On the basis of this formula, the minimum panel dimension required to induce failure of a dolerite sill to date would never have been under-estimated. However, in two instances, this dimension would have been over-estimated by up to 45 m. In view of these discrepancies and of the complex nature of the equation, it is not considered as an improvement on the Effective Span Elastic Thin Plate model, defined by Eqn. (VIII.7).

VIII.5.3.2 Analytical solution

If the $D_p/t_D$ ratio influences the load distribution it follows from Eqn. (VIII.14) that this influence must be expressed through the externally applied load, $q_e$, (since the term 'pgdp' is independent of the $D_p/t_D$ ratio). As a first step in assessing this potential influence, the well established analytical solution for calculating load transfer between gravity loaded multiple roof laminae with built-in ends has been utilized (Obert and Duvall (1967), Chapter 20). This solution is based on the assumptions that:

i) the load per unit length, $q_2$, divided by the flexural rigidity, i.e. $q_2/E_1I_2$, for the upper beam is greater than that of the lower beam (I is the moment of inertia, $E$ is the modulus of elasticity). That is, the upper beam will load the lower beam and the lower beam will partially support the upper beam;

ii) the coefficient of friction between the two beams is zero;
iii) the deflections of the two beams are equal at any point along the interface;

iv) the upper beam loads the lower beam with a uniform load per unit length;

v) the lower beam supports the upper beam with an equal load per unit length;

vi) both beams are the same length and width.

Then, the load transfer, $q_t$, between the two beams is given by the equation -

$$q_t = \frac{q_2 E_1 I_1 - q_1 E_2 I_2}{E_1 I_1 + E_2 I_2}, \quad (N) \quad (VIII.21)$$

where subscript '1' denotes lower beam (dolerite sill), subscript '2' denotes upper beam (shale and sandstone). Now $q_t$ is analogous to $q_e$, the external load acting on a dolerite sill, so setting:

$$q_t = q_e,$$

and approximating $\rho_1 = \rho_2$,

and $E_1 = 10E_2$, then;

$$q_e = \frac{\rho_2 g t_2 \cdot 10E_2 \cdot t_1^3}{12} - \frac{\rho_2 g t_1 \cdot E_2 \cdot t_2^3}{12}$$

$$= \frac{10E_2 t_1^3}{12} + \frac{E_2 t_2^3}{12}.$$
If deadweight loading were assumed at the interface, as is the case with the elastic thin plate models, then the load transfer, \( q_{e_{DW}} \), would be:

\[
q_{e_{DW}} = 0.2g t_2 \quad \text{(N)}
\]

Thus, the ratio between the actual load transfer, \( q_e \), and the deadweight load transfer, \( q_{e_{DW}} \), is:

\[
\frac{q_e}{q_{e_{DW}}} = \frac{10t_1^3 t_2 - t_1 t_2^3}{(10t_1^3 + t_2^3)}
\]

The influence of the depth:thickness ratio, \( \frac{D_d}{t_D} \), on this load transfer can be equated by expressing \( \frac{D_d}{t_D} \), as:

\[
\frac{D_d}{t_D} = \frac{t_1 + t_2}{t_1} = k
\]

Therefore, \( t_2 = t_1 (k-1) \)

Substituting Eqn. (VIII.26) into Eqn. (VIII.24) yields:

\[
\frac{q_e}{q_{e_{DW}}} = \frac{10(k-1) - (k-1)^3}{10(k-1) + (k-1)^4}
\]

This relationship between \( \frac{q_e}{q_{e_{DW}}} \) and \( \frac{D_d}{t_D} \) is plotted in Fig. VIII.12. For a \( \frac{D_d}{t_D} \) ratio between 1 and 1.6, a negligible error results if the external load is equated to the deadweight of the overlying strata. But,
FIGURE VIII.12 Relationship between load transfer at stratum interface and depth of interface.
for an increasing $D_p/t_d$ ratio greater than 1.6, the external load becomes an increasingly smaller fraction of the deadweight load until, at $D_p/t_d = 4$, the external load is almost zero.

Thus, on the basis of the analytical solution, the $D_p/t_d$ ratio can have a significant influence on the external load acting on a dolerite sill. In particular, when $D_p/t_d > 1.6$ the external load acting on a sill can no longer be approximated to be the deadweight load of the overlying strata and can, in fact, be neglected when $D_p/t_d > 4$.

In the light of this finding, Eqn. (VIII.17) can be updated to give the average total load acting on a dolerite sill as

$$q_s = K_8 \left[ \left( \frac{10(k-1)-(k-1)^3}{10(k-1)+(k-1)^4} \right) t_c + \frac{t_d}{2} \right], \quad (N)$$  (VIII.28)

where $K_8 = 08$, a constant ($01=02$)

and $k = \frac{D_p}{t_d}$

On the basis of this equation, the effective critical stress associated with all longwall panels located beneath dolerite sills to date has been re-calculated and is plotted in Fig. VIII.13. Comparison with Fig. VIII.11 confirms that the critical stress level decreases with increasing $D_p/t_d$ ratio.

More importantly, however, based on the limited data available, when $D_p/t_d > 1.6$ the critical stress level associated with failure points appears to be constant. The effects of weathering may still account for the lower critical stress levels associated with failure points where $D_p/t_d < 1.6$. In view of the significance of these findings, an attempt to confirm, or at least
FIGURE VIII.13 Critical stress levels based on analytically derived dolerite load distribution.
support, the findings using a numerical solution has been considered justified.

VIII.5.3.3 Numerical solution

The boundary element program 'DINCL' (section 9.2.2) was utilized to obtain the numerical solutions. This program considers the contacts between the various stratum to be bonded. Thus, in the case under discussion, the dolerite sill and the overlying stratum behave as a single beam and not as two beams having frictionless contacts as assumed in the preceding analytical analysis. Consequently, the overall flexural rigidity (EI) of the strata is increased and so smaller displacements are induced by any given load.

Since considerable computing costs are associated with this program and since solutions were only required to verify the analytical analysis, the number of cases analysed was limited to five. These five cases have been numbered '1' to '5' in Fig. VIII.13 and represent a wide spread of $D_p/t_p$ ratios for both failed and unfailed cases. For each case both the external load, $q_e$, and the total load, $q_{caj}$, acting on a sill were computed. Similarly, these loads were calculated analytically using Eqns. (VIII.22) and (VIII.28), respectively. The ratio between these loads and the corresponding loads, $q_{DP}$ and $q_{DP'}$, as assumed in the elastic thin plate model are presented in Table VIII.6. Also recorded in this Table is the effective span critical stress, $\sigma_c'$, calculated on the basis of \(^{\circ}\) analytically and numerically derived total loads. The following points pertain to the limited results recorded in Table VIII.6:

1) Up to a $D_p/t_p$ ratio of at least 2.13 there is close agreement between the analytical solutions and the numerical solutions. However, in the two cases where the $D_p/t_p$ ratio exceeded 2.13, the agreement is poor.
TABLE VIII.6  Ratios of external, $q_e$, and total loads, $q_{cal}$, derived both analytically and numerically, with corresponding loads, $q_{edw}$ and $q_{pd}$, as assumed in the elastic thin plate model.

<table>
<thead>
<tr>
<th>Case</th>
<th>$D_N$</th>
<th>$e_D$</th>
<th>$q_e$</th>
<th>$q_{cal}$</th>
<th>$q_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$q_{edw}$</td>
<td>$q_{pd}$</td>
<td>Analytical</td>
<td>Numerical</td>
<td>Analytical</td>
</tr>
<tr>
<td>DNC 410</td>
<td>1.88</td>
<td>0.86</td>
<td>0.85</td>
<td>0.67</td>
<td>0.70</td>
</tr>
<tr>
<td>(Unfailed)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sigma I</td>
<td>1.44</td>
<td>0.97</td>
<td>0.92</td>
<td>0.64</td>
<td>0.66</td>
</tr>
<tr>
<td>(failed)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sigma 4</td>
<td>2.13</td>
<td>0.76</td>
<td>0.80</td>
<td>0.44</td>
<td>0.63</td>
</tr>
<tr>
<td>(failed)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C'brook I</td>
<td>2.79</td>
<td>0.43</td>
<td>0.77</td>
<td>0.46</td>
<td>0.71</td>
</tr>
<tr>
<td>(failed)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C'brook IV</td>
<td>2.97</td>
<td>0.35</td>
<td>0.77</td>
<td>0.40</td>
<td>0.62</td>
</tr>
<tr>
<td>(unfailed)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
ii) Although up to a $D_p/t_D$ ratio of about 1.6, the external load acting on a sill is very similar to that as assumed in the elastic thin plate model, the total load acting on a sill is significantly less than that assumed in the model.

iii) The numerical solutions indicate that a relationship exists between the $D_p/t_D$ ratio and the external load which acts on a sill. However, the solutions do not indicate that a relationship exists between this ratio and the total load which acts on a sill.

iv) Utilizing numerically computed total loads, a good correlation exists between the effective span critical stress, $\sigma'_c$, and the $D_g/t_g$ ratio. On this basis, the $D_p/t_D$ ratio may be proportional to the mass strength, or competence, of a dolerite sill.

VIII.5.3.4 Conclusions

An appreciation of the loading conditions associated with a dolerite sill is an essential prerequisite for the formulation of any analytical model. The foregoing analysis has illustrated that the load acting on a sill is significantly different to that previously assumed. In particular, the depth:thickness ratio, $D_p/t_D$, of a sill appears to have a significant influence on the behaviour of a sill.

VIII.6 Conclusions Concerning Elastic Thin Plate Models

Denkhaus (1964) noted that a model must contain all the essential features of the prototype but must also be simple enough to lend itself to logical reasoning, that is, in most cases to mathematical treatment. This usually requires certain assumptions, for example, the relationship between movement and stress. In the case of the elastic thin plate
models, a relationship between failure and stress was assumed on the basis that the mechanical properties of all massive dolerite sills were similar. At the time that it was derived, the elastic thin plate model contained the essential features of the prototype in that a close agreement existed between elastic theory and measured displacement.

Subsequent field observations and a review of the model in the light of the significantly greater amount of data now available have shown that the original Elastic Thin Plate Model, described by Eqn. VIII.1, is unreliable under most conditions associated with local dolerite sills. However, field observations in conjunction with a mathematical review of the loading conditions associated with a sill have enabled the original model to be refined to the 'Effective Span Elastic Thin Plate Model', defined by:

\[ S = \sqrt{1165t_D - 935 \frac{t_D^2}{P_D}} + 2t_D \tan(\theta - 90) \text{ (m)} \] (VIII.29)

It is doubtful whether this model can be considered as an elastic thin plate model. Although the model has been developed from the original elastic thin plate model through a number of logical refinements, it has also been derived on the basis of field data. Furthermore, it is to be questioned whether the elastic thin plate theory as applied to massive dolerite sills is valid. Measured displacements often do not agree with those prescribed by elastic thin plate theory whilst limited measurements indicate that the actual failure mode of a sill is also incompatible with this theory.

Nevertheless, although the model may not describe the failure mode of a sill it provides an accurate means of calculating the minimum panel dimension required to induce failure of a massive dolerite sill. At this stage it is not considered worthwhile to attempt to drive a new model since:
1) in view of the material variations which occur in geology it is unlikely that a more accurate formula than that defined by Eqn. (VIII.29) could be developed to calculate the minimum panel dimension;

ii) indications are that the behaviour of a dolerite sill is influenced significantly by the structural geology of the sill, and this structure is difficult to account for in 'analytically' based models;

iii) the actual failure mode of a dolerite sill, on which such a model needs to be based, has yet to be determined accurately;

iv) as illustrated by Fig. VIII.14, such a model should be based on thick plate theory. This theory introduces sophisticated equations, which make solutions cumbersome and time-consuming to use. Thus, two of the major advantages of such models, namely simplicity and speed, are lost. Furthermore,

v) the widespread use of computers and the introduction of powerful numerical techniques during recent years have put a much wider range of more sophisticated models at the disposal of the engineer. The availability of these relatively accurate models renders the simpler 'analytically' based models almost redundant.

Thus, it is considered that for the foreseeable future the minimum dimension of 'total' extraction panels beneath massive dolerite sills should be calculated on the basis of Eqn. VIII.29. This dimension can be computed quickly for conditions which fall in the range of those from which the model was derived utilizing the curves plotted in Fig. VIII.15. The effective unsupported span at which a sill should fail is read from Fig. VIII.15(a). To this value is added the increase in span required to compensate for the presence of a parting between the base of the dolerite sill
<table>
<thead>
<tr>
<th>GEOMETRY</th>
<th>$\frac{S_{f}(\text{max})}{S_{f}(\text{min})}$</th>
<th>$\frac{S_{f}(\text{min})}{C_{0}}$</th>
<th>THEORY</th>
</tr>
</thead>
<tbody>
<tr>
<td>FACE ADVANCE &lt;= FACE LENGTH</td>
<td>$\geq 8$</td>
<td>$&lt; 4$</td>
<td>THIN BEAM</td>
</tr>
<tr>
<td>FACE ADVANCE = FACE LENGTH</td>
<td>$\leq 2$</td>
<td>$\geq 4$</td>
<td>THIN PLATE</td>
</tr>
<tr>
<td>FACE ADVANCE &gt;= FACE LENGTH</td>
<td>$\geq 8$</td>
<td>$\leq 4$</td>
<td>THICK BEAM</td>
</tr>
</tbody>
</table>

**FIGURE VIII.14** Selection of an analytical model on the basis of mining geometry.
(a) Effective unsupported span.

(b) Parting span.

FIGURE VIII.15 Graphic method for calculating the minimum panel dimension.
and the mining horizon. This latter value is read from Fig. VIII.15(b), the sum of the two values being the required minimum panel dimension.

A feature of Fig. VIII.15(a) is that the effective span, $S_{\text{eff}}$, as defined by the equation:

$$S_{\text{eff}} = \sqrt{1165t_D - 935 \frac{t_D^2}{D_D}} \quad (\text{m})$$  \hspace{1cm} (VIII.30)

is a maximum at $D_D/t_D$ ratio of 1.605, whilst the effective span, $S_{\text{eff}}$, becomes insensitive to increasing $D_D/t_D$ ratio once this ratio exceeds about 2.2. In the light of preceding discussions it is assumed that the $D_D/t_D$ ratio is a compensating factor for applying a thin plate model to a set of conditions which approximate to a composite thick plate. Research into this aspect and the whole concept of the $D_D/t_D$ ratio is still in progress.

A further noteworthy feature is that the minimum effective span required to induce dolerite failure can be approximated in terms of the thickness and geographic location of a sill. This feature is illustrated in Fig. VIII.14 and has been tabulated in Table VIII.7.

It can be concluded that if the theory of dolerite behaviour is to be advanced the actual failure mode of a sill must be established. This information is required regardless of whether an analytical or a numerical approach is adopted to modelling dolerite behaviour. The information is also required because the failure mode of a sill can have a significant influence on mine design, mine support methods and mining operations, especially in thick seam mining operations where strata displacement may be considerable because of large mining heights. Only very limited information relating to this aspect of dolerite behaviour can
be derived from previous investigations since the progress of failure through a sill was monitored rarely.

**TABLE VIII.7** Approximate minimum effective panel dimensions in terms of geographic location and thickness of a dolerite sill as derived from Fig. VIII.14.

<table>
<thead>
<tr>
<th>Location</th>
<th>Minimum Effective Panel Dimension</th>
<th>Shear Strain Factor $S_{eff}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>DNC</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Sigma</td>
<td>1.8</td>
<td>1.8</td>
</tr>
<tr>
<td>Coalbrook</td>
<td>1.6</td>
<td>1.6</td>
</tr>
</tbody>
</table>

Consequently, a new research program into this subject has been initiated. Early findings indicate that the failure mode of a sill may be significantly different to that described by beam and plate theories, with failure being initiated by excessive shear stresses rather than bending stresses. It is considered worthwhile to review briefly these findings, especially since they may have a significant influence on future research investigations.

**VIII.7 Shear Failure Model of a Dolerite Sill**

**VIII.7.1 Structural geology of dolerite sills**

Galvin (1978) initiated an investigation into the structural geology of dolerite sills which had two aims, namely:

1) to determine whether the assumption associated with the elastic thin plate models that 'all sills
be derived from previous investigations since the progress of failure through a sill was monitored rarely.

TABLE VIII.7 Approximate minimum effective panel dimensions in terms of geographic location and thickness of a dolerite sill as derived from Fig. VIII.14.

<table>
<thead>
<tr>
<th>Location</th>
<th>Minimum Effective Panel Dimension</th>
<th>( \frac{S_{	ext{eff}}}{t_D} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>DNC</td>
<td></td>
<td>1,5</td>
</tr>
<tr>
<td>Sigma</td>
<td></td>
<td>1,8</td>
</tr>
<tr>
<td>Coalbrook</td>
<td></td>
<td>1,6</td>
</tr>
</tbody>
</table>

Consequently, a new research program into this subject has been initiated. Early findings indicate that the failure mode of a sill may be significantly different to that described by beam and plate theories, with failure being initiated by excessive shear stresses rather than bending stresses. It is considered worthwhile to review briefly these findings, especially since they may have a significant influence on future research investigations.

VIII.7 Shear Failure Model of a Dolerite Sill

VIII.7.1 Structural geology of dolerite sills

Galvin (1978) initiated an investigation into the structural geology of dolerite sills which had two aims, namely:

i) to determine whether the assumption associated with the elastic thin plate models that 'all sills
possess similar mechanical properties' was valid, and

ii) to determine suitable input parameters for numerical solutions.

This investigation was supplemented by Hepworth (1981) who conducted detailed joint surveys at two locations where dolerite sills were freshly exposed. These investigations revealed that dolerite sills are highly jointed rock masses. Typically, one well-defined near-horizontal joint set and two to three well-defined near-vertical joint sets occur within a sill, along with secondary near 45 degree jointing and occasional shear zones, Tables VIII.8 and VIII.9. Major joints are continuous over many tens of metres whilst minor joints continue en-echelon or die out altogether. Typically, joint surfaces are undulating. On a smaller scale, major irregularities may have wavelengths up to 1 m and amplitudes up to 100 mm, whilst minor irregularities may have wavelengths of 50 mm or less and amplitudes of 4 mm or less.

Average joint spacing bears little resemblance to actual joint spacing, which may range from tens of joints per metre to 1 joint per 3 or 4 m, Plates VIII.1 to VIII.4. Three types of joint opening have been recognised by Galvin (1980), namely, tightly closed joints, infilled closed joints and infilled open joints. Frequently, joint spacing is high in the immediate vicinity of infilled open joints and this gives rise to a weak, often water-bearing, highly jointed zone of rock material, Plates VIII.3 and VIII.4. These zones are expected to have an over-riding influence on the structural stability of the rock mass. Although the mechanical properties of these zones and other joint types have yet to be determined, it is apparent from the observations and measurements made by Galvin (1978,
TABLE VIII.8  Joint orientation and spacing measured in the Ingogo dolerite sill at Ngagane Quarry, Newcastle, Natal.

<table>
<thead>
<tr>
<th>Pole</th>
<th>Plane</th>
<th>Dominance at various localities</th>
<th>Average Joint Spacing No./m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dip</td>
<td>Azimuth</td>
<td>1</td>
</tr>
<tr>
<td>P1</td>
<td>38°</td>
<td>327°</td>
<td>*</td>
</tr>
<tr>
<td>P2</td>
<td>64°</td>
<td>352°</td>
<td>*</td>
</tr>
<tr>
<td>P3</td>
<td>68°</td>
<td>256°</td>
<td>*</td>
</tr>
<tr>
<td>P4</td>
<td>58°</td>
<td>250°</td>
<td></td>
</tr>
<tr>
<td>P5</td>
<td>62°</td>
<td>125°</td>
<td>*</td>
</tr>
<tr>
<td>P6</td>
<td>84°</td>
<td>255°</td>
<td>*</td>
</tr>
<tr>
<td>P7</td>
<td>59°</td>
<td>328°</td>
<td>*</td>
</tr>
<tr>
<td>P8</td>
<td>40°</td>
<td>200°</td>
<td></td>
</tr>
</tbody>
</table>

TABLE VIII.9  Joint orientation and spacing measured in the B4 dolerite sill in the East incline shaft, Bosjesspruit Collieries.

<table>
<thead>
<tr>
<th>Pole</th>
<th>Plane</th>
<th>Average Joint Spacing (No./m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dip</td>
<td>Azimuth</td>
</tr>
<tr>
<td>P1</td>
<td>87°</td>
<td>152°</td>
</tr>
<tr>
<td>P2</td>
<td>85°</td>
<td>226°</td>
</tr>
<tr>
<td>P3</td>
<td>6°</td>
<td>66°</td>
</tr>
<tr>
<td>P4</td>
<td>76°</td>
<td>77°</td>
</tr>
</tbody>
</table>
PLATE VIII.1
Near-vertical and near-horizontal infilled joints in a 250 mm diameter core.

PLATE VIII.2
Jointing in the Ingogo dolerite sill, Nqaqane Quarry, Natal.
PLATE VIII.3
Zone of dry infilled joints.

PLATE VIII.4
Zone of wet infilled joints.
1980) and Hepworth (1981) at five locations in three coalfields, that the mechanical properties of the various dolerite sills overlying South African coal measures are similar.

Furthermore, in view of the very high density of jointing in a dolerite sill, it is still considered feasible to model a sill as an homogeneous isotropic body. The significance of dominant filled open joints may be evaluated by incorporating suitably spaced and defined joints (discontinuities) into such models. However, the strength and elastic properties of the rock mass and rock material must be refined. In particular, the rock mass cannot be considered to sustain high tensile stresses, whilst the elastic modulus of the rock material in compression must be reduced to compensate for the infilling along joint planes.

VIII.7.2 Observations at Sigma Collieries

The first recorded failure of a dolerite sill over a longwall panel was that associated with the mining of Panel 4 at Sigma Collieries (Hardman, 1971). In this instance, six anchors were installed in each of seven boreholes located in the vicinity of the region where the initial failure of the 32 m thick dolerite sill occurred. At a face advance of 142 m the lower 16,5 m to 24,4 m of the sill caved but the remaining 7,6 m to 15,5 m of the sill only failed at a face advance of 180 m, Fig. VIII.6.

Hardman's (1971) report was carefully analysed when reviewing literature relating to this research and has formed the basis for a further report by Galvin (1978). In this latter report, the concept of the shear failure mode of a dolerite sill was proposed. This proposal centres on the deduction that one of the seven boreholes was located in the centre of the region of collapse.
whilst one intersected the edge of the region, Fig. VIII.16. A comparison between anchor movements in each of these two boreholes revealed that, whilst initially the deflection of the bridging dolerite sill increased elastically with increasing face advance, failure of the sill occurred as a result of sliding at the abutments of the undermined sill. Rapid subsidence of the centre of the sill and the edges of the sill occurred at the same instance and at the same rate. In addition, bulking, both within the sill and within the overlying sandstone and shale stratum, was minimal at the centre of the sill and at the three stationary abutments. However, considerable bulking occurred within the upper 9 m of the dolerite sill along the face abutment, giving rise to a 'hump' in the surface subsidence profile. Galvin (1980) attributed this bulking to the continual advancement of the face abutment, which prevented the development of a well-defined sliding plane and, instead, gave rise to a shear zone of considerable lateral extent, Fig. VIII.16. The location of surface cracks in conjunction with a review of caving angles and the investigations into the structural geology of dolerite sills supported the shear failure concept.

Further support for the shear failure concept came as a result of surface subsidence observations above longwall Panel III at Sigma Collieries. In this instance the dolerite sill was 45 m thick and failed at a face advance of 182 m. No borehole observations or surface levelling measurements were made but the surface subsidence is well illustrated in Plates VII.5 to VIII.7.

From these Plates it would appear that a 'plug' type failure of the dolerite sill occurred. Wagner (1980, personal communication) proposed that this failure mode could be explained by equating the driving force, $W_D$, causing the plug of dolerite to slide to the clamping
FIGURE VIII.16 Proposed shear failure mode of dolerite sill overlying panel 4 at Sigma Colliery.
PLATE VIII.5 Areal view of initial and subsequent failure (looking East).

PLATE VIII.6 Surface definition of dolerite sill failure above starting face of panel. Note the uniform subsidence within the subsidence basin.
PLATE VIII.7 Abrupt 0.75 m step in surface profile defining perimeter of subsidence basin.
PLATE VIII.7 Abrupt + 0.75 m step in surface profile defining perimeter of subsidence basin.
force, $R$, resisting the sliding of the plug. That is, if

$$W_D = \rho g V_p,$$

the weight of the plug (N), where

$V_p$ is the volume of the plug (m$^3$),

and

$$R = \mu \sigma_H C_A,$$

the lateral forces (N) acting on the circumference of the plug, where

$\mu$ is the coefficient of friction along sliding planes,

$\sigma_H$ is the horizontal stress (N/m$^2$), and

$C_A$ is the circumferential area (m$^2$),

then failure occurs when

$$\rho g V_p > \mu \sigma_H C_A$$  \hspace{1cm} \text{(VIII.31)}

In order to verify Eqn. (VIII.31) the following information is required:

i) the horizontal stress, $\sigma_H$, which exists in a sill, and/or

ii) the dimensions of the plug of dolerite which fails.

Attempts were made by the author to measure the tectonic stresses in a dolerite sill using a CSIR triaxial strain cell. Measurements were made at a depth of 100 m near the base of a 50 m thick portion of the dolerite sill at Coalbrook Collieries. However, a wide range of results, which were all less than the sensitivity range of the measuring equipment, were obtained. It was concluded that this type of measurement could not be made at shallow depth in strata of very high elastic modulus. Consequently, the author proposed to verify Eqn. (VIII.31) by solving the equation for known geometries to obtain $\sigma_H$. 
This approach was limited by the lack of borehole information concerning the progress of failure through a sill. In fact, at the time, the only information available from borehole anchor movements was that associated with Panel 4 at Sigma Collieries. An attempt was made to deduce the information from surface levelling observations but with no significant success due to the large interval between levelling stations. However, it was reasoned that perhaps the required information could be obtained from a surface levelling and drilling program, conducted over a longwall panel at DNC where the dolerite sill was considered to be in a transitional state of failure \((0.1 \ M_y \leq V_z \leq 0.3 \ M_y)\).

VIII.7.3 Observations over panels 481 and 491 at DNC

Longwall panels 481 and 491, mined between 1970 and 1973, are adjacent panels separated by two lines of 20 m wide chain pillars. Dolerite failure above both panels has been described as transitional since a maximum of only 580 mm surface subsidence was recorded above panel 481, whilst surface subsidence above panel 491 only increased from a maximum of 300 mm to 920 mm sometime between 1976 and 1978. In addition, surface subsidence in excess of 300 mm has only occurred over a limited portion of each panel. Since the original surface topography was fairly uniform and most of the original levelling stations were still intact, it was decided to survey the panels on a 10 m station interval as opposed to the previous surveys which were conducted on a 70 m station interval.

Although sharp steps in the surface topography would have been denuded with the passing of time, this survey still detected plug-like subsidence basins over both panels, Fig. VIII.17. These basins were not only characterised by steeply dipping sides, but also by
FIGURE VIII.17 Surface subsidence over panels 481 and 491 at DNC.
relatively flat bottoms. In view of these features, a borehole was diamond drilled through the sill above panel 481 at the location shown in Fig. VIII.17(a). This borehole was then logged for cavities using both a one-arm and three-arm caliper logger. The results of this logging and that of the core logging are recorded in Fig. VIII.18.

A 480 mm wide cavity was detected at a depth of 46.4 m whilst a 220 mm wide cavity was detected at the base of the sill. Despite the zones of intense jointing which existed within the sill and the fact that over 580 mm of surface subsidence had already taken place, the walls of the borehole were remarkably smooth throughout the sill. This observation is a further indication that bulking within a subsided plug of dolerite is minimal.

Prior to the malfunction of the borehole camera, a 150 degree portion of the 480 mm wide cavity was photographed, Plate VIII.8. From this photograph it appears that separation occurred along a well defined near-horizontal joint plane. The core recovered from this location confirms the observation, Fig. VIII.18.

This observation, together with those relating to Sigma 4, Fig. VIII.6, suggest that prior to initial failure, a dolerite sill may behave as a number of distinct plates. In the case of Sigma 4, the lower half to three quarters of the sill subsided as a distinct plate, with negligible bulking occurring within this plate. Later, the remaining portion of the sill together with the overlying strata also subsided as a distinct plate, again with negligible bulking occurring within the plate. It is reasonable to assume that the +150 mm of bulking which did result occurred at the interface of the two plates. Similarly, the borehole observations over panel DNC 481 have revealed that the lower two thirds of the sill subsided as a distinct
**FIGURE VIII.18** Log of borehole drilled through dolerite sill overlying Panel 481 at DNC.
Plate VIII.8 A 150 degree view of a 480 mm wide cavity detected in the dolerite sill overlying Panel 481 at DMC.
plate, with negligible bulking occurring within this plate. Reference to Plate VIII.8, especially to the left of the photograph, highlights that, should the remainder of the sill subside, it is highly unlikely that both plates would interlock perfectly and so bulking would also occur at the interface.

VIII.7.4 *Horizontal stress distribution*

The observation that a dolerite sill may behave as a number of distinct plates is supported further by measurements recently made at Bomjesspruit Collieries. In this instance, a 52 m thick sill also failed as a number of distinct plates, with the upper 15 m thickness of the sill bridging over an effective area exceeding 100 m x 300 m.

On the basis of the limited information available to date, it has not been possible to accurately verify Eqn. (VIII.31) by calculating the horizontal stress associated with known situations. Values calculated range from a horizontal to vertical stress ratio of 0.92 to 1.75 at Sigma Collieries, and a ratio of 0.83 to 1.11 at DNC. Nevertheless, it is significant that these ratios differ greatly to those commonly associated with South African coal measures where no dolerite sills occur of 0.1 to 0.4 (Salamon and Oravecz, 1975) and are closer to those of 1.0 measured in dolerite dykes by Gay (1979).

Whilst a research programme involving a closely spaced pattern of boreholes is required to confirm the 'shear failure mode' and the 'distinct plate' concepts, these two concepts appear to explain a number of past anomalies concerning dolerite sill behaviour. In the first instance, the shear failure mode concept together with the presence of well developed vertical and horizontal joints within a sill may account for the
'distinct plate' concept. Approximating the shape of the plug shown in Plate VII.5 to be rectangular, the driving force leading to the formation of a plug, namely the weight, $W_D$, of the plug of undermined strata, is defined by the equation -

$$W_D = L_{\text{eff}} \times W_{\text{eff}} \times t_v \times \rho \times g, \quad (N) \quad \text{(VIII.32)}$$

where $L_{\text{eff}}$ is the effective panel length (face advance) (m), $W_{\text{eff}}$ is the effective panel width (m), $t_v$ is the thickness of the plug (m), $\rho$ is the density (kg/m$^3$), and $g$ is the gravitational acceleration constant (m/s$^2$).

Similarly, the clamping force, $R$, which is the sum of the horizontal forces acting on the circumference of the potential plug, is defined by the equation -

$$R = 2(L_{\text{eff}} + W_{\text{eff}}) \times t_v \times \mu \times \sigma_H, \quad (N) \quad \text{(VIII.33)}$$

where $\mu$ is the coefficient of friction across a joint plane, and $\sigma_H$ is the horizontal stress in strata (N/m$^2$).

Eqsns. (VII.31) and (VIII.32) can be simplified, respectively, to -

$$W_D = KgL_{\text{eff}} \quad \text{(VIII.34)}$$

and

$$R = K_{10}L_{\text{eff}} + K_{11}. \quad \text{(VIII.35)}$$

($K_i$, $i = 1$ to 14 being constants).
Thus,

\[
\frac{R}{W_D} = K_{12} + \frac{K_{13}}{L_{\text{eff}}}
\]  

(VIII.36)

From Eqns. (VIII.33) to (VIII.36) it can be concluded that:

i) the probability of a shear failure occurring increases with increasing face advance. This is because the ratio between the clamping force, \( R \), resisting plug development and the driving force, \( W_D \), leading to plug development decreases with increasing face advance, Eqn. (VIII.36);

ii) the development of a plug is not influenced by the thickness of the undermined strata but only by the effective face advance. Thus, the vertical extent of a plug may be influenced by near-horizontal jointing;

iii) the clamping force, \( R \), is a function of the coefficient of friction along joint planes. Thus, the lateral extent of a plug may be influenced significantly by near-vertical jointing.

These three points provide the basis for simply explaining the 'distinct plate' concept. Geometrical considerations dictate that with increasing face advance the driving forces leading to a shear failure increase at a greater rate than the clamping forces. Furthermore, the clamping forces can be reduced significantly by the presence of well developed jointing. Consequently, a plug failure may occur up to some well-defined horizon. A re-distribution of horizontal stresses will occur, resulting in an increase in the horizontal stress acting in the remaining intact strata. Reference to Eqn. (VIII.33) shows that this has the
effect of increasing the clamping forces acting in this strata and so a further increase in face advance is necessary if the strata is to fail as a plug. Thus, the bridging strata may fail in shear as a number of discrete plates, each plate failure being separated by a considerable period of time. This failure mode contrasts with the sudden en masse bending failure prescribed by elastic thin plate theory.

Other anomalies which, perhaps, could be explained by the 'shear failure' concept include:

i) The capability of a 7.6 to 15.5 m thickness of dolerite "ill overlain by 36 m of other strata (Sigma 4) to t-.ge over an area exceeding 155 m x 165 m. The concentration of horizontal stress acting in the bridging strata serve to clamp it together, analogous to clamping together a pack of cards between two fingers. It is interesting to note that similar behaviour has been observed above copper stopes at O'Kiep Copper Mine (Genis, 1980, personal communication).

ii) The minimal increase in the rate of surface subsidence observed over Sigma 4 when the lower two-thirds of the sill subsided, Fig. VIII.6. Again, due to the increase in the horizontal stress acting through the remaining bridging strata as a result of this collapse. This, in turn, explains:

iii) The relative absence of high abutment stress at DNC despite surface subsidence observations which indicate that a sill has not failed. A lack of surface subsidence may not necessarily preclude the collapse of a large portion of a sill. In particular, if the minimum panel dimension is sub-critical, as appears to be the case at DNC (section VIII.5.2), it is plausible that an initial
plug failure could easily occur towards the base of a sill, thus relieving the abutment stresses but also increasing the competence of the remaining portion of the uncaved strata.

iv) The 'hump' in the surface subsidence profile which always occurs at about the face advance at which a dolerite sill first fails to surface, Fig. VIII.16.

VIII.7.5 Conclusions concerning the shear failure model

The 'shear failure' model has been deduced on the basis of very limited information and a number of uncertainties exist concerning its validity. One of the most important of these is the lack of surface definition of a plug type failure over a number of longwall panels. In these instances, failure has been evident in the form of a saucer-shape basin. This behaviour may be explained in terms of the nature and thickness of the strata overlying a sill, since these parameters could effectively mask a plug type failure. However, a more plausible explanation is thought to be that illustrated in Fig. VIII.19. That is, a sill failure may be induced by shear stresses when the sill is thick, or by bending or shear stresses when the sill is thin. Thus, a shear failure of a portion of a thick sill may result in either a shear failure or a bending failure of the remaining portion of the sill, thereby accounting for the two modes of surface subsidence.

The conjectured failure mode illustrated in Fig. VIII.19 would appear to provide a foundation for explaining many past unanswerable questions and anomalies. Therefore, it is considered that the model justifies the time and expense required to verify it. Such an investigation, in any case, is necessary if the theory of dolerite behaviour is to be advanced.
Verification of the model requires the monitoring of the development of failure through a sill. This may be achieved using a large number of boreholes equipped with multiple position borehole extensometers. The boreholes need to be located regularly within the anticipated region of collapse whilst borehole extensometers must be spaced regularly throughout the sill and be monitored continuously. In addition, the investigation needs to be supplemented with numerical analysis. Such analysis should permit an evaluation of the significance of jointing on the stability of a dolerite sill. In this regard, it is worth noting that limited numerical analysis to date incorporating joint elements into the boundary element program 'MINAP' supports the shear failure mode observed over Panel 4 at Sigma Colliery. However, more accurate input data is required before any degree of reliance can be placed on these solutions.
VIII.8 Conclusions

Currently, the most accurate means of calculating the minimum panel dimension required to induce failure of a massive dolerite sill is provided by the 'Effective Span Elastic Thin Plate Model'. This model is defined by the equation -

\[
S = \sqrt{\frac{1165t_D - \frac{935t_D^2}{D_D}}{D_D} + 2t_p \tan(\beta - 90)} \text{ (m)} \quad \text{(VIII.37)}
\]

where:
- \( S \) is the minimum panel dimension (m),
- \( t_D \) is the thickness of dolerite sill (m),
- \( D_D \) is the depth to base of sill (m),
- \( t_p \) is the thickness of parting between base of sill and seam (m),
- \( \beta \) is the caving angle of strata between seam and base of sill,
- with the values 1165 and 935 having units of metres.

The equation has yet to be verified for sills exceeding 75 m in thickness or sills exceeding a depth (to the base) of 140 m. Furthermore, a good appreciation of the caving angle, \( \beta \), is required in order to obtain an accurate solution, especially when the parting thickness, \( t_p \), is large.

Nevertheless, this problem will be encountered with any analytical model. Consequently, in view of the material variations which occur in geology, it can be concluded that it is unlikely that the minimum panel dimension required to induce failure of a massive dolerite sill can be calculated more accurately by an alternative 'analytical' model to that defined by Eqn. (VIII.37).

However, it is most doubtful whether this model, and elastic thin plate theory in general, can account entirely for the failure mode of a sill. Research currently in progress indicates that failure progresses through the full thickness of a sill in a number of discrete stages. Each stage is
characterised by the subsidence of a large slab or 'plate' of
dolerite. Rather than reducing the capacity of the remaining
portion of the sill to bridge across mine workings, as
specified by elastic thin plate theory, the subsidence of
such a plate appears to increase this capacity. It is
proposed that the redistribution of horizontal stresses that
occurs after a plate subsidence accounts for this behaviour.
Furthermore, subsidence of the lower most plate or plates is
due to shearing along joint planes at or near the plate
abutments. Depending on factors such as vertical:horizontal
stress ratio, ultimate plate thickness, and the distribution
of major geologic discontinuities within the sill, the
failure of the thin plate which ultimately results may be due
to either excessive bending stresses (elastic thin plate
theory) or excessive shear stresses at the plate abutments.

It is important to conduct further research into determining
the failure mode of a massive dolerite sill since this may
have a significant influence on mine design, selection of
mining equipment, and mining operations. For example, the
failure mode of a sill may account for the negligible adverse
influence which 'apparently' unfailed massive dolerite sills
have on 'total' extraction operations at DNC. A knowledge of
the failure mode may also be extremely valuable in
controlling surface subsidence, especially in thick seam
mining.
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