite is clearly true here. It is possible that this is a result of the fact that the stress on the pillars between the two drives being excavated is more severe than on any pillar in the three-by-three scenario, and therefore these pillars are more sensitive to the beneficial effects of the backfill. Thus, when designing the sequencing of the mine, it is advisable to follow up extraction of a drive immediately with backfilling, particularly if it is necessary to extract two drives adjacent to one another.

7.4.5 Optimum sequence

Based on the observations so far, the one-by-one sequence appears to have the best results with regards to stability. However, this sequence is impractical, as it would require the highwall miner to be idle for long amounts of time while waiting for the previous drive to be backfilled. The ideal sequence appears to be one where spaces between drives remain as large as possible, while not sacrificing productivity of the miner. Therefore the optimum solution would appear to be a panel where two drives are in operation at once: one is being backfilled and one is being excavated. However, the distance between these two drives must be as large as possible. In a 10-drive panel, this is possible with a five-by-five sequence, where there is always a distance of five drives between the two drives in operation. This was simulated using the following sequence: Extract and backfill drive 1, Extract and backfill drive 6, Extract and backfill drive 2, Extract and backfill drive 7, Extract and backfill drive 3, Extract and backfill drive 8, Extract and backfill drive 4, Extract and backfill drive 9, Extract and backfill drive 5, Extract and backfill drive 10. The plasticity state plot for this simulation is shown in Figure 103.
Figure 103: Plasticity state for 5-by-5 sequence.

Figure 103 shows that, as expected, failure zones in this simulation are fewer than in most other sequences. The only sequence with less failure zones is the one-by-one sequence, which has been deemed too impractical. Therefore, this five-by-five sequence appears to be a good candidate for the optimum mine and backfill sequence.
7.5 Backfill material
In order to determine the effect of the backfill material, a simulation was done with radically different properties for the backfill material. A cohesionless material such as loose sand would save a lot of costs in material preparation. Considering the state of the backfill material from India, which itself was almost cohesionless, it is valid to do a simulation using loose sand. Using widely available indicative data regarding the properties of loose sand, the following parameters were chosen:

- Friction angle ($\phi$) 35°
- Young’s modulus ($E$) 20 MPa
- Poisson ratio ($\nu$) 0.3
- Cohesion ($c$) 0 MPa

7.5.1 Failure
The plasticity state plot for the simulation using loose sand as backfill material is shown in Figure 104.

Comparing this plot to the plasticity state plot for the simulation using FCM as backfill material (Figure 96) shows an obvious difference. Unsurprisingly, the backfill material itself is failing - this is to be expected, as its strength is very low. However, the surrounding pillars also show a significant increase in failure compared to Figure 96. Failure in the backfilled drives occurs immediately after backfilling. By contrast, the pillars themselves don't start failing until the last mining pass, as shown in Figure 105.
7.5.2 Displacement

Displacement in the x-direction, shown in Figure 106, follows the same distribution pattern as the simulation with FCM-material as backfill (Figure 77). However, the magnitude of the displacement is slightly increased for the loose sand. The maximum displacement towards the left is 1.88 mm while the maximum displacement towards the right is 1.94 mm. Compared to a maximum displacement of 1.65 mm for FCM-material, this results in an increase in horizontal displacement by approximately 15% compared to the FCM-backfilled state. However, compared to the non-backfilled state, the reduction in displacement is merely 9%.

Figure 106: X-displacement in the pillars, after backfill (backfill material loose sand).

Figure 107 shows the vertical displacement for the backfilling with loose sand scenario.
The difference in vertical displacement shown is more significant than the horizontal displacement. A maximum downwards displacement of 9.2 mm is a 400% increase. However, the minimum downwards displacement in the pillars is also lower: 2 mm for the loose sand versus 1 mm for the FCM material. Overall there is a much steeper displacement gradient within the loose sand: it seems like it is being packed down, with material from the top of the drive settling towards the bottom. Upwards displacement of the floor is also slightly decreased: from 1.6 to about 1.5 - a reduction of about 5%. This is possibly due to the increased downwards movement in the backfilled drives, which itself is likely a result of the lack of cohesion in the material.

7.5.3 Stress
The stress state plot for the loose sand simulation, shown in Figure 108, is very similar to the stress state plot for the FGM material simulation (Figure 82). The major difference is the much lower stress in the backfilled drives, which is caused by the fact that the loose sand is inherently much weaker than the FGM material. The remaining stress components are mostly the same. However, since most of the pillars have failed in this simulation, the pillars have clearly lost some of the strength they had in the FGD simulation.
7.5.4 Effect of stiffness

As discussed in section 4.3, Donovan and Karfakis (2001) suggested that the backfill material must be of equal or greater stiffness than the coal. This is not the case with the FCM backfill material that is used in these simulations: the Young’s modulus used for coal is 3.47 GPa, while the Young’s modulus used for the backfill material is merely 0.32 GPa. For this reason, an additional simulation was done with the level of stiffness for the backfill material increased to 3.5 GPa. This resulted in the plasticity state plot seen in Figure 109.
As with Figure 96, in the plot for the simulation with the lower modulus of elasticity, failure is concentrated mostly around drives 3, 6, and 9, which are the last drives to be excavated, but the prevalence of the failure zones is significantly reduced. The reduction is such that it is likely that the pillars will be stable even at a smaller pillar size. This is an important result, because it shows that an increase in stiffness of the backfill material could allow for a further decrease in pillar size, and therefore an even greater increase in recovery.

Figures 110 and 111 show the x-displacement and z-displacement, respectively. These plots show the same patterns as in Figures 77 and 78, but the magnitude of displacement has changed slightly. The maximum overall displacement in the pillars in a horizontal direction has increased by a small amount, about 4.3%, while the compression in the vertical direction has been slightly reduced by about 3%.

![Contour Of X-Displacement]

Figure 110: X-displacement for backfill material with increased stiffness, after backfill.
Figure 111: Z-displacement for backfill material with increased stiffness, after backfill.

The stress contour plot can be seen in Figure 112. This figure shows a significant reduction in stresses compared to Figure 82. There is a similar pattern of stress distribution, but the zones of maximum stress in the pillars are closer to 6 MPa than the 6.9 MPa seen in Figure 82. The increased stiffness of the backfill material appears to have the effect of distributing the stress more evenly and relieving the pillars of some stress, thus making the pillars less susceptible to failure.

Figure 112: Contour of minor principal stress for backfill material with increased stiffness, after backfill.
7.6 Partial backfill

The reasons to examine the possibility of partial backfill are twofold: first, it would be financially beneficial to backfill a portion of the drive. It could be possible that filling only part of the drive will supply the pillars with sufficient confinement to increase their strength. The second reason is more practical: in reality it will be nearly impossible to completely backfill the entire drive from top to bottom. For a large part of the drive, the top of the backfill material will not be in contact with the roof of the drive. Therefore, it is useful to understand how this impacts the backfilling process and its efficiency.

Two modes of partial backfill were examined:

- partial backfill in the vertical direction
- partial backfill in the horizontal direction.

It should be noted that, due to the geometrical constraints of the FLAC model, neither of these models are entirely realistic: the model simulates the surface of the backfill material as being parallel to the dip of the seam, whereas in reality the surface of a fluid backfill material would be perfectly horizontal. However, the following simulations still serve to give a general indication of the effectiveness of partial backfilling.

7.6.1 Partial backfill in the vertical direction

Backfill was simulated as being half the drive height throughout the length of the drive. Additional parameters were kept the same.

7.6.1.1 Failure

Initial simulations were carried out at half the total fill height. The plasticity state plot is shown in Figure 113.

![FLAC3D 4.00](image)

**Figure 113:** Plasticity state plot after backfilling halfway in the vertical direction.
This figure shows a large amount of failure in the pillars. However, while the amount of failure is higher than the amount of failure in the fully backfilled drives (Figure 96), it is also lower than the amount of failure in the non-backfilled drives (Figure 65). Clearly the partial backfill does have some benefit for the stability of the pillars.

7.6.1.2 Displacement

Figures 114 and 115 show the horizontal and vertical displacement for the half-backfilled simulations, respectively.

**Figure 114:** X-displacement in the pillars, after backfilling halfway in the vertical direction.

**Figure 115:** Z-displacement in the pillars, after backfilling halfway in the vertical direction.
Horizontal displacement follows a similar pattern to that displayed in the full-backfilled simulation (Figure 77), but the magnitude is slightly increased: from a maximum in either direction of about 1.65 to 1.77 is an increase of approximately 7%. However, the displacement is still about 15% less than if there was no backfill at all (Figure 75).

Vertical displacement is also mostly similar in magnitude to the fully backfilled state (Figure 78). However, the magnitude of downwards displacement of backfill material is about 25% of that at full backfill height. This is to be expected as the total weight of the backfill is now much lower.

7.6.1.3 Stress
The contour of minor principal stress for the partial backfilling in the vertical direction is shown in Figure 116.

![FLAC3D 4.00](image)

**Figure 116: Contour of minor principal stress, after backfilling halfway in the vertical direction.**

The distribution of the stress in the pillars shown here is fairly similar to the distribution displayed for the full backfill scenario (Figure 82). However, in terms of stress magnitude, it resembles the non-backfill scenario (Figure 81) more. There is a significant increase in stress in the pillars compared to the full-backfill scenario, and the plasticity state plot indicates a significant decrease in shear failure compared to the non-backfill scenario. Therefore, it must be concluded that, in the vertical direction at least, even partial backfill imparts some strength to the web pillars.

7.6.2 Partial backfill in the horizontal direction
For partial backfill in the horizontal direction, the backfill material was simulated as filling only the back half of the drive, with the backfill depth being half the total drive depth. This leaves the front half of the drives entirely empty.

7.6.2.1 Failure
Figure 117 shows the plasticity state plot of the simulation at the usual 150 m into the model (100 m into the drive).
Figure 117: Plasticity state plot after backfilling halfway in the horizontal direction ($y = 150\,\text{m}$).

Because the backfill only starts at 125 m into the drive, the drives at this point are still empty. For this reason, an additional plot is displayed in Figure 118 at a cutting plane 200 m into the model (150 m into the drive). Both of these plots show a significant increase in failure compared to the fully-backfilled plasticity state plot (Figure 96). Moreover, they show very little, if any, decrease in failure compared to the non-backfilled plasticity state plot (Figure 67). It would seem, therefore, that partial backfilling in the horizontal direction does not provide much advantage compared to not backfilling at all.

Figure 118: Plasticity state plot after backfilling halfway in the horizontal direction ($y = 200\,\text{m}$).
7.6.2.2 Displacement

Figures 119 and 120 show the x-displacement in the model at a cutting plane at 200 m and 150 m in the y direction, respectively.

**Figure 119:** X-displacement in the pillars, after filling halfway in the horizontal direction (y = 200 m).

**Figure 120:** X-displacement in the pillars, after filling halfway in the horizontal direction (y = 150 m).

Comparing the 200 m plot to the corresponding plots for the non-backfilled and completely backfilled scenarios (Figures 75 and 77 respectively), it can be seen that not only is there an increase in displacement compared to the completely backfilled situation, as would be expected, but the amount of displacement is actually slightly higher than the non-backfilled scenario. However, this could be due to the fact that the cutting planes were taken...
50 m apart. The cutting plane at 150 m shows an increase in displacement compared to the fully backfilled state, but a very slight decrease in displacement compared to the non-backfilled state. This corresponds with the expectations. However, the decrease in displacement is so minimal (approximately 5% in either direction), that it would likely not be worth the trouble of backfilling at all.

The Z-displacement plots are shown in Figures 121 and 122.

![Figure 121: Z-displacement in the pillars, after filling halfway in the horizontal direction (y = 200 m).](image1)

![Figure 122: Z-displacement in the pillars, after filling halfway in the horizontal direction (y = 150 m).](image2)
These plots show similar results. Upwards displacement in the drive floors is significantly decreased for the 200 m cutting plane compared to the fully backfilled state (Figure 78), but this is again likely due to the 50 m difference between the cutting planes: due to the incline, the drive floor towards the back of the drive will experience upwards movement to a lesser degree than the floor towards the front of the drive. The plot at cutting plane 150 m shows that there is barely any difference in displacement compared to the non-backfilled state (Figure 76).

7.6.2.3 Stress
The stress state plots for cutting planes at 200 m and 150 m are shown in Figures 123 and 124, respectively. The plot at 150 m is virtually the same as the plot for the non-backfilled state in Figure 81. The plot incorporating the backfill at 200 m is mostly similar to the fully backfilled state in Figure 82; however, the magnitude of the minimum principal stress is increased by approximately 11%. This, again, is likely due to the 50 m distance between the two plots, with the overburden depth increasing along with the horizontal mining depth.

Figure 123: Contour of minor principal stress after backfilling halfway in the horizontal direction ($y = 200$ m).
From the stress plots as well as the plasticity state plots and the displacement plots it can be concluded that partial backfilling in the horizontal direction is not a desirable method of backfilling.

7.7 Multi-lift mining

The final set of simulations focused on the possibility of multi-lift mining. For these simulation, all parameters were kept equal, except for the drive height: the total seam height of 4.5 m was divided into one section of 3 m thick and one section of 1.5 m thick. Two scenarios were simulated: multi-lift mining in an upwards direction and multi-lift mining in a downwards direction. The first lift excavates and backfills the seam up to 3 m high; the remaining 1.5 m is done during the second lift.

7.7.1 Failure

Figures 125 and 126 show the plasticity state plots after the first pass and second lift, respectively. Figure 125 shows no signs of failure, which is unsurprising as the initial width to height ratio of the pillars is now much higher due to the relative shortening of the drive height.
Figure 125: Plasticity state plot for multi-lift simulation, after first lift, upward.

Figure 126 shows clear signs of failure, but the pattern is very different from the pattern in the single-pass simulation (Figure 68). Like the single-pass simulation, failure here is focused around the drives that were the last to be excavated. However, failure is concentrated around the top portion of the pillars. The drive walls are subject to less failure, but the drive roofs appear to have become less stable. The magnitude of the failure zones is comparable between the two scenarios.

Figure 126: Plasticity state plot for multi-lift simulation, after second lift, upward.
7.7.2 Displacement

Figures 127 - 130 show the horizontal and vertical displacement for the first and second pass scenarios.

**Figure 127:** X-displacement in the pillars, after backfilling first lift, upward.

**Figure 128:** X-displacement in the pillars, after backfilling second lift, upward.
Comparing these to the displacement plots for the single pass scenario without backfill (Figures 75 and 76), both first lift and second lift scenarios show a decrease in horizontal displacement, as expected. The maximum upwards displacement is slightly increased in the first pass scenario compared to the non-backfill scenario, but is comparable to the non-backfill scenario after the second pass has been completed. The reduction in maximum upwards displacement that occurs during the second pass is likely a result of increased downwards pressure exerted by the overlying backfill material.
Comparing Figures 127 - 130 with the single-lift backfilled plots (Figures 77 and 78), the multiple-lift plots show a decrease in maximum x-displacement compared to the single-lift backfilled plots, both after the first lift and after the second lift. The ultimate reduction in displacement, however, is small: between 7 and 10% in either direction. Maximum z-displacement is decreased in the downwards direction for the multiple-lift backfilled scenario compared to the single-lift backfilled scenario. However, in the upwards direction, displacement is slightly increased after the first lift, and at a comparable level to the single-lift scenario after the second pass. Obviously the smaller initial drive height results in an initial increase in upwards displacement. However, after backfilling has taken place, the weight of the overlying backfill reduces the upwards displacement again.

The maximum downwards displacement in the backfill material has significantly decreased after the second lift compared to the single pass scenario, and even compared to the first lift situation in the multiple pass scenario. The total reduction in upwards displacement in the backfill material after the second pass compared to the single pass scenario is approximately 66%. However, in both cases the displacement is a matter of (fractions of) millimeters, and even a reduction as significant as 66% is therefore not necessarily of interest. Considering the results from Figure 126, it seems likely that the reduction in downwards displacement may be linked to the failure of the roof rock.

7.7.3 Stress

Figures 131 and 132 show the contour of the minor principal stress after backfilling the first and second lift, respectively. Comparing Figure 131 to its counterpart for the single-pass scenario, Figure 82, the magnitude of stress is more or less the same, as is the stress distribution throughout the panel.

![FLAC3D 4.00](image)

**Figure 131:** Contour of minor principal stress after backfilling first lift, upward.

After the second lift, however, it can be seen in Figure 132 that there is a slight increase in minimum principal stress in the pillars of approximately 4%. However, the stress upon the backfill material has decreased significantly almost by a factor 5. This is possibly a result of the failure in the upper part of the backfilled drives, as seen in Figure 126.
Figure 132: Contour of minor principal stress after backfilling second lift, upward.

7.7.4 Multi-lift mining in the downward direction

Multi-lift mining in the upward direction has the disadvantage of requiring the highwall miner to be positioned on top of the backfill material, creating additional requirements for the backfill material. Moreover, there is a risk of the highwall miner extracting backfill material from the drive floor; this can be damaging to the equipment. Additionally, in some cases the local circumstances may simply not allow for upward mining. For this reason, an additional simulation was carried out for multi-lift mining in the downward direction. This simulation first extracts the upper 1.5 meters of coal, then the lower 3 meters, followed by backfilling the entire drive. Because it is not possible to backfill the upper 1.5 meters before extracting the lower 3 meters, it is possible that this will cause additional instability. As with the previous simulation, this simulation follows the three-by-three sequence where two drives are skipped between each drive being excavated and backfilled in every pass.

Figure 133 shows the plasticity state plots for this simulation.
This simulation clearly shows failure in all of the pillars. The extraction of the top part of the ten drives, which cannot be backfilled before the lower part is extracted, has clearly negatively affected the stability of the pillars to the extent that they are no longer stable. For this reason, multi-lift mining and backfilling in the downward direction cannot be considered advisable.

The displacement and stress plots for this simulation can be found in Appendix P.
7.8 Overview of results

For a mining height of 4.5 m, as is the case at West Bokaro, a pillar width of 2.9 m offers the most obvious advantage between a backfilled and a non-backfilled state. Although failure still occurs for the backfilled state, the magnitude is much decreased for the non-backfilled state. A larger pillar width results in stable pillars, even without backfill and therefore backfill may not seem advantageous at a larger width. However, this does not take into account the factor of safety. For the non-backfilled state, all the pillars will be exposed and unsupported. However, the backfilled pillars will experience support from the confinement that is offered by the surrounding backfill material. Although the coal pillars themselves are 2.9m wide, the surrounding backfill material means that no 2.9m of coal pillar is ever exposed on both sides during a backfill scenario.

The simulations for mining sequence clearly favored mining sequences with many passes. Rather than spreading out the excavated drives along the panel before backfilling, it is advisable to backfill immediately after mining a drive, before moving on to the adjacent drive. In this way, stress on individual pillars is minimized, as no pillar is ever exposed on both sides at the same time, and there is at all times only one drive that is empty, so that the stress is focused on that single spot rather than spread throughout the panel.

The most important result from the backfill material simulations is that a backfill material that offers merely confinement but lacks any cohesion is inadequate. The lack of cohesion and strength in the backfill material results in failure not only of the backfill material itself, but within failure of the pillars, as well. The slight decrease in displacement compared to not backfilling at all is not enough to justify using a cohesionless material such as loose sand to backfill the drives.

Partial backfilling is not a desirable option. Although there is a slight increase in strength in the pillars as a result of partial backfilling in the vertical direction, it is insufficient to make a real difference in comparison to the non-backfilled state. However, the fact that partial backfilling to 50% of the drive height was still somewhat effective, would suggest that a small gap at the top of the fill, which is likely to be unavoidable during backfilling, will not unduly compromise the efficacy of the backfill. Partial backfilling in the horizontal direction, on the other hand, does not seem to contribute any support at all.

Multiple-pass mining certainly seems to be a viable possibility in order to reduce failure in the pillars. However, when the seam is thin enough to mine in one pass, it is probable that the additional expense of mining the seam in two passes would be excessive. After all, the overall reduction in displacement is still relatively low.

Table 10 shows an overview of the simulation results and their relative changes compared to the initial simulation. Although most of the results are fairly mixed - reductions in some places and increases in others - there are two standouts as far as performance is concerned. First, it is abundantly clear that the loose sand performs very poorly as a backfill material, causing quite a significant increase in displacement. On the other hand, double pass mining clearly works well to reduce displacement. The slight increase in stress in the pillars is also a good sign, because it suggests an increase in strength. Furthermore the differences between the various mine-to-backfill sequences are mostly minimal.
<table>
<thead>
<tr>
<th></th>
<th>Max. x-displacement (pos) (mm)</th>
<th>Min. x-displacement (mm)</th>
<th>Maximum horizontal extension (mm)</th>
<th>Max. z-displacement (mm)</th>
<th>Min. z-displacement (mm)</th>
<th>Maximum vertical compression (mm)</th>
<th>Min. minor principal stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PW 2.9 m, 3 by 3, FCM, full backfill, single pass (= standard)</td>
<td>1.67</td>
<td>-1.65</td>
<td>3.32</td>
<td>15.99</td>
<td>-2.28</td>
<td>18.27</td>
<td>-6.87</td>
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<td>15.56</td>
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<td>N/A</td>
<td>+8.6%</td>
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<tr>
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<td>-1.68</td>
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<td>-1.59</td>
<td>17.99</td>
<td>-6.78</td>
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<td>-0.5%</td>
<td>+2.6%</td>
<td>-30.1%</td>
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</tr>
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<td>2 by 2 sequence</td>
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<td>-1.66</td>
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<td>15.88</td>
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<td>18.14</td>
<td>-6.88</td>
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<td></td>
<td>Comparison to standard</td>
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<td>+5.1%</td>
<td>+2.3%</td>
<td>+0%</td>
<td>+0.9%</td>
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<td>18.3</td>
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<td>+2.1%</td>
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<td>+3.9%</td>
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<tr>
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<td>-2.29</td>
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<td>-6.84</td>
</tr>
<tr>
<td></td>
<td>Comparison to standard</td>
<td>+2.5%</td>
<td>+3.0%</td>
<td>+2.7%</td>
<td>-0.1%</td>
<td>+0.7%</td>
<td>+0%</td>
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<td>18.33</td>
<td>-6.83</td>
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<td>+9.3%</td>
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<td>-1.7%</td>
<td>+0.4%</td>
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<td>Loose sand BF</td>
<td>1.94</td>
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<td>15.14</td>
<td>-9.24</td>
<td>24.38</td>
<td>-6.98</td>
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<td></td>
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<td>+17%</td>
<td>+13.9%</td>
<td>+15.3%</td>
<td>+5.3%</td>
<td>+305%</td>
<td>+33.4%</td>
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<td>Increased stiffness</td>
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<td>-1.71</td>
<td>3.46</td>
<td>16.56</td>
<td>-1.13</td>
<td>17.69</td>
<td>-6.51</td>
</tr>
<tr>
<td></td>
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<td>+3.7%</td>
<td>+4.3%</td>
<td>+3.5%</td>
<td>+50.3%</td>
<td>-3.2%</td>
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<td>1.78</td>
<td>-1.77</td>
<td>3.55</td>
<td>15.70</td>
<td>-0.54</td>
<td>16.24</td>
<td>-7.49</td>
</tr>
<tr>
<td></td>
<td>Comparison to standard</td>
<td>+6.9%</td>
<td>+7.1%</td>
<td>+7.02%</td>
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<td>Partial backfill (horizontal)</td>
<td>2.01</td>
<td>-1.92</td>
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<td>+16.1%</td>
<td>+18.4%</td>
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<td>Double-pass (upward)</td>
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<td>-1.54</td>
<td>3.03</td>
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<td>-0.77</td>
<td>16.40</td>
<td>-7.20</td>
</tr>
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<td></td>
<td>Comparison to standard</td>
<td>-10.3%</td>
<td>-6.8%</td>
<td>-8.6%</td>
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<td>-66.3%</td>
<td>-10.3%</td>
</tr>
</tbody>
</table>
### Table 10: Overview of displacement and stress results compared to initial simulations at pillar width 2.9 m, 3 by 3 sequence, full backfill, single pass.

<table>
<thead>
<tr>
<th></th>
<th>Max. x-displacement (pos) (mm)</th>
<th>Min. x-displacement (mm)</th>
<th>Maximum horizontal extension (mm)</th>
<th>Max. z-displacement (mm)</th>
<th>Min. z-displacement (mm)</th>
<th>Maximum vertical compression (mm)</th>
<th>Min. minor principal stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Double-pass (downward)</strong></td>
<td>1.85</td>
<td>-1.87</td>
<td>3.72</td>
<td>15.33</td>
<td>-1.96</td>
<td>17.29</td>
<td>-7.12</td>
</tr>
<tr>
<td><strong>Comparison to standard</strong></td>
<td><strong>+10.8%</strong></td>
<td><strong>+13.3%</strong></td>
<td><strong>+12%</strong></td>
<td><strong>-4.1%</strong></td>
<td><strong>-14%</strong></td>
<td><strong>-5.4%</strong></td>
<td><strong>+3.6%</strong></td>
</tr>
</tbody>
</table>

Some aspects of the model do not make much sense. For instance, the lack of significant displacement, even in the case of severe failure, is odd. Even at its worst, the displacement in the model amounts to no more than a few centimeters. If this were truly a representation of reality, most of the results of this thesis would be moot because the pillars stay in place even despite massive failure. However, the lack of significant displacement should not be considered a reason to disregard all the results. The model is flawed because it relies on a large amount of assumptions, many of which are probably not entirely correct. However, even if the data are not fully accurate, the general trends displayed by the model - such as the point at which a pillar fails, or when an increase in displacement occurs, however small - can still serve as meaningful predictors of the true behavior of the rock mass.
8. Discussion

This section will discuss the results of the simulations and attempt to answer the research questions defined at the beginning of this thesis:

1. What are the most important factors affecting the efficiency of backfilling?
2. What effect do the various factors have on the efficiency of backfilling?
3. What method should be used to implement backfill technology?
4. Is West Bokaro a suitable location for the implementation of backfill technology?

8.1 Factors affecting backfill efficiency

Pillar width, mine and backfill sequencing, backfill material, partial backfilling and multi-lift mining are the five factors that have been discussed in this thesis. The choice for these factors was based on existing literature and the simulations attempted to expand on the information that was already available.

8.1.1 Pillar width

The simulation results indicate that variation in pillar width has the most pronounced effect on the total stability of the mine site. Calculations using the Mark-Bieniawski equation result in a pillar width of 3.9 m for a factor of safety of 1, while the Sheorey method advocates a pillar width of 4.5 m. In a non-backfill scenario, the FLAC simulations suggest that the pillar width could be as low as 3.4 m, which gives a factor of safety of 0.9. However, with backfilling, the simulation results suggest that pillar width could be reduced to 2.9 m, which gives a factor of safety of 0.8.

This means that the theoretical increase in recovery could be 26% compared to a non-backfill scenario using the Mark-Bieniawski equation to calculate pillar width, or up to 36% compared to a non-backfill scenario using the Sheorey equation. However, such reductions in pillar size would also require a reduction in the factor of safety to about 0.8. Thus, the practical possibility of actually applying this technique will be dependent on the local regulations with regards to pillar width. If a requirement exists for a minimum factor of safety of 1 - thus a minimum pillar width of 3.9 - then backfilling is unlikely to have any beneficial effect beyond an increase in stability, as no increase in recovery can be achieved through reduction of the pillar size. Moreover, there are often regulations in place which require the height to width ratio of the pillars to be 1 at minimum. In this case, the pillar width prescribed by regulations would be even larger - 4.5 m minimum - and backfilling would be even less effective.

Hume and Searle (1998) suggest the possibility of 100% recovery by replacing all of the coal with backfill. This does not seem advisable, as the main function of the backfill has shown to be increasing pillar strength through confinement, rather than through supporting the weight of the overburden. Moreover, it is unlikely that such a technique would be legally permitted.

Although the pillars clearly show signs of being crushed and swelling in the center, the lack of significant displacement even in cases of severe failure is puzzling. It is not clear whether this is the result of faulty input data or whether this is actually representative for the rock mass. Considering the high compressive strength and stiffness of the coal, it is possible that the pillars at West Bokaro are simply capable of resisting much higher stresses than anticipated. As described in Section 4.3, Donovan and Karfakis (2004) advocate deliberately allowing the pillars to fail, thereby activating the confining potential of the backfill. The lack of significant
displacement suggest that this could be one of the mechanisms that causes the rock mass to remain stable despite obvious failure of the pillars. However, there are too many uncertainties in the current model to say anything definitive about this.

The four remaining factors - sequence, backfill material, partial backfilling and multi-lift mining - do not have a direct effect on the recovery, but they can improve pillar stability which should allow for a minimal pillar width and thus a maximized recovery.

8.1.2 Sequencing of mining and backfill
For the mining and backfilling sequence, many more combinations are possible than demonstrated in this thesis. Of the sequences simulated, a one-by-one sequence appeared to be the most desirable. This is likely due to the fact that, in this sequence, at any given time only one drive was in an excavated, non-backfilled state. This indicates that having multiple drives open at a time will have a negative effect on pillar stability, and backfilling should occur directly after mining to ensure optimum stability. Moreover, the excavation of the next drive should wait until backfilling of the previous drive has been completed. However, this is not a practical solution, as that would mean that the highwall miner would be idle for long amounts of time. Thus, a compromise should be created that allows the miner to continue, but leaves a maximum amount of space between drives that are being excavated and drives that are being backfilled. One solution could be to switch the extraction and backfilling processes between two separate panels, such that at any given moment only one of these operations is occurring in a single panel. Another solution could be to keep both processes in the same panel, but maximize the space between them. Thus, in a 10-drive panel, the five-by-five sequence could be a good intermediate solution between stability and efficiency.

8.1.3 Backfill material
The backfill material is of prime importance. The simulations have shown that adequate cohesion and stiffness are both imperative to the efficiency of a backfill material. Based on the test results and the simulations, the material supplied as the intended backfill material to be used at West Bokaro appears to be wholly unsuitable for the task. The lack of cohesion, stiffness and strength have all been shown to be detrimental to the material’s efficiency in backfilling and it seems unlikely that this material, which pulverized immediately after testing, should hold up as support in an excavated drive. Problems are likely to be exacerbated by the presence of water, as during the testing process the material desintegrated upon contact with water. However, another type of fly ash composite material would likely be more suitable for use as a backfill material. FCM materials have been successfully used in backfilling in underground mines and the simulations show that a FCM material with adequate strength and cohesion can offer significant increases in stability.

8.1.4 Partial backfilling
Although partial backfilling in the vertical direction has been shown to have a small positive effect on the stability of the pillars, it is not recommended as the benefits are minimal when compared to the costs. Deliberate partial backfilling would be fraught with difficulties - partial backfilling in a horizontal sense would be possible only in downward-dipping seams, but the effects are negligible, while partial backfilling in the horizontal direction would be possible only for perfectly flat seams. However, when backfilling a drive, it may be unavoidable that the fill will not reach all the way to the roof. This is unlikely to significantly compromise the efficiency of the backfill. These findings are in accordance with those of Cai (1983), as discussed in Section 4.3: the backfill itself does not support the overburden or bear any vertical load until deformation of the roof has occurred. The fill cannot rigidly support the total weight of overburden and acts only as a secondary support system.
8.1.5 Multiple-lift mining
Mining in multiple lifts is a plausible possibility, and results in an increase in stability and a decrease in displacement. However, the risk of roof failure is greatly increased compared to the single-pass scenario, and care must be taken to prevent trapped equipment or roof collapse. Moreover, the model does not take into account the additional weight of the highwall miner on top of the backfill material. If multiple-pass mining is considered, extra attention must be paid to the backfill material, which must be strong enough to support the load of the highwall miner on top of it. Moreover, there is the risk of damaging the highwall miner if it starts extracting backfill material. From a practical standpoint, multiple lift mining while incorporating backfill would make more sense if mining occurs in a downwards direction, starting with the upper layer and ending with the lower. This would require the construction of a bench for the highwall miner to stand on, which is subsequently removed when it has to extract the lower layer. However, this would mean that backfilling could only take place once the entire drive has been extracted, and simulations have shown that the stability of the pillars is excessively compromised by leaving the top layers of the drives open while continuing with the excavation of the bottom layers.

8.2 Backfill methods
The method of delivery is a key issue. Although hydraulic delivery seems like the most obvious choice, there are other options. Depending on the properties of the overburden, fluid placement through boreholes drilled from the top of the drive is a possibility, and would make it easier to install a bulkhead to seal off the drive before backfilling it. However, in accordance with the findings of Hume and Searle (1998) as discussed in Section 4.3.3.1.4, the most cost-efficient and effective method of delivery would probably be to somehow alter the highwall miner itself in such a way that it can place the backfill on retreat. One possibility would be to use the augers in reverse to place the backfill material. However, that would only be suitable for non-fluid backfill materials, and as the simulations have shown, dry backfill materials which lack cohesion are not suitable for use as backfill material in a highwall mining scenario. For fluid cement-type material, it would theoretically be possible to install a hydraulic delivery system on the highwall miner. The use of such a method would be limited to sites with downwards-dipping seams. However, this would require a significant overhaul of the highwall miner and is not currently a viable option.

Thus, the most efficient methods of delivery are currently through hydraulic means. Local geography will be key to determining the exact method of delivery: through boreholes drilled into the overburden, or through pipelines installed throughout the drives. As a result of a decrease of the costs associated with it, paste backfill could also be a viable option.

8.3 Suitability of West Bokaro mine for application of backfill technology
The situation at West Bokaro offers an intriguing possibility for the implementation of backfill in highwall mining. The compressive strength measured in the samples that were collected on-site was very high, and although the samples were few, the high strength of the coal corresponds to the data that is known about coal from that region. This high coal strength, along with coal seams that are clearly defined and have a slight dip in a downwards direction combine to create favorable conditions for both highwall mining and backfilling. Backfill material can be brought in at low cost which is imperative from a feasibility standpoint.

The simulations show that a significant increase in recovery can occur as a result of backfilling. However, this requires significant shrinking of the pillar size. As the usual regulations stipulate that the factor of safety must be 1 at minimum, it is not clear whether such a reduction can be made. This is a problem that will not be unique to West Bokaro: regulations worldwide with regards to the factor of safety are based on the dimensions of free-
standing pillars. In the case of backfill, the pillars will never be free-standing, but rather will always be surrounded by backfill material on at least one side. Thus it may be necessary to define a new way of formulating a factor of safety that takes into account the additional support provided to the pillars by the backfill material.

The proposed type of backfill at West Bokaro, as well as the delivery system (hydraulic through pipeline) has already been chosen. However, it would be worthwhile to reconsider these options. The backfill material supplied to the author appeared in every aspect to be unsuitable for use as such. The simulations have shown that cohesion, stiffness and strength of the backfill material, which was severely lacking in the samples supplied, is an essential contributor to the increased stability it offers. Moreover, the material lacks the strength to offer adequate support through confinement. An additional problem is the long drying time that this backfill material requires. Mining in the drives adjacent to the backfilled drives cannot commence until the backfill material has fully set. This would mean that a 28-day waiting period would have to be implemented between each pass. Other methods of delivery, such as hydraulic delivery through boreholes drilled through the overburden, could possibly eliminate the need for backfill material with such a slow drying speed.

In short, West Bokaro as a mine site appears to be eminently suitable for the implementation of backfilling in highwall mining. However, the backfilling material that is currently chosen does not.

**8.4 Feasibility**

Although it has been shown that backfill can be an effective tool to increase recovery in highwall mining, the question remains whether it is also feasible. This will depend largely on the the cost of fill required to produce one extra ton of coal, as described in Section 4.5. Due to the fact that the data required to investigate this was not available, little can be said on this matter in the present study. However, much will depend on the percentage by which the recovery can be increased. Based on the simulations completed in this thesis, that would not appear to be much. The use of backfill allows for the reduction of the pillar size by only 0.5 m from 3.4 m to 2.9 m, which results in an increase of recovery of just 15%. This is unlikely to be enough to overcome the expense of backfilling. However, comparing the simulation results to the empirical formulae currently employed to calculate pillar strength, the increase in recovery can theoretically be up to 36%. Such a scenario would be much more likely to be economically attractive.
9. Conclusion

The main research question of this thesis was:

“Can backfilling be an effective tool to increase recovery in highwall mining?”

It is clear from the simulations that backfilling can be effective as a method of increasing recovery for highwall mining. However, the probability of success will depend on many factors, which should all be given due consideration. Between the simulations done here, the difference between the pillar width at which backfilling is effective, and the pillar width at which backfilling is not necessary, is quite small - the simulations showed an improvement in pillar stability as a result of backfill, but the total gain in terms of pillar width was no more than 0.5 m. However, when compared to the theoretical methods currently used to calculate pillar stability, an increase in recovery of up to 36% could be achieved. Whether it is realistically possible to implement such a decrease in pillar width, however, is entirely dependent on local regulations concerning factor of safety. If such regulations cannot be altered, backfill could still be used as a means to improve stability, but will have little effect in the way of increasing recovery.

• The backfill strengthens the pillars mostly through confinement, but it also takes on a small amount of load of its own. For this reason it is important that the backfill is of adequate stiffness, strength, and cohesion to fulfill such a purpose. The simulations show that stability of the pillars is hindered when the backfill material is cohesionless. Moreover, the simulations have shown that an increase in stiffness of the backfill material is directly proportional to pillar stability and can, by itself, lead to a smaller required pillar width and increase in recovery.

• Although the simulations showed that a one-by-one mining-to-backfilling sequence is the most effective from a geomechanical point of view, this will likely not be practical. For the sake of efficiency, a sequence that excavates more drives in a single pass would be more suitable. However, the increased risk of pillar instability should be taken into account. Backfilling each drive immediately after excavation will ensure the highest stability. Moreover, it is advisable to always leave at least one drive intact or backfilled between two drives that are being excavated.

• Deliberate partial backfilling is not recommended in either direction. While partial backfilling in the vertical direction offers a small amount of support, partial backfill in the horizontal offers no advantages at all at all. However, a small gap at the top of the drive is unlikely to be severely problematic, as the simulations showed that most of the support offered by the backfill is imparted upon the pillars through the sides, rather than via the roof of the drive.

• Multi-pass backfilling can be beneficial to overall stability, but there is an increased risk of instability in the roof that should be taken into account. Moreover, the ability of the backfill material to support the weight of the highwall miner on top of it should be ascertained beforehand. The strength requirements for this purpose are likely to be much more stringent - and the backfill material more costly - than it would be for regular backfill applications.
The West Bokaro mine possesses many qualities that make it favorable as a site to implement backfilling. However, **the chosen backfill material does not appear to be suitable for that purpose.** It would be advisable to reconsider the choice of material.

The crux of the findings of this thesis is that backfilling can be used as a tool to increase recovery in highwall mining. However, whether it will be feasible in any given situation is highly dependent on local circumstances. In the case of West Bokaro Mine, the exact economical data for that site are not known, and therefore, although it is likely that the use of backfill will increase recovery, it is not possible to make a judgment on whether or not it will ultimately be profitable.
10. Recommendations

Although this thesis has attempted to make judgments on specific situations, much data was not available and as a result, many assumptions had to be made. Much of the geomechanical input data for the FLAC model was derived either from the laboratory tests done on the very limited amount of samples available, or data collected from previous reports on the mine site. This has a negative effect on the overall reliability of the results. This report should therefore not be considered definitive by any means, but rather as a baseline for additional research. Based on the research done in this thesis, the following recommendations are made with regards to sampling, modeling, and implementation:

Sampling
A large amount of samples should be obtained in situ to gain a better understanding of the true rock properties of the coal as well as the overburden. The strength of the coal measured in the lab was much higher than what could have been expected based on data available from previous cores. It is possible that large parts of the original cores were damaged which compromised their compressive strength; regardless, it seems unlikely that the data from previous drilling should be representative of the ore body as a whole, since tests done in this thesis resulted in a compressive strength almost twice that of any previous measurement. The model will become much more reliable when reliable data can be used as input. Ideally some in-situ measurements could be taken of the rock properties by means of an extensometer or similar equipment.

Modeling

- Additional refinements can be added to the model. The overburden group of the model can be divided into the various strata as indicated in the geological profile. Additionally, the number of blocks, particularly in the barrier pillars, can be increased to optimize the level of accuracy.

- The model is currently unable to simulate upward-dipping seams. This would require a new group and a significant change in geometry at the front of the model (particularly the ‘wedge’ group), but it would be useful to determine the effect of the orientation of the seam on the behavior of the model.

- The model is based on the Mohr Coulomb yield criterion, which in many cases is oversimplified. Depending on the situation, some of the other yield criteria available in FLAC may be more suitable. For jointed rock masses, the Hoek-Brown criterion would be more useful. Additionally, the double yield model might be more suitable for the backfill material if it is fluid and cement-like.

- It might be useful to examine the effect of varying pillar widths within a single panel. However, this would require a rewrite of a large part of the FISH code.

- Although the model allows for the modeling of roof coal, this has not been simulated in this thesis. However, this would be an interesting factor to examine.
Implementation

• Applying backfill in highwall mining can be beneficial to overall recovery, but the supportive effect of the backfill is not so large that it would be unequivocally feasible. It is imperative that costs be kept to a minimum to ensure that a profit can be made.

• During the initial stages of backfilling, it would be very useful to install some extensometers, to monitor the behavior of the rock in stages when it is still difficult to predict. These data could also be used to make the FLAC model more reliable.

• Backfill material should possess sufficient strength and cohesion. The testing done for this thesis would suggest that the backfill sample sent from India is unsuitable for that purpose.

• From a geomechanical point of view a one-by-one sequence would be best. A sequence that incorporates more excavations in a single pass is more efficient, but is also more susceptible to failure. These considerations should be taken into account.

• Deliberate partial backfilling should not be attempted, either horizontally or vertically. However, since the backfill material is not load-bearing for the most part, it is not imperative that the backfill fill the drive all the way to the top.

• Based on the findings of this thesis, the recommendations for West Bokaro specifically are as follows:

1. Ideal pillar width should be 2.9 m.

2. Best sequence, in the case of a 10-drive panel, would be a 5-by-5 sequence with simultaneous mining and backfilling.

3. A fly ash composite material could be suitable for use, but adequate strength and stiffness must be ensured.

4. Partial backfilling is not recommended.

5. Multiple lift mining is possible in an upward direction. However, care must be taken to ensure the stability of the roof.
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Appendices

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Appendix A. Geological Map of Jharkhand

Figure 1: Geological map of Jharkhand (Government of Jharkhand).
Appendix B. Summary of coal seams at West Bokaro

Table 1. Summary of Banji Village Seam Model

<table>
<thead>
<tr>
<th>Unit</th>
<th>Thickness (m)</th>
<th>Average Thickness (m)</th>
<th>Approximate Depth (m)</th>
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<tr>
<td>Seam 10</td>
<td>1–3</td>
<td>1.5</td>
<td>14</td>
</tr>
<tr>
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<td>0.5</td>
<td>—</td>
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<td>Seam 9</td>
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</tr>
<tr>
<td>Seam 7/Seam 8 IB</td>
<td>7–13</td>
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</tr>
<tr>
<td>Seam 7</td>
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<td>8.0</td>
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<tr>
<td>Seam 6/7 IB</td>
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</tr>
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<td>Seam 6</td>
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<td>60</td>
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<tr>
<td>Seam 5/6 IB</td>
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<td>—</td>
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<td>Seam 5</td>
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Table 2. Summary of Quarry D Seam Model

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<th>Unit</th>
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<td>3–4</td>
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<td>69</td>
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<tr>
<td>Seam 5</td>
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Table 3. Summary of Quarry SEB Seam Model

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Tables 1-3: Summary of coal seam thicknesses at West Bokaro Mine (Agapito Associates 2005).
Appendix C. Generalized lithology log for Quarry SEB

Figure 2: Generalized lithology log for Quarry SEB (Agapito Associates 2005).
Appendix D. Results of FEM analysis for backfilling highwall drives with FGD material

Table 1. Material properties for FEM

<table>
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<tr>
<th>No.</th>
<th>Material</th>
<th>Young's Modulus, Mpa</th>
<th>Strength, Mpa</th>
<th>Poisson's Ratio</th>
<th>Density, g/cm³</th>
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<tr>
<td>1</td>
<td>Roof Rock</td>
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<td>Coal</td>
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<td>3</td>
<td>Fire clay</td>
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Table 2. Displacement and stress results before backfilling with FGD material

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<th>Case</th>
<th>$\sigma_1$, kpa</th>
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<th>Displacement, cm</th>
<th>Stress, kpa</th>
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<tbody>
<tr>
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<td>137.9</td>
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<tr>
<td>2</td>
<td>689.5</td>
<td>276.8</td>
<td>6.36</td>
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Table 3. Displacement and stress results after backfilling with FGD material

<table>
<thead>
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<th>Displacement, cm</th>
<th>Stress, kpa</th>
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Table 4. Displacement and stress results after backfilling with FGD material and removal of coal web

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<th>Poisson's Ratio</th>
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<tr>
<td>1</td>
<td>168.9</td>
<td>0.25</td>
<td>4.14</td>
<td>366.1</td>
</tr>
<tr>
<td>2</td>
<td>344.7</td>
<td>0.25</td>
<td>3.36</td>
<td>324.8</td>
</tr>
<tr>
<td>3</td>
<td>689.5</td>
<td>0.25</td>
<td>3.26</td>
<td>315.8</td>
</tr>
<tr>
<td>4</td>
<td>3447.5</td>
<td>0.25</td>
<td>3.18</td>
<td>303.4</td>
</tr>
<tr>
<td>5</td>
<td>4481.8</td>
<td>0.25</td>
<td>3.17</td>
<td>302.7</td>
</tr>
<tr>
<td>6</td>
<td>5171.3</td>
<td>0.25</td>
<td>3.17</td>
<td>302.0</td>
</tr>
</tbody>
</table>

Tables 4-7: Results of Sweigard and Wang (2006)'s FEM models on backfilling highwall drives with FGD material.
### Appendix E. Rock Tunneling Quality Index Q

**Table 8: Classification of individual parameters used in the Tunneling Quality Index Q (Hoek 2007).**

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>VALUE</th>
<th>NOTES</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. ROCK QUALITY DESIGNATION</td>
<td>RQD</td>
<td></td>
</tr>
<tr>
<td>A. Very poor</td>
<td>0 - 25</td>
<td>1. Where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q.</td>
</tr>
<tr>
<td>B. Poor</td>
<td>25 - 50</td>
<td></td>
</tr>
<tr>
<td>C. Fair</td>
<td>50 - 75</td>
<td></td>
</tr>
<tr>
<td>D. Good</td>
<td>75 - 90</td>
<td>2. RQD intervals of 5, i.e. 100, 95, 90 etc. are sufficiently accurate.</td>
</tr>
<tr>
<td>E. Excellent</td>
<td>90 - 100</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>2. JOINT SET NUMBER</th>
<th>Jn</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Massive, no or few joints</td>
<td>0.5 - 1.0</td>
<td></td>
</tr>
<tr>
<td>B. One joint set</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>C. One joint set plus random</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>D. Two joint sets</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>E. Two joint sets plus random</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>F. Three joint sets</td>
<td>9</td>
<td>1. For intersections use (3.0 × Jn)</td>
</tr>
<tr>
<td>G. Three joint sets plus random</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>H. Four or more joint sets, random, heavily jointed, ‘sugar cube’, etc.</td>
<td>15</td>
<td>2. For portals use (2.0 × Jn)</td>
</tr>
<tr>
<td>J. Crushed rock, earthlike</td>
<td>20</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>3. JOINT ROUGHNESS NUMBER</th>
<th>Jr</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>a. Rock wall contact</td>
<td></td>
<td></td>
</tr>
<tr>
<td>b. Rock wall contact before 10 cm shear</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A. Discontinuous joints</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>B. Rough and irregular, undulating</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>C. Smooth undulating</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>D. Slickensided undulating</td>
<td>1.5</td>
<td>1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.</td>
</tr>
<tr>
<td>E. Rough or irregular, planar</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>F. Smooth, planar</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>G. Slickensided, planar</td>
<td>0.5</td>
<td>2. Jr = 0.5 can be used for planar, slickensided joints having lineations, provided that the lineations are oriented for minimum strength.</td>
</tr>
<tr>
<td>c. No rock wall contact when sheared</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H. Zones containing clay minerals thick enough to prevent rock wall contact (nominal)</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>J. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact (nominal)</td>
<td>1.0</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>4. JOINT ALTERATION NUMBER</th>
<th>Ja</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>a. Rock wall contact</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A. Tightly healed, hard, non-softening, impermeable filling</td>
<td>0.75</td>
<td>1. Values of ϕr, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.</td>
</tr>
<tr>
<td>B. Unaltered joint walls, surface staining only</td>
<td>1.0</td>
<td>25 - 35</td>
</tr>
<tr>
<td>C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.</td>
<td>2.0</td>
<td>25 - 30</td>
</tr>
<tr>
<td>D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)</td>
<td>3.0</td>
<td>20 - 25</td>
</tr>
<tr>
<td>E. Softening or low-friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1 - 2 mm or less)</td>
<td>4.0</td>
<td>8 - 16</td>
</tr>
</tbody>
</table>
4. Joint Alteration Number  
   \[ J_a \]  
   \( \text{for degrees (approx.)} \)

b. Rock wall contact before 10 cm shear

<table>
<thead>
<tr>
<th></th>
<th>( J_a )</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>F. Sandy particles, clay-free, disintegrating rock etc.</td>
<td>4.0</td>
<td>25 - 30</td>
</tr>
<tr>
<td>G. Strongly over-consolidated, non-softening clay mineral fillings (continuous &lt; 5 mm thick)</td>
<td>6.0</td>
<td>16 - 24</td>
</tr>
<tr>
<td>H. Medium or low over-consolidation, softening clay mineral fillings (continuous &lt; 5 mm thick)</td>
<td>8.0</td>
<td>12 - 16</td>
</tr>
<tr>
<td>J. Swelling clay fillings, i.e. montmorillonite, (continuous &lt; 5 mm thick). Values of ( J_a ) depend on percent of swelling clay-size particles, and access to water.</td>
<td>8.0 - 12.0</td>
<td>6 - 12</td>
</tr>
</tbody>
</table>

c. No rock wall contact when sheared

<table>
<thead>
<tr>
<th></th>
<th>( J_a )</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>K. Zones or bands of disintegrated or crushed rock and clay</td>
<td>6.0</td>
<td></td>
</tr>
<tr>
<td>L. Rock and clay (see G, H and J for clay conditions)</td>
<td>8.0</td>
<td></td>
</tr>
<tr>
<td>M. Zones or bands of silty-or sandy-clay, small clay fraction, non-softening</td>
<td>5.0</td>
<td></td>
</tr>
<tr>
<td>N. Thick continuous zones or bands of clay</td>
<td>10.0 - 13.0</td>
<td></td>
</tr>
<tr>
<td>O. Zones or bands of silty- or sandy-clay, small clay fraction, non-softening</td>
<td>6.0 - 24.0</td>
<td></td>
</tr>
</tbody>
</table>

5. Joint Water Reduction  
   \( J_w \)  
   approx. water pressure (kgf/cm²)

<table>
<thead>
<tr>
<th></th>
<th>( J_w )</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Dry excavation or minor inflow i.e. &lt; 5 l/m locally</td>
<td>1.0</td>
<td>&lt; 1.0</td>
</tr>
<tr>
<td>B. Medium inflow or pressure, occasional outwash of joint fillings</td>
<td>0.66</td>
<td>1.0 - 2.5</td>
</tr>
<tr>
<td>C. Large inflow or high pressure in competent rock with unfilled joints</td>
<td>0.5</td>
<td>2.5 - 10.0</td>
</tr>
<tr>
<td>D. Large inflow or high pressure at blasting, decaying with time</td>
<td>0.33</td>
<td>2.5 - 10.0</td>
</tr>
<tr>
<td>E. Exceptionally high inflow or pressure at blasting, decaying with time</td>
<td>0.2 - 0.1</td>
<td>&gt; 10</td>
</tr>
<tr>
<td>F. Exceptionally high inflow or pressure at blasting, decaying with time</td>
<td>0.1 - 0.05</td>
<td>&gt; 10</td>
</tr>
</tbody>
</table>

6. Stress Reduction Factor  
   \( SRF \)

a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated

<table>
<thead>
<tr>
<th></th>
<th>( SRF )</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)</td>
<td>10.0</td>
<td></td>
</tr>
<tr>
<td>B. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth &lt; 50 m)</td>
<td>5.0</td>
<td></td>
</tr>
<tr>
<td>C. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth &gt; 50 m)</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td>E. Single shear zone in competent rock (clay free), (depth of excavation &lt; 50 m)</td>
<td>5.0</td>
<td></td>
</tr>
<tr>
<td>F. Single shear zone in competent rock (clay free), (depth of excavation &gt; 50 m)</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>G. Loose open joints, heavily jointed or 'sugar cube', (any depth)</td>
<td>5.0</td>
<td></td>
</tr>
</tbody>
</table>

**Table 8:** Classification of individual parameters used in the Tunneling Quality Index Q (Hoek 2007) (cont.).
### Table 6: Classification of individual parameters in the Tunnelling Quality Index $Q$ (After Barton et al 1974).

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>VALUE</th>
<th>SRF</th>
<th>NOTES</th>
</tr>
</thead>
<tbody>
<tr>
<td>b. Competent rock, rock stress problems</td>
<td>$\sigma_c/\sigma_1$</td>
<td>$\sigma_t/\sigma_1$</td>
<td>2. For strongly anisotropic virgin stress field (if measured): when $5&lt;\sigma_1/\sigma_3&lt;10$, reduce $\sigma_c$</td>
</tr>
<tr>
<td>H. Low stress, near surface</td>
<td>&gt; 200</td>
<td>&gt; 13</td>
<td>2.5</td>
</tr>
<tr>
<td>J. Medium stress</td>
<td>200 - 10</td>
<td>13 - 0.66</td>
<td>1.0</td>
</tr>
<tr>
<td>K. High stress, very tight structure</td>
<td>10 - 5</td>
<td>0.66 - 0.33</td>
<td>0.5 - 2</td>
</tr>
<tr>
<td>L. Mild rockburst (massive rock)</td>
<td>5 - 2.5</td>
<td>0.33 - 0.16</td>
<td>5 - 10</td>
</tr>
<tr>
<td>M. Heavy rockburst (massive rock)</td>
<td>&lt; 2.5</td>
<td>&lt; 0.16</td>
<td>10 - 20</td>
</tr>
<tr>
<td>c. Squeezing rock, plastic flow of incompetent rock under influence of high rock pressure</td>
<td></td>
<td></td>
<td>3. Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).</td>
</tr>
<tr>
<td>c. Squeezing rock, plastic flow of incompetent rock</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>N. Mild squeezing rock pressure</td>
<td>5 - 10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>O. Heavy squeezing rock pressure</td>
<td>10 - 20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>d. Swelling rock, chemical swelling activity depending on presence of water</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P. Mild swelling rock pressure</td>
<td>5 - 10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Q. Heavy swelling rock pressure</td>
<td>10 - 15</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**ADDITIONAL NOTES ON THE USE OF THESE TABLES**

When making estimates of the rock mass Quality ($Q$), the following guidelines should be followed in addition to the notes listed in the tables:

1. When borehole core is unavailable, $RQD$ can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relationship can be used to convert this number to $RQD$ for the case of clay free rock masses: $RQD = 115 - 3.3 J_v$ (approx.), where $J_v = \text{total number of joints per m}^3$ ($0 < RQD < 100$ for $35 > J_v > 4.5$).

2. The parameter $J_n$ representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed, these parallel ‘joints’ should obviously be counted as a complete joint set. However, if there are few ‘joints’ visible, or if only occasional breaks in the core are due to these features, then it will be more appropriate to count them as ‘random’ joints when evaluating $J_T$.

3. The parameters $J_r$ and $J_a$ (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of $J_r/J_a$ is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of $J_r/J_a$ should be used when evaluating $Q$. The value of $J_r/J_a$ should in fact relate to the surface most likely to allow failure to initiate.

4. When a rock mass contains clay, the factor SRF appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.

5. The compressive and tensile strengths ($\sigma_c$ and $\sigma_t$) of the intact rock should be evaluated in the saturated condition if this is appropriate to the present and future in situ conditions. A very conservative estimate of the strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

---

**Table 8: Classification of individual parameters used in the Tunneling Quality Index $Q$ (Hoek 2007) (cont.).**
### A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Range of values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength of intact rock material</td>
<td>&gt;10 MPa</td>
</tr>
<tr>
<td>Point load strength index</td>
<td>4 - 10 MPa</td>
</tr>
<tr>
<td>Uniaxial comp. strength</td>
<td>2 - 4 MPa</td>
</tr>
<tr>
<td>Linet load strength index</td>
<td>1 - 2 MPa</td>
</tr>
<tr>
<td>For this low range - uniaxial compressive preferred</td>
<td></td>
</tr>
<tr>
<td>Rating</td>
<td>15</td>
</tr>
<tr>
<td>Drill core Quality (RQD)</td>
<td>90% - 100%</td>
</tr>
<tr>
<td>Rating</td>
<td>20</td>
</tr>
<tr>
<td>Spacing of discontinuities</td>
<td>&gt; 2 m</td>
</tr>
<tr>
<td>Rating</td>
<td>20</td>
</tr>
<tr>
<td>Condition of discontinuities (See E)</td>
<td>Very rough surfaces</td>
</tr>
<tr>
<td>Slightly rough surfaces</td>
<td>Slightly weathered walls</td>
</tr>
<tr>
<td>Slightly weathered walls</td>
<td>Slickensided surfaces or Gouge &lt; 5 mm thick or Separation 1-5 mm Continuous</td>
</tr>
<tr>
<td>Rating</td>
<td>30</td>
</tr>
<tr>
<td>Groundwater materials</td>
<td>None</td>
</tr>
<tr>
<td>Inflow per 10 m tunnel length (l/m)</td>
<td>&lt; 10</td>
</tr>
<tr>
<td>General conditions</td>
<td>Completely dry</td>
</tr>
<tr>
<td>Rating</td>
<td>15</td>
</tr>
<tr>
<td>Strike and dip orientations</td>
<td>Tunnels &amp; mines</td>
</tr>
<tr>
<td>Ratings</td>
<td>0</td>
</tr>
<tr>
<td>Class number</td>
<td>I</td>
</tr>
<tr>
<td>Description</td>
<td>Very good rock</td>
</tr>
<tr>
<td>Cohesion of rock mass (kPa)</td>
<td>&gt; 400</td>
</tr>
<tr>
<td>Friction angle of rock mass (deg)</td>
<td>&gt; 45</td>
</tr>
<tr>
<td>E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY CONDITIONS</td>
<td></td>
</tr>
<tr>
<td>Discontinuity length (persistence)</td>
<td>&lt; 1 m</td>
</tr>
<tr>
<td>Rating</td>
<td>6</td>
</tr>
<tr>
<td>Separation (aperture)</td>
<td>None</td>
</tr>
<tr>
<td>Rating</td>
<td>6</td>
</tr>
<tr>
<td>Roughness</td>
<td>Very rough</td>
</tr>
<tr>
<td>Rating</td>
<td>6</td>
</tr>
<tr>
<td>Infilling (gouge)</td>
<td>None</td>
</tr>
<tr>
<td>Rating</td>
<td>6</td>
</tr>
<tr>
<td>Weathering</td>
<td>Unweathered</td>
</tr>
<tr>
<td>Rating</td>
<td>6</td>
</tr>
<tr>
<td>F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**</td>
<td></td>
</tr>
<tr>
<td>Strike perpendicular to tunnel axis</td>
<td>Strike parallel to tunnel axis</td>
</tr>
<tr>
<td>Drive with dip - Dip 45 - 90°</td>
<td>Drive with dip - Dip 20 - 45°</td>
</tr>
<tr>
<td>Very favourable</td>
<td>Very favourable</td>
</tr>
<tr>
<td>Drive against dip - Dip 45-90°</td>
<td>Drive against dip - Dip 20-45°</td>
</tr>
<tr>
<td>Fair</td>
<td>Unfavourable</td>
</tr>
<tr>
<td>Drive against dip - Dip 45-90°</td>
<td>Drive against dip - Dip 20-45°</td>
</tr>
<tr>
<td>Dip 0-20 - Irrespective of strike*</td>
<td>Fair</td>
</tr>
</tbody>
</table>

**Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.

**Modified after Wickham and Yalcin (1973).

### Table 9: Rock Mass Rating System (Hoek 2007)
### Appendix G. Geological Strength Index

**GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS** (Hoek and Marinos, 2000)

From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis.

**Figure 3:** Geological Strength Index for jointed rocks (Hoek 2007).

<table>
<thead>
<tr>
<th>STRUCTURE</th>
<th>DECREASING INTERLOCKING OF ROCK PIECES</th>
<th>DECREASING SURFACE QUALITY</th>
<th>VERY GOOD</th>
<th>GOOD</th>
<th>FAIR</th>
<th>POOR</th>
<th>VERY POOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>INTACT OR MASSIVE - intact rock specimens or massive in situ rock with few widely spaced discontinuities</td>
<td></td>
<td>N/A</td>
<td>90</td>
<td>80</td>
<td>70</td>
<td>60</td>
<td>50</td>
</tr>
<tr>
<td>BLOCKY - well interlocked undisturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets</td>
<td></td>
<td>N/A</td>
<td>80</td>
<td>70</td>
<td>60</td>
<td>50</td>
<td>40</td>
</tr>
<tr>
<td>VERY BLOCKY - interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets</td>
<td></td>
<td>N/A</td>
<td>70</td>
<td>60</td>
<td>50</td>
<td>40</td>
<td>30</td>
</tr>
<tr>
<td>BLOCKY/DISTURBED/SEAMY - folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity</td>
<td></td>
<td>N/A</td>
<td>60</td>
<td>50</td>
<td>40</td>
<td>30</td>
<td>20</td>
</tr>
<tr>
<td>DISINTEGRATED - poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces</td>
<td></td>
<td>N/A</td>
<td>50</td>
<td>40</td>
<td>30</td>
<td>20</td>
<td>10</td>
</tr>
<tr>
<td>LAMINATED/SHEARED - Lack of blockiness due to close spacing of weak schistosity or shear planes</td>
<td></td>
<td>N/A</td>
<td>40</td>
<td>30</td>
<td>20</td>
<td>10</td>
<td></td>
</tr>
</tbody>
</table>

Table 5: Characterisation of blocky rock masses on the basis of interlocking and joint conditions.
The development of rock engineering

Determination of the strength of an in situ rock mass by laboratory type testing is generally not practical. Hence this strength must be estimated from geological observations and from test results on individual rock pieces or rock surfaces which have been removed from the rock mass. This question has been discussed extensively by Hoek and Brown (1980) who used the results of theoretical (Hoek, 1968) and model studies (Brown, 1970, Ladanyi and Archambault, 1970) and the limited amount of available strength data, to develop an empirical failure criterion for jointed rock masses. Hoek (1983) also proposed that the rock mass classification system of Bieniawski could be used for estimating the rock mass constants required for this empirical failure criterion. This classification proved to be adequate for better quality rock masses but it soon became obvious that a new classification was required for the very weak tectonically disturbed rock masses associated with the major mountain chains of the Alps, the Himalayas and the Andes.

The Geological Strength Index (GSI) was introduced by Hoek in 1994 and this Index was subsequently modified and expanded as experience was gained on its application to practical rock engineering problems. Marinos and Hoek (2000, 2001) published the chart reproduced in Figure 8 for use in estimating the properties of heterogeneous rock masses such as flysch (Figure 9).

**Figure 4:** GSI for heterogeneous rock masses such as flysch (Hoek 2007).
Appendix H. UCS result curves for rock and backfill samples

**Figure 5:** Stress-strain curve for India coal sample #1

**Figure 6:** Transverse strain vs axial strain for India coal sample #1
Figure 7: Stress-strain curve for India coal sample #2

Figure 8: Transverse strain vs axial strain for India coal sample #2
**Figure 9:** Stress-strain curve for India coal sample #2 (second time)

**Figure 10:** Transverse strain vs axial strain for India coal sample #2 (second time)
Figure 11: Stress-strain curve for India coal sample #3

Figure 12: Transverse strain vs axial strain for India coal sample #3
Figure 13: Stress-strain curve for India coal sample #4

Figure 14: Transverse strain vs axial strain for India coal sample #4
Figure 15: Stress-strain curve for India roof rock sample #2 (wet)

Figure 16: Transverse strain vs axial strain for India roof rock sample #2 (wet)
Figure 17: Stress-strain curve for India roof rock sample #2 (dry)

Figure 18: Transverse strain vs axial strain for India roof rock sample #2 (dry)
**Figure 19:** Stress-strain curve for India roof rock sample #3 (wet)

**Figure 20:** Transverse strain vs axial strain for India roof rock sample #3 (wet)
Figure 21: Stress-strain curve for India roof rock sample #3 (dry)

Figure 22: Transverse strain vs axial strain for India roof rock sample #3 (dry)
Figure 23: Stress-strain curve for Australian foam backfill sample #4 (wet)

Figure 24: Transverse strain vs axial strain for Australian foam backfill sample (wet)
Figure 25: Stress-strain curve for Australian foam backfill sample #4 (dry)

Figure 26: Transverse strain vs axial strain for Australian foam backfill sample (dry)
Figure 27: Stress-strain curve for Australian foam backfill sample #5

Figure 28: Stress-strain curve for Indian backfill sample

\[ y = 0.4902x - 2.4494 \]

\[ y = 0.0147x - 0.0232 \]
Figure 29: Stress-strain curve for American backfill sample (wet)

Figure 30: Transverse strain vs axial strain for American backfill sample (wet)
Figure 31: Stress-strain curve for American backfill sample (dry)

Figure 32: Transverse strain vs axial strain for American backfill sample (dry)
Appendix I. XRF test of backfill material

The geomechanical tests yielded some unexpected results for the Indian backfill material. For this reason, it was decided to carry out some chemical tests on this material to determine its exact composition. Initial tests were carried out using XRF analysis of a pressed powder sample of the backfill material. The results of this test are listed in Table 10.

The main components revealed by this test are not unusual for fly ash composite materials: silicon dioxide and aluminium oxide make up the vast majority, with smaller amounts of iron oxide, calcium oxide, titanium dioxide, and potassium oxide. However, some of the trace compounds, such as rubidium oxide, dysprosium oxide and gallium oxide were rather unusual. Considering the fact that these elements were present in both samples, it seems unlikely that either of the samples were somehow contaminated. The provenance of these materials is not entirely clear.

<table>
<thead>
<tr>
<th>Compound Name</th>
<th>Conc. wt(%)</th>
<th>Absolute Error (%)</th>
<th>Compound Name</th>
<th>Conc. wt(%)</th>
<th>Absolute Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO2</td>
<td>56.103</td>
<td>0.1</td>
<td>SiO2</td>
<td>56.29</td>
<td>0.1</td>
</tr>
<tr>
<td>Al2O3</td>
<td>29.803</td>
<td>0.1</td>
<td>Al2O3</td>
<td>30.719</td>
<td>0.1</td>
</tr>
<tr>
<td>Fe2O3</td>
<td>6.283</td>
<td>0.07</td>
<td>Fe2O3</td>
<td>5.732</td>
<td>0.07</td>
</tr>
<tr>
<td>CaO</td>
<td>2.336</td>
<td>0.05</td>
<td>CaO</td>
<td>2.435</td>
<td>0.05</td>
</tr>
<tr>
<td>TiO2</td>
<td>2.123</td>
<td>0.04</td>
<td>TiO2</td>
<td>2.055</td>
<td>0.04</td>
</tr>
<tr>
<td>K2O</td>
<td>1.024</td>
<td>0.03</td>
<td>K2O</td>
<td>1.053</td>
<td>0.03</td>
</tr>
<tr>
<td>MgO</td>
<td>0.822</td>
<td>0.03</td>
<td>MgO</td>
<td>0.726</td>
<td>0.03</td>
</tr>
<tr>
<td>P2O5</td>
<td>0.476</td>
<td>0.02</td>
<td>P2O5</td>
<td>0.407</td>
<td>0.02</td>
</tr>
<tr>
<td>F</td>
<td>0.308</td>
<td>0.02</td>
<td>BaO</td>
<td>0.113</td>
<td>0.01</td>
</tr>
<tr>
<td>SO3</td>
<td>0.141</td>
<td>0.01</td>
<td>SO3</td>
<td>0.088</td>
<td>0.009</td>
</tr>
<tr>
<td>Na2O</td>
<td>0.124</td>
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<td>Na2O</td>
<td>0.082</td>
<td>0.009</td>
</tr>
<tr>
<td>BaO</td>
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<td>0.01</td>
<td>ZrO2</td>
<td>0.066</td>
<td>0.008</td>
</tr>
<tr>
<td>MnO</td>
<td>0.061</td>
<td>0.007</td>
<td>MnO</td>
<td>0.065</td>
<td>0.008</td>
</tr>
<tr>
<td>CeO2</td>
<td>0.068</td>
<td>0.01</td>
<td>Dy2O3</td>
<td>0.039</td>
<td>0.006</td>
</tr>
<tr>
<td>MnO</td>
<td>0.061</td>
<td>0.007</td>
<td>Cr2O3</td>
<td>0.023</td>
<td>0.005</td>
</tr>
<tr>
<td>Cl</td>
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<td>0.005</td>
<td>Cl</td>
<td>0.016</td>
<td>0.004</td>
</tr>
<tr>
<td>Cr2O3</td>
<td>0.024</td>
<td>0.005</td>
<td>CuO</td>
<td>0.013</td>
<td>0.003</td>
</tr>
<tr>
<td>CuO</td>
<td>0.012</td>
<td>0.003</td>
<td>NiO</td>
<td>0.013</td>
<td>0.003</td>
</tr>
<tr>
<td>NiO</td>
<td>0.01</td>
<td>0.003</td>
<td>Y2O3</td>
<td>0.008</td>
<td>0.003</td>
</tr>
<tr>
<td>Y2O3</td>
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<td>0.003</td>
<td>Rb2O</td>
<td>0.007</td>
<td>0.003</td>
</tr>
<tr>
<td>ZnO</td>
<td>0.009</td>
<td>0.003</td>
<td>ZnO</td>
<td>0.007</td>
<td>0.003</td>
</tr>
<tr>
<td>Rb2O</td>
<td>0.009</td>
<td>0.003</td>
<td>Ga2O3</td>
<td>0.006</td>
<td>0.002</td>
</tr>
<tr>
<td>Nb2O5</td>
<td>0.007</td>
<td>0.002</td>
<td>Nb2O5</td>
<td>0.005</td>
<td>0.002</td>
</tr>
<tr>
<td>Co3O4</td>
<td>0.004</td>
<td>0.002</td>
<td>PbO</td>
<td>0.004</td>
<td>0.002</td>
</tr>
<tr>
<td>Ga2O3</td>
<td>0.004</td>
<td>0.002</td>
<td>Co3O4</td>
<td>0.004</td>
<td>0.002</td>
</tr>
</tbody>
</table>

Table 10: XRF results for Indian backfill material.
Appendix J. Front and side views of FLAC model

Figure 33: Front view of FLAC model.
Figure 34: Side view of FLAC model.
Appendix K. DAT file for FLAC model

;----------------------------------------------------------------------------------------------------- PILLAR WIDTH 3.4 – Backfill
;-----------------------------------------------------------------------------------------------------
new
set fish safe off

; NEW DAT FILE BEHORENDE BIJ CODE.FIS

; BBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBB
; BBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBB
; BBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBBB

def input_geom

sim = 'simFCMBFPW3_4NEW'
; 1) holes
  thbottomhalfdrive = 2.25
  thtophalfdrive = 2.25
  throofcoal = 1.
  wdrive = 3.5
  wpillar = 3.4
  wbarrier = 15.
  ldrive = 250.
  ndrives = 10.
; 2) geology
  thfootwall = 80.
  thoverburden = 140.
  wbench = 50.
  lvirgin = 300.
  alpha = 5
  beta = 60.
; 3) mesh refinement = NUMBER OF CELLS/BLOCKS
  mdrive = 8
  mpillar = 8
  mbarrier = 16
  nlvirgin = 12
  nthoverburden = 10
  mbench = 6
  nthfootwall = 10
  nthoverburden = 10
  nlnonbackfill = 20
  nlbackfill = 20
  nthbottomhalfdrive = 6
  nthtophalfdrive = 6
  nthroofcoal = 1
; 4) ratios
  rwdrive = 1
  rwpillar = 1
  rwbarrier = .8
  rlvirgin = 1.2
  rthoverburden = 1.5
  rwbench = 0.8
  rthfootwall = .7
  rlnonbackfill = 1
  rlbbackfill = 1
  rthbottomhalfdrive = 1
  rthtophalfdrive = 1
  rthroofcoal = 1
; 5) densities
  ob_density = 2300
  coal_density = 1550
  backfill_density = 1800
  fbackfill = 0.5

; Indicate presence or absence of roof coal: if roof coal is present, then we say roofcoal = y; else roofcoal = n
roofcoal = 0
end

@input_geom

# mesh_geom definition

@mesh_geom

# gen_groups definition

@gen_groups

model mohr

set _phi 43.83
set _coh 3.097e6
set young 4.2257e9
set poisson 0.2
get_KG

prop friction _phi cohesion _coh bulk _K shear _G density ob_density &
range group 'overburden'

prop friction _phi cohesion _coh bulk _K shear _G density ob_density &
range group 'trench_overburden'

prop friction _phi cohesion _coh bulk _K shear _G density ob_density &
range group 'trench_wedge'

prop friction _phi cohesion _coh bulk _K shear _G density ob_density &
range group 'footwall'

set _phi 40.19
set _coh 1.208e6
set young 3.468e9
set poisson 0.208
get_KG

prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'trench_bottomhalf'

prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'trench_tophalf'

prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'trench_roofcoal'

prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'barrier1'

prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'barrier2'

prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'webpillar'
Appendix K - DAT file for FLAC model

```plaintext
;prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
;range group 'roofcoal'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdrivefront1'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdrivefront2'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdrivefront3'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdrivefront4'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdrivefront5'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdrivefront6'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdrivefront7'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdrivefront8'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdrivefront9'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdrivefront10'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdrivefront11'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdrivefront12'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdrivefront13'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdrivefront14'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdrivefront15'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdrivefront16'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdrivefront17'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdrivefront18'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdrivefront19'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdrivefront20'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdriveback1'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdriveback2'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdriveback3'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdriveback4'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdriveback5'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdriveback6'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdriveback7'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdriveback8'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdriveback9'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'bottomhalfdriveback10'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdriveback11'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdriveback12'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdriveback13'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdriveback14'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdriveback15'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdriveback16'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdriveback17'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdriveback18'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdriveback19'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdriveback20'
```

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Appendix K - DAT file for FLAC model

; range group 'bottomhalfdriveback15'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdriveback16'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdriveback17'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdriveback18'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdriveback19'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'bottomhalfdriveback20'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'tophalfdriveback'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'tophalfdrivefront1'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'tophalfdrivefront2'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'tophalfdrivefront3'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'tophalfdrivefront4'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'tophalfdrivefront5'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'tophalfdrivefront6'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'tophalfdrivefront7'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'tophalfdrivefront8'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'tophalfdrivefront9'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'tophalfdrivefront10'
; range group 'tophalfdrivefront11'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
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; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
; range group 'tophalfdrivefront13'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
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; range group 'tophalfdrivefront15'
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
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; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
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range group 'tophalfdriveback1'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'tophalfdriveback2'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'tophalfdriveback3'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'tophalfdriveback4'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'tophalfdriveback5'
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range group 'tophalfdriveback6'
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range group 'tophalfdriveback7'
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range group 'tophalfdriveback8'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
range group 'tophalfdriveback9'

Appendix K - DAT file for FLAC model
Appendix K - DAT file for FLAC model

prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
  range group 'tophalfdriveback10' &
; prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
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;prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
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;prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
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  range group 'tophalfdriveback19'
;prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
  range group 'tophalfdriveback20'
prop friction _phi cohesion _coh bulk _K shear _G density coal_density &
  range group 'virgin'

; %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
; %%%%% BOUNDARY CONDITIONS %%%%%
; %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
fix x range x xminleft xminright
fix x range x xmaxleft xmaxright
fix y range y yminfront yminbehind
fix y range y ymaxfront ymaxbehind
fix z range z zminunder zminabove

; %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
; %%%%% SET GRAVITY
; %%%%%%%%%%%%%%%%%
set gravity 0 0 -10.

; %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
; %%%%% ASSUME K0=0.5
; %%%%%%%%%%%%%%%%%
set k0 0.5

@get_stresses
initial sxx sxmax grad 0. 0. sxgrad
initial syy sxmax grad 0. 0. sygrad
initial szz szmax grad 0. 0. szgrad

; %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
; %%%%% SET HISTORIES
; %%%%%%%%%%%%%%%%%
hist unbal
hist ratio

; %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
; #### SOLVE INITIAL ####
; #### EQUILIBRIUM ####
set mech ratio 5e-5
solve
save simFCMBFPW3_4NEWINEQ.sav
res simFCMBFPW3_4NEWINEQ.sav

; %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

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Appendix K - DAT file for FLAC model

; #### EXCAVATE THE ####
; #### TRENCH AND SOLVE ####
; ###################################################################

@excavate_trench
save simFCMBFPW3_4NEWTREX.sav
res simFCMBFPW3_4NEWTREX.sav

; ###################################################################
; #### EXCAVATE THE DRIVE ####
; #### AND FOLLOW WITH BACKFILL ####
; #### SOLVE ####
; ###################################################################

; #### DRIVES 1,4,7,10 #######
@setittoone
@excavate_drive
@backfill_drive

; #### DRIVES 2,5,8 #######
@addonetoi
@excavate_drive

@addonetoi
@backfill_drive

; #### DRIVES 3,6,9 #######
@addonetoi
@addonetoi
@excavate_drive

@addonetoi
@addonetoi
@addonetoi
@addonetoi
@backfill_drive

;################################################################### END OF SIMULATION
###################################################################
def ini_array
    x = get_array(1000)
    y = get_array(1000)
    z = get_array(1000)
    xhw = get_array(1000)
    yhw = get_array(1000)
    zhw = get_array(1000)
    xbf = get_array(1000)
    ybf = get_array(1000)
    zbf = get_array(1000)
    xeod = get_array(1000)
    yeod = get_array(1000)
    zeod = get_array(1000)
    xv = get_array(1000)
    yv = get_array(1000)
    zv = get_array(1000)
end
@end_array;

### Define Mesh geometry ###

def mesh_geom
    input:
    1) holes
    thbottomhalfdrive = thickness of bottom half of drive
    thtophalfdrive = thickness of top half of drive
    throofcoal = thickness of layer of coal in roof
    wdrive = width of the drive
    wpillar = width of the pillar
    wbarrier = width of barrier pillar
    ldrive = length of the drive
    ndrives = number of drives
    2) geology
    thfootwall = footwall thickness at drive entry
    thoverburden = overburden thickness at drive entry
    wbench = width of bench/trench
    lvirgin = width of virgin rock
    alpha = seam dip
    beta = highwall angle
    3) mesh refinement = NUMBER OF CELLS/BLOCKS
    nwdrive
    npillar
    nwpillar
    nwbarrier
    nlvirgin
    nthoverburden
    nwbench
    nthfootwall
    nlnonbackfill
    nlbackfill
    nthbottomhalfdrive
    nthtophalfdrive
    nthroofcoal
    4) ratios
    rwdrive
    rwpillar
    rwbarrier
    rlvirgin
    rthoverburden
    rwbench
    rthfootwall
    rlnonbackfill
    rlbackfill
    rthbottomhalfdrive
    rthtophalfdrive
    rthroofcoal
    rwedge
    5) densities
; ob_density = density of overburden
; coal_density = density of coal

; fbackfill = fraction of drive that is backfilled

; convert degrees to radians
alpha = alpha*(pi/180)
beta = beta*(pi/180)

; Define all points in the model (grid)

### START OF PLANE I
### FIRST COLUMN OF COORDINATES ON PLANE I

x(1)=0.
y(1)=0.
z(1)=0.

x(12*ndrives+13)=x(1)
y(12*ndrives+13)=y(1)
z(12*ndrives+13)=z(1)+thfootwall

x(2)=x(1)
y(2)=y(1)
z(2)=z(12*ndrives+13)+(wbench*tan(alpha))

x(3)=x(2)
y(3)=y(2)
z(3)=z(2)+((thbottomhalfdrive/cos(alpha))

x(4)=x(3)
y(4)=y(3)
z(4)=z(4)+((thtophalfdrive/cos(alpha))

x(5)=x(4)
y(5)=y(4)
z(5)=z(4)+((throofcoal/cos(alpha))

x(6)=x(5)
y(6)=y(5)
z(6)=thfootwall+((thbottomhalfdrive/sin(alpha+beta))*sin(beta))+((thtophalfdrive/sin(alpha+beta))

### SECOND COLUMN OF COORDINATES ON PLANE I

x(7)=x(1)+0.5*wbarrier
y(7)=y(1)
z(7)=x(1)

x(12*ndrives+14)=x(12*ndrives+13)+0.5*wbarrier
y(12*ndrives+14)=y(12*ndrives+13)
z(12*ndrives+14)=z(12*ndrives+13)

x(8)=x(2)+0.5*wbarrier
y(8)=y(2)
z(8)=z(2)

x(9)=x(3)+0.5*wbarrier
y(9)=y(3)
z(9)=z(3)

x(10)=x(4)+0.5*wbarrier
y(10)=y(4)
z(10)=z(4)
x(11)=x(5)+0.5*wbarrier
y(11)=y(5)
z(11)=z(5)

x(12)=x(6)+0.5*wbarrier
y(12)=y(6)
z(12)=z(6)

; LOOP THAT STARTS WITH THE RIGHT HAND SIDE OF THE DRIVE(i)
i=1
loop while i<= ndrives
  x(12*i+1)=x(12*i-5)+wdrive
  y(12*i+1)=y(12*i-5)
  z(12*i+1)=z(1)
  x(12*ndrives+(13+(2*i)))=x(12*i+1)
  y(12*ndrives+(13+(2*i)))=y(12*i+1)
  z(12*ndrives+(13+(2*i)))=z(12*ndrives+14)

  x(12*i+2)=x(12*i+1)
  y(12*i+2)=y(12*i+1)
  z(12*i+2)=z(2)
  x(12*i+3)=x(12*i+2)
  y(12*i+3)=y(12*i+2)
  z(12*i+3)=z(3)
  x(12*i+4)=x(12*i+3)
  y(12*i+4)=y(12*i+3)
  z(12*i+4)=z(4)
  x(12*i+5)=x(12*i+4)
  y(12*i+5)=y(12*i+4)
  z(12*i+5)=z(5)
  x(12*i+6)=x(12*i+5)
  y(12*i+6)=y(12*i+5)
  z(12*i+6)=z(6)

; RIGHT HAND SIDE OF THE PILLAR(i)
  X(12*i+7)=x(12*i+1)+wpillar
  y(12*i+7)=y(12*i+1)
  z(12*i+7)=z(1)
  x(12*ndrives+(14+(2*i)))=x(12*i+7)
  y(12*ndrives+(14+(2*i)))=y(12*i+7)
  z(12*ndrives+(14+(2*i)))=z(12*ndrives+14)

  x(12*i+8)=x(12*i+7)
  y(12*i+8)=y(12*i+7)
  z(12*i+8)=z(2)
  x(12*i+9)=x(12*i+8)
  y(12*i+9)=y(12*i+8)
  z(12*i+9)=z(3)
  x(12*i+10)=x(12*i+9)
  y(12*i+10)=y(12*i+9)
  z(12*i+10)=z(4)
  x(12*i+11)=x(12*i+10)
  y(12*i+11)=y(12*i+10)
  z(12*i+11)=z(5)
  x(12*i+12)=x(12*i+11)
  y(12*i+12)=y(12*i+11)
  z(12*i+12)=z(6)

; i INCREASES BY 1
Appendix L - FISH file for FLAC model

```plaintext
i=i+1
end_loop

; RIGHT HAND SIDE OF THE LAST DRIVE

x(12*ndrives+1)=x(12*ndrives-5)+wdrive
y(12*ndrives+1)=y(12*ndrives-5)
z(12*ndrives+1)=z(1)

x(14*ndrives+13)=x(12*ndrives+1)
y(14*ndrives+13)=y(12*ndrives+1)
z(14*ndrives+13)=z(12*ndrives+13)

x(12*ndrives+2)=x(12*ndrives+1)
y(12*ndrives+2)=y(12*ndrives+1)
z(12*ndrives+2)=z(2)

x(12*ndrives+3)=x(12*ndrives+2)
y(12*ndrives+3)=y(12*ndrives+2)
z(12*ndrives+3)=z(3)

x(12*ndrives+4)=x(12*ndrives+3)
y(12*ndrives+4)=y(12*ndrives+3)
z(12*ndrives+4)=z(4)

x(12*ndrives+5)=x(12*ndrives+4)
y(12*ndrives+5)=y(12*ndrives+4)
z(12*ndrives+5)=z(5)

x(12*ndrives+6)=x(12*ndrives+5)
y(12*ndrives+6)=y(12*ndrives+5)
z(12*ndrives+6)=z(6)

; RIGHT HAND SIDE OF THE WHOLE MODEL (HALFWAY THROUGH RIGHT SIDE BARRIER PILLAR)

x(12*ndrives+7)=x(12*ndrives+1)+0.5*wbarrier
y(12*ndrives+7)=y(12*ndrives+1)
z(12*ndrives+7)=z(1)

x(14*ndrives+14)=x(12*ndrives+7)
y(14*ndrives+14)=y(12*ndrives+7)
z(14*ndrives+14)=z(12*ndrives+13)

x(12*ndrives+8)=x(12*ndrives+7)
y(12*ndrives+8)=y(12*ndrives+7)
z(12*ndrives+8)=z(2)

x(12*ndrives+9)=x(12*ndrives+8)
y(12*ndrives+9)=y(12*ndrives+8)
z(12*ndrives+9)=z(3)

x(12*ndrives+10)=x(12*ndrives+9)
y(12*ndrives+10)=y(12*ndrives+9)
z(12*ndrives+10)=z(4)

x(12*ndrives+11)=x(12*ndrives+10)
y(12*ndrives+11)=y(12*ndrives+10)
z(12*ndrives+11)=z(5)

x(12*ndrives+12)=x(12*ndrives+11)
y(12*ndrives+12)=y(12*ndrives+11)
z(12*ndrives+12)=z(6)

;######################################################################## END OF PLANE I
########################################################################

;######################################################################## START OF PLANE II########################################################################
; FIRST COLUMN OF COORDINATES ON PLANE II
```
Appendix L - FISH file for FLAC model

```
xhw(1)=0.
yhw(1)=0+wbench
zhw(1)=0.

xhw(2)=xhw(1)
yhw(2)=yhw(1)
zhw(2)=zhw(1)+thfootwall

xhw(3)=xhw(2)
yhw(3)=yhw(2)+(thbottomhalfdrive/sin(alpha+beta))*cos(beta)
zhw(3)=zhw(2)+(thbottomhalfdrive/sin(alpha+beta))*sin(beta)

xhw(4)=xhw(3)
yhw(4)=yhw(3)+(thtophalfdrive/sin(alpha+beta))*cos(beta)
zhw(4)=zhw(3)+(thtophalfdrive/sin(alpha+beta))*sin(beta)

xhw(5)=xhw(4)
yhw(5)=yhw(4)+(throofcoal/sin(alpha+beta))*cos(beta)
zhw(5)=zhw(4)+(throofcoal/sin(alpha+beta))*sin(beta)

xhw(6)=xhw(5)
yhw(6)=yhw(5)+thoverburden/tan(beta)
zhw(6)=zhw(5)+thoverburden

; SECOND COLUMN OF COORDINATES ON PLANE II

xhw(7)=xhw(1)+0.5*wbarrier
yhw(7)=yhw(1)
zhw(7)=zhw(1)

xhw(8)=xhw(2)+0.5*wbarrier
yhw(8)=yhw(2)
zhw(8)=zhw(2)

xhw(9)=xhw(3)+0.5*wbarrier
yhw(9)=yhw(3)
zhw(9)=zhw(3)

xhw(10)=xhw(4)+0.5*wbarrier
yhw(10)=yhw(4)
zhw(10)=zhw(4)

xhw(11)=xhw(5)+0.5*wbarrier
yhw(11)=yhw(5)
zhw(11)=zhw(5)

xhw(12)=xhw(6)+0.5*wbarrier
yhw(12)=yhw(6)
zhw(12)=zhw(6)

; LOOP THAT STARTS WITH THE RIGHT HAND SIDE OF THE DRIVE(i)
i=1
loop while i<= ndrives
  xhw(12*i+1)=xhw(12*i-5)+wdrive
  yhw(12*i+1)=yhw(12*i-5)
  zhw(12*i+1)=zhw(12*i-5)

  xhw(12*i+2)=xhw(12*i+1)
  yhw(12*i+2)=yhw(12*i+1)
  zhw(12*i+2)=zhw(12*i+1)+thfootwall

  xhw(12*i+3)=xhw(12*i+2)
  yhw(12*i+3)=yhw(12*i+2)+(thbottomhalfdrive/sin(alpha+beta))*cos(beta)
  zhw(12*i+3)=zhw(12*i+2)+(thbottomhalfdrive/sin(alpha+beta))*sin(beta)

  xhw(12*i+4)=xhw(12*i+3)
  yhw(12*i+4)=yhw(12*i+3)+(thtophalfdrive/sin(alpha+beta))*cos(beta)
  zhw(12*i+4)=zhw(12*i+3)+(thtophalfdrive/sin(alpha+beta))*sin(beta)

  xhw(12*i+5)=xhw(12*i+4)
  yhw(12*i+5)=yhw(12*i+4)+(throofcoal/sin(alpha+beta))*cos(beta)
  zhw(12*i+5)=zhw(12*i+4)+(throofcoal/sin(alpha+beta))*sin(beta)
```
Appendix L - FISH file for FLAC model

\[
\begin{align*}
\text{xhw}(12*i+6) &= \text{xhw}(12*i+5) \\
\text{yhw}(12*i+6) &= \text{yhw}(12*i+5) + \text{thoverburden}/\tan(\beta) \\
\text{zhw}(12*i+6) &= \text{zhw}(6)
\end{align*}
\]

; RIGHT SIDE OF THE PILLAR(i)

\[
\begin{align*}
\text{xhw}(12*i+7) &= \text{xhw}(12*i+1) + \text{wpillar} \\
\text{yhw}(12*i+7) &= \text{yhw}(12*i+1) \\
\text{zhw}(12*i+7) &= \text{zhw}(12*i+1)
\end{align*}
\]

\[
\begin{align*}
\text{xhw}(12*i+8) &= \text{xhw}(12*i+7) \\
\text{yhw}(12*i+8) &= \text{yhw}(12*i+7) \\
\text{zhw}(12*i+8) &= \text{zhw}(12*i+7) + \text{thfootwall}
\end{align*}
\]

\[
\begin{align*}
\text{xhw}(12*i+9) &= \text{xhw}(12*i+8) \\
\text{yhw}(12*i+9) &= \text{yhw}(12*i+8) + (\text{thbottomhalfdrive}/\sin(\alpha + \beta)) \cdot \cos(\beta) \\
\text{zhw}(12*i+9) &= \text{zhw}(12*i+8) + (\text{thbottomhalfdrive}/\sin(\alpha + \beta)) \cdot \sin(\beta)
\end{align*}
\]

\[
\begin{align*}
\text{xhw}(12*i+10) &= \text{xhw}(12*i+9) \\
\text{yhw}(12*i+10) &= \text{yhw}(12*i+9) + (\text{thtophalfdrive}/\sin(\alpha + \beta)) \cdot \cos(\beta) \\
\text{zhw}(12*i+10) &= \text{zhw}(12*i+9) + (\text{thtophalfdrive}/\sin(\alpha + \beta)) \cdot \sin(\beta)
\end{align*}
\]

\[
\begin{align*}
\text{xhw}(12*i+11) &= \text{xhw}(12*i+10) \\
\text{yhw}(12*i+11) &= \text{yhw}(12*i+10) + (\text{throofcoal}/\sin(\alpha + \beta)) \cdot \cos(\beta) \\
\text{zhw}(12*i+11) &= \text{zhw}(12*i+10) + (\text{throofcoal}/\sin(\alpha + \beta)) \cdot \sin(\beta)
\end{align*}
\]

\[
\begin{align*}
\text{xhw}(12*i+12) &= \text{xhw}(12*i+11) \\
\text{yhw}(12*i+12) &= \text{yhw}(12*i+11) + \text{thoverburden}/\tan(\beta) \\
\text{zhw}(12*i+12) &= \text{zhw}(6)
\end{align*}
\]

; i INCREASES BY 1

\[
\begin{align*}
i &= i+1
\end{align*}
\]

end_loop

; RIGHT HAND SIDE OF THE LAST DRIVE

\[
\begin{align*}
\text{xhw}(12*\text{ndrives}+1) &= \text{xhw}(12*\text{ndrives}-5) + \text{wdrive} \\
\text{yhw}(12*\text{ndrives}+1) &= \text{yhw}(12*\text{ndrives}-5) \\
\text{zhw}(12*\text{ndrives}+1) &= \text{zhw}(12*\text{ndrives}-5)
\end{align*}
\]

\[
\begin{align*}
\text{xhw}(12*\text{ndrives}+2) &= \text{xhw}(12*\text{ndrives}+1) \\
\text{yhw}(12*\text{ndrives}+2) &= \text{yhw}(12*\text{ndrives}+1) \\
\text{zhw}(12*\text{ndrives}+2) &= \text{zhw}(12*\text{ndrives}+1) + \text{thfootwall}
\end{align*}
\]

\[
\begin{align*}
\text{xhw}(12*\text{ndrives}+3) &= \text{xhw}(12*\text{ndrives}+2) + (\text{thbottomhalfdrive}/\sin(\alpha + \beta)) \cdot \cos(\beta) \\
\text{yhw}(12*\text{ndrives}+3) &= \text{yhw}(12*\text{ndrives}+2) + (\text{thbottomhalfdrive}/\sin(\alpha + \beta)) \cdot \sin(\beta)
\end{align*}
\]

\[
\begin{align*}
\text{xhw}(12*\text{ndrives}+4) &= \text{xhw}(12*\text{ndrives}+3) \\
\text{yhw}(12*\text{ndrives}+4) &= \text{yhw}(12*\text{ndrives}+3) + (\text{thtophalfdrive}/\sin(\alpha + \beta)) \cdot \cos(\beta) \\
\text{zhw}(12*\text{ndrives}+4) &= \text{zhw}(12*\text{ndrives}+3) + (\text{thtophalfdrive}/\sin(\alpha + \beta)) \cdot \sin(\beta)
\end{align*}
\]

\[
\begin{align*}
\text{xhw}(12*\text{ndrives}+5) &= \text{xhw}(12*\text{ndrives}+4) \\
\text{yhw}(12*\text{ndrives}+5) &= \text{yhw}(12*\text{ndrives}+4) + (\text{throofcoal}/\sin(\alpha + \beta)) \cdot \cos(\beta) \\
\text{zhw}(12*\text{ndrives}+5) &= \text{zhw}(12*\text{ndrives}+4) + (\text{throofcoal}/\sin(\alpha + \beta)) \cdot \sin(\beta)
\end{align*}
\]

\[
\begin{align*}
\text{xhw}(12*\text{ndrives}+6) &= \text{xhw}(12*\text{ndrives}+5) \\
\text{yhw}(12*\text{ndrives}+6) &= \text{yhw}(12*\text{ndrives}+5) + \text{thoverburden}/\tan(\beta) \\
\text{zhw}(12*\text{ndrives}+6) &= \text{zhw}(6)
\end{align*}
\]

; RIGHT HAND SIDE OF THE WHOLE MODEL (HALFWAY THROUGH RIGHT SIDE BARRIER PILLAR)

\[
\begin{align*}
\text{xhw}(12*\text{ndrives}+7) &= \text{xhw}(12*\text{ndrives}+1) + 0.5 \times \text{wbarrier} \\
\text{yhw}(12*\text{ndrives}+7) &= \text{yhw}(12*\text{ndrives}+1) \\
\text{zhw}(12*\text{ndrives}+7) &= \text{zhw}(12*\text{ndrives}+1)
\end{align*}
\]

\[
\begin{align*}
\text{xhw}(12*\text{ndrives}+8) &= \text{xhw}(12*\text{ndrives}+7) \\
\text{yhw}(12*\text{ndrives}+8) &= \text{yhw}(12*\text{ndrives}+7) \\
\text{zhw}(12*\text{ndrives}+8) &= \text{zhw}(12*\text{ndrives}+7) + \text{thfootwall}
\end{align*}
\]
Appendix L - FISH file for FLAC model

```
xhw(12*ndrives+9)=xhw(12*ndrives+8)
yhw(12*ndrives+9)=yhw(12*ndrives+8)+(thbottomhalfdrive/sin(alpha+beta))*cos(beta)
zhw(12*ndrives+9)=zhw(12*ndrives+8)+(thbottomhalfdrive/sin(alpha+beta))*sin(beta)

xhw(12*ndrives+10)=xhw(12*ndrives+9)
yhw(12*ndrives+10)=yhw(12*ndrives+9)+(thtophalfdrive/sin(alpha+beta))*cos(beta)
zhw(12*ndrives+10)=zhw(12*ndrives+9)+(thtophalfdrive/sin(alpha+beta))*sin(beta)

xhw(12*ndrives+11)=xhw(12*ndrives+10)
yhw(12*ndrives+11)=yhw(12*ndrives+10)+(throofcoal/sin(alpha+beta))*cos(beta)
zhw(12*ndrives+11)=zhw(12*ndrives+10)+(throofcoal/sin(alpha+beta))*sin(beta)

xhw(12*ndrives+12)=xhw(12*ndrives+11)
yhw(12*ndrives+12)=yhw(12*ndrives+11)+thoverburden/tan(beta)
zhw(12*ndrives+12)=zhw(6)

;##################################################### END OF PLANE II
######################################################

;##################################################### START OF PLANE III
######################################################

; FIRST COLUMN OF COORDINATES ON PLANE III

xbf(1)=0.
ybf(1)=0+(wbench+(1-fbackfill)*ldrive)
zbf(1)=0.

xbf(2)=xbf(1)
ybf(2)=ybf(1)
zbf(2)=zbf(1)+(thfootwall-(ldrive*tan(alpha)))+(fbackfill*ldrive)*tan(alpha)

xbf(3)=xbf(2)
ybf(3)=ybf(2)+thbottomhalfdrive*sin(alpha)
zbf(3)=zbf(2)+thbottomhalfdrive*cos(alpha)

xbf(4)=xbf(3)
ybf(4)=ybf(3)+thtophalfdrive*sin(alpha)
zbf(4)=zbf(3)+thtophalfdrive*cos(alpha)

xbf(5)=xbf(4)
ybf(5)=ybf(4)+throofcoal*sin(alpha)
zbf(5)=zbf(4)+throofcoal*cos(alpha)

xbf(6)=xbf(5)
ybf(6)=ybf(5)+thoverburden*sin(alpha)
zbf(6)=zbf(6)

; SECOND COLUMN OF COORDINATES ON PLANE III

xbf(7)=xbf(1)+0.5*wbarrier
ybf(7)=ybf(1)
zbf(7)=zbf(1)

xbf(8)=xbf(2)+0.5*wbarrier
ybf(8)=ybf(2)
zbf(8)=zbf(2)

xbf(9)=xbf(3)+0.5*wbarrier
ybf(9)=ybf(3)
zbf(9)=zbf(3)

xbf(10)=xbf(4)+0.5*wbarrier
ybf(10)=ybf(4)
zbf(10)=zbf(4)

xbf(11)=xbf(5)+0.5*wbarrier
ybf(11)=ybf(5)
zbf(11)=zbf(5)
```

xbf(12)=xbf(6)+0.5*wbarrier
ybf(12)=ybf(6)
zbf(12)=zbf(6)

; LOOP THAT STARTS WITH THE RIGHT HAND SIDE OF THE DRIVE(i)
i=1
loop while i<= ndrives
  xbf(12*i+1)=xbf(12*i-5)+wdrive
  ybf(12*i+1)=ybf(12*i-5)
  zbf(12*i+1)=zbf(12*i-5)
  xbf(12*i+2)=xbf(12*i+1)
  ybf(12*i+2)=ybf(12*i+1)
  zbf(12*i+2)=zbf(12*i+1)+(thfootwall-(ldrive*tan(alpha)))+(fbackfill*ldrive)*tan(alpha)
  xbf(12*i+3)=xbf(12*i+2)
  ybf(12*i+3)=ybf(12*i+2)+thbottomhalfdrive*sin(alpha)
  zbf(12*i+3)=zbf(12*i+2)+thbottomhalfdrive*cos(alpha)
  xbf(12*i+4)=xbf(12*i+3)
  ybf(12*i+4)=ybf(12*i+3)+thtophalfdrive*sin(alpha)
  zbf(12*i+4)=zbf(12*i+3)+thtophalfdrive*cos(alpha)
  xbf(12*i+5)=xbf(12*i+4)
  ybf(12*i+5)=ybf(12*i+4)+throofcoal*sin(alpha)
  zbf(12*i+5)=zbf(12*i+4)+throofcoal*cos(alpha)
  xbf(12*i+6)=xbf(12*i+5)
  ybf(12*i+6)=ybf(12*i+5)+thoverburden*sin(alpha)
  zbf(12*i+6)=zbf(6)
end_loop

; RIGHT SIDE OF THE PILLAR(i)
xbf(12*i+7)=xbf(12*i+1)+wpillar
ybf(12*i+7)=ybf(12*i+1)
zbiff(12*i+7)=zbiff(12*i+1)
xbf(12*i+8)=xbf(12*i+7)
ybf(12*i+8)=ybf(12*i+7)
zbiff(12*i+8)=zbiff(12*i+7)+(thfootwall-(ldrive*tan(alpha)))+(fbackfill*ldrive)*tan(alpha)
xbf(12*i+9)=xbf(12*i+8)
ybf(12*i+9)=ybf(12*i+8)+thbottomhalfdrive*sin(alpha)
zbiff(12*i+9)=zbiff(12*i+8)+thbottomhalfdrive*cos(alpha)
xbf(12*i+10)=xbf(12*i+9)
ybf(12*i+10)=ybf(12*i+9)+thtophalfdrive*sin(alpha)
zbiff(12*i+10)=zbiff(12*i+9)+thtophalfdrive*cos(alpha)
xbf(12*i+11)=xbf(12*i+10)
ybf(12*i+11)=ybf(12*i+10)+throofcoal*sin(alpha)
zbiff(12*i+11)=zbiff(12*i+10)+throofcoal*cos(alpha)
xBF(12*i+12)=xBF(12*i+11)
ybf(12*i+12)=ybf(12*i+11)+thoverburden*sin(alpha)
zbiff(12*i+12)=zbiff(6)

; i INCREASES BY 1
i=i+1
end_loop

; RIGHT HAND SIDE OF THE LAST DRIVE
xbf(12*ndrives+1)=xbf(12*ndrives-5)+wdrive
ybf(12*ndrives+1)=ybf(12*ndrives-5)
zbf(12*ndrives+1)=zbf(12*ndrives-5)
xbf(12*ndrives+2)=xbf(12*ndrives+1)
ybf(12*ndrives+2)=ybf(12*ndrives+1)
zbf(12*ndrives+2)=zbf(12*ndrives+1)+(thfootwall-(ldrive*tan(alpha)))+(fbackfill*ldrive)*tan(alpha)
Appendix L - FISH file for FLAC model

\[\text{xbf}(12*\text{ndrives}+3) = \text{xbf}(12*\text{ndrives}+2)\]
\[\text{ybf}(12*\text{ndrives}+3) = \text{ybf}(12*\text{ndrives}+2) + \text{thbottomhalfdrive}*\sin(\alpha)\]
\[\text{zbf}(12*\text{ndrives}+3) = \text{zbf}(12*\text{ndrives}+2) + \text{thbottomhalfdrive}*\cos(\alpha)\]

\[\text{xbf}(12*\text{ndrives}+4) = \text{xbf}(12*\text{ndrives}+3)\]
\[\text{ybf}(12*\text{ndrives}+4) = \text{ybf}(12*\text{ndrives}+3) + \text{thtophalfdrive}*\sin(\alpha)\]
\[\text{zbf}(12*\text{ndrives}+4) = \text{zbf}(12*\text{ndrives}+3) + \text{thtophalfdrive}*\cos(\alpha)\]

\[\text{xbf}(12*\text{ndrives}+5) = \text{xbf}(12*\text{ndrives}+4)\]
\[\text{ybf}(12*\text{ndrives}+5) = \text{ybf}(12*\text{ndrives}+4) + \text{throofcoal}*\sin(\alpha)\]
\[\text{zbf}(12*\text{ndrives}+5) = \text{zbf}(12*\text{ndrives}+4) + \text{throofcoal}*\cos(\alpha)\]

\[\text{xbf}(12*\text{ndrives}+6) = \text{xbf}(12*\text{ndrives}+5)\]
\[\text{ybf}(12*\text{ndrives}+6) = \text{ybf}(12*\text{ndrives}+5) + \text{thoverburden}*\sin(\alpha)\]
\[\text{zbf}(12*\text{ndrives}+6) = \text{zbf}(6)\]

; RIGHT HAND SIDE OF THE WHOLE MODEL (HALFWAY THROUGH RIGHT SIDE BARRIER PILLAR)

\[\text{xbf}(12*\text{ndrives}+7) = \text{xbf}(12*\text{ndrives}+1) + 0.5*\text{wbarrier}\]
\[\text{ybf}(12*\text{ndrives}+7) = \text{ybf}(12*\text{ndrives}+1)\]
\[\text{zbf}(12*\text{ndrives}+7) = \text{zbf}(12*\text{ndrives}+1)\]

\[\text{xbf}(12*\text{ndrives}+8) = \text{xbf}(12*\text{ndrives}+7)\]
\[\text{ybf}(12*\text{ndrives}+8) = \text{ybf}(12*\text{ndrives}+7)\]
\[\text{zbf}(12*\text{ndrives}+8) = \text{zbf}(12*\text{ndrives}+7) + (\text{thfootwall} - (\text{ldrive}*\tan(\alpha))) + (\text{fbackfill} + \text{ldrive})*\tan(\alpha)\]

\[\text{xbf}(12*\text{ndrives}+9) = \text{xbf}(12*\text{ndrives}+8)\]
\[\text{ybf}(12*\text{ndrives}+9) = \text{ybf}(12*\text{ndrives}+8) + \text{thbottomhalfdrive}*\sin(\alpha)\]
\[\text{zbf}(12*\text{ndrives}+9) = \text{zbf}(12*\text{ndrives}+8) + \text{thbottomhalfdrive}*\cos(\alpha)\]

\[\text{xbf}(12*\text{ndrives}+10) = \text{xbf}(12*\text{ndrives}+9)\]
\[\text{ybf}(12*\text{ndrives}+10) = \text{ybf}(12*\text{ndrives}+9) + \text{thtophalfdrive}*\sin(\alpha)\]
\[\text{zbf}(12*\text{ndrives}+10) = \text{zbf}(12*\text{ndrives}+9) + \text{thtophalfdrive}*\cos(\alpha)\]

\[\text{xbf}(12*\text{ndrives}+11) = \text{xbf}(12*\text{ndrives}+10)\]
\[\text{ybf}(12*\text{ndrives}+11) = \text{ybf}(12*\text{ndrives}+10) + \text{throofcoal}*\sin(\alpha)\]
\[\text{zbf}(12*\text{ndrives}+11) = \text{zbf}(12*\text{ndrives}+10) + \text{throofcoal}*\cos(\alpha)\]

\[\text{xbf}(12*\text{ndrives}+12) = \text{xbf}(12*\text{ndrives}+11)\]
\[\text{ybf}(12*\text{ndrives}+12) = \text{ybf}(12*\text{ndrives}+11) + \text{thoverburden}*\sin(\alpha)\]
\[\text{zbf}(12*\text{ndrives}+12) = \text{zbf}(6)\]

;########################################################################### END OF PLANE III

;########################################################################### START OF PLANE IV

; FIRST COLUMN OF COORDINATES ON PLANE IV

\text{xeod}(1)=0.\]
\text{yeod}(1)=0 + (\text{wbench} + \text{ldrive})\]
\text{zeod}(1)=0.\]

\text{xeod}(2) = \text{xeod}(1)\]
\text{yeod}(2) = \text{yeod}(1)\]
\text{zeod}(2) = \text{zeod}(1) + (\text{thfootwall} - (\text{ldrive})*\tan(\alpha))\]

\text{xeod}(3) = \text{xeod}(2)\]
\text{yeod}(3) = \text{yeod}(2)\]
\text{zeod}(3) = \text{zeod}(2) + \text{thbottomhalfdrive}/\cos(\alpha)\]

\text{xeod}(4) = \text{xeod}(3)\]
\text{yeod}(4) = \text{yeod}(3)\]
\text{zeod}(4) = \text{zeod}(3) + \text{thtophalfdrive}/\cos(\alpha)\]

\text{xeod}(5) = \text{xeod}(4)\]
\text{yeod}(5) = \text{yeod}(4)\]
zeod(5)=zeod(4)+throofcoal/cos(alpha)

xeod(6)=xeod(5)
yeod(6)=yeod(5)
zeod(6)=z(6)

; SECOND COLUMN OF COORDINATES ON PLANE IV

xeod(7)=xeod(1)+0.5*wbarrier
yeod(7)=yeod(1)
zeod(7)=zeod(1)

xeod(8)=xeod(2)+0.5*wbarrier
yeod(8)=yeod(2)
zeod(8)=zeod(2)

xeod(9)=xeod(3)+0.5*wbarrier
yeod(9)=yeod(3)
zeod(9)=zeod(3)

xeod(10)=xeod(4)+0.5*wbarrier
yeod(10)=yeod(4)
zeod(10)=zeod(4)

xeod(11)=xeod(5)+0.5*wbarrier
yeod(11)=yeod(5)
zeod(11)=zeod(5)

xeod(12)=xeod(6)+0.5*wbarrier
yeod(12)=yeod(6)
zeod(12)=zeod(6)

; LOOP THAT STARTS WITH THE RIGHT HAND SIDE OF THE DRIVE(i)

i=1
loop while i<= ndrives
  xecd(12*i+1)=xeod(12*i-5)+wdrive
  yeod(12*i+1)=yeod(12*i-5)
  zeod(12*i+1)=zeod(12*i-5)

  xecd(12*i+2)=xeod(12*i+1)
  yeod(12*i+2)=yeod(12*i+1)
  zeod(12*i+2)=zeod(12*i+1)+(thfootwall-(ldrive*tan(alpha)))

  xecd(12*i+3)=xeod(12*i+2)
  yeod(12*i+3)=yeod(12*i+2)
  zeod(12*i+3)=zeod(12*i+2)+thbottomhalfdrive/cos(alpha)

  xecd(12*i+4)=xeod(12*i+3)
  yeod(12*i+4)=yeod(12*i+3)
  zeod(12*i+4)=zeod(12*i+3)+thtophalfdrive/cos(alpha)

  xecd(12*i+5)=xeod(12*i+4)
  yeod(12*i+5)=yeod(12*i+4)
  zeod(12*i+5)=zeod(12*i+4)+throofcoal/cos(alpha)

  xecd(12*i+6)=xeod(12*i+5)
  yeod(12*i+6)=yeod(12*i+5)
  zeod(12*i+6)=zeod(6)

; RIGHT SIDE OF THE PILLAR(i)

xeod(12*i+7)=xeod(12*i+1)+wpillar
yeod(12*i+7)=yeod(12*i+1)
zeod(12*i+7)=zeod(12*i+1)

xeod(12*i+8)=xeod(12*i+7)
yeod(12*i+8)=yeod(12*i+7)
zeod(12*i+8)=zeod(12*i+7)+(thfootwall-(ldrive*tan(alpha)))

xeod(12*i+9)=xeod(12*i+8)
yeod(12*i+9)=yeod(12*i+8)
zeod(12*i+9)=zeod(12*i+8)+thbottomhalfdrive/cos(alpha)
Appendix L - FISH file for FLAC model

```
xecd(12*i+10)=xeod(12*i+9)
yecd(12*i+10)=yecd(12*i+9)
zedc(12*i+10)=zedc(12*i+9)+thtophalfdrive/cos(alpha)

xeod(12*i+11)=xeod(12*i+10)
yecd(12*i+11)=yecd(12*i+10)
zedc(12*i+11)=zedc(12*i+10)+throofcoal/cos(alpha)

xeod(12*i+12)=xeod(12*i+11)
yecd(12*i+12)=yecd(12*i+11)
zedc(12*i+12)=zedc(6)

; i INCREASES BY 1

i=i+1
end_loop

; RIGHT HAND SIDE OF THE LAST DRIVE

xeod(12*ndrives+1)=xeod(12*ndrives-5)+wdrive
yecd(12*ndrives+1)=yecd(12*ndrives-5)
zedc(12*ndrives+1)=zedc(12*ndrives-5)

xeod(12*ndrives+2)=xeod(12*ndrives+1)
yecd(12*ndrives+2)=yecd(12*ndrives+1)
zedc(12*ndrives+2)=zedc(12*ndrives+1)+(thfootwall-(ldrive*tan(alpha)))

xeod(12*ndrives+3)=xeod(12*ndrives+2)
yecd(12*ndrives+3)=yecd(12*ndrives+2)
zedc(12*ndrives+3)=zedc(12*ndrives+2)+thbottomhalfdrive/cos(alpha)

xeod(12*ndrives+4)=xeod(12*ndrives+3)
yecd(12*ndrives+4)=yecd(12*ndrives+3)
zedc(12*ndrives+4)=zedc(12*ndrives+3)+thtophalfdrive/cos(alpha)

xeod(12*ndrives+5)=xeod(12*ndrives+4)
yecd(12*ndrives+5)=yecd(12*ndrives+4)
zedc(12*ndrives+5)=zedc(12*ndrives+4)+throofcoal/cos(alpha)

xeod(12*ndrives+6)=xeod(12*ndrives+5)
yecd(12*ndrives+6)=yecd(12*ndrives+5)
zedc(12*ndrives+6)=zedc(6)

; RIGHT HAND SIDE OF THE WHOLE MODEL (HALFWAY THROUGH RIGHT SIDE BARRIER PILLAR)

xeod(12*ndrives+7)=xeod(12*ndrives+1)+0.5*wbarrier
yecd(12*ndrives+7)=yecd(12*ndrives+1)
zedc(12*ndrives+7)=zedc(12*ndrives+1)

xeod(12*ndrives+8)=xeod(12*ndrives+7)
yecd(12*ndrives+8)=yecd(12*ndrives+7)
zedc(12*ndrives+8)=zedc(12*ndrives+7)+(thfootwall-(ldrive*tan(alpha)))

xeod(12*ndrives+9)=xeod(12*ndrives+8)
yecd(12*ndrives+9)=yecd(12*ndrives+8)
zedc(12*ndrives+9)=zedc(12*ndrives+8)+thbottomhalfdrive/cos(alpha)

xeod(12*ndrives+10)=xeod(12*ndrives+9)
yecd(12*ndrives+10)=yecd(12*ndrives+9)
zedc(12*ndrives+10)=zedc(12*ndrives+9)+thtophalfdrive/cos(alpha)

xeod(12*ndrives+11)=xeod(12*ndrives+10)
yecd(12*ndrives+11)=yecd(12*ndrives+10)
zedc(12*ndrives+11)=zedc(12*ndrives+10)+throofcoal/cos(alpha)

xeod(12*ndrives+12)=xeod(12*ndrives+11)
yecd(12*ndrives+12)=yecd(12*ndrives+11)
zedc(12*ndrives+12)=zedc(6)
```
Appendix L - FISH file for FLAC model

;############################################################################### END OF PLANE IV
;###############################################################################

;############################################################################### START OF PLANE V
;###############################################################################
; FIRST COLUMN OF COORDINATES ON PLANE V

xv(1)=0.
yv(1)=0+(wbench+ldrive+lvirgin)
zv(1)=0.

xv(2)=xv(1)
yv(2)=yv(1)
zv(2)=zv(1)+((thfootwall-((ldrive+lvirgin)*tan(alpha))))

xv(3)=xv(2)
yv(3)=yv(2)
zv(3)=zv(2)+thbottomhalfdrive/cos(alpha)

xv(4)=xv(3)
yv(4)=yv(3)
zv(4)=zv(3)+thtophalfdrive/cos(alpha)

xv(5)=xv(4)
yv(5)=yv(4)
zv(5)=zv(4)+throofcoal/cos(alpha)

xv(6)=xv(5)
yv(6)=yv(5)
zv(6)=z(6)

; SECOND COLUMN OF COORDINATES ON PLANE V

xv(7)=xv(1)+0.5*wbarrier
yv(7)=yv(1)
zv(7)=zv(1)

xv(8)=xv(2)+0.5*wbarrier
yv(8)=yv(2)
zv(8)=zv(2)

xv(9)=xv(3)+0.5*wbarrier
yv(9)=yv(3)
zv(9)=zv(3)

xv(10)=xv(4)+0.5*wbarrier
yv(10)=yv(4)
zv(10)=zv(4)

xv(11)=xv(5)+0.5*wbarrier
yv(11)=yv(5)
zv(11)=zv(5)

xv(12)=xv(6)+0.5*wbarrier
yv(12)=yv(6)
zv(12)=zv(6)

; LOOP THAT STARTS WITH THE RIGHT HAND SIDE OF THE DRIVE(i)

i=1
loop while i<= ndrives
xv(12*i+1)=xv(12*i-5)+wdrive
yv(12*i+1)=yv(12*i-5)
zv(12*i+1)=zv(12*i-5)

xv(12*i+2)=xv(12*i+1)
yv(12*i+2)=yv(12*i+1)
zv(12*i+2)=zv(12*i+1)+((thfootwall-((ldrive+lvirgin)*tan(alpha))))

xv(12*i+3)=xv(12*i+2)

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yv(12*i+3)=yv(12*i+2)
zv(12*i+3)=zv(12*i+2)+thbottomhalfdrive/cos(alpha)

xv(12*i+4)=xv(12*i+3)
yv(12*i+4)=yv(12*i+3)
zv(12*i+4)=zv(12*i+3)+thtophalfdrive/cos(alpha)

xv(12*i+5)=xv(12*i+4)
yv(12*i+5)=yv(12*i+4)
zv(12*i+5)=zv(12*i+4)+throofcoal/cos(alpha)

xv(12*i+6)=xv(12*i+5)
yv(12*i+6)=yv(12*i+5)
zv(12*i+6)=zv(6)

; RIGHT SIDE OF THE PILLAR(i)

xv(12*i+7)=xv(12*i+1)+wpillar
yv(12*i+7)=yv(12*i+1)
zv(12*i+7)=zv(12*i+1)

xv(12*i+8)=xv(12*i+7)
yv(12*i+8)=yv(12*i+7)
zv(12*i+8)=zv(12*i+7)+{thfootwall-((ldrive+lvirgin)*tan(alpha))}

xv(12*i+9)=xv(12*i+8)
yv(12*i+9)=yv(12*i+8)
zv(12*i+9)=zv(12*i+8)+thbottomhalfdrive/cos(alpha)

xv(12*i+10)=xv(12*i+9)
yv(12*i+10)=yv(12*i+9)
zv(12*i+10)=zv(12*i+9)+thtophalfdrive/cos(alpha)

xv(12*i+11)=xv(12*i+10)
yv(12*i+11)=yv(12*i+10)
zv(12*i+11)=zv(12*i+10)+throofcoal/cos(alpha)

xv(12*i+12)=xv(12*i+11)
yv(12*i+12)=yv(12*i+11)
zv(12*i+12)=zv(6)

; i INCREASES BY 1

i=i+1
end_loop

; RIGHT HAND SIDE OF THE LAST DRIVE

xv(12*ndrives+1)=xv(12*ndrives-5)+wdrive
yv(12*ndrives+1)=yv(12*ndrives-5)
zv(12*ndrives+1)=zv(12*ndrives-5)

xv(12*ndrives+2)=xv(12*ndrives+1)
yv(12*ndrives+2)=yv(12*ndrives+1)
zv(12*ndrives+2)=zv(12*ndrives+1)+{thfootwall-((ldrive+lvirgin)*tan(alpha))}

xv(12*ndrives+3)=xv(12*ndrives+2)
yv(12*ndrives+3)=yv(12*ndrives+2)
zv(12*ndrives+3)=zv(12*ndrives+2)+thbottomhalfdrive/cos(alpha)

xv(12*ndrives+4)=xv(12*ndrives+3)
yv(12*ndrives+4)=yv(12*ndrives+3)
zv(12*ndrives+4)=zv(12*ndrives+3)+thtophalfdrive/cos(alpha)

xv(12*ndrives+5)=xv(12*ndrives+4)
yv(12*ndrives+5)=yv(12*ndrives+4)
zv(12*ndrives+5)=zv(12*ndrives+4)+throofcoal/cos(alpha)

xv(12*ndrives+6)=xv(12*ndrives+5)
yv(12*ndrives+6)=yv(12*ndrives+5)
zv(12*ndrives+6)=zv(6)
Appendix L - FISH file for FLAC model

; RIGHT HAND SIDE OF THE MODEL (HALF OF RIGHT-HAND BARRIER PILLAR)

xv(12*ndrives+7)=xv(12*ndrives+1)+0.5*wbarrier
yv(12*ndrives+7)=yv(12*ndrives+1)
zv(12*ndrives+7)=zv(12*ndrives+1)

xv(12*ndrives+8)=xv(12*ndrives+7)
yv(12*ndrives+8)=yv(12*ndrives+7)
zv(12*ndrives+8)=zv(12*ndrives+7)+(thfootwall-((ldrive+1*virgin)*tan(alpha)))

xv(12*ndrives+9)=xv(12*ndrives+8)
yv(12*ndrives+9)=yv(12*ndrives+8)
zv(12*ndrives+9)=zv(12*ndrives+8)+thbottomhalfdrive/cos(alpha)

xv(12*ndrives+10)=xv(12*ndrives+9)
yv(12*ndrives+10)=yv(12*ndrives+9)
zv(12*ndrives+10)=zv(12*ndrives+9)+thtophalfdrive/cos(alpha)

xv(12*ndrives+11)=xv(12*ndrives+10)
yv(12*ndrives+11)=yv(12*ndrives+10)
zv(12*ndrives+11)=zv(12*ndrives+10)+throofcoal/cos(alpha)

xv(12*ndrives+12)=xv(12*ndrives+11)
yv(12*ndrives+12)=yv(12*ndrives+11)
zv(12*ndrives+12)=zv(6)

;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;

; ####### DETERMINE BOUNDARIES ##############
xminleft = x(1) - 0.01
xminright = x(1) + 0.01
xmaxleft = x((12*ndrives)+8) - 0.01
xmaxright = x((12*ndrives)+8) + 0.01
yminfront = y(1) - 0.01
yminbehind = y(1) + 0.01
ymaxfront = yv(5) - 0.01
ymaxbehind = yv(5) + 0.01
zminunder = z(1) - 0.01
zminabove = z(1) + 0.01
; ####### END DETERMINE BOUNDARIES ##############

end

;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;

; ##### DEFINE ALL GROUPS ######
; ######
; ######
;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;

def gen_one_brick
(n1_,n2_,n3_,r1_,r2_,r3_,x0_,y0_,z0_,x1_,y1_,z1_,x2_,y2_,z2_,x3_,y3_,z3_,x4_,y4_,z4_,x5_,y5_,z5_,x6_,y6_,z6_,x7_,y7_,z7_,group_)

global gn1 = n1_
global gn2 = n2_
global gn3 = n3_
global gr1 = r1_
global gr2 = r2_
global gr3 = r3_
global gx0 = x0_
global gy0 = y0_
global gz0 = z0_
global gx1 = x1_
global gy1 = y1_
global gz1 = z1_
global gx2 = x2_
global gy2 = y2_
global gz2 = z2_

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global gx3 = x3_
global gy3 = y3_
global gz3 = z3_
global gx4 = x4_
global gy4 = y4_
global gz4 = z4_
global gx5 = x5_
global gy5 = y5_
global gz5 = z5_
global gx6 = x6_
global gy6 = y6_
global gz6 = z6_
global gx7 = x7_
global gy7 = y7_
global gz7 = z7_
global ggroup = group_

command
  generate zone brick size @gn1 @gn2 @gn3 ratio @gr1 @gr2 @gr3 p0 @gx0 @gy0 @gz0 p1 @gx1 @gy1 @gr1 p2 @gx2 @gy2 @gr2 p3 @gx3 @gy3 @gr3 p4 @gx4 @gy4 @gr4 p5 @gx5 @gy5 @gr5 p6 @gx6 @gy6 @gr6 p7 @gx7 @gy7 @gr7 group @ggroup
end_command
end

def gen_one_wedge
  (n1_, n2_, n3_, r1_, r2_, r3_, x0_, y0_, z0_, x1_, y1_, z1_, x2_, y2_, z2_, x3_, y3_, z3_, x4_, y4_, z4_, x5_, y5_, z5_, gro
  up_)
  
global gn1 = n1_
global gn2 = n2_
global gn3 = n3_
global gr1 = r1_
global gr2 = r2_
global gr3 = r3_
global gx0 = x0_
global gy0 = y0_
global gz0 = z0_
global gx1 = x1_
global gy1 = y1_
global gz1 = z1_
global gx2 = x2_
global gy2 = y2_
global gz2 = z2_
global gx3 = x3_
global gy3 = y3_
global gz3 = z3_
global gx4 = x4_
global gy4 = y4_
global gz4 = z4_
global gx5 = x5_
global gy5 = y5_
global gz5 = z5_
global ggroup = group_

command
  generate zone uwedge size @gn1 @gn2 @gn3 ratio @gr1 @gr2 @gr3 p0 @gx0 @gy0 @gz0 p1 @gx1 @gy1 @gr1 p2 @gx2 @gy2 @gr2 p3 @gx3 @gy3 @gr3 p4 @gx4 @gy4 @gr4 p5 @gx5 @gy5 @gr5 group @ggroup
end_command
end

def gen_groups

; #define footwall group #define footwall group
;

; #define left barrier pillar #define left barrier pillar

rx = rwbarrier
nx = nwbarrier/2

gen_one_brick(nx, nwbench, nthfootwall, rx, rwbench, rthfootwall, x(1), y(1), z(1), x(7), y(7), z(7), xhw(1), yhw(1), zhw(1), x(12*ndrives+13), y(12*ndrives+13), z(12*ndrives+13), xhw(7), yhw(7), zhw
Appendix L - FISH file for FLAC model

\[ (((2*i-1)*6)+8), zhw(((2*i-1)*6)+8), x(12*ndrives+(14+(2*i))), y(12*ndrives+(14+(2*i))), z(12*ndrives+(14+(2*i))) \]
\[ xhw(((2*i-1)*6)+14), yhw(((2*i-1)*6)+14), zhw(((2*i-1)*6)+14), 'footwall' \]

```plaintext
; gen_one_brick(nx,nlnonbackfill,nthfootwall,rx,rlonbackfill,rthfootwall,xhw((2*i-1) *6)+7), yhw(((2*i-1)*6)+7), zhw(((2*i-1)*6)+7), x((12*ndrives+1)*14), y((12*ndrives+1)*14), z((12*ndrives+1)*14), xhw((2*i-1) *6)+8), yhw(((2*i-1)*6)+8), zhw(((2*i-1)*6)+8), 'footwall'
; gen_one_brick(nx,nlnonbackfill,nthfootwall,rx,rlonbackfill,rthfootwall,xhw(((2*i-1)*6)+7), yhw(((2*i-1)*6)+7), zhw(((2*i-1)*6)+7), x((12*ndrives+1)*14), y((12*ndrives+1)*14), z((12*ndrives+1)*14), xhw((2*i-1) *6)+8), yhw(((2*i-1)*6)+8), zhw(((2*i-1)*6)+8), 'footwall'
; gen_one_brick(nx,nlnonbackfill,nthfootwall,rx,rlonbackfill,rthfootwall,xhw(((2*i-1)*6)+7), yhw(((2*i-1)*6)+7), zhw(((2*i-1)*6)+7), x((12*ndrives+1)*14), y((12*ndrives+1)*14), z((12*ndrives+1)*14), xhw((2*i-1) *6)+8), yhw(((2*i-1)*6)+8), zhw(((2*i-1)*6)+8), 'footwall'
```

```plaintext
i=i+1
end_loop
```

; ###### END OF LOOP ######

; ####### RIGHT SIDE BARRIER PILLAR #######

```plaintext
rx = 1/rwbarrier
nx = nbARRIER/2
```

```plaintext
; gen_one_brick(nx,nwbench,nthfootwall,rx,rbwbench,rthfootwall,x(12*ndrives+1), y(12*ndrives+1), z(12*ndrives+1), xhw((2*i-1)*6)+7), yhw(((2*i-1)*6)+7), zhw(((2*i-1)*6)+7), x((12*ndrives+1)*14), y((12*ndrives+1)*14), z((12*ndrives+1)*14), xhw((2*i-1) *6)+8), yhw(((2*i-1)*6)+8), zhw(((2*i-1)*6)+8), 'footwall'
; gen_one_brick(nx,nlnonbackfill,nthfootwall,rx,rlonbackfill,rthfootwall,xhw((2*i-1)*6)+7), yhw(((2*i-1)*6)+7), zhw(((2*i-1)*6)+7), x((12*ndrives+1)*14), y((12*ndrives+1)*14), z((12*ndrives+1)*14), xhw((2*i-1) *6)+8), yhw(((2*i-1)*6)+8), zhw(((2*i-1)*6)+8), 'footwall'
; gen_one_brick(nx,nlnonbackfill,nthfootwall,rx,rlonbackfill,rthfootwall,xhw(((2*i-1)*6)+7), yhw(((2*i-1)*6)+7), zhw(((2*i-1)*6)+7), x((12*ndrives+1)*14), y((12*ndrives+1)*14), z((12*ndrives+1)*14), xhw(((2*i-1) *6)+8), yhw(((2*i-1)*6)+8), zhw(((2*i-1)*6)+8), 'footwall'
; gen_one_brick(nx,nlnonbackfill,nthfootwall,rx,rlonbackfill,rthfootwall,xhw(((2*i-1)*6)+7), yhw(((2*i-1)*6)+7), zhw(((2*i-1)*6)+7), x((12*ndrives+1)*14), y((12*ndrives+1)*14), z((12*ndrives+1)*14), xhw(((2*i-1) *6)+8), yhw(((2*i-1)*6)+8), zhw(((2*i-1)*6)+8), 'footwall'
```

```plaintext
; gen_one_brick(nx,nlnonbackfill,nthfootwall,rx,rlonbackfill,rthfootwall,xhw(((2*i-1)*6)+7), yhw(((2*i-1)*6)+7), zhw(((2*i-1)*6)+7), x((12*ndrives+1)*14), y((12*ndrives+1)*14), z((12*ndrives+1)*14), xhw(((2*i-1) *6)+8), yhw(((2*i-1)*6)+8), zhw(((2*i-1)*6)+8), 'footwall'
; gen_one_brick(nx,nlnonbackfill,nthfootwall,rx,rlonbackfill,rthfootwall,xhw(((2*i-1)*6)+7), yhw(((2*i-1)*6)+7), zhw(((2*i-1)*6)+7), x((12*ndrives+1)*14), y((12*ndrives+1)*14), z((12*ndrives+1)*14), xhw(((2*i-1) *6)+8), yhw(((2*i-1)*6)+8), zhw(((2*i-1)*6)+8), 'footwall'
; gen_one_brick(nx,nlnonbackfill,nthfootwall,rx,rlonbackfill,rthfootwall,xhw(((2*i-1)*6)+7), yhw(((2*i-1)*6)+7), zhw(((2*i-1)*6)+7), x((12*ndrives+1)*14), y((12*ndrives+1)*14), z((12*ndrives+1)*14), xhw(((2*i-1) *6)+8), yhw(((2*i-1)*6)+8), zhw(((2*i-1)*6)+8), 'footwall'
```

```plaintext
; gen_one_brick(nx,nlnonbackfill,nthfootwall,rx,rlonbackfill,rthfootwall,xhw(((2*i-1)*6)+7), yhw(((2*i-1)*6)+7), zhw(((2*i-1)*6)+7), x((12*ndrives+1)*14), y((12*ndrives+1)*14), z((12*ndrives+1)*14), xhw(((2*i-1) *6)+8), yhw(((2*i-1)*6)+8), zhw(((2*i-1)*6)+8), 'footwall'
```

```plaintext
i=i+1
end_loop
```

; ###### END OF RIGHT SIDE BARRIER PILLAR ######

; ####################################################################

; #### END OF FOOTWALL GROUP ######
Appendix L - FISH file for FLAC model

; ###########################################################################
; ###### LEFT BARRIER PILLAR ######

rx = 1/rwbarrier
nx = nwbarrier/2
r wedge = 1/rwbench

generate_wedge(nwbench,nx,nwbench,rwbench,rx,rwedge,x(12*ndrives+14),y(12*ndrives+14),z(12*ndrives+14),xhw(8),yhw(8),zkw(8),x(12*ndrives+13),y(12*ndrives+13),z(12*ndrives+13),x(8),y(8),z(8),xhw(2),yhw(2),zkw(2),x(2),y(2),z(2),'trench_wedge')

; ###### END OF LEFT BARRIER PILLAR ######

; ###### NOW THE LOOP STARTING WITH DRIVE 1 ######

; ### FIRST FOR 'DRIVES' ###

i=1
loop while i <= ndrives
rx = 1/rwdrive
nx = nwdrive

generate_wedge(nwbench,nx,nwbench,rwbench,rx,rwedge,x(12*ndrives+(13+(2*i))),y(12*ndrives+(13+(2*i))),z(12*ndrives+(13+(2*i))),xhw(((2*i)-1)*6)+8),yhw(((2*i)-1)*6)+8),zkw(((2*i)-1)*6)+8),x(12*ndrives+(12+(2*i))),y(12*ndrives+(12+(2*i))),z(12*ndrives+(12+(2*i))),x(((2*i)-1)*6)+8),y(((2*i)-1)*6)+8),z(((2*i)-1)*6)+8),xhw(((2*i)-1)*6)+2),yhw(((2*i)-1)*6)+2),z(((2*i)-1)*6)+2),'trench_wedge')

i=i+1
end_loop

; ### NOW FOR 'PILLARS' ###

i=1
loop while i < ndrives
rx = 1/rwpillar
nx = npillar

generate_wedge(nwbench,nx,nwbench,rwbench,rx,rwedge,x(12*ndrives+(14+(2*i))),y(12*ndrives+(14+(2*i))),z(12*ndrives+(14+(2*i))),xhw(((2*i)-1)*6)+14),yhw(((2*i)-1)*6)+14),zkw(((2*i)-1)*6)+14),x(12*ndrives+(13+(2*i))),y(12*ndrives+(13+(2*i))),z(12*ndrives+(13+(2*i))),x(((2*i)-1)*6)+14),y(((2*i)-1)*6)+14),z(((2*i)-1)*6)+14),xkw(((2*i)-1)*6)+8),ykw(((2*i)-1)*6)+8),zkw(((2*i)-1)*6)+8),x((12*ndrives+13+(2*i))),y((12*ndrives+13+(2*i))),z((12*ndrives+13+(2*i))),'trench_wedge')

i=i+1
end_loop

; ###### END OF LOOP ######

; ###### RIGHT SIDE BARRIER PILLAR ######

rx = rwbarrier
nx = nwbarrier/2

; generate_wedge(nwbench,nx,nwbench,rwbench,rx,rwedge,x(14*ndrives+14),y(14*ndrives+14),z(14*ndrives+14),xhw(12*ndrives+8),yhw(12*ndrives+8),zkw(12*ndrives+8),x(14*ndrives+13),y(14*ndrives+13),z(14*ndrives+13),x(12*ndrives+8),y(12*ndrives+8),z(12*ndrives+8),xkw(12*ndrives+2),ykw(12*ndrives+2),zkw(12*ndrives+2),x((12*ndrives+2)),y((12*ndrives+2)),z((12*ndrives+2)),'trench_wedge')

; ###### END OF RIGHT SIDE BARRIER PILLAR ######

; ###########################################################################
; ###########################################################################

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```plaintext
; #%%%%%%%%%%%%%%%%%%%%%%%%%%
; ###### CONTINUE W/ BOTTOM HALF DRIVE ######
; #%%%%%%%%%%%%%%%%%%%%%%%%%%

; ###### LEFT BARRIER PILLAR ######

rx = rwbarrier
nx = nwbarrier/2

gen_one_brick(nx,nwbench,nthbottomhalfdrive,rx,rxbench,rthbottomhalfdrive,x(2),y(2),z(2),x(8),y(8),z(8),xhw(2),yhw(2),zhw(2),x(3),y(3),z(3),xhw(8),yhw(8),zhw(8),xhw(3),yhw(3),zhw(3),x(9),y(9),z(9),xhw(9),yhw(9),zhw(9),'trench_bottomhalf')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(2),yhw(2),zhw(8),xhf(2),yhf(2),zhf(2),xhw(3),yhw(3),zhw(3),xhf(3),yhf(3),zhf(3),'barrier1')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(2),yhw(8),zhw(8),xhf(2),yhf(8),zhf(8),xhf(3),yhf(3),zhf(3),'barrier1')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(8),yhw(8),zhw(8),xhf(8),yhf(8),zhf(8),xhf(3),yhf(3),zhf(3),'barrier1')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(8),yhw(8),zhw(8),xhf(8),yhf(8),zhf(8),xhf(3),yhf(3),zhf(3),'barrier1')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(8),yhw(8),zhw(8),xhf(8),yhf(8),zhf(8),xhf(3),yhf(3),zhf(3),'barrier1')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(8),yhw(8),zhw(8),xhf(8),yhf(8),zhf(8),xhf(3),yhf(3),zhf(3),'barrier1')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(8),yhw(8),zhw(8),xhf(8),yhf(8),zhf(8),xhf(3),yhf(3),zhf(3),'barrier1')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(8),yhw(8),zhw(8),xhf(8),yhf(8),zhf(8),xhf(3),yhf(3),zhf(3),'barrier1')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(8),yhw(8),zhw(8),xhf(8),yhf(8),zhf(8),xhf(3),yhf(3),zhf(3),'barrier1')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(8),yhw(8),zhw(8),xhf(8),yhf(8),zhf(8),xhf(3),yhf(3),zhf(3),'barrier1')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(8),yhw(8),zhw(8),xhf(8),yhf(8),zhf(8),xhf(3),yhf(3),zhf(3),'barrier1')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(8),yhw(8),zhw(8),xhf(8),yhf(8),zhf(8),xhf(3),yhf(3),zhf(3),'barrier1')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(8),yhw(8),zhw(8),xhf(8),yhf(8),zhf(8),xhf(3),yhf(3),zhf(3),'barrier1')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(8),yhw(8),zhw(8),xhf(8),yhf(8),zhf(8),xhf(3),yhf(3),zhf(3),'barrier1')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(8),yhw(8),zhw(8),xhf(8),yhf(8),zhf(8),xhf(3),yhf(3),zhf(3),'barrier1')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(8),yhw(8),zhw(8),xhf(8),yhf(8),zhf(8),xhf(3),yhf(3),zhf(3),'barrier1')

; ###### END OF LEFT BARRIER PILLAR ######

; ###### NOW THE LOOP STARTING WITH DRIVE 1 ######

### FIRST FOR "DRIVES" ###

i=1
loop while i <= ndrives

rx = rwdrive
nx = ndrive

nnameb = 'bottomhalfdriveback' + string(int(i))
nnamef = 'bottomhalfdrivefront' + string(int(i))

gen_one_brick(nx,nwbench,nthbottomhalfdrive,rx,rxbench,rthbottomhalfdrive,x((2*i-1)*6)+2),y(((2*i-1)*6)+2),z(((2*i-1)*6)+2),xhw(((2*i-1)*6)+2),yhw(((2*i-1)*6)+2),zhw(((2*i-1)*6)+2),x((2*i-1)*6)+8),y((2*i-1)*6)+8),z((2*i-1)*6)+8),xhw((2*i-1)*6)+8),yhw((2*i-1)*6)+8),zhw((2*i-1)*6)+8),x((2*i-1)*6)+3),y((2*i-1)*6)+3),z((2*i-1)*6)+3),xhw((2*i-1)*6)+3),yhw((2*i-1)*6)+3),zhw((2*i-1)*6)+3),x((2*i-1)*6)+9),y((2*i-1)*6)+9),z((2*i-1)*6)+9),xhw((2*i-1)*6)+9),yhw((2*i-1)*6)+9),zhw((2*i-1)*6)+9),'trench_bottomhalf')

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(((2*i-1)*6)+2),yhw(((2*i-1)*6)+2),xhw(((2*i-1)*6)+8),yhw(((2*i-1)*6)+8),zhw(((2*i-1)*6)+8),x(((2*i-1)*6)+2),y((2*i-1)*6)+2),z((2*i-1)*6)+2),xhw(((2*i-1)*6)+3),y((2*i-1)*6)+3),z((2*i-1)*6)+3),xhw(((2*i-1)*6)+3),yhw(((2*i-1)*6)+3),zhw(((2*i-1)*6)+3),x((2*i-1)*6)+9),y((2*i-1)*6)+9),z((2*i-1)*6)+9),xhw((2*i-1)*6)+9),yhw((2*i-1)*6)+9),zhw((2*i-1)*6)+9),nnamef)

gen_one_brick(nx,nlnonbackfill,nthbottomhalfdrive,rx,rlnonbackfill,rthbottomhalfdrive,xhw(((2*i-1)*6)+3),yhw(((2*i-1)*6)+3),xhw(((2*i-1)*6)+9),yhw(((2*i-1)*6)+9),zhw(((2*i-1)*6)+9),x(((2*i-1)*6)+2),y((2*i-1)*6)+2),z((2*i-1)*6)+2),xhw(((2*i-1)*6)+8),y((2*i-1)*6)+8),z((2*i-1)*6)+8),xhw(((2*i-1)*6)+3),y((2*i-1)*6)+3),z((2*i-1)*6)+3),xhw(((2*i-1)*6)+3),yhw(((2*i-1)*6)+3),zhw(((2*i-1)*6)+3),x(((2*i-1)*6)+9),y((2*i-1)*6)+9),z((2*i-1)*6)+9),xhw((2*i-1)*6)+9),yhw((2*i-1)*6)+9),zhw((2*i-1)*6)+9),nnamef)

i=i+1
```

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Appendix L - FISH file for FLAC model

end_loop

; ###### NOW FOR "PILLARS" ######

i=1
loop while i < ndrives
rx = rwpillar
nx = nwpillar

gen_one_brick(nx,nwbench,nthbottomhalfdrive,rx,rwbench,rthbottomhalfdrive,x(((2*i-1)*6)+8),y(((2*i-1)*6)+8),z(((2*i-1)*6)+8),x(((2*i-1)*6)+14),y(((2*i-1)*6)+14),z(((2*i-1)*6)+14),xh(((2*i-1)*6)+9),yh(((2*i-1)*6)+9),zh(((2*i-1)*6)+9),x(((2*i-1)*6)+15)/x(((2*i-1)*6)+15),y(((2*i-1)*6)+15),z(((2*i-1)*6)+15),xh(((2*i-1)*6)+15),yh(((2*i-1)*6)+15),zh(((2*i-1)*6)+15),'trench_bottomhalf')

gen_one_brick(nx,nwbench,nthbottomhalfdrive,rx,rwbench,rthbottomhalfdrive,x(((2*i-1)*6)+8),y(((2*i-1)*6)+8),z(((2*i-1)*6)+8),x(((2*i-1)*6)+14),y(((2*i-1)*6)+14),z(((2*i-1)*6)+14),xh(((2*i-1)*6)+9),yh(((2*i-1)*6)+9),zh(((2*i-1)*6)+9),x(((2*i-1)*6)+15)/x(((2*i-1)*6)+15),y(((2*i-1)*6)+15),z(((2*i-1)*6)+15),xh(((2*i-1)*6)+15),yh(((2*i-1)*6)+15),zh(((2*i-1)*6)+15),'trench_bottomhalf')

i=i+1
end_loop

; ###### END OF LOOP ######

; ###### RIGHT SIDE BARRIER PILLAR ######

rx = rwbarrier
nx = nwbarrier

gen_one_brick(nx,nwbench,nthtophalfdrive,rx,rwbench,rthtophalfdrive,x(((2*i-1)*6)+8),y(((2*i-1)*6)+8),z(((2*i-1)*6)+8),x(((2*i-1)*6)+14),y(((2*i-1)*6)+14),z(((2*i-1)*6)+14),xh(((2*i-1)*6)+9),yh(((2*i-1)*6)+9),zh(((2*i-1)*6)+9),x(((2*i-1)*6)+15)/x(((2*i-1)*6)+15),y(((2*i-1)*6)+15),z(((2*i-1)*6)+15),xh(((2*i-1)*6)+15),yh(((2*i-1)*6)+15),zh(((2*i-1)*6)+15),'trench_bottomhalf')

gen_one_brick(nx,nwbench,nthtophalfdrive,rx,rwbench,rthtophalfdrive,x(((2*i-1)*6)+8),y(((2*i-1)*6)+8),z(((2*i-1)*6)+8),x(((2*i-1)*6)+14),y(((2*i-1)*6)+14),z(((2*i-1)*6)+14),xh(((2*i-1)*6)+9),yh(((2*i-1)*6)+9),zh(((2*i-1)*6)+9),x(((2*i-1)*6)+15)/x(((2*i-1)*6)+15),y(((2*i-1)*6)+15),z(((2*i-1)*6)+15),xh(((2*i-1)*6)+15),yh(((2*i-1)*6)+15),zh(((2*i-1)*6)+15),'trench_bottomhalf')

i=i+1
end_loop


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```
; ##### END OF RIGHT SIDE BARRIER PILLAR #####

; ;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;
; ;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;
; ;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;
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; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ;

; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ;

; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ;
```

### FIRST FOR "DRIVES" ###

```bash
i=1
loop while i <= ndrives
  rx = rwdrive
  nx = nwdrive

  nnameb = 'topleveldriveback' + string(int(i))
  nnamef = 'topleveldrivefront' + string(int(i))

  gen_one_brick(nx, nwbond, nt HuffPost drive, rx, rwbond, rth HuffPost drive, xwod (((2*i-1)*6)+3), y (((2*i-1)*6)+3), z (((2*i-1)*6)+3), x(((2*i-1)*6)+9), y(((2*i-1)*6)+9), z(((2*i-1)*6)+9), xwod (((2*i-1)*6)+6), y(((2*i-1)*6)+6), z(((2*i-1)*6)+6), x(((2*i-1)*6)+12), y(((2*i-1)*6)+12), z(((2*i-1)*6)+12), nnamef)

  gen_one_brick(nx, nbackfill, nt HuffPost drive, rx, rbackfill, rth HuffPost drive, xwod (((2*i-1)*6)+3), y(((2*i-1)*6)+3), z(((2*i-1)*6)+3), x(((2*i-1)*6)+9), y(((2*i-1)*6)+9), z(((2*i-1)*6)+9), xwod (((2*i-1)*6)+6), y(((2*i-1)*6)+6), z(((2*i-1)*6)+6), x(((2*i-1)*6)+12), y(((2*i-1)*6)+12), z(((2*i-1)*6)+12), xwod (((2*i-1)*6)+12), y(((2*i-1)*6)+12), z(((2*i-1)*6)+12), nnamef)
```
gen_one_brick(nx, nlvirgin, nthtophalfdrive, rx, rlvirgin, rthtophalfdrive, x((2*i-1)*6)+9), y(((2*i-1)*6)+9), x((2*i-1)*6)+15), y(((2*i-1)*6)+15), z(((2*i-1)*6)+15), xhw(((2*i-1)*6)+10), yhw(((2*i-1)*6)+10), z(((2*i-1)*6)+10), 'virgin')

i=i+1
end_loop

; ###### END OF LOOP ######

; ###### RIGHT SIDE BARRIER PILLAR ######

rx = 1/rwbarrier
nx = nwbarrier/2

gen_one_brick(nx, nwbench, nthtophalfdrive, rx, rwbench, rthtophalfdrive, x((12*ndrives)+3), y((12*ndrives)+3), x((12*ndrives)+9), y((12*ndrives)+9), x((12*ndrives)+15), y((12*ndrives)+15), z(((12*ndrives)+16), xhw(((12*ndrives)+9), yhw(((12*ndrives)+9), xhw(((12*ndrives)+15), yhw(((12*ndrives)+15), xhw(((12*ndrives)+16), yhw(((12*ndrives)+16), xhw(((12*ndrives)+16), yhw(((12*ndrives)+16), 'trench_tophalf')

gen_one_brick(nx, nlnobackfill, nthtophalfdrive, rx, rlnobackfill, rthtophalfdrive, x((2*i-1)*6)+10), y(((2*i-1)*6)+10), x(((2*i-1)*6)+6), y(((2*i-1)*6)+6), x(((2*i-1)*6)+16), y(((2*i-1)*6)+16), 'webpillar')

i=i+1
end_loop

; ###### END OF LOOPS ######

Appendix L - FISH file for FLAC model

*6)+4), zbf(((2*i-1)*6)+4), xeoed(((2*i-1)*6)+9), yedood(((2*i-1)*6)+9), zedood(((2*i-1)*6)+9), xedood(((2*i-1)*6)+9), xedood(((2*i-1)*6)+10), yedood(((2*i-1)*6)+10), zbf(((2*i-1)*6)+10), xedood(((2*i-1)*6)+10), yedood(((2*i-1)*6)+10), zbf(((2*i-1)*6)+10), namemb)

; ### NOW FOR "PILLARS" ###

i=1
loop while i < ndrives
rx = rwpillar
nx = nwpillar
gen_one_brick(nx, nwbench, nthtophalfdrive, rx, rwbench, rthtophalfdrive, x((2*i-1)*6)+9), y(((2*i-1)*6)+9), x((2*i-1)*6)+15), y(((2*i-1)*6)+15), z(((2*i-1)*6)+15), xhw(((2*i-1)*6)+10), yhw(((2*i-1)*6)+10), z(((2*i-1)*6)+10), xhw(((2*i-1)*6)+15), yhw(((2*i-1)*6)+15), z(((2*i-1)*6)+15), xhw(((2*i-1)*6)+16), yhw(((2*i-1)*6)+16), 'trench_tophalf')

i=i+1
end_loop

; ####### RIGHT SIDE BARRIER PILLAR #######

rx = 1/rwbarrier
nx = nwbarrier/2

gen_one_brick(nx, nwbench, nthtophalfdrive, rx, rwbench, rthtophalfdrive, x((12*ndrives)+3), y((12*ndrives)+3), x((12*ndrives)+9), y((12*ndrives)+9), x((12*ndrives)+15), y((12*ndrives)+15), x((12*ndrives)+16), y((12*ndrives)+16), 'trench_tophalf')

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```fishtext
((12*ndrives)+9),xbf((12*ndrives)+9),ybf((12*ndrives)+9),zbf((12*ndrives)+9),xhw((12*ndrives)+9),yhw((12*ndrives)+9),zhw((12*ndrives)+9),barrier2)

geon_brick(nx,nltbackfill,nthtophalfdrive,rx,rlbackfill,rthtophalfdrive,xbf((12*ndrives)+9),ybf((12*ndrives)+9),zbf((12*ndrives)+9),xhw((12*ndrives)+9),yhw((12*ndrives)+9),zhw((12*ndrives)+9),barrier2)
l
geon_brick(nx,nltvirgin,nthtophalfdrive,rx,rlvirgin,rthtophalfdrive,xeod((12*ndrives)+9),yv((12*ndrives)+9),zeod((12*ndrives)+9),xeod((12*ndrives)+9),yv((12*ndrives)+9),zeod((12*ndrives)+9),virgin)
l
if roofcoal = 1 then
    nameroofdrive = 'roofcoal'
    namerooftrench = 'trench_roofcoal'
else
    nameroofdrive = 'overburden'
    namerooftrench = 'trench_overburden'
endif

geon_brick(nx,nwbench,nthroofcoal,rx,rwbench,rthroofcoal,x(4),y(4),z(4),x(10),y(10),z(10),xhw(4),yhw(4),z(4),x(10),yhw(10),z(10),xhw(10),yhw(5),z(5),x(11),y(11),z(11),xhw(11),yhw(11),z(11),namerooftrench)

geon_brick(nx,nlnonbackfill,nthroofcoal,rx,rlnonbackfill,rthroofcoal,xhw(4),yhw(4),z(4),xhw(10),yhw(10),z(10),xbf(4),ybf(4),zbf(4),xbf(10),ybf(10),zbf(10),xbf(10),ybf(10),zbf(10),xbf(10),ybf(10),zbf(10),nameroofdrive)

geon_brick(nx,nltbackfill,nthroofcoal,rx,rlbackfill,rthroofcoal,xbf(4),ybf(4),zbf(4),xbf(10),ybf(10),zbf(10),xbf(10),ybf(10),zbf(10),ybf(10),zbf(10),zbf(10),ybf(10),zbf(10),ybf(10),zbf(10),nameroofdrive)

geon_brick(nx,nltvirgin,nthroofcoal,rx,rlvirgin,rthroofcoal,xeod(4),yv(4),zv(4),xeod(4),yv(4),zv(4),xeod(10),yv(10),zv(10),xeod(10),yv(10),zv(10),xeod(10),yv(10),zv(10),nameroofdrive)

#### END OF LEFT BARRIER PILLAR ####

#### NOW THE LOOP STARTING WITH DRIVE 1 ####

i=1
loop while i <= nndrives
rx = rwdrive
nx = nwdrive
```

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gen_one_brick(nx, nwbench, rthroofoal, rx, rbench, rthroofoal, x(((2*i-1)*6)+4), y(((2*i-1)*6)+4), z(((2*i-1)*6)+4), x(((2*i-1)*6)+10), y(((2*i-1)*6)+10), z(((2*i-1)*6)+10), xhw(((2*i-1)*6)+5), yhw(((2*i-1)*6)+5), zhw(((2*i-1)*6)+5), x(((2*i-1)*6)+10), y(((2*i-1)*6)+10), z(((2*i-1)*6)+10), xhw(((2*i-1)*6)+10), yhw(((2*i-1)*6)+10), zhw(((2*i-1)*6)+10), xhw(((2*i-1)*6)+10), yhw(((2*i-1)*6)+10), zhw(((2*i-1)*6)+10), namerooftrench)

i=i+1
end_loop

; ###### END OF LOOP ######
Appendix L - FISH file for FLAC model

; ############### RIGHT SIDE BARRIER PILLAR ###############

\[ rx = 1/rwbarrier \]
\[ nx = mbarrier/2 \]

gen_one_brick(nx, nwbench, nthroofcoal, rwbench, rthroofcoal, x((12*ndrives)+4), y((12*ndrives)+4), z((12*ndrives)+4), yh(x((12*ndrives)+10), y((12*ndrives)+10), z((12*ndrives)+10)), xh((12*ndrives)+5), yh(x((12*ndrives)+5)), z((12*ndrives)+5)), xhw((12*ndrives)+10), yhw((12*ndrives)+10), zhw((12*ndrives)+10), yh
((12*ndrives)+5), zhw((12*ndrives)+5), xhw((12*ndrives)+10), yhw((12*ndrives)+10), zhw((12*ndrives)+10))}

; ###### END OF RIGHT SIDE BARRIER PILLAR ######

; #################################################################

; ########### END OF ROOF COAL ##############

; #################################################################

; #################################################################

; ######## CONTINUE WITH OVERBURDEN #########

; #################################################################

; ###### LEFT BARRIER PILLAR ######

\[ rx = rwbarrier \]
\[ nx = mbarrier/2 \]

gen_one_brick(nx, nwbench, nthoverburden, rwbench, rthoverburden, x(5), y(5), z(5), x(11), y(11), z(11), xhw(5), yhw(5), zhw(5), x(6), y(6), z(6), xhw(11), yhw(11), zhw(11), xhw(6), yhw(6), zhw(6), x(12), y(12), z(12), xhw(12), yhw(12), 'trench_overburden')

gen_one_brick(nx, nlnonbackfill, nthoverburden, rlnonbackfill, rthoverburden, xhw(5), yhw(5), zhw(5), xhw(11), yhw(11), zhw(11), xhw(6), yhw(6), zhw(6), xhw(12), yhw(12), zhw(12), xhw(11), yhw(11), zhw(11), 'overburden')

gen_one_brick(nx, nlnonbackfill, nthoverburden, rlnonbackfill, rthoverburden, xbw(5), ybw(5), zbw(5), xbw(11), ybw(11), zbw(11), xbw(6), ybw(6), zbw(6), xbw(12), ybw(12), zbw(12), 'overburden')

gen_one_brick(nx, nlvirgin, nthoverburden, rlnvirgin, rthoverburden, xeod(5), yvod(5), zeod(5), xeod(12), yvod(12), zeod(12), 'overburden')

; ###### END OF LEFT BARRIER PILLAR ######

; ###### NOW THE LOOP STARTING WITH DRIVE 1 ######

; ### FIRST FOR "DRIVES" ###
i = 1
while i <= ndrives
  rx = rwdrive
  nx = nwdrive
  gen_one_brick(nx, nwbench, nthoverburden, rx, nwbench, rthoverburden, x(((2*i-1)*6)+5), y(((2*i-1)*6)+5), z(((2*i-1)*6)+5), xv(((2*i-1)*6)+5), yv(((2*i-1)*6)+5), zv(((2*i-1)*6)+5),
  gen_one_brick(nx, nlnonbackfill, nthoverburden, rx, rlnonbackfill, rthoverburden, x(((2*i-1)*6)+5), y(((2*i-1)*6)+5), z(((2*i-1)*6)+5),
  gen_one_brick(nx, nlbrick, nthoverburden, rx, nlbrick, rthoverburden, x(((2*i-1)*6)+5), y(((2*i-1)*6)+5), z(((2*i-1)*6)+5),
  gen_one_brick(nx, nwinfill, nthoverburden, rx, rwinfill, rthoverburden, x(((2*i-1)*6)+5), y(((2*i-1)*6)+5), z(((2*i-1)*6)+5),
end_loop

### NOW FOR "PILLARS" ###

i = 1
while i <= ndrives
  rx = rwpillar
  nx = npillar
  gen_one_brick(nx, nwbench, nthoverburden, rx, nwbench, rthoverburden, x(((2*i-1)*6)+6), y(((2*i-1)*6)+6), z(((2*i-1)*6)+6),
  gen_one_brick(nx, nlnonbackfill, nthoverburden, rx, rlnonbackfill, rthoverburden, x(((2*i-1)*6)+6), y(((2*i-1)*6)+6), z(((2*i-1)*6)+6),
  gen_one_brick(nx, nlbrick, nthoverburden, rx, nlbrick, rthoverburden, x(((2*i-1)*6)+6), y(((2*i-1)*6)+6), z(((2*i-1)*6)+6),
  gen_one_brick(nx, nwinfill, nthoverburden, rx, rwinfill, rthoverburden, x(((2*i-1)*6)+6), y(((2*i-1)*6)+6), z(((2*i-1)*6)+6),
end_loop

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i=i+1
end_loop

; ###### END OF LOOP ######

; ###### RIGHT SIDE BARRIER PILLAR ######

rx = 1/rwbarrier
nx = nbarrier/2

gen_one_brick(nx,nwbench, nthoverburden,rx, rwbench, rthoverburden,x((12*ndrives)+5), y((12*ndrives)+5), z((12*ndrives)+5), xh((12*ndrives)+5), yh((12*ndrives)+5), zh((12*ndrives)+5), x((12*ndrives)+6), y((12*ndrives)+6), z((12*ndrives)+6), xhw((12*ndrives)+11), yhw((12*ndrives)+11), zhw((12*ndrives)+11), x((12*ndrives)+12), y((12*ndrives)+12), z((12*ndrives)+12), 'trench_overburden')
gen_one_brick(nx,nnonbackfill, nthoverburden,rx, rlnonbackfill, rthoverburden,xh((12*ndrives)+5), yh((12*ndrives)+5), zh((12*ndrives)+5), xhw((12*ndrives)+11), yhw((12*ndrives)+11), zb((12*ndrives)+11), xeod((12*ndrives)+5), yeod((12*ndrives)+5), zeod((12*ndrives)+5), xv((12*ndrives)+5), yv((12*ndrives)+5), zv((12*ndrives)+5), 'overburden')
gen_one_brick(nx, nlbackfill, nthoverburden, rx, rlbackfill, rthoverburden, xbf((12*ndrives)+5), ybf((12*ndrives)+5), zbf((12*ndrives)+5), xbw((12*ndrives)+11), ybw((12*ndrives)+11), zbw((12*ndrives)+11), 'overburden')
gen_one_brick(nx, nlvirgin, nthoverburden, rx, rlvirgin, rthoverburden, xeo((12*ndrives)+5), yeo((12*ndrives)+5), zeo((12*ndrives)+5), xeod((12*ndrives)+11), yeod((12*ndrives)+11), zeod((12*ndrives)+11), 'overburden')

def get_cohesion
alfa = 0.25*pi - 0.5*_phi*pi/180.
_coh = 0.5*UCS*tan(alfa)
end
def get_KG
_K = young/(3.*(1.-2.*poisson))
_G = 0.5*young/(1.+poisson)
end
def get_stresses
; Compute stress gradients and stress maximums
; Parameters used:
; ob_density
; k0
; szgrad = -ob_density*zgrav
; sxgrad = k0*szgrad
; szmax  = -szgrad*(z(6)-z(1))
; sxmax  = -sxgrad*(z(6)-z(1))
end

def excavate_trench
; EXCAVATE TRENCH
model null range group trench_bottomhalf
model null range group trench_tophalf
model null range group trench_roofcoal
model null range group trench_overburden
model null range group trench_wedge
set mech ratio 5e-5
solve
end_command
end

def setittoone
i=1
end

def addonetoi
i=i+1
end

def excavate_drive
loop while i<=ndrives
nnamebf = 'bottomhalfdrivefront'+string(int(i))
nnamebb = 'bottomhalfdriveback'+string(int(i))
nnameft = 'tophalfdrivefront'+string(int(i))
nnamebt = 'tophalfdriveback'+string(int(i))
nnamesave = sim+'drive'+string(int(i))
command
model null range group nnamebf
model null range group nnamebb
model null range group nnameft
model null range group nnamebt
set mech ratio 5e-5
solve
save nnamesave
end
end_command
i=i+3
end_loop
i=1
end

def backfill_drive

loop while i<=ndrives
nnamebf = 'bottomhalfdrivefront'+string(int(i))
nnamebb = 'bottomhalfdriveback'+string(int(i))
nnamef = 'tophalfdrivefront'+string(int(i))
nnameb = 'tophalfdriveback'+string(int(i))
nnamesave = sim+'backfill'+string(int(i))
command
    mo mo range group nnamebf
    mo mo range group nnamebb
    mo mo range group nnamef
    mo mo range group nnameb
    set _phi 29.6
    set _coh 1.17e6
    set young 0.317e9
    set poisson 0.35
    get_KG
    prop friction _phi cohesion _coh bulk _K shear _G density backfill_density &
    range group nnamebf
    prop friction _phi cohesion _coh bulk _K shear _G density backfill_density &
    range group nnamebb
    prop friction _phi cohesion _coh bulk _K shear _G density backfill_density &
    range group nnamef
    prop friction _phi cohesion _coh bulk _K shear _G density backfill_density &
    range group nnameb
    set mech ratio 5e-5
    solve
    save nnamesave
end_command
i=i+3
end_loop
i=1
end
Appendix M. Virtual triaxial test

; Triaxial test of in situ coal
new
set fish safe off
title
  Triaxial test of coal
gen zone cylind p0 0 0 0 p1 1 0 0 p2 0 4 0 p3 0 0 1 size 12 30 12

group sample

gen zone reflect norm 1,0,0
gen zone reflect norm 0,0,1

model mohr

fix x y z range y -.1 .1
fix x y z range y 3.9 4.1
ini yvel 2.5e-6 range y -.1 .1
ini yvel -2.5e-6 range y 3.9 4.1

ini density 1550.
prop friction 40.19
prop cohesion 1.208e6
prop bulk 1.976e9
prop shear 1.436e9

def confinement
  global _sig3 = -20.e6 ; Enter confinement in Pa, compression is negative
  command
    apply sxx _sig3 szz _sig3 range cyl end1 0,0,0 end2 0,4,0 radius 1
  endcommand
@end

def ax_stress
  local stre = 0
  local pnt = gp_head
  loop while pnt # null
    if gp_ypos(pnt) < 0.1 then
      str = str + gp+yfunbal(pnt)
    endif
    pnt = gp_next(pnt)
  endloop
  ax_stress = str / pi
end ; Cylinder radius = 1

def ax_strain ; Length of core is 4, compressed from both sides. So the original length is 2
  ipt = gp_near(0,0,0)
  _ydisp = gp_ydisp(ipt)
  ax_strain = _ydisp / 2
end

hist nstep 3
hist add id=1 fish #ax_strain
hist add id=2 fish #ax_stress
hist add id=3 gp xdisp 1,1,0
hist label 1 “Axial strain”
hist label 2 “Axial stress”
hist label 3 “Radial strain”

plot create view sig1_vs_e1
plot add hist 2 yreversed vs 1
plot create view e3_vs_e1
plot add hist 3 yreversed vs 1

step 4000

save confinement20MPa.sav
Appendix N. Major and intermediate principal stress progression plots

Figure 35: Major principal stress progression for roof zone

Figure 36: Intermediate principal stress progression for roof zone
Figure 37: Major principal stress progression for pillar zone

Figure 38: Intermediate principal stress progression for pillar zone
Figure 39: Major principal stress progression for drive zone

Figure 40: Intermediate principal stress progression for drive zone
Appendix 0. Displacement and stress plots for sequence simulations

Figure 41: X-displacement for one-by-one sequence

Figure 42: Z-displacement for one-by-one sequence
Figure 43: Contour of minor principal stress for one-by-one sequence

Figure 44: X-displacement for two-by-two sequence
Figure 45: Z-displacement for two-by-two sequence

Figure 46: Contour of minor principal stress for two-by-two sequence
Figure 47: X-displacement for four-by-four sequence

Figure 48: Z-displacement for four-by-four sequence
Figure 49: Contour of minor principal stress for four-by-four sequence

Figure 50: X-displacement for modified three-by-three sequence
**Figure 51**: Z-displacement for modified three-by-three sequence

**Figure 52**: Contour of minor principal stress for modified three-by-three sequence
**Figure 53**: X-displacement for modified two-by-two sequence  

**Figure 54**: Z-displacement for modified two-by-two sequence
Figure 55: Contour of minor principal stress for modified two-by-two sequence.

Figure 56: X-displacement for modified two-by-two sequence with direct backfilling.
Figure 57: Z-displacement for modified two-by-two sequence with direct backfilling.

Figure 58: Contour of minor principal stress for modified two-by-two sequence with direct backfilling.
Figure 59: X-displacement for five-by-five sequence.

Figure 60: Z-displacement for five-by-five sequence.
**Figure 61:** Contour of minor principal stress for five-by-five sequence.