FIGURE VII.8 Correlation between specific surface area and pozzolanic activity for PFA from five power stations.
other hand up to several metres of thixotropic ashfill layers have been observed within a 100 m distance of strong ashfill layers. It is obvious therefore, that a number of other parameters must also control the in-situ quality of ashfill.

Many of these parameters are associated with the method of placement of ashfill underground and the environment into which the ashfill is placed. The more important are:

i) Ashfill is placed in layers. Typically, a 40 mm layer of ashfill is placed in a two hour period once every 12 hours. Furthermore, one in every five layers, on the average, consists of coarse, well-draining, non-cementing clinker ash. The effect of placing ashfill in layers on a frequent basis is to compact previously deposited layers before significant cementing has occurred. Water squeezed out of the layers in the densification process drains away through the coarse ash layers. Particle density and interparticle contact is increased and a higher strength ashfill is achieved than that achieved from a 20 per cent slurry concentration (uncompacted) in the laboratory.

ii) Ashfill layers are of considerable lateral extent. This factor can have a significant effect on ashfill drainage if long delays occur in ashfilling operations and in-situ ashfill layers cement. The permeability of cemented ashfill is of the order of $10^{-4}$ cm/sec which, for most practical purposes, is considered impervious. Thus, when ashfilling operations are resumed pore water in fine ashfill layers deposited on top of cemented ashfill can only drain laterally. Therefore, most of the pore
water may be retained in the ashfill giving rise to thixotropic ashfill layers.

iii) Ashfill areas are usually undulating. Since the primary reason for ashfilling at the present time is to dispose of ash little consideration is given to the damming of water in the ashfill areas. Ashfill deposited in these low lying areas remains saturated and thixotropic.

iv) The ashfill environment is at a lower temperature and at a higher humidity than the laboratory environment. Since an excess of water is already deposited in ashfill areas, neither of these two factors has a beneficial influence on ashfill strength.

It is apparent that variations in in-situ ashfill quality at Springfield Colliery can be attributed to a complex and random interaction of factors such as ash type, ash segregation during placement, slurry concentration, thickness of individual ashfill layers, ratio of fine ashfill layers (PFA) to coarse ashfill layers (clinker), frequency of ashfill placement and depositional environment. However, in the light of laboratory findings all these factors can be reduced to two parameters, namely specific surface area and water:PFA ratio. This latter parameter is the one requiring the most control if a good quality ashfill is to be achieved using present ash disposal techniques. Recommended means of controlling this parameter are the subject of a report by Galvin (1978).

Finally, the potential exists at Faringfield Colliery to generate a further 8 million tonnes of reserves from bord and pillar workings in thick coal seams utilizing previously placed ashfill. The greatest obstacle to the success of this operation is the possibility of
liquefaction of thixotropic ashfill layers triggered by blasting. However, Gilbert (1976) reports that soils exhibiting all of the following physical properties are most susceptible to liquefaction:

i) The percentage of silt and clay-size particles is less than 10 per cent.

ii) The particle diameter at 60 per cent passing on the grain size curve, $D_{60}$, is between 0.2 and 1.0 mm.

iii) The uniformity coefficient, $D_{60}/D_{10}$, is between 2 and 5.

iv) The blow count from standard penetration is less than 15, indicating that the material is loosely deposited.

Gilbert (1976) also reports that other investigators report similar findings. Investigations into thixotropic ashfill layers have shown that none of the four preceding conditions are satisfied. Furthermore, it is considered that the distribution of coarse ash layers throughout ashfill and the limited amount of cementing within thixotropic layers will limit the development of liquefaction. The presence of thixotropic ashfill layers is expected to be of far greater significance when consideration is given to the operation of mechanised equipment on top of ashfill.

VII.6.2 Improvement of the pozzolanic activity of PFA

Reference to Fig. VII.5 shows that ashfill derived from Field 1 PFA has only about 30 per cent of the strength of ashfill derived from Field 3 PFA. Since over 60 per cent of all ash produced is Field 1 PFA, whilst less than 3 per cent is Field 3 PFA, the overall quality of in-situ ashfill could be improved significantly if the
pozzolanic activity of Field 1 PFA could be improved. Research results presented in Fig. VII.2 and Table VII.1 show that the only significant difference between Field 1 PFA and Field 3 PFA is the grain size distribution and thus, the specific surface area. Modification of the grain size distribution by grinding is, therefore, an obvious means of attempting to improve the pozzolanic activity of Field 1 PFA.

In the light of electron microscope studies of Field 1 PFA, grinding may lead to an increase in specific surface area in one of the following three manners:

i) by reducing the size of clusters of fused spheres;

ii) by breaking hollow spheres to effectively double the surface area of particles;

iii) by breaking pherospheres to release smaller spheres contained inside.

Each of these operations may also result in an increase in the 'free' lime content. The results of grinding tests are shown in Fig. VII.9. The strength of an ashfill prepared from Field 1 PFA at a slurry concentration of 60 per cent solids by weight was effectively doubled when the ash was ground wet for 30 seconds, and trebled when ground for 60 seconds. Only a minimal further increase in strength was gained by grinding ash for longer than 60 seconds. It is also significant that grinding trebled the strength of ashfills prepared at slurry concentrations of 20 per cent and 40 per cent solids by weight. Furthermore, for a given ashfill strength, grinding resulted in an increase in the modulus of deformation. For ground Field 1 PFA the correlation between these two parameters is defined by the equation -
FIGURE VII.9 The effect of wet grinding of Field 1 PFA on ashfill strength.
However, no significant change in grain size distribution after grinding could be established. A number of reasons may account for this observation, including:

i) Grain size analysis is based on Stokes Law and requires a knowledge of the specific gravity of the grains. The grain size analysis of ground PFA did not take into consideration any change in the specific gravity of crushed spheres.

ii) PFA is only sized after it has been ground wet and then dried. Thus, the effects of grinding may be disguised because some cementing of particles has occurred as a result of the wetting and drying process.

iii) The effect of grinding PFA may not be to reduce particle size but, rather, to produce a more intimate mix of ash particles, free lime, and water.

These uncertainties are currently being researched further.

It should be appreciated that grinding times are a function of the type of mill used. Results presented in Fig. VII.9 were obtained using a laboratory model of the recently developed centrifugal mill, (Bradley et al., 1974, 1975). This mill achieves a much faster rate of grind than other conventional mills.

Further increases in the pozzolanic activity of PFA may be achieved by the addition of lime to the PFA. Provided sufficient water is available, the addition of lime extends the hydration period of ashfill, thus
increasing the ashfill strength and modulus of deformation. The influence of lime content on ashfill strength is recorded in Table VII.3. It can be seen that, in the short term at least, ashfills derived from lime deficient PFA's benefit most from the addition of lime to the mix.

In this regard, it should be noted that the lime deficient ash recorded in Table VII.3 was derived from a liquefaction plant. Although the addition of 15 per cent lime by weight increased the ashfill strength by 411 per cent, this strength still did not exceed the strength of ashfill derived from unmodified Field 3 Grootvlei PFA. Whilst this aspect must be researched further it appears that it may not be practically nor economically feasible to utilize ash produced by liquefaction plants as a form of pillar support in bord and pillar workings.

VII.6.3 Improving and utilizing in-situ ashfill quality

In-situ and laboratory studies have highlighted deficiencies in current ashfill placement techniques which must be overcome if full advantage is to be gained from the potential support properties of ashfill. In particular, ashfill quality may be improved by:

1) Placing ashfill at very high slurry concentrations.

2) Impacting ashfill against coal pillars, thereby compacting the ashfill and thus further increasing its strength and modulus of deformation.

3) The incorporation of a grinding circuit into an ashfill reticulation system to improve the pozzolanic activity of the more abundant Field 1 and Field 2 PFA's. (Economic considerations, Table 8.1 rule out the use of additives).
iv) The placement of a homogenous ashfill utilizing PFA as a cementing agent and clinker as an aggregate.

**TABLE VII.3** The effect of lime addition on ashfill strength

<table>
<thead>
<tr>
<th>Ash Type</th>
<th>Slurry Conc.</th>
<th>% Lime Added</th>
<th>Age (days)</th>
<th>Strength Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grootvlei Power Station Field 1</td>
<td>20 and 40</td>
<td>1</td>
<td>38</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>148</td>
<td>78%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>148</td>
<td>89%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>148</td>
<td>167%</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>1</td>
<td>38</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>148</td>
<td>32%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>148</td>
<td>90%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>148</td>
<td>108%</td>
</tr>
<tr>
<td>Sasol I Liquefaction Plant</td>
<td>50</td>
<td>5</td>
<td>30</td>
<td>11%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>75</td>
<td>78%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10</td>
<td>75</td>
<td>167%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15</td>
<td>75</td>
<td>156%</td>
</tr>
</tbody>
</table>

Studies have also shown that ashfill gains strength very slowly. Laboratory studies reveal that good quality ashfill continues to gain strength rapidly for 3 months after preparation. Underground observation and samples indicate that in a mine environment this period is nearer 6 months. Consequently, mining operations need to be conducted in such a manner that the strength...
properties of ashfill are not required to control strata displacement within 6 months of ashfill placement.

VII.7 Conclusions

The in-situ quality of PFA derived ashfill is primarily a function of the pozzolanic activity of the PFA, ashfill slurry concentration and time. In turn, the pozzolanic activity of PFA is primarily a function of the specific surface area and 'free' lime content of the material. These parameters should not be less than 2 000 cm$^2$/g and 2.5 percent respectively if good quality ashfill is to be achieved. PFA's produced by local power stations equipped with electrostatic precipitators usually satisfy these requirements but considerable doubt surrounds the suitability of PFA's produced by liquefaction plants. This aspect requires further research.

The quality of ashfill derived from either Field 3 or Field 4 PFA, the more pozzolanic active PFA's, improves markedly with increasing slurry concentration. However, ashfilling experiences indicate that similar benefits may be achieved by ensuring adequate in-situ drainage and compaction of ashfill. The pozzolanic activity of the more abundant but less pozzolanic active Field 1 and Field 2 PFA's can be doubled or even trebled by grinding the ash.

Ashfill gains strength slowly and mining operations need to be conducted in such a manner that the strength properties of ashfill are not required to control strata displacement within at least six months of ashfill placement. The modulus of deformation of ashfill is a direct indication of ashfill strength and thus the quality of ashfill.
APPENDIX VIII

RESEARCH INTO THE BEHAVIOUR OF MASSIVE DOLERITE SILLS

VIII.1 Introduction

The strength and elastic modulus of massive dolerite sills (<30 m thick) which overlie a large proportion of South African coal reserves are an order of magnitude greater than those associated with other overlying stratum. Consequently, at the shallow depth of local mining the behaviour of massive dolerite sills has an over-riding influence on the manner in which stresses are distributed around mine workings, especially when 'total' extraction mining methods are employed. An assessment of this influence, which can only be made on the basis of local mining experiences, has been conducted in a number of stages since 1960. But recent mining experiences have highlighted that this research is still inadequate, especially if 'total' extraction thick seam mining methods are to be implemented in coalfields where no previous experience of 'total' extraction mining beneath massive dolerite sills exists. This Appendix details additional research which has been conducted for the purpose of developing the theory of dolerite behaviour so that 'total' extraction thick seam mining methods may be implemented successfully in the South Rand and Vereeniging-Sasolburg Coalfields.

VIII.2 Previous Research Findings

VIII.2.1 Elastic thin plate model

Investigations into the behaviour of massive dolerite sills overlying 'total' extraction panels (pillar extraction and longwall panels) were initiated in 1965
at Durban Navigation Collieries in the Klip River Coalfield and extended to Sigma Collieries in the Vereeniging-Sasolburg Coalfield in 1967. The formulation of a mathematical model of dolerite behaviour by Salamon et al. (1972) culminated the investigations. Various aspects of these investigations have been reported by Salamon (1966), Oravecz (1966), Hardman (1968a), Hardman (1968b), Oravecz (1968) and Hardman (1971), and have been summarised concisely by Salamon et al. (1972). The more important and relevant findings and conclusions were:

1. The strength and elastic modulus of dolerite are an order of magnitude greater than that of normal coal measure rocks. Table VIII.1 records the typical range in the value of these two parameters for shale, sandstone, coal and dolerite stratum.

TABLE VIII.1 Typical range in strength and elastic modulus of South African coal measure rocks, as determined on 25.4 mm diameter, 76.2 mm long specimens.

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Strength (MPa) Range</th>
<th>Typical</th>
<th>Elastic Modulus (GPa) Range</th>
<th>Typical</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shale</td>
<td>60 - 80</td>
<td>70</td>
<td>0.5 - 13.0</td>
<td>3</td>
</tr>
<tr>
<td>Sandstone</td>
<td>40 - 100</td>
<td>70</td>
<td>4.0 - 15.2</td>
<td>7</td>
</tr>
<tr>
<td>Coal</td>
<td>15 - 40</td>
<td>30</td>
<td>0.4 - 4.1</td>
<td>2</td>
</tr>
<tr>
<td>Dolerite</td>
<td>250 - 390</td>
<td>300</td>
<td>50.0 - 98.5</td>
<td>70</td>
</tr>
</tbody>
</table>

2. Prior to failure of a massive dolerite sill caving of the roof strata only extends up to (or near) the base of the sill. As a result of this discontinuous subsidence, a gap forms between the top of the caved material and the base of the uncaved strata.
3. The possibility of a sudden and violent failure of a dolerite sill can be excluded.

4. After the initial failure of a dolerite sill, the progress of subsidence keeps pace with the progress of mining.

5. A close agreement exists between measured displacements and those derived from elastic theory. This supports the concept that a massive dolerite sill behaves as an elastic thin plate, at least up to a span which corresponds to its failure. Accordingly, this model offers a simple means for comparing different mining situations. The maximum stress in a uniformly loaded, infinitely long rectangular plate is proportional to the intensity of the load, to the square of the span, $S$, and inversely proportional to the square of the thickness of the plate, $t$. As the load intensity is approximately proportional to the depth, $D_D$, of the base of the dolerite below surface, the maximum stress in the sill is proportional to the factor $D_D S^2/t^2$, where $t_D$ is the thickness of the dolerite sill. If it is required to compare two mining situations then the dimensionless factor, $\varnothing$, can be formed such that-

$$\varnothing = \left(\frac{S_1t_{D_1}}{3^2t_{D_1}}\right)^2 \frac{D_{D_1}}{D_{D_2}}$$

(VIII.1)

Theoretically, if $\varnothing = 1$, two geometries are identical from the point of view of magnitude of stress. Therefore, assuming that all sills fail at the same magnitude of stress, henceforth referred to as the critical stress, and having determined $S_1$ from experience, Eqn. (VIII.1) can be used to calculate the minimum panel dimension, $S_2$, required to induce failure of a sill.
The elastic thin plate model was formulated on the basis of the behaviour of a 70 m thick dolerite sill overlying longwall panel 410 at DNC (Durban Nativation Collieries). Surface subsidence and borehole observations over this panel indicated that the 230 m face length was almost sufficient to cause failure of the sill. The model was 'verified' by incorporating the DNC 410 (panel 410) results into Eqn. (VIII.1), to calculate the minimum panel dimension required to induce failure of a 32 m thick sill overlying longwall panel 4 at Sigma Collieries (Sigma 4).

This calculation predicted that failure of the sill overlying the 192 m wide Sigma 4 panel would occur at a face advance of 149 m. In fact, the lower 16.5 to 24.4 m of the 32 m thick sill caved at a face advance of 142 m. However, the remaining 15.5 to 7.6 m of the sill only failed at a face advance of 181 m. Nevertheless in the light of:

i) the reasonable agreement between subsidence values predicted on the basis of elastic theory and those actually measured over DNC 410 and Sigma 4;

ii) the reasonable agreement between the predicted face advance at which failure of the sill overlying Sigma 4 would occur, and the actual face advance, and

iii) the lack of any additional field results,

it was considered reasonable to accept the simple model.

Between 1966 and 1976 a further seven longwall panels at DNC and three longwall panels at Sigma Collieries were dimensioned on the basis of the model. The predicted
result, that is, failure or non-failure of a sill, was achieved in each instance at Sigma Collieries but at DNC the expected failure of the sill over one panel never occurred whilst over two panels, failure only occurred some months after the completion of mining. Nevertheless, no further consideration was given to the model until 1977 when longwall mining was initiated at Coalbrook Collieries.

VIII.2.2 Refined elastic thin plate model

The longwall panels at Coalbrook Collieries are overlain by the same massive dolerite sill which overlies the longwall panels at the nearby Sigma Collieries. The face width of the first longwall panel at Coalbrook Collieries was 210 m, and failure of the dolerite sill was expected to occur at a face advance of about 170 m. However, failure only occurred at a face advance of 251 m. As a consequence, mining operations were adversely affected by the high abutment stresses induced by the unfailed sill, especially after the face advance had exceeded 150 m.

Following this experience, a review of the elastic thin plate model was undertaken by Wagner and Galvin (1977). Particular attention was given to critical stress, \( \sigma_c \), defined as:

\[
\sigma_c = \frac{k_y D_0 S^2}{t_0^2} \text{ (MPa)} \quad \text{(VIII.2)}
\]

where \( k_y \) is a (specific weight) constant (MN/m\(^3\)).

The researchers concluded that if the model was reasonably correct, the critical stress associated with each case where a sill had failed should be near constant. The results of this review, plotted in Fig. VIII.1, clearly indicate that this condition did not hold. In
result, that is, failure or non-failure of a sill, was achieved in each instance at Sigma Collieries but at DNC the expected failure of the sill over one panel never occurred whilst over two panels, failure only occurred some months after the completion of mining. Nevertheless, no further consideration was given to the model until 1977 when longwall mining was initiated at Coalbrook Collieries.

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\]

where \( K_y \) is a (specific weight) constant \((MN/m^3)\)

The researchers contended that if the model was reasonably correct, the critical stress associated with each case where a sill had failed should be near constant. The results of this review, plotted in Fig. VIII.1, clearly indicate that this condition did not hold. In
X FAILED (SURFACE SUBSIDENCE, $V_Z > 0.3$ MINING HEIGHT, $M_H$)

X TRANSITIONAL ($0.1 M_H < V_Z < 0.3 M_H$)

O UNFAILED ($V_Z < 0.1 M_H$)

FIGURE VIII.1 Review of elastic thin plate model - 1977 data.
fact, it would appear that one of the two sets of data on which the model was derived, namely DNC 410, is in itself, an anomaly.

An analysis of 1977 data, Table VIII.2, by Wagner and Galvin (1977), revealed that the three high points in Fig. VIII.1, namely DNC 410, Sigma 4 and Coalbrook 1, represented situations where a dolerite sill occurred at depth. The depth:thickness ratio, \( D_D/t_D \), of the sill for these situations was respectively, 2.0, 2.19 and 2.79. All other failed or transitional (refer Fig. VIII.1 for definition) cases had a depth:thickness ratio of 1.5 or less. Therefore, the critical stress, \( \sigma_C \), was plotted against the depth:thickness ratio, \( D_D/t_D \), to determine if a relationship existed between these two parameters, Fig. VIII.2.

On the basis of Fig. VIII.2 the following relationship was derived—

\[
\sigma_C = K_Y \left( 1400 \frac{D_D}{t_D} - 800 \right) \quad \text{(MPa)} \tag{VIII.3}
\]

Substituting this expression for \( \sigma_C \) into Eqn. (VIII.2) yielded—

\[
S = \sqrt{1400t_D - \frac{800t_D^2}{D_D}} \quad \text{(m)} \tag{VIII.4}
\]

where \( S \) is the minimum panel dimension (m) required to induce failure of a sill.

---

* The numerical values 1400 and 800 have units of metres. Similarly, all numerical values plotted on the ordinate axis in Figs. VIII.2, VIII.5 and VIII.11 and contained in Eqns. (VIII.4), (VIII.6), (VIII.19) and (VIII.30) have units of metres.
TABLE VIII.2 Data available in 1977 concerning longwall mining beneath massive dolerite sills.

<table>
<thead>
<tr>
<th>Case</th>
<th>D_D (m)</th>
<th>t_D (m)</th>
<th>S (m)</th>
<th>σ_c (MPa)</th>
<th>D_D</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>DNC 411-6/8</td>
<td>75</td>
<td>71</td>
<td>200*</td>
<td>595kY</td>
<td>1.07</td>
<td>Failed</td>
</tr>
<tr>
<td>DNC 426/8</td>
<td>76</td>
<td>71</td>
<td>215</td>
<td>697kY</td>
<td>1.07</td>
<td>Failed</td>
</tr>
<tr>
<td>DNC 410-6/8</td>
<td>83</td>
<td>75</td>
<td>220</td>
<td>714kY</td>
<td>1.11</td>
<td>Transitional</td>
</tr>
<tr>
<td>DNC 456/8</td>
<td>83</td>
<td>75</td>
<td>220</td>
<td>714kY</td>
<td>1.11</td>
<td>Transitional</td>
</tr>
<tr>
<td>DNC 410</td>
<td>140*</td>
<td>70*</td>
<td>230</td>
<td>151kY</td>
<td>2.00</td>
<td>Unfailed</td>
</tr>
<tr>
<td>DNC 412-6/8</td>
<td>75</td>
<td>71</td>
<td>168</td>
<td>105kY</td>
<td>1.11</td>
<td>Unfailed</td>
</tr>
<tr>
<td>DNC 436/8</td>
<td>75</td>
<td>71</td>
<td>120*</td>
<td>708kY</td>
<td>1.11</td>
<td>Unfailed</td>
</tr>
<tr>
<td>DNC 476/8</td>
<td>75</td>
<td>71</td>
<td>128*</td>
<td>243kY</td>
<td>1.06</td>
<td>Unfailed</td>
</tr>
<tr>
<td>Sigma 4</td>
<td>70</td>
<td>32</td>
<td>149*</td>
<td>151kY</td>
<td>2.19</td>
<td>Failed</td>
</tr>
<tr>
<td>Sigma I</td>
<td>60</td>
<td>40</td>
<td>166</td>
<td>103kY</td>
<td>1.50</td>
<td>Failed</td>
</tr>
<tr>
<td>Sigma II</td>
<td>42</td>
<td>35</td>
<td>168</td>
<td>968kY</td>
<td>1.20</td>
<td>Failed</td>
</tr>
<tr>
<td>Sigma III</td>
<td>60</td>
<td>45</td>
<td>182</td>
<td>981kY</td>
<td>1.33</td>
<td>Failed</td>
</tr>
<tr>
<td>Sigma 1</td>
<td>71</td>
<td>33</td>
<td>100</td>
<td>652kY</td>
<td>2.20</td>
<td>Unfailed</td>
</tr>
<tr>
<td>Sigma 2</td>
<td>72</td>
<td>33</td>
<td>100</td>
<td>661kY</td>
<td>2.18</td>
<td>Unfailed</td>
</tr>
<tr>
<td>Coalbrook 1</td>
<td>111</td>
<td>41</td>
<td>210</td>
<td>291kY</td>
<td>2.71</td>
<td>Failed</td>
</tr>
</tbody>
</table>

D_D - depth to base of dolerite sill  
D_D - thickness of dolerite sill  
S - minimum panel dimension  
σ_c - critical stress (elastic thin plate model, Eqn. (VIII.2))  
* - data since updated
FIGURE VIII.2 Derivation of depth:thickness formula.
At the time the researchers concluded that the term \( Dp/tp \) accounted for the effects of weathering on the strength of a dolerite sill. At greater depth a sill would be less weathered and, thus, more competent than a sill near surface. Consequently, a greater stress would be required to induce failure of the deeper sill. Circumstances at the time did not warrant further development of this concept, with the result that Eqn. (VIII.4) has been used extensively to date for designing longwall panel dimensions. However, this model has not proven applicable to all conditions.

VIII.3 Current Investigations and Theories

VIII.3.1 The need for further research

In early 1980 longwall mining operations beneath a massive dolerite sill commenced at Bosjesspruit Collieries in the Highveld Coalfield. On the basis of the refined elastic thin plate model, Eqn. (VIII.4), the minimum panel dimension required to break the sill was calculated to be 177 m. However, investigations since the formulation of this model had indicated that the parting thickness between a seam and a dolerite sill also influences the minimum panel dimension. An attempt was made to approximate this influence and the minimum panel dimension was revised to 200 m. For technical and operating reasons, the face length of the first longwall panel was extended to 222 m. Nevertheless, the dolerite sill did not fail until the face had advanced over 700 m.

The lack of success in inducing a dolerite sill to fail when longwall mining operations are extended to a new coalfield raises serious doubts about the validity of the refined elastic thin plate model. In particular, it is reasonable to question whether the behaviour of the massive dolerite sill which overlies proposed thick seam
mining panels in the South Rand Coalfield can be predicted sufficiently accurately by the model. Therefore, it is necessary that the behaviour of massive dolerite sills be reviewed in the light of experience gained from the longwall mining of more than 25 panels beneath such sills during the last 15 years.

VIII.3.2 Approach to current research

Unfortunately, although more than 25 longwall panels have been extracted beneath massive dolerite sills, very little detailed information about the failure mode of sills has been obtained. Invariably, surface subsidence observations have been made over the panels but in only two instances have borehole extensometers been spaced at regular intervals throughout a sill. Consequently, investigations to date have relied heavily on surface subsidence observations and, to a much lesser extent, on measurements and observations made in surface boreholes over under-mined panels. Thus, the development and, in particular, the verification of new models and theories, is restricted by the time interval associated with the instrumenting and mining of further longwall panels.

Nevertheless, a re-analysis of measurement and observations presented in literature in the light of experiences gained during the 15 years of longwall mining beneath dolerite sills, in conjunction with recent measurements and observations, does enable some advances to be made in the theory of the behaviour of dolerite sills. The approach which has been adopted in this regard is outlined in Fig. VIII.3 with the more important findings being discussed in the remainder of this Appendix.
Re-analyse measurements and observations presented in literature

Effective Span Elastic Thin Plate Model. (VIII.4)

Review of anomalies and limitations associated with Elastic Thin Plate Model. (VIII.5)

Conclusions concerning Elastic Thin Plate Models. (VIII.6)

Possible combination of the two models.

Conclusions (VIII.8)

FIGURE VIII.3 Approach adopted to developing further the theory of dolerite behaviour.
VIII.4 Effective Span Elastic Thin Plate Model

One of the first points to emerge from the re-analysis of all available information relating to dolerite behaviour is that the effective unsupported span of a sill is influenced significantly by the caving angle, $\beta$, and the parting thickness, $t_p$, of the strata between a seam and a sill. By definition, the caving angle, $\beta$, is measured relative to the plane of the seam ahead of the face, Fig. VIII.4. The caving angle associated with the strata overlying longwall panels at Durban Navigation, Sigma, Coalbrook and Bosjesspruit Collieries is recorded in Table VIII.3. Since the angle is calculated by noting the face position at which a borehole anchor subsided rapidly, some error may exist in the angle when the strata caves in large blocks rather than as a granular material. However, with the exception of Bosjesspruit Collieries relatively consistent caving angles have been recorded for any one colliery and errors can be considered minimal.

Relevant important points arising from Table VIII.3 concerning the caving angle, $\beta$, are:

i) the angle is not 90 degrees as assumed in previous elastic thin plate models but may range from about 103 degrees to in excess of 140 degrees,

ii) the angle is greater before a sill fails than after a sill fails, and

iii) the angle is greater for more competent strata.

On the basis of the limited information contained in Table VIII.3, the following caving angles to the base of a sill have been assigned to each location, Table VIII.4.
Figure VIII.4 Influence of caving angle, $\beta$, and parting thickness, $t_p$, on unsupported dolerite span, $S_{eff}$. 
### TABLE VIII.3 Caving angles of strata beneath dolerite sills.

<table>
<thead>
<tr>
<th>Panel</th>
<th>Borehole</th>
<th>Anchor*</th>
<th>Strata</th>
<th>Condition of Dolerite</th>
<th>Caving Angle (β)</th>
<th>Average Angle (β)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DNC 410</td>
<td>2</td>
<td>A1</td>
<td>Interspersed</td>
<td>Unfailed</td>
<td>107°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>A1</td>
<td>sandstone and shaly</td>
<td>Unfailed</td>
<td>110°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>A2</td>
<td></td>
<td>Unfailed</td>
<td>111°</td>
<td>110°</td>
</tr>
<tr>
<td>DNC 481</td>
<td>1</td>
<td>A1</td>
<td>Interspersed</td>
<td>Unfailed</td>
<td>115°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>A3</td>
<td>sandstone and shaly</td>
<td>Unfailed</td>
<td>125°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>A4</td>
<td></td>
<td>Unfailed</td>
<td>121°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>A1</td>
<td>Interspersed</td>
<td>Unfailed</td>
<td>114°</td>
<td></td>
</tr>
<tr>
<td>Sigma 4</td>
<td>1</td>
<td>A2</td>
<td>sandstone</td>
<td>Unfailed</td>
<td>118°</td>
<td></td>
</tr>
<tr>
<td>Sigma I</td>
<td>2</td>
<td>A3</td>
<td></td>
<td>Unfailed</td>
<td>115°</td>
<td>114°</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>A4</td>
<td></td>
<td>Unfailed</td>
<td>120°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>A2</td>
<td></td>
<td>Failed</td>
<td>106°</td>
<td></td>
</tr>
<tr>
<td>Sigma 4</td>
<td>5</td>
<td>A3</td>
<td></td>
<td>Failed</td>
<td>108°</td>
<td>106°</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>A4</td>
<td></td>
<td>Failed</td>
<td>105°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>A2</td>
<td></td>
<td>Failed</td>
<td>103°</td>
<td></td>
</tr>
<tr>
<td>Sigma I</td>
<td>3</td>
<td>A3</td>
<td></td>
<td>Failed</td>
<td>106°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>A2</td>
<td></td>
<td>Failed</td>
<td>108°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>A5</td>
<td></td>
<td>Failed</td>
<td>106°</td>
<td></td>
</tr>
<tr>
<td>Coalbrook</td>
<td>SL14</td>
<td>A3</td>
<td>Interspersed</td>
<td>Failed</td>
<td>112°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>A4</td>
<td>sandstone and shaly</td>
<td>Failed</td>
<td>108°</td>
<td>106°</td>
<td></td>
</tr>
<tr>
<td>Bosjes-</td>
<td>E1</td>
<td>A3</td>
<td>Competent</td>
<td>Unfailed</td>
<td>141°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>A4</td>
<td>massive sandstone</td>
<td>Unfailed</td>
<td>130°</td>
<td>133°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>A5</td>
<td>stratum</td>
<td>Unfailed</td>
<td>128°</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>A6</td>
<td>Interspersed</td>
<td>Nearest failure</td>
<td>114° to 119°</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>W1</td>
<td>A3</td>
<td>sandstone with sub-ordinate</td>
<td>No dolerite</td>
<td>117°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>A4</td>
<td></td>
<td></td>
<td></td>
<td>115°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>A5</td>
<td></td>
<td></td>
<td></td>
<td>118°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>A6</td>
<td></td>
<td></td>
<td></td>
<td>108°</td>
<td></td>
</tr>
</tbody>
</table>

* Only anchors which were located at least three times the mining height above the seam have been considered.
TABLE VIII.4 Caving angle, $\beta$, to the base of a dolerite sill.

<table>
<thead>
<tr>
<th>Colliery</th>
<th>Caving Angle $\beta$ (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DNC</td>
<td>110</td>
</tr>
<tr>
<td>Sigma</td>
<td>117</td>
</tr>
<tr>
<td>Coalbrook</td>
<td>110</td>
</tr>
<tr>
<td>Bosjesspruit</td>
<td>119</td>
</tr>
</tbody>
</table>

Consequently, the effective span, $S_{eff}$, over which an unfailed sill bridges is not equal to the panel dimension, $S$, but is defined by the equation—

$$S_{eff} = S - 2t_{p}\tan(\beta - 90) \quad (\text{m}) \quad (\text{VIII.5})$$

It should be appreciated that, because the caving line slopes towards the goaf, the perimeter of a bridging sill rests on undermined strata. This strata constitutes a soft support and yields away from the weight of the bridging overlying strata, thus effectively increasing the span of the bridging sill. However, this increase is difficult to assess and for the task at hand the effective span, $S_{eff}$, as defined by Eqn. (VIII.5) is considered sufficiently accurate.

Therefore, the refined elastic thin plate model has been modified to take into account these findings. The caving angles recorded in Table VIII.4 have been utilized in re-calculating the critical stress associated with each longwall panel mined to date, Fig. VIII.5. Because additional borehole information has been obtained in the interim period, slightly different values to those previously associated with some longwall panels have been used in the re-calculations. The latest available information is recorded in Table VIII.5.
FIGURE VIII.5 Derivation of Effective Span Elastic Thin Plate Model.
### TABLE VIII.3 Data relating to longwall panels and elastic thin plate models.

<table>
<thead>
<tr>
<th>Case</th>
<th>Dp (m)</th>
<th>dp (m)</th>
<th>tp (m)</th>
<th>smin (m)</th>
<th>Mh (m)</th>
<th>Vmax (m)</th>
<th>θ (deg)</th>
<th>S_eff (m)</th>
<th>c的主题前缀</th>
<th>c的主题前缀</th>
<th>θ (degrees)</th>
<th>State</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>DNC 410</td>
<td>137</td>
<td>73</td>
<td>64</td>
<td>91</td>
<td>230</td>
<td>1.88</td>
<td>1x1.5</td>
<td>130</td>
<td>164</td>
<td>1160</td>
<td>689</td>
<td>505</td>
<td>462 U.F.</td>
</tr>
<tr>
<td>DNC 412-6/8</td>
<td>75</td>
<td>71</td>
<td>4</td>
<td>135</td>
<td>168</td>
<td>1.06</td>
<td>2x1.5</td>
<td>50</td>
<td>110</td>
<td>420</td>
<td>72</td>
<td>38</td>
<td>119 U.F.</td>
</tr>
<tr>
<td>DNC 414-6/8</td>
<td>83</td>
<td>74</td>
<td>9</td>
<td>97</td>
<td>190</td>
<td>1.12</td>
<td>2x1.5</td>
<td>139</td>
<td>110</td>
<td>547</td>
<td>216</td>
<td>120</td>
<td>U.F.</td>
</tr>
<tr>
<td>DNC 416-6/8</td>
<td>83</td>
<td>74</td>
<td>9</td>
<td>97</td>
<td>180</td>
<td>1.12</td>
<td>2x1.5</td>
<td>141</td>
<td>110</td>
<td>109</td>
<td>491</td>
<td>181</td>
<td>100 U.P.</td>
</tr>
<tr>
<td>DNC 418-6/8</td>
<td>83</td>
<td>74</td>
<td>9</td>
<td>97</td>
<td>115</td>
<td>1.12</td>
<td>2x1.5</td>
<td>61</td>
<td>110</td>
<td>40</td>
<td>200</td>
<td>30</td>
<td>17 U.F.</td>
</tr>
<tr>
<td>DNC 436/8</td>
<td>75</td>
<td>71</td>
<td>4</td>
<td>135</td>
<td>155</td>
<td>1.06</td>
<td>2x1.5</td>
<td>126</td>
<td>110</td>
<td>57</td>
<td>357</td>
<td>48</td>
<td>25 U.P.</td>
</tr>
<tr>
<td>DNC 456/8</td>
<td>83</td>
<td>75</td>
<td>8</td>
<td>98</td>
<td>120</td>
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<td>2x1.5</td>
<td>220</td>
<td>110</td>
<td>119</td>
<td>714</td>
<td>326</td>
<td>179 U.F.</td>
</tr>
<tr>
<td>DNC 554</td>
<td>99</td>
<td>74</td>
<td>25</td>
<td>135</td>
<td>180</td>
<td>1.34</td>
<td>2x1.5</td>
<td>77</td>
<td>110</td>
<td>82</td>
<td>566</td>
<td>121</td>
<td>75 U.F.</td>
</tr>
<tr>
<td>DNC 410-6/8</td>
<td>83</td>
<td>75</td>
<td>8</td>
<td>99</td>
<td>220</td>
<td>1.11</td>
<td>2x1.5</td>
<td>520</td>
<td>110</td>
<td>148</td>
<td>714</td>
<td>326</td>
<td>177 U.P.</td>
</tr>
<tr>
<td>DNC 481</td>
<td>94</td>
<td>74</td>
<td>20</td>
<td>100</td>
<td>195</td>
<td>1.27</td>
<td>2x1.5</td>
<td>580</td>
<td>110</td>
<td>122</td>
<td>651</td>
<td>256</td>
<td>155 T</td>
</tr>
<tr>
<td>DNC 491</td>
<td>94</td>
<td>74</td>
<td>20</td>
<td>100</td>
<td>190</td>
<td>1.27</td>
<td>2x1.5</td>
<td>920</td>
<td>110</td>
<td>117</td>
<td>620</td>
<td>236</td>
<td>143 T</td>
</tr>
<tr>
<td>DNC 411-6/8</td>
<td>75</td>
<td>71</td>
<td>4</td>
<td>135</td>
<td>240</td>
<td>1.06</td>
<td>2x1.5</td>
<td>1420</td>
<td>110</td>
<td>142</td>
<td>859</td>
<td>299</td>
<td>157 T</td>
</tr>
<tr>
<td>DNC 426/8</td>
<td>76</td>
<td>71</td>
<td>4</td>
<td>135</td>
<td>214</td>
<td>1.07</td>
<td>2x1.5</td>
<td>1060</td>
<td>110</td>
<td>116</td>
<td>690</td>
<td>202</td>
<td>108 T</td>
</tr>
<tr>
<td>Sigma I</td>
<td>69</td>
<td>31</td>
<td>38</td>
<td>97</td>
<td>100</td>
<td>2.23</td>
<td>2.3</td>
<td>51</td>
<td>117</td>
<td>48</td>
<td>718</td>
<td>156</td>
<td>129 T</td>
</tr>
<tr>
<td>Sigma 2</td>
<td>72</td>
<td>33</td>
<td>39</td>
<td>20</td>
<td>170</td>
<td>2.18</td>
<td>2.3</td>
<td>49</td>
<td>117</td>
<td>50</td>
<td>661</td>
<td>166</td>
<td>128 T</td>
</tr>
<tr>
<td>Sigma 4</td>
<td>68</td>
<td>32</td>
<td>38</td>
<td>25</td>
<td>181</td>
<td>2.13</td>
<td>2.3</td>
<td>1240</td>
<td>117</td>
<td>156</td>
<td>2175</td>
<td>1606</td>
<td>1228 T</td>
</tr>
<tr>
<td>Sigma I</td>
<td>62</td>
<td>43</td>
<td>19</td>
<td>52</td>
<td>166</td>
<td>1.44</td>
<td>2.7</td>
<td>1283</td>
<td>117</td>
<td>113</td>
<td>923</td>
<td>428</td>
<td>280 F</td>
</tr>
<tr>
<td>Sigma II</td>
<td>55</td>
<td>40</td>
<td>15</td>
<td>60</td>
<td>168</td>
<td>1.38</td>
<td>2.7</td>
<td>1141</td>
<td>117</td>
<td>107</td>
<td>970</td>
<td>393</td>
<td>250 F</td>
</tr>
<tr>
<td>Sigma III</td>
<td>60</td>
<td>45</td>
<td>15</td>
<td>50</td>
<td>182</td>
<td>1.33</td>
<td>2.7</td>
<td>1400</td>
<td>117</td>
<td>131</td>
<td>981</td>
<td>509</td>
<td>344 F</td>
</tr>
<tr>
<td>Coalbrook II</td>
<td>105</td>
<td>40</td>
<td>65</td>
<td>49</td>
<td>121</td>
<td>2.63</td>
<td>2.5-3.0</td>
<td>230</td>
<td>110</td>
<td>85</td>
<td>960</td>
<td>478</td>
<td>244 F</td>
</tr>
<tr>
<td>Coalbrook IV</td>
<td>110</td>
<td>37</td>
<td>73</td>
<td>60</td>
<td>150</td>
<td>2.97</td>
<td>2.5-3.0</td>
<td>280</td>
<td>110</td>
<td>106</td>
<td>1307</td>
<td>908</td>
<td>755 F</td>
</tr>
<tr>
<td>Coalbrook I</td>
<td>106</td>
<td>38</td>
<td>68</td>
<td>49</td>
<td>210</td>
<td>3.79</td>
<td>2.5-3.0</td>
<td>1420</td>
<td>110</td>
<td>174</td>
<td>3237</td>
<td>2231</td>
<td>1831 1026 F</td>
</tr>
<tr>
<td>Bosjespruit I</td>
<td>52</td>
<td>52</td>
<td>52</td>
<td>120</td>
<td>222</td>
<td>1.0</td>
<td>3.0</td>
<td>450</td>
<td>119</td>
<td>89</td>
<td>948</td>
<td>152</td>
<td>76 6 T</td>
</tr>
</tbody>
</table>

**Definitions:**
- Dp: depth to base of sill.
- dp: thickness of sill.
- tp: thickness of strata overlying sill.
- smin: thickness of parting between seam and sill.
- Mh: minimum panel dimension (when sill broke).
- Vmax: maximum surface subsidence.
- θ: caving angle.
- S_eff: effective minimum panel dimension.
- c: State = U.F. unfailed (Vmax < 0.1 Mh).
- θ: transitional (0.1 Mh ≤ Vmax < 0.3 Mh).
- θ: Failed (Vmax ≥ 0.3 Mh).
- c: Elastic Thin Plate Model, Eqn. (VII.2).
- c: Average Deadweight Load, Effective Span Elastic Thin Plate Model, Eqn. (VIII.18).
- c: Load Transfer, Effective Span Elastic Thin Plate Model, based on Eqn. (VIII.28).
In the light of this information, the following relationship has been derived from Fig. VIII.5 -

\[ \sigma_c' = K \left( \frac{D_D S_{eff}}{t_D^2} \right) = K \gamma \left( \frac{1165 D_D}{t_D} - 935 \right) \text{ (MPa)} \text{ (VIII.6)} \]

where \( \sigma_c' = \sigma_c \left( \frac{S_{eff}}{S} \right)^2 \), the effective critical stress.

Thus,

\[ S = \sqrt{1165t_D - 935 \frac{t_D^2}{D_D}} + 2t_p \tan(8-90) \text{ (m)} \text{ (VIII.7)} \]

The difference between the actual span at which a sill failed and that as calculated from Eqn. (VIII.7) ranges from one under-estimation of only 3 m (Sigma 4) to an over-estimation of 33 m (Sigma II). A noteworthy feature of the model is that for Bosjesspruit Collieries the effective unsupported span of the dolerite sill at which failure is calculated to occur is 109 m. The present face width of 222 m results in only an 89 m effective unsupported span. On the basis of Eqn. (VIII.7), a face width of 242 m would be required to induce the sill to fail. However, these figures must be accepted with caution since, in view of the large parting thickness, \( t_p \), they are extremely sensitive to variations in the caving angle, \( \beta \), which has yet to be determined accurately.

VIII.5 Anomalies Associated with Elastic Thin Plate Models

The elastic thin plate model was derived on the basis of limited data and was intended as a quick and simple 'analytical' solution to a rather complex problem. Although the model has been refined and developed in the light of further information, it still represents a simple solution. Therefore, it is not surprising that a number of anomalies are associated with the model, the more important of which require noting.
VIII.5.1 Failure mechanism

Whilst strata movement throughout the full thickness of a dolerite sill during failure has only been monitored twice, the limited information so obtained in conjunction with borehole observations using a borehole camera, has shown that failure of a dolerite sill occurs in a number of discrete steps separated by a considerable period of time. This behaviour, illustrated in Fig. VIII.6, can be shown to be incompatible with the models based on the elastic thin plate concept.

Consider the most general case as depicted in Fig. VIII.7. On the basis of elastic thin plate theory the effective critical stress, $\sigma_c'$, acting in the bridging dolerite plate is given by the equation -

$$
\sigma_c' = K_Y \frac{(D-D-h_c)(S-2tp\tan(\beta-90)-h_c\tan(\alpha-90)-2s)}{(t_D-h_c)^2} \quad \text{(MPa)}
$$

(VIII.8)

Two situations can be recognised, namely:

i) $\alpha = 90^\circ$, $s = 0$ : effective span elastic thin plate model

Then,

$$
\sigma_c' = K_Y \frac{(D_D-h_c)(S-2tp\tan(\beta-90))^2}{(t_D-h_c)^2} = K_Y \frac{S_{eff}^2}{(D_D-h_c)^2}
$$
FIGURE VIII.6 Development of failure within the dolerite sill overlying Panel 4 at Sigma Colliery (Sigma 4).

FIGURE VIII.7 Geometry associated with the caving of a dolerite sill.
and as \( h_c \to D \),
- \( D_0 - h_c = K_2 \), (constant)
- \( \text{Seff} = K_3 \), (constant)
- \( t_D - h_c = 0 \).

and so \( \sigma_c' \to \infty \).

Expressed simply, caving (of the base) of a sill, initiated by inducing the effective critical stress, \( \sigma_c' \), in the sill, results in an increase in the stress acting in the sill. Thus, caving should propagate rapidly through the sill.

ii) \( \alpha > 90^\circ \), \( s \geq 0 \)

The limited information obtained to date indicates that, typically, \( \alpha = 115^\circ \), and \( s = 20-30 \text{ m} \)

Thus, the effective span, \( \text{Seff} \), is given by the equation:

\[
\text{Seff} = S - 2t_D \tan(\beta - 90) - 2h_c \tan(\alpha - 90) - 2s \quad (\text{m})
\]  

(VIII.9)

If for \( h_c = t_D \), \( \text{Seff} > 0 \), then the caving lines will not intersect within the sill and a similar analysis to that conducted in (i) can be performed to show that when caving is initiated the total thickness of the sill should fail.

Alternatively, if, for \( h_c = t_D \), \( \text{Seff} \leq 0 \), then the caving lines will intersect within the sill. Assume that at a mining span, \( S \), the critical stress, \( \sigma_c' \), is reached and caving is initiated. Also assume that at this span the caving lines defined by the angle, \( \alpha \), intersect at a point \( P \) defined by \( h_c = P \). The location of point \( P \) can be found by solving—
\[ S_{\text{eff}} = S - 2t_p \tan(\beta - 90) - 2h_c \tan(\alpha - 90) - 2s = 0 \]  
\text{(VIII.10)}

i.e., \[ S - K_4 - K_5 h_c - K_6 = 0 \]

\[ S - K_5 h_c - K_7 = 0 \]

where \( K_i, \ i = 1 \) to 14 are constants

and \( K_4 = 2t_p \tan(\beta - 90) \),
\( K_5 = 2 \tan(\alpha - 90) \),
\( K_6 = 2s \),
\( K_7 = K_4 + K_6 \).

Thus,

\[ h_c = \frac{S - K_7}{K_5} = p \]

But, once the effective critical stress, \( \sigma_c' \), is reached at a mining span \( S \) and caving is initiated then as:

\[ h_c + p, \]
\( D_p - h_c \) constant,
\( S - K_5 h_c - K_7 = 0 \),

and \( (t_D - h_c) \) constant.

Thus, from Eqn. (VIII.8)

\[ \sigma_c' = 0 \]

That is, the initiation of caving effectively decreases the stress acting in the sill to below the critical level and so caving should be arrested. But immediately the mining span is increased to \( S + \Delta S \), the effective critical stress level, \( \sigma_c' \), is again reached and further caving is initiated. Thus, failure of the sill should
progress smoothly from the base upwards at a rate which is directly proportional to the face advance. Experience, as illustrated by Fig. VIII.6, does not support this failure mode.

VIII.5.2 DNC dolerite failures

All information currently available relating to the geometry of longwall panels beneath massive dolerite sills has been recorded in Table VIII.5. The critical stress level associated with each of the elastic thin plate models reviewed has also been recorded for each longwall panel. Reference to the Table and to Fig. VIII.5 reveals a number of anomalies concerning the effective critical stress required to induce failure of the dolerite sill at DNC. For example, the sill failed totally over panels 481 and 491, but remained intact over panel 414. But the critical or maximum stress associated with each of these panels was similar. Furthermore, the behaviour is inconsistent with the depth:thickness ratio, $D_g/t_D$, concept.

An immediate reaction to these anomalies is to doubt the validity of the model. However, a closer examination of Table VIII.5 suggests that the anomalies are attributable to sub-critical panel dimensions. For example, the effective span of panel 411 was 142 m, decreasing to 122 m and 117 m for panels 481 and 491 respectively, and 119 m and 109 m for panels 413 and 414 respectively. Since the geology and geometry of the superincumbent strata associated with each of these panels is very similar, it is reasonable to conclude that effective spans in the range of about 115 to 130 m are sub-critical.

A re-analysis of the development of surface subsidence over panel 411 supports this conclusion. Fortunately, the face width of this panel was increased in a number
of steps and so the influence of effective span on surface subsidence can be assessed, Fig. VIII.8. It is apparent that an effective span of at least 130 m is required to induce substantial subsidence of the sill. However, even an effective span of over 140 m was still not sufficient to induce total collapse of the sill during mining operations. Total collapse only occurred at a later date when caving was reactivated during the extraction of a lower seam.

VIII.5.3 Depth:Thickness ratio, Dp/tD

When the significance of the Dp/tD ratio was first noted in 1977, it was concluded that the term reflected the influence of weathering on the strength of a dolerite sill. However, this current review of dolerite behaviour indicates that the term is also a measure of the load acting on a dolerite sill. An analytical technique has been employed to establish quickly and simply the potential influence of the Dp/tD ratio on dolerite behaviour. On the basis of the trend analysis provided by these results, a limited number of more accurate numerical calculations have been performed utilizing the boundary element program 'MINAP' (refer section 9.2.2). An appreciation of the loading conditions associated with the elastic thin plate models is an essential prerequisite for evaluating the significance of the Dp/tD ratio.

VIII.5.3.1 Loading of an elastic thin plate

The elastic thin plate theory so far presented in this chapter (and defined by Eqns. (VIII.1) to (VIII.9)) assumes that:
<table>
<thead>
<tr>
<th>FACE WIDTH (m)</th>
<th>160</th>
<th>193</th>
<th>210</th>
<th>230</th>
<th>240</th>
</tr>
</thead>
<tbody>
<tr>
<td>EFFECTIVE SPAN (m)</td>
<td>62</td>
<td>95</td>
<td>112</td>
<td>132</td>
<td>142</td>
</tr>
</tbody>
</table>

**FIGURE VIII.8** Influence of effective span on surface (dolerite) subsidence at DNC.
i) the plate is weightless, and
ii) the plate is loaded uniformly by a load equal to $\rho gdD$ where

$\rho$ is the density of strata (kg/m$^3$),  
g is the gravitational acceleration (m/s$^2$),  
$D_D$ is the depth to base of dolerite (m).

This loading condition, illustrated in Fig. VIII.9, is unrealistic.

Consider the case where a sill outcrops, Fig. VIII.10. The load, $q_s$, acting at any point in the sill is a linear function of depth and is given by the equation -

$$q_s = \rho gd_D, \quad (N) \quad \text{(VIII.11)}$$

where $d_D$ is the vertical distance (m) from top of sill.

Thus, the load assumed in the elastic thin plate models,

$$q_s = \rho dD, \quad (N) \quad \text{(VIII.12)}$$

only acts at the base of the sill. The average load acting on the sill is, in fact,

$$\overline{q_s} = \frac{\rho gd_D}{2}, \quad (N) \quad \text{(VIII.13)}$$

in the more general case of a sill overlain by strata, the load acting at any point in a sill is given by the equation -

$$q_s = q_e + \rho gd_D, \quad (N) \quad \text{(VIII.14)}$$
FIGURE VIII.9 Loading condition associated with elastic thin plate models.

FIGURE VIII.10 Load distribution within a sill due to the self-weight of the sill.
where $q_e$ is the externally applied load (N) of the overlying strata on the sill. If this load is assumed to be the dead-weight of the overlying strata, then

$$q_s = \frac{1}{2} g t_c + \rho_1 g \delta_D, \quad (N) \quad (VIII.15)$$

where $t_c$ is the thickness of overlying strata (m),

$\rho_1$ is the density of sill (kg/m$^3$),

and $\rho_2$ is the density of overlying strata (kg/m$^3$).

For $t_1 = \delta_2 = 0$,

$$q_s = g \varphi (t_c + \delta_D) \quad (VIII.16)$$

thus, the average load, $q_s$, acting on the sill is

$$q_s = g \varphi (t_c + \frac{\delta_D}{2}) \quad (N) \quad (VIII.17)$$

on the basis of this equation the critical stress associated with a dolerite sill is given by the equation:

$$\sigma'' = \frac{ho_2}{t_1} \frac{S^2_{\text{eff}}}{(t_2^2 - t_0^2)} \quad (MPa) \quad (VIII.18)$$

A revision of the elastic thin plate model on the basis of Eqn. (VIII.18) yields, from Fig. VII.11, the formula:

$$\sigma'' = k_y \left[ 1023 \frac{D_D}{E_D} - 928 \right] \quad (MPa) \quad (VIII.19)$$

Equating equations (VIII.18) and (VIII.19), the minimum critical panel dimension, $S$, is given by the equation:
\( V_{z} > 0.3 M_{h} \)  
\( 0.1 M_{h} \leq V_{z} \leq 0.3 M_{h} \)  
\( V_{z} < 0.1 M_{h} \)  

**FIGURE VIII.11** Derivation of average deadweight load, Effective Span Elastic Thin Plate Model.
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Name of thesis  The mining of South African thick coal seams - rock mechanics and mining considerations  1981

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