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Slope stability considerations of a major excavation in rock: the Mt Newman case study

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SLOPE STABILITY CONSIDERATIONS OF

A MAJOR EXCAVATION IN ROCK

– THE MT NEWMAN CASE STUDY

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STATEMENT

I hereby certify that the work presented in this thesis has not been submitted for a degree to any other university or similar institution.

P A GRAY

The work presented in this thesis is part of a major project concerning geotechnical and pit wall design aspects of the Mt Newman open pit mine at Mt Whaleback in Western Australia. The project was commissioned by Mt Newman Mining Company Pty Ltd and the writer gratefully acknowledges the permission of the Company to publish material relevant to this thesis.

The writer was the only one involved with the preparation of this thesis and is solely responsible for the ideas and concepts presented in it. However, he acknowledges gratefully the support and encouragement of the Department of Civil and Mining Engineering, University of Wollongong. In particular, the writer would like to express his gratitude to his supervisor, Dr Robin Chowdhury, Associate Professor, for many useful discussions, for effective supervision of research and for continual help, advice and encouragement.

The writer was also the technical director the the Mt Newman Project and the work for that Project was either performed by him or at his direction. He was responsible for taking major initiatives and decisions and for developing innovative techniques. The writer would however, like to acknowledge the contributions of his co-workers and others to the Mt Newman Project. In particular, the writer appreciates the assistance of Keith Preston for work on engineering geology, Tony Purdon for assistance with computer graphics modelling and Lawrence Leung for work on cross hole seismic tomography. The writer would also like to acknowledge the work of the CSIRO Division of Geomechanics in connection with structural geology, and the work of Australian Groundwater Consultants in connection with groundwater studies. The writer was the principle author of the report submitted to the Company about this Project.

The writer would also like to thank Denise Paine and Michael Wilyman who respectively typed the manuscript and assisted with the figures.

CONTENTS

This thesis is concerned with the slope stability considerations of the North Wall at the Mt Newman open pit mine in Western Australia. This is the largest open pit iron ore mine in the world and the design of the North Wall presented an immense challenge. The results of detailed investigations and analyses into the geological and geotechnical factors which influence large-scale pit wall stability and subsequent design are presented in this thesis.

Based on conventional mapping and drilling the geology has been studied including a detailed consideration of structural geology. Moreover, engineering geological aspects were considered comprehensively. Particular attention was given to the implications of geology, with three-dimensional consideration of geological structure, in relation to the geometry of the North Wall. These detailed considerations are synthesised into a clear picture of how the geological structure and engineering geological characteristics will influence slope stability considerations which are most significant for decisions in respect of mine design. These include potential failure mechanisms as well as factor of safety for bench-scale and overall stability.

Innovative techniques have been used successfully to assist in providing a clear definition of the geological and geotechnical picture at Mt Newman. These include cross hole seismic tomographic imaging to define geological structure on a large scale, detailed three-dimensional computer graphics modelling in order to determine the shape and plunge of potentially unstable structures, and collection of all borehole geophysical data including gamma sonic and neutron logs with assay data in order to verify lithological interpretations.

This investigation has also been concerned with a critical appraisal of previous geotechnical work at Mt Newman and, in particular, the shear strength test results. On the basis of this appraisal a new comprehensive testing programme was developed in order to define the shear strength of the rock masses and especially of the sheared and disturbed Jeerinah Shales and Fault Shales.

The shear strength testing programme was developed on the basis of modern concepts concerning the behaviour of rock masses. Reliability of data was considered before finalising shear strength parameters for different types of rock mass.

Failures which have occurred at Mt Newman were studied and appropriate slope stability analyses conducted. From these studies back analysed values of shear strength parameters were obtained and compared to corresponding values from laboratory measurements.

Stability analyses for the North Wall were made using several limit equilibrium models considering slip surfaces of circular and non-circular shape. Before considering the potential for major overall failures in this way, potential failure mechanisms were considered on the basis of structural geology and engineering geological parameters. More importantly, sections were chosen for analysis in the following different ways: (a) conventional north-south sections, (b) sections normal to the pit wall, and (c) sections in the maximum plunge direction. Consideration was given to shear strength anisotropy wherever appropriate.

Stability analyses were also made considering potential bench-scale instability using planar failure and wedge failure models.

Having assessed the relative importance of various factors and the interaction of geological and geotechnical engineering parameters, consideration was given to the economies of several design options for the North Wall. An assessment of the alternatives indicated that two options were significantly better than others from the point of view of stability and economics. These two options have therefore been combined together to formulate the extraction sequence for the North Wall in the short term. In the longer term, the decision process involved in making the final choice of pit wall design will be based primarily, although not exclusively, on the results of an actual field trial involving the North Wall excavation. (Note that no failures in the Jeerinah Formation have, as yet, occurred.)

A cautious approach to the mine design and extraction sequence has been recommended based on an appreciation of the fact that high shear strengths for the Jeerinah Formation measured in laboratory tests have yet to be proven in the field. The strategy associated with this cautious approach is the incorporation of a toe buttress of variable height. Decisions to finally remove the toe buttress would be taken in due course based on further observations of the performance of the North Wall excavation coupled with engineering judgement.

The whole process of analysis and synthesis described in this thesis, which is primarily relevant to the Mt Newman mining operations, has significant implications for future open pit mining elsewhere in Australia and the world.

CHAPTER 1

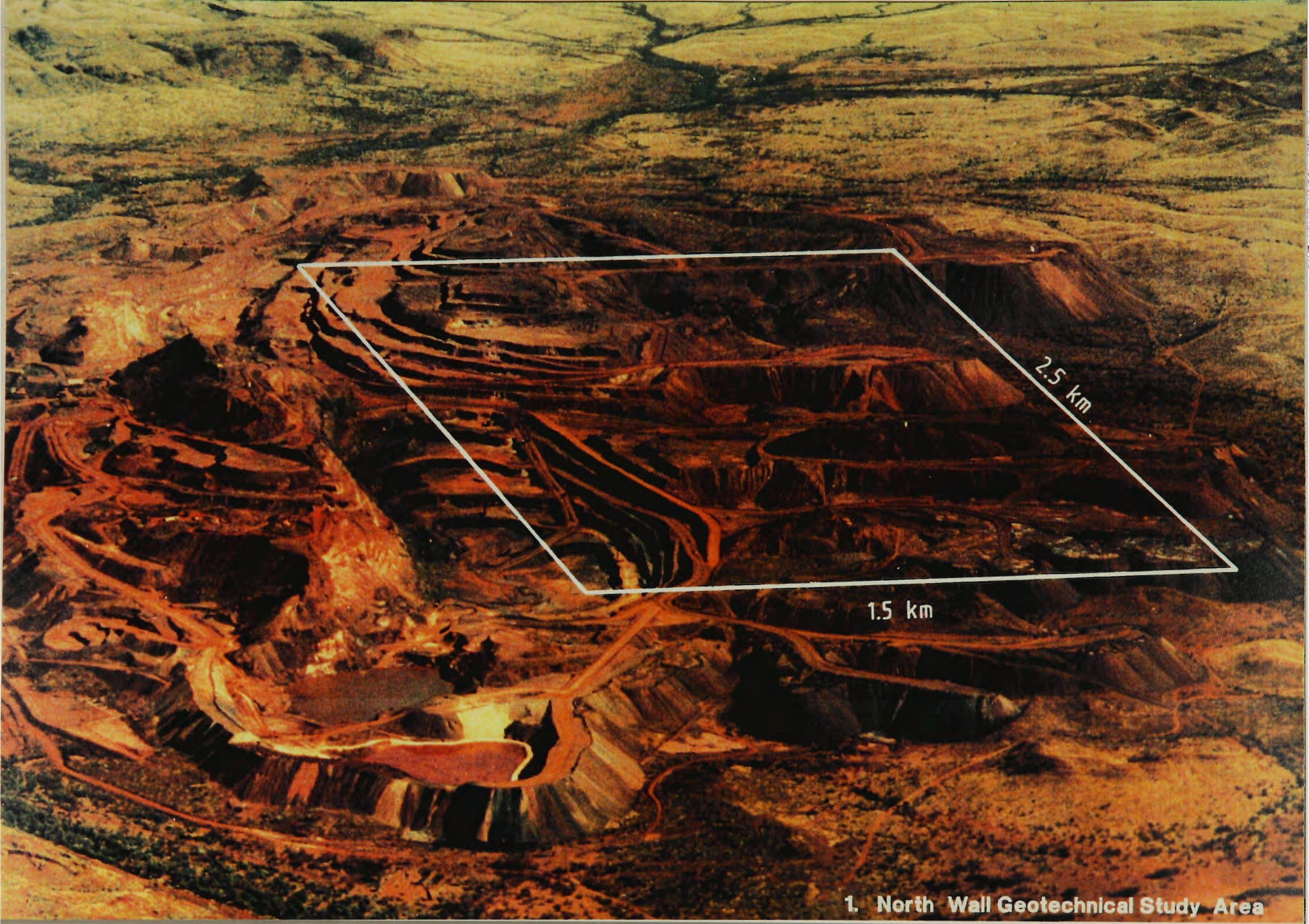
INTRODUCTION AND SCOPE

1.1 AIM OF THIS THESIS

The research work presented in this thesis is the result of detailed investigations into the pit wall design of the North Wall of Mt Whaleback open pit mine, which is the largest single pit iron ore mine in the world. A brief description of this mining project is given in Chapter 3. The major aim was to identify the significant parameters which influence slope stability of such a major rock excavation and, in doing so, to use a range of new techniques in order to gain a better insight into the total geotechnical picture at Mt Whaleback.

It should also be stated here that Mt Whaleback is not only one of the largest open pit mines in the world but is also one of the most structurally complex. In addition, previous stability analyses of the North Wall of Mt Whaleback had indicated potential slope failures of up to 60 million tonnes in size. Therefore it was of crucial importance to determine the salient factors controlling pit wall stability and design and these are presented in this thesis. These factors are somewhat different to the traditional view of slope stability and pit wall design. Given the size and complexity of the Mt Whaleback operations, these factors are also of interest to the open pit mining industry generally.

The aim of the original research effort was to investigate an optimum wall design which was thought to lie between two extreme alternative wall designs. These were the so-called 'Northern Option' and the 'Southern Option' details of which are given in Chapters 10 and 11. For the North Wall, these two wall designs covered an area approximately 2.5km x 1.5km and it rapidly became apparent that insufficient borehole information and insufficient face exposures were available in this area in order to provide the basic geological information required.



1. North Wall Geotechnical Study Area

1.2 LIMITATIONS OF CONVENTIONAL DRILLING

In order to try and obtain more information a detailed drilling programme was undertaken consisting of 15 fully cored diamond holes with a total meterage of 2108m and 51 percussion holes with a total meterage of 7385m. However, even this amount of drilling together with existing holes, was insufficient to provide detailed geological information over the whole area and so borehole locations were chosen very carefully. Where available, these boreholes were located in 'windows' in the waste dumps on the North Wall and were also concentrated at the eastern end of the North Wall where existing pit walls were close to final pit limits and decisions were, therefore, required to be made urgently.

In addition, all relevant pit faces were mapped in order to provide information about the overall geotechnical picture.

A review of the information likely to be available for this research project after completion of this drilling, indicated that even this information would be insufficient in order to cover the area in detail. The cost of even more drilling to provide sufficient supplementary information would be prohibitive. A re-assessment of the mining constraints on the Northern and Southern Options also indicated that the Southern Option presented many operational difficulties and a management decision was taken to concentrate the area of investigation between the Northern Option and the then existing pit limit designs such that maximum ore could be extracted.

1.3 SCOPE OF INVESTIGATIONS

In order to maximize the amount of information obtained from drilling and face mapping and gain knowledge of the values of relevant parameters sufficient for accurate geotechnical analyses, conventional and innovative data gathering and interpretation techniques have been used as follows:

- For identification or confirmation of lithology, detailed geophysical logs on all holes including gamma, sonic, neutron and caliper measurements;

- Detailed assay sampling and testing on all holes in order to confirm the extent of ore and waste and to complement information of lithology and stratigraphy;

- Cross hole seismic tomographic imaging of the structural geology in order to confirm large scale structures, this was extremely important for understanding folding in the Jeerinah Formation. It would be extremely expensive to get the same information by commissioning more drillholes;

- Intensive groundwater monitoring and in–situ testing to determine field parameters such as permeability and transmissivity and to identify perched watertables and any artesian conditions;

- Careful testing of materials which had undergone complex changes and disturbance during their geological history in order to determine realistic shear strength values. (The information previously available was only for shear strength of intact rocks or for the residual shear strength along joints);

- Sophisticated three dimensional computer graphics modelling of complex fold structures enabling the correct fold styles to be determined;

- Back analysis of all previous failures on Mt Whaleback enabling alternative determination of realistic values of field shear strength parameters which could be compared to corresponding values obtained from laboratory testing;

- Stability analyses in areas of potentially unstable structures using modern methods of slope stability analyses. Limit equilibrium methods developed by Spencer and Sarma were found to be particularly useful;

- In order to determine the most appropriate cross sections for slope stability analyses 3-dimensional computer graphics was used. The sections used were of three types: (1) the conventional mine sections originally planned two decades back which in this case were North-South (the North Wall extends approximately East-West), (2) sections which are normal to the wall, and (3) sections taken in the maximum plunge direction of potentially unstable structures in the North Wall (approximately NE-SW direction);

- An economic assessment of various pit options incorporating removal costs as well as capital equipment;

- Pit option selection on the basis of both localised face stability as well as overall pit slope stability and this required 3-D consideration of geological structures and slope stability.

CHAPTER 2

IMPORTANT CONSIDERATIONS FOR THE STABILITY OF EXCAVATED ROCK SLOPE

2.1 INTRODUCTION

2.2 GEOMETRIC PROBLEMS

2.3 ROCK STRENGTH

2.4 NORMAL STRESS RANGE

2.5 JOINT ORIENTATION AND STRUCTURAL GEOLOGY

2.6 GROUNDWATER

2.7 THE RELEVANCE OF PROGRESSIVE FAILURE

2.1 INTRODUCTION

Open pit mining has created the largest excavated rock slopes in the world and rock slope stability is a very important consideration for any large scale surface mining operation. Open pit mining is also one of the most cost–effective ways of extracting large volumes of minerals from the ground and there are a large number of different techniques and equipment employed to do this. Traditionally for very soft rocks, continuous mining systems are employed such as scrapers or bucket wheel excavators. For weak to medium strength rocks overlying coal seams, draglines are the most economical way of removing overburden. Nevertheless for traditional open–pit mines in weak to very strong rocks, truck and shovel operations are used and these give the most flexibility of any mining system and are hence the most common.

The particular method used for open pit mining is important because it determines to a large extent the shape of the pit walls and hence has a major influence on rock slope stability. Except for some special cases of footwall mining, truck and shovel operations have benches and berms of varying heights and widths. These heights and widths are determined by four major factors being:

- the reach of the equipment
- the shape and dip of the orebody
- safety considerations from falling rocks, and
- rock slope stability.

It can therefore be seen that rock slope stability is only one consideration in open pit mining, albeit a very important one. Some of the most important factors for rock slope stability as they specifically relate to open pit mining are outlined in this Chapter.

2.2 GEOMETRIC PROBLEMS

Open pit rock slopes are designed to follow the shape of the orebody as closely as possible up to a maximum practical overall slope angle of approximately 55° - 60°. Since Mines Department regulations will normally not allow overhanging or near–vertical batter faces, this means that maximum batter face angles are generally 70°.

Bench heights are dependent upon the equipment used and upon the regulations which in Australia normally allow a maximum bench height of 20m (except in special circumstances). Berm widths can typically range from 4 – 25m; therefore in a practical sense, it is very difficult to get an overall slope of greater than 55° - 60°.

In terms of rock slope stability, berms are primarily included to catch small rock falls and increase the safety of miners working below these slopes. However in some cases (see Ref 2.28) berms actually decrease stability because they enable surface water to penetrate the rock slope. In the case of Koolan Island iron ore mine in Western Australia, surface water was penetrating the footwall through the berms and creating major washouts and pit wall instability (Ref 2.39). This problem was simply solved by creating a berm–less footwall slope.

For many open pit mines, the orebody is an irregular shape and is dipping at some preferred orientation. The pit slope which is below this orebody and generally follows its dip is called the footwall, whereas the opposite slope is called the hanging wall. Both the footwall and the hangingwall often have very significant structural implications based purely on this geometry. For example, the major structural orientation in the footwall normally (but not always) follows the dip of the orebody. Therefore stability problems of footwall slopes are often associated with exposing or 'daylighting' this major structural orientation in batter faces.

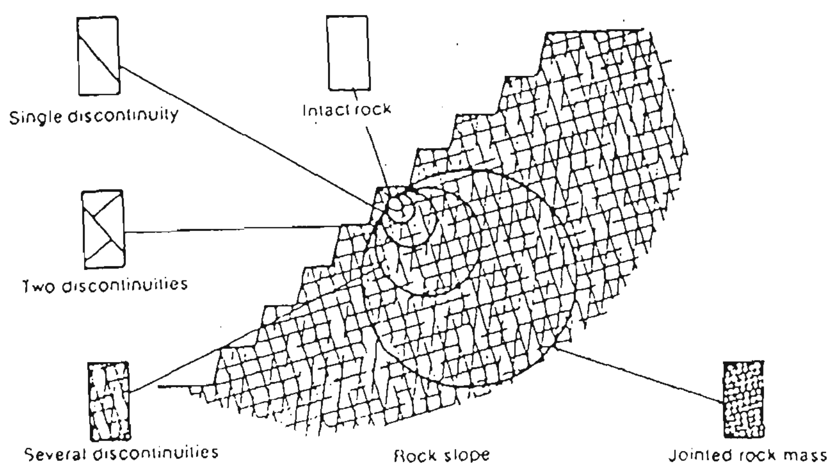
On the other hand, the major structural orientation in the hangingwall often dips back into the slope and therefore toppling failures are more likely than sliding failures. There are, however, exceptions to this. A subject of this thesis is an example. At Mt Newman a classic hangingwall situation does not apply. Because of complex geology involving a major fault the structure is not favourable to a toppling type failure on the North Wall. This will become clear in subsequent Chapters.

The above examples demonstrate that some rock slope stability problems are simply controlled by their geometrical relationships to the orebody and the structural orientation within the rocks. However this is only applicable in a general sense and more specific stability considerations are described in the following sections.

2.3 ROCK STRENGTH

The strength of rocks has been covered in great detail in the literature, but since rocks are anisotropic and have a wide range of strengths, it is important to identify which rock strength is relevant to a particular open pit stability problem. It is therefore important to firstly consider the scale of the problem. Hoek (Ref 2.37) has summarised scale as being an important factor as shown below.

Figure 2.1: Simplified representation of the influence of scale on a type of rock mass behaviour model.



On a small scale of typically a few centimetres, it is the intact strength of the rock material which is important. This assumes that there are no discontinuities in such a small rock sample. On a larger scale of a few metres, it is normally the predominant discontinuity direction which controls rock strength since a greater number of these predominant discontinuities are encountered than any other discontinuities. On a very large scale of the whole slope, it is a combination of many discontinuities and weak rock bridges that control rock stability and these points have been covered at great length in the literature (eg. Refs 2.3, 2.4, 2.6, 2.8, 2.10, 2.20, 2.22, 2.35, 2.47, 2.83).

The above model of rock strength is correct in a simplistic sense, but there are many other important parameters which govern rock strength as applied to open pits and these are described below.

2.3.1 Rock Mass Strength

The strength of rock masses is a subject which has been discussed at great length in the literature and involves the strength of both the rock substance (intact rock) as well as the strength along discontinuities in the rock. The relative significance of each of these two factors is still the subject of some debate but it is probably true to say that their significance should be treated separately for each problem.

The rock mass strength is controlled by movement along discontinuities as well as by translation, rotation, shearing and or crushing of the intact rock blocks which are separated by these discontinuities. In addition, the strength of rock samples determined from laboratory tests can be significantly different from field strength depending on specimen size, loading rate, moisture content, intermediate principal stress and on the orientation of the failure surface with respect to the specimen.

Therefore it can be seen that rock mass strength is very complex and is difficult to determine from laboratory tests alone. Essentially there are three factors which control rock mass strength. These are: (1) the intact strength, (2) the discontinuity strength, and (3) the complex interaction of these two components. The factors controlling rock mass strength are discussed in the following sections.

2.3.2 Intact Strength

Intact rock strength, that is the strength of rock samples with no discontinuities within them, is relatively straight forward and normally characterised by elastic isotropic behaviour with a brittle mode of failure. The intact strength of a rock sample is conventionally defined either by its uniaxial compressive strength (UCS) or by its triaxial strength.

The intact strength of rocks, as defined by UCS, has a very wide range from greater than 400 MPa to less than 1 MPa (Refs 2.9, 2.15, 2.20, 2.22, 2.37, 2.48, 2.84). There are very few examples of excavated rock slopes greater than 400m high, therefore the shear stresses in excavated rock slopes seldom exceed 10 MPa (Refs 2.14, 2.16, 2.19, 2.33, 2.44, 2.45, 2.51, 2.78, 2.87). Consequently the intact strength will be exceeded only for very soft rocks.

Therefore for all practical purposes, the strength of the discontinuities has a major influence on rock mass strength and hence rock slope stability in medium to strong rocks. In some instances, rock mass behaviour will still be controlled by discontinuities even in soft rocks.

The brittle mode of failure of intact rocks has been related to the Griffith (Ref 2.31) theory of rupture which was originally designed for other brittle materials such as glass. Griffith's theory is that fractures are initiated when the tensile strength of the material is exceeded at the ends of microscopic cracks in the material.

These cracks could be at crystal boundaries or grain boundaries or other microscopic weaknesses. Hoek (Ref 2.38) summarised the implication of Griffith theory in terms of the conventional Mohr failure envelope as being

$$\tau = 2(|\sigma_t|(|\sigma_t| + \sigma'))^{1/2} \quad (2.1)$$

where τ = shear strength, σ' is the effective normal stress and $|\sigma_t|$ is the absolute value of the tensile strength of the material. However, this was relevant to tensile stress fields.

For rocks subjected to compressive stress where frictional strength is important, the Griffith theory has been modified and the shear strength is defined by:

$$\tau = 2|\sigma_t| + \sigma' \mathrm{Tan}\,\phi' \quad (2.2)$$

where ϕ' is the angle of friction along the fractures.

Hoek (Ref 2.37) has shown that both the original Griffith theory and the modified theory do not predict failure of intact rock very well. The original Griffith theory underestimates strength at high normal stresses whereas the modified theory overestimates strength.

The above problems with the Griffith failure criterion for rocks have led others to develop alternative failure criteria (Refs 2.10, 2.32, 2.33, 2.34, 2.36, 2.49) and the most widely used at present is the Hoek Brown failure criterion.

The Hoek Brown failure criterion can be used both for intact rock and for jointed rock masses and takes the form:

$$\sigma_1' = \sigma_3' + (m\sigma_c\sigma_3' + s\sigma_c^2)^{1/2} \tag{2.3}$$

where

σ_1' = is the major principal stress at failure

σ_3' = is the minor principal stress at failure

σ_c = is the uniaxial compressive strength of the intact rock

and,

m & s = are empirical constants

Hoek (Ref 2.38) has described how this criterion can be effectively used for intact rock, joint surfaces and for jointed rock masses and in each case it produces a curved failure envelope. In addition, Hoek also demonstrated how the Hoek Brown failure criterion could be used to model the shear strength of clays in order to demonstrate its very wide applicability to geological materials. He fitted his failure criterion to published strength data on intact London clay which gave values of σ_c = 212 kPa, m = 6.475 and s = 1 with a correlation coefficient of 0.98 and indicated that this failure criterion works well over a wide range of material strengths.

Johnson (Refs 2.49) has also developed an empirical failure criterion for intact rocks which is of the form:

$$\sigma_1' = \left(\frac{M}{B}\sigma_3' + 1\right)^B \tag{2.4}$$

where M and B are rock constants. Johnson derived values for these for a wide range of intact rocks and clays. There is no significant difference between Johnson's criterion and the Hoek Brown criterion for intact rocks. However, his criterion is not applicable to jointed rock masses.

Therefore, for all practical purposes Hoek Brown criterion has much wider applicability than Johnson's criterion.

The strength of intact rocks can therefore adequately be described by failure criterion which produces a curved failure envelope at high normal stresses. However, since the normal stresses in open pits rarely exceed 10 MPa, intact rock behaviour in this relatively low stress range is relevant. It has been shown that in this low stress range the failure envelope is linear. Therefore for open pits and particularly for Mt Newman, a linear Mohr-Coloumb failure criterion quite adequately represents intact rock behaviour (Refs 2.42, 2.63). Nevertheless, as discussed in Chapter 8, both criteria were examined for this thesis.

For many weak rocks failure through intact material would theoretically occur in a direction predicted by the Mohr-Coloumb theory (inclination of $[45 + \emptyset/2]$ to the direction of minor principal stress), and there is no preferred failure direction determined by discontinuities. Considering overall slope failure, the slip surface would be curved rather than planer on a gross scale. However, on a small scale, the exact shape of the failure surface will follow the weakest link in the rock mass which in practice may be clay-coated slickensided shears or similar surfaces at irregular orientations. For stronger rocks the orientation and shear strength of discontinuities becomes much more significant in controlling the shape of the failure surface.

2.3.3 Strength of Discontinuities

The strength of discontinuities is normally the weakest link of rock mass strength and has been described at length by many authors (Refs 2.3, 2.5, 2.8, 2.25, 2.33, 2.47, 2.53, 2.55, 2.56, 2.57, 2.58, 2.79). Discontinuities in this context refer to any substantial and distinct weakness surface in the rock mass including joints, bedding, cleavage, foliation, shears, faults, etc. The strength of these discontinuities is therefore governed by the frictional properties of the surface contacts which could be the rock substance itself or a coating material, the interlocking strength caused by roughness and waviness and any cohesive strength which for discontinuities is normally very low or zero. One way to describe discontinuity strength is that proposed by Barton (Refs 2.3, 2.6, 2.8)

$$\tau = \sigma' \mathrm{Tan}(\phi_b' + \mathrm{JRC}\,\mathrm{Log}_{10}(\mathrm{JCS}/\sigma')) \tag{2.5}$$

where

τ = peak shear strength
σ' = effective normal stress
JRC = joint roughness coefficient
JCS = joint wall compressive strength
ϕ_b' = basic friction angle of smooth planar discontinuities

Other systems have also been proposed to classify the strength of discontinuities (Refs 2.36, 2.58, 2.59) and most of these exhibit a curved failure envelope with increasing normal stress. For discontinuity surfaces, the roughness and waviness will generally be 'over-riden' by shear movement at low normal stresses but be sheared through at higher normal stresses. Hence this will produce a curved failure envelope.

It should also be pointed out that both Barton's failure criterion and the Hoek Brown failure criterion show very similar strength envelopes when fitted to the same data set (Ref 2.38) and therefore their use really depends on the data available. Barton's criterion requires a knowledge of the basic friction angle, the joint roughness coefficient and the joint wall compressive strength, whereas the Hoek Brown criterion requires a knowledge of the unconfined compressive strength and the values of m and s constants. Hoek has simplified this process of fitting the Hoek Brown curves to laboratory data by using a program called LABDATA (Ref 2.41).

The strength of discontinuities therefore depends on the materials in and on either side of the discontinuity surface, the roughness of the surface and the normal stress across it. Two extreme examples of discontinuity strength would be a rough joint surface in fresh granite and a slickensided, 'clay coated' shear in mudstone.

The former surface would have an extremely high strength with a failure envelope which is curved at high normal stresses. It would exhibit an initial frictional strength on a Mohr diagram of approximately 60° – 70° but this frictional strength would reduce at higher normal stresses and become closer to its basic frictional strength. However the second example described above would be a smoother surface and be close to its residual strength. A simple linear plot with typical values of about $c = 0$, $\emptyset = 10°$ would adequately represent this discontinuity strength.

The first example would generally pose no problem in excavated rock slopes since the initial frictional strength (60° – 70°) is almost the same as the overall batter face angle. The second example would often pose a significant problem for rock slope stability, not so much in terms of identifying its material strength, but in identifying the location of the discontinuity itself. Often they are not picked up in boreholes (because they are very thin or are washed out by drilling, etc) and therefore the prediction of rock slope stability in advance of mining becomes a difficult task. As well as their strength, the orientation of these discontinuities is the other factor which has an important effect on rock slope stability and this is discussed in Section 5.

2.3.4 Strength of Jointed Rock Masses

The previous sections have been concerned with the strength both of intact rocks and of discontinuities but a simple combination of the two approaches will not produce the correct rock mass strength particularly for the assessment of open pit stability.

Traditional approaches to this problem have been to take the field mapping information from the site geologists and then combine this with some strength parameter (normally the unconfined compressive strength) and then produce a scale of field rock strength. The two most common ways to do this are either by using the CSIR rock mass rating system (Refs 2.13) or by using the Norwegian Geotechnical Institute's Q rating system (Ref 2.7). Both of these classification systems are similar and have been reviewed by Hoek (Ref 2.38), Singh (Ref 2.76) and others and they were originally designed for underground rock mechanics applications and, therefore, they have several disadvantages for excavated rock slopes.

° **Rock Mass Rating RMR System**

The CSIR rock mass rating system is given by

$$RMR = UCS + RQD + J_s + J_c + GW + RA \tag{2.6}$$

where UCS is the unconfined compressive strength, RQD is the rock quality designation, J_s is the joint spacing, J_c is the condition of discontinuities (ie their roughness and separation), GW is the groundwater conditions, RA is the rating adjustment for joint orientations. Laubscher (Refs 2.54) modified Bieniawski's original RMR rating which had the same five classification parameters (UCS, RQD, J_s, J_c and GW) but sub-divides these further into sub-classes with new ranges and ratings for intact strength, joint spacing and condition of joints. The modified RMR or MBR (modified basic RMR) has a scale of 0–20 (very poor rock), 21–40 (poor rock), 41–60 (fair rock), 61–80 (good rock) and 81–100 (very good rock).

Some of the disadvantages with the RMR system are that it does not take into account the confining stress in the rock nor does it explicitly relate the number of joint sets although these are indirectly considered by other parameters. Considerable weight is also given to the block size since both RQD and joint spacing are classification parameters.

° **The Norwegian Geotechnical Institute Q System**

The Q system is given by

$$Q = RQD/J_N \cdot J_R/J_A \cdot J_W/SRF \tag{2.7}$$

where RQD is the rock quality designation, J_N is the joint set number, J_R is the joint roughness number, J_A is the joint alteration number, J_W is the joint water reduction factor and SRF is the stress reduction factor.

Basically the Q system is based on three major aspects affecting the rock mass which are rock block size (RQD/J_N), joint shear strength (J_R/J_A) and confining stress (J_W/SRF). The range of values for the Q system is 0.001 for extremely poor rock to 1000 for excellent intact rock.

Some of the disadvantages with the Q system are that the rock material strength is not taken into account directly and the SRF is used to modify this as appropriate. Also the orientation of joints is not considered since the number of joint sets is considered to be more important for movement of rock blocks.

Both the RMR system and the Q system have been related to each other. Bieniawski (Ref 2.13) gave the relationship as:

$$RMR = 9 \ln Q + 44 \tag{2.8}$$

Singh (Ref 2.76) developed slope adjustment values for use for slope stability or blasting purposes and also found that the logarithmic correlation between the slope adjustment values in the RMR and Q system was

$$RMR = 19 \ln Q + 26 \tag{2.9}$$

which is slightly different from their direct relationship shown above.

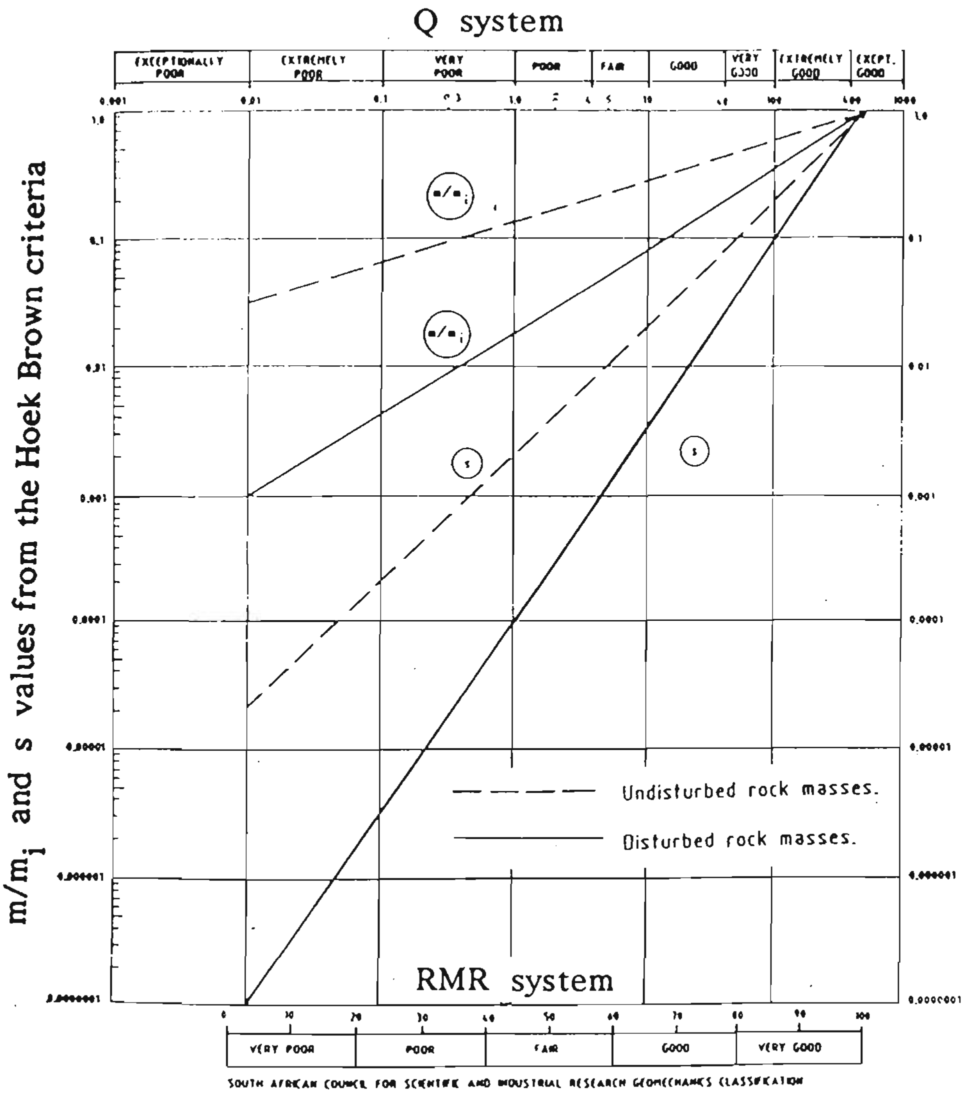
Hoek (Ref 2.42) has critically reviewed both of the above classification systems and pointed out that some of the parameters they use are not rock properties and others are not representative of rock mass characteristics. For example, both the systems include the effect of groundwater whereas this is totally independent of the rock itself and stability calculations should be performed in terms of effective stress anyway. Therefore although groundwater pressures are important, they are not a fundamental rock property.

Also both systems use RQD which is a measure of the number of intact pieces of core over 10cm long obtained from rock core. The relevance of this to rock slope faces on a scale of hundreds of metres in questionable. Both classification systems were originally designed for use in underground tunnel applications and have been subsequently adapted for use with rock slopes (Ref 2.7, 2.11).

However in a large scale failure of a rock slope, the failure surface may be 10–50m behind the rock face. Therefore the rock mass characteristics of the rock near the failure surface will be different to that at the exposed rock face which has been subjected to both blasting and stress relief. The rock classification systems are undoubtedly a useful tool to relate measurable field parameters to rock behaviour and strength but for large open pit slopes they do not yet provide the complete answer to rock mass strength. The combination of discontinuity measurements with geophysics appears to hold considerable promise in defining a rock mass classification system more realistically.

The Hoek Brown failure criterion can be related to the Q and the RMR systems and in this way they can produce a general indication of rock mass strength. The figure below shows a range of m/m_i and s values from the Hoek Brown criteria related to Q and RMR both for disturbed and undisturbed rock masses. In this case the m_i value is simply taken from Hoek's table for intact rock (Ref 2.37, 2.38, 2.63).

Figure 2.2: Relationship between Q and RMR rock classification systems and Hoek and Brown empirical constants m and s.



° **Rock Mass Strength from Laboratory Tests**

Many laboratory tests and field measurements have been carried out to determine rock mass strength (Refs 2.4, 2.12, 2.24, 2.30, 2.32, 2.46, 2.53, 2.76). Traditionally laboratory tests have involved the compression of an assemblage of pre-cut square blocks with their man-made discontinuities at various angles to the compressive stress (Ref 2.55). These tests indicate that the weakest strength is obtained when the discontinuities are orientated at about 30° to the major principal stress direction. However it is interesting to note that at other orientations to the principal stress direction, failure develops in one of the following ways:

- where the discontinuities are parallel and at right angles to the principal stress direction, failure is traditional shear failure at approximately (45 – Ø/2) to the direction of major principal stress and involves mainly shearing of intact material.

- where the discontinuities are orientated between about 0–30° to the major principal stress directions, a shear zone failure develops which involves shearing through some blocks and movement along discontinuities.

- where the discontinuities are orientated at greater than about 30° to the major principal stress directions, a kink band failure develops where there is complete block rotation between two failure surfaces.

The above laboratory tests were conducted by Ladanyi and Archambault (Ref 2.55) and graphically demonstrate the complex failure behaviour of even very regular laboratory models. In practice, actual failure of rock masses is even more complex.

Some laboratory test work (Ref 2.73) has shown that very highly compacted granulated rock behaves in a very similar way to intact rock. This granulated rock was produced by heating coarse grained marble at 600°C causing the cementing minerals to fracture due to thermal expansion. The results (Refs 2.24, 2.73) quoted by Hoek (Ref 2.38) clearly show that the tightly compacted marble is only slightly weaker than the intact marble and demonstrates that the interlocking and interaction of particle or rock block behaviour is a crucial factor controlling rock mass strength. It should be noted that the stress ranges for this series of tests was much higher than is encountered in rock slopes (ie. UCS = 82.3 MPa, m = 7.39 s = 0.65 for granulated marble) In any case such material would be unlikely to cause problems for rock slope stability in open pit mines.

2.4 NORMAL STRESS RANGE

One of the crucial decisions to make in relation to rock slope stability is 'what rock mass strength is significant?' For most rock slopes it is only the strength of the rock mass in the normal stress range of 0 – 10 MPa which needs to be considered and generally the range 0 – 5 MPa is quite adequate for most practical purposes. Therefore only the materials that will fail in this stress range are significant.

Table 2.1 lists a range of typical rock slopes heights in Australia with the approximate maximum normal stress range. It can be seen that the maximum normal stress is only 9 MPa for deep open pit slopes, whereas for open strip coal mines the maximum normal stress is typically only 2 MPa. However, if the typical size of failure in such mines is considered rather than the overall slope height, the maximum normal stress is much less, normally less than 1 MPa, as shown in Table 2.1.

Table 2.1: Maximum Normal Stresses for Typical Open Pits in Australia

Rock Slope Type	Examples		Max Depth*	Approx Max Normal Stress
Deep Open Pit	Mt Newman	Overall slope Double bench	350m 30m	9 MPa 0.75 MPa
Open Pit **	Saxonvale, Koolan Island, Coronation Hill, Iron Monarch etc	Overall slope Double bench	100 – 200m 20 – 24m	2.5 – 5 MPa 0.5 – 0.6 MPa
Open Strip Coal	Most Bowen Basin Mines	Overall slope Typical failure height	60 – 100m 30m	1.2 – 2 MPa 0.6 MPa

* Planned or actual depth. There are obviously exceptions to this and much deeper pits exist (eg. Bouganville or Bingham Canyon). However even these only have maximum normal stress in the order of 25 MPa. Moreover, overall slope failures are much less frequent than bench scale failures. In this case a double bench failure has been used as a typical example.

** A considerable number of shallow open pit mines (less than 100m) also exists particularly for gold operations. In this case, the normal stress ranges are consequently lower.

This table is based on simple gravitational overburden stresses, however, high horizontal stresses may exist leading to higher values for normal stresses than those shown in Table 2.1. In addition, by using Sarma's method of stability analysis (Ref 2.74) it can be shown that the actual normal stresses on the failure plane can be different to those generated from simple overburden considerations alone. However, the basic premise remains that normal stresses within rock slopes are generally low.

In order to determine the effect of the stress range on rock strength, careful triaxial tests are essential. A detailed series of triaxial tests were carried out for Bouganville Copper Mine on a range of weathered andesite samples (Refs 2.46). These test results were very similar to detailed triaxial test results conducted for Mt Newman (Ref 2.63 and Chapter 8 of this thesis) on disturbed Jeerinah Shale samples.

For both Bouganville and Newman the strength of highly weathered or disturbed samples was several orders of magnitude less than the intact strength of the same material, and in the case of Newman this disturbed strength could quite adequately be represented by a simple linear Tau – Sigma $_n$ plot at these low normal stress levels (Ref 2.41).

The usefulness of the above tests is really limited to the situations where failure will occur through the rock mass. For large rock slopes, failure often occurs on some pre–existing discontinuity which is 'daylighting' at the toe of the slope (Ref 2.30, 2.57) and therefore it is the strength and the orientation of such discontinuities which is of fundamental importance for rock slope stability.

It is the experience of the writer that rock slope failures in Australia almost always occur where structure or discontinuities are 'daylighted' at or near the toe of the slip or in the face of the slope. Examples of this range from simple undercut bedding on a batter face, to exposing predominant joint sets, to faults or shear zones occurring near the toe, or having an in–dipping stratigraphic contact exposed in a pit face.

Some examples of this are listed in Table 2.2 and they range from failures in very strong quartzites at Koolan Island iron ore mine in Western Australia to highly weathered overburden materials at Riverside coal mine in Queensland and Ora Banda gold mine in Western Australia.

Table 2.2 Examples of Failures Caused by 'Daylighted' Discontinuities at the Toe

Mine	Location	Failure Type	Predominant Discontinuity Daylighting Near Toe
Koolan Island WA Iron Ore	Hanging Wall	Wedge	Two joint sets in very strong quartzite
Newman WA Iron Ore	South Wall	Complex bedding slides	Bedding on medium strength shales
Saxonvale NSW Coal	East Wall	Large wedge	Bedding on medium to weak tuffs and mudstones
Riverside Qld Coal	Highwall	Slide on in–dipping contact	Base of Paleochannel/ Shears in weak clay
Ora Banda WA Gold	South Wall	Irregular wedge	Steep, joint set/ shear zone sub–parallel to face in clay

It is interesting to note that even in very weak overburden materials at Riverside and Ora Banda (ie. UCS less than 5 MPa), failure is still controlled by weak discontinuities daylighting at or near the toe. All of these examples listed in Table 2.2 involve the exposure of some predominant weakness surface at or near the toe of the failure and therefore discontinuity controlled failures are not solely restricted to strong rocks.

Stress analysis studies of open pit slopes (Ref 2.33, 2.44, 2.78) also clearly show that high shear stresses are generated at the toe of a slope whereas tensile stresses are generated near the crest. Failure surfaces near the crest of a slope are normally sub-vertical, normal stresses on them are low, movement is therefore tensile rather than in shear, and hence the roughness of the failure surface at the crest of the slope is of little significance. For this reason it is relatively simple for a failure surface to propagate near the crest of slope since only the tensile strength of the weakest link need be exceeded. A predominant discontinuity at the crest of a slope is therefore not normally required for failure to occur. Numerous examples of this are available at mines around Australia. For example, at Mt Newman, WA, many failures have occurred where tensile failure has developed in the upper part of the failure surface along a series of complex joint sets forming an extremely 'rough' failure surface near the crest of the slope.

However the opposite applies at the toe of a slope. Here normal stresses are relatively high, shear stresses are at a maximum and it is the writer's experience that predominant, relatively smooth discontinuities are normally required for failure to occur. Failure through intact rock bridges is possible but unlikely given the high intact strength of the rock compared to the relatively low shear stresses imposed on them.

These points are discussed further in Chapters 8, 9 and 10.

2.5 JOINT ORIENTATION AND STRUCTURAL GEOLOGY

The orientation of discontinuities in the rock mass has long been recognised as being a major factor controlling rock slope stability. The orientation of this discontinuity data commonly refers to detailed scanline or window mapping of joints. However, it should also be mentioned that there are other significant weakness zones in the rock mass which have a controlling influence on rock slope stability. These are faults, shear zones or bedding which generally have a much greater persistence than joints.

In the case of Mt Newman, the Whaleback Fault Zone has a displacement of approximately 600m and is a large scale shear zone with clay and slickensided surfaces. Therefore, these large scale features need to be mapped and considered in the assessment of slope stability as well as the normally smaller scale joint data.

Joint orientation has been used for a long time to define the potential for rock slope failure (Refs 2.26, 2.37, 2.43, 2.66, 2.67, 2.68, 2.75, 2.80). The techniques used to define joint orientation are normally hemispherical (or stereographic) methods being either polar or equitorial projection methods (Refs 2.23, 2.69, 2.81).

In Chapter 4 the discontinuity data is plotted using the equitorial equal area projection method for convenience but other methods can be used. Essentially, these techniques define the orientation of joint sets and the variation of individual joints about a mean pole orientation. These mean poles and the variations about them, can be used to identify potential rock slope failures. This is commonly achieved by identifying rock wedges which are formed by two or more intersecting joint sets.

Statistical methods are often used to define the variability in joint set orientations (Ref 2.50, 2.52, 2.67, 2.82) and this variability can be subsequently used in the statistical analysis of rock slope stability (Ref 2.18, 2.60, 2.70). Rock slope stability analysis can then be undertaken in terms of two dimensional multi-planar or step path failure modes or three dimensional rock wedge or rock block failure modes (Refs 2.26, 2.37, 2.40, 2.68).

A rock wedge in this instance is a mass of rock which is bounded by two free faces (ie. an inclined batter face and a horizontal berm) whereas a rock block is normally defined as a mass of rock bounded by only one free face (ie. a batter face). The stability of rock blocks has also attracted considerable research effort (Refs 2.27, 2.85) which is based on the concept of 'key blocks'. Key blocks are blocks of rock which because of their position and geometry are required to move before any other blocks of rock in a rock face can move. Therefore if they key block is stabilized, all other blocks of rocks are stabilized in the rock face.

The key block concept works well for underground openings but it has limitations when it is applied to rock slopes. Specifically, the key block technique is based on scanline or window mapping of exposed rock faces and therefore does not include considerations of possible deep-seated rock slope instability. Thus key blocks techniques are likely to identify only small scale batter face instability but not larger scale instability.

2.5.1 Kinematic Admissability

The kinematic admissability of a mechanism of rock slope stability must be considered where failure is controlled by discontinuities and this is often checked for potential rock wedge failures. However, such considerations may also be important for more complex failure surfaces. In general, the potential failure surface must normally (but not always) dip at an inclination greater than the friction angle. For simple planar failure it must also dip at less than the slope angle, but for complex failure surfaces (ie. curved, irregular, stepped, etc) the failure surface can dip at greater than the slope angle near the top of the failure mass. The key point here is that the major weakness in a rock mass may be a particular joint set, but if this is dipping at an angle smaller than the friction angle, its significance to potential instability decreases.

Priest (Ref 2.70) has analysed the kinematic stability of both rock wedges and blocks, including the effects of all possible unstable blocks in a rock face. This can be conveniently done by plotting joint orientations as well as the batter face angle and the friction angle together on a stereoplot.

In this way the potentially unstable rock blocks can be quickly identified. For a rock face which has actual discontinuity scanline data available, individual discontinuities can be plotted on face maps and unstable blocks identified using similar principles as with the stereoplots, but with the aid of computer processing (Ref 2.27, 2.70, 2.85). By producing these maps, the inherent variability of discontinuities is considered whereas with stereoplots only the mean pole is normally used.

Another way to consider the variability of discontinuity data is to use statistical methods to determine the probability of failure. These methods can also include the variability of rock strength, groundwater pressures and other parameters in order to produce an overall chart of the probability of failure versus slope angle (Refs 2.60).

It is the writer's experience that the conventional probabilistic approach to determine rock slope failure in open pit mines only works well where there is detailed structural information available, and where the discontinuities have a consistent orientation both within the pit face and in unmined areas (Ref 2.30, 2.63). Methods have been introduced for alternative consideration of uncertainties in relation to discontinuity data, notably by using fuzzy set theory (Ref 2.65) but to date these methods have not been fully developed. In any case, consideration of such approaches is outside the scope of this thesis.

For large complex rock slopes where the failure surface can be a combination of different discontinuities the toe of the failure surface is often defined by predominant in–dipping discontinuities (Refs 2.30). However, the dip of this failure plane could be less than friction angle since material higher up in the failure mass often creates a driving force to cause failure. Therefore traditional stereographic stability methods may often overlook the potential for such a failure and therefore it is extremely important that the structural geology be known in detail. This applies in the case of Mt Newman as will be described in Chapter 4.

The structural geology of large open pit mine slopes is extremely important because the orientation of discontinuities within the rock mass is controlled by it. In the case of complex slopes where discontinuity systems are highly variable a detailed knowledge of the structural geology is essential.

For example, folding on a small scale can indicate large scale folding which may be unrecognised in currently exposed pit faces (eg. the shape of parasitic folding). In addition, the direction of cleavage will often indicate the dip of asymmetric folding or vice versa. Therefore knowledge of the structural geology will indicate the likely orientation of discontinuities within the rock slope on a large scale which may not be available from borehole or face mapping information.

Since large scale rock failures often involve deep seated failure surfaces, structural geology can indicate the likely orientation of the failure surface which may or may not be similar to measurements taken on exposed pit faces. This particularly applies to large complex slopes such as Mt Newman in Western Australia and Bingham Canyon in Utah, USA.

In summary, joint orientation is an important parameter which directly affects rock slope stability. On a small scale of tens of centimetres to several metres, the orientation of individual joints is the controlling factor. On a larger scale, it is the orientation of many joints which is the controlling factor on rock slope stability and the best way to determine these is to have a detailed knowledge of the structural geology (Refs 2.63, 2.71).

It should also be pointed out that there are other characteristics of rock joints apart from orientation which are important for rock slope stability including frequency, persistence, spacing, roughness, waviness (large scale roughness), coatings and frictional properties. However, these parameters all go to make up the strength of rock joints and are not considered further in this discussion.

2.5.2 **Weathering**

The degree of weathering also controls rock strength and hence stability of open pit rock slopes. The traditional weathering classifications (after ISRM) are as follows:

Term	**Description**
Fresh	No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surfaces.
Slightly Weathered	Discolouration indicates weathering of rock material and discontinuity surfaces. All rock material may be discoloured by weathering and may be somewhat weaker externally than in its fresh condition.
Moderately Weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.
Highly Weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.
Completely Weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.
Residual Soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

It can be seen that only the completely weathered (CW) rocks have all remnant structure completely destroyed by weathering and essentially exhibit soil like engineering properties. Residual soils exhibit no characteristics of rock as described above.

For rock slopes in extremely dry environments with no groundwater pressures, this high degree of weathering can actually be advantageous for stability since all remnant structure has been destroyed and, therefore, potential for failure along a predominant discontinuity does not need to be considered. In this case the intact material strength remains low but this is very similar to the overall material strength and is sufficient for stability of individual batter faces (Ref 2.29).

For rocks with a lesser degree of weathering (ie. highly weathered and above) remnant structure does play an important role. Specifically the remnant structure does create weaknesses within the rock mass which will be preferentially followed by a potential failure surface.

Weathering also affects the actual condition of discontinuity surfaces since they are often the natural pathway for leaching within the rock mass. The degree of weathering can therefore be greater next to these discontinuities and if any movement has occurred slickensided surfaces can be present. Therefore although the overall rock may only be highly weathered, its strength may be quite low due to the reduction in strength on its discontinuities.

2.6 GROUNDWATER

Groundwater has been discussed at great length in the literature in relation to rock slopes (Refs 2.1, 2.2) and the basic concepts of flow and effective stress are well known.

Rock slopes are highly variable but in almost all open pit mines flow is restricted to the discontinuities in the rock mass, and it is this which is important for slope stability analysis. Exceptions to this are flow through solution cavities in limestone quarries or flow through washout cavities in laterite (eg. Groote Eylandt mine, Northern Territory) but in general it is pressure in the water contained in the discontinuities which is important.

Given the above, the volume of water contained in the rock mass is generally very small particularly in tight or highly stressed rock masses. This means that even a small volume change can cause a large change in pressure and this is an important point to consider both for recharge and for dewatering programmes and for dewatering programmes and for analysis of slope stability as well. Consequently, it only requires a small inflow of water to make a rock slope unstable, conversely, drainage of only a small quantity of water may lead to sufficient increase in factor of safety for the slope to remain stable.

Rock masses also commonly have a highly anisotropic permeability which can result in considerable variability in groundwater levels (note groundwater levels for Newman described later in Chapter 7 of this thesis). It is the experience of the writer that idealised drawdown curves shown in text books are rarely developed in actual rock slopes. Nevertheless, uplift pressures caused by groundwater have a significant effect on rock slope stability. Numerous stability analyses can be found in the literature, but as a rule of thumb, stability of rock slopes can be increased by as much as 30% by effective dewatering (depressurization). This is typically achieved by installing horizontal drainage holes, but can also be achieved by vertical pumped holes or drainage galleries (eg. Twin Buttes, Arizona, USA).

Groundwater also has other effects adverse to stability quite apart from creating uplift pressures on the failure surface. The first important factor is a decrease in material strength due to an increased moisture content. This is particularly true for soft sedimentary rocks (eg. claystones, tuffs) which, when saturated, may become quite soft. It may be noted that the reduction in stress next to open pit slope faces enables additional water to be absorbed by the rock itself (eg. Saxonvale Mine, NSW, Ref 2.85).

The second important effect of groundwater on stability of open pit mines is erosion and slaking. In tropical areas of Australia, excessive rainfall in the wet season is a significant problem and produces large volumes of water running off pit slope faces. Batter faces enable this surface water to run off rapidly but berms reduce the velocity of flow almost instantaneously. Therefore berms create the ideal location for groundwater recharge. If porous or friable rock is present it often creates extensive erosional problems which can lead to large scale rock slope failures (eg. Koolan Island, Ref 2.28). It is therefore important that berms as well as pit crests are well graded and sealed with clay in areas of excessive rainfall to prevent groundwater recharge and erosion.

Slaking or the effects of wetting and drying cause rocks to degrade but this is generally restricted to the exposed rocks in pit faces and is more directly related to surface water rather than to groundwater. Nevertheless this can be a significant problem and cause undercutting of rock faces and subsequent degradation of batter faces.

Groundwater is therefore an extremely important factor in rock slope stability. Due to the highly anisotropic nature of rocks groundwater tables should not be assumed, but should be measured with piezometers and not with standpipes. Groundwater pressures can change rapidly in short distances in rock slopes since rocks are rarely isotropic porous media. Therefore the groundwater pressure near the potential failure surface should be measured directly rather than based on an average standpipe pressure.

The reader is referred to Chapter 7 for more details on the Groundwater at Mt Newman.

2.7 PROGRESSIVE FAILURE

Slope excavated in cohesive soils and soft rocks have a potential for 'progressive failure'. An understanding of the relevant mechanisms is, therefore, of tremendous interest. Failure may initiate at some locations along a potential slip surface involving a decrease of shear strength below the peak value at these locations. This decrease is associated with a re-adjustment of shear and normal stresses within the earth or rock mass and with consequent enlargement of the zone of over-stress and local failure. The concepts concerning progressive failure have been discussed by Skempton (Ref 2.88, 2.89 and 2.90), Bishop (Ref 2.91 and 2.92) and Bjerrum (Ref 2.93). Factors contributing to progressive failure include non-uniform stress distribution, non-uniform strain-distribution, strain-softening characteristics of soils and rocks as well as recoverable strain energy. Studies of progressive failure have been reviewed by Chowdhury (Ref 2.18) and further references may be found in Bertoldi (Ref 2.94) and Chowdhury (2.95 and 2.96).

Failures of slopes excavated in hard rock always occur along major discontinuities or within weakened zones (eg. Refs 2.97, 2.98, 2.99, 2.100). These failures are, relatively speaking, simultaneous rather than progressive. This was confirmed by literature sources and is borne out by the writer's own experience in Australia and from discussions with co-workers around the world. However, the shear strength along discontiniuties may already have been reduced during past geological processes. Similarly the shear strength within weak zones may be close to residual values even before excavation. In so far as excavation of slopes within hard rock environments may result in further decrease of the shear strength of weak zones, progressive failure may be involved.

However, this is a subject which has not received much attention probably because of the importance of many other aspects of much greater significance such as the characterisation of rock mass strength and the understanding of engineering geological aspects of slope stability.

As discussed in Chapter 8, back analysis of failures which have occurred at Mt Newman has not revealed any significant mechanism of progressive failure.

As far as potential instability of the North Wall is concerned the primary considerations are again the engineering geological aspects, the rock mass shear strength of the weakened zones (eg. the Whaleback Fault Zone) and shear strength along discontinuities. The potential for progressive failure is of secondary importance in establishing a future wall design because the main aim is to avoid any significant failure until most of the ore has been excavated.

The exploration of potential progressive failure with associated developments could be the subject of a separate major research project. It was considered totally outside the scope of this present thesis.

The writer has, however, direct experience of analytical studies concerning progressive failure in overconsolidated clays involving both limit equilibrium and finite element studies (Refs 2.101, 2.102 and 2.104). The writer also has had experience of potential instability involving excavated slopes in soft rock environments (Ref 2.103).

CHAPTER 3

THE MT WHALEBACK MINING PROJECT

The Mt Whaleback mining project is one of the largest single pit iron ore mines in the world and is currently operated by BHP–Utah International – Iron Ore Division. The mine is located at Newman in the Pilbara region of Western Australia approximately 1100km north–east of Perth.

The history of the region dates back to 1896 when a mapping expedition led by Aubrey Newman, named a peak at the eastern end of the Opthalmia range, Mt Newman and a cattle station was established there in 1901. However it was not until 1957 that the un–named, humpback hill 21km further south of Mt Newman was discovered. This of course was Mt Whaleback. A geologist named Stan Hilditch discovered high grade iron ore at Mt Whaleback and in 1960 Hilditch staked claims to Mt Whaleback and surrounding areas.

During 1963, Amax Inc of the USA became interested in Mt Whaleback and subsequently invited CSR to form a joint venture company to test the reserves. These reserves were subsequently proved to be 1.4 billion tonnes of mineable ore at greater than 63.5% iron. This makes Mt Whaleback the largest known deposit of high grade hematite in Australia. By 1965 the Japanese had signed a letter of intent to purchase iron ore and in 1966 BHP became a major purchaser of iron ore and also became a shareholder and operator of the mining company, then called Mt Newman Mining Company Pty Ltd (MNM Co).

A township was established at Mt Whaleback and was called Newman as well as a heavy duty rail line to Port Hedland, 426km to the north. The iron ore mining and railing operations commenced in January 1969 and the mine is currently producing 32 million tonnes of ore per year with a total material movement in excess of 100 million tonnes.

Initial mining development commenced in an area known as the East Pit since this had the lowest stripping ratio. Today the East Pit is well developed and final pit slopes have been established on the South Wall. However the North Wall is less well established and final pit limits have only been developed at the extreme eastern end of the East Pit.

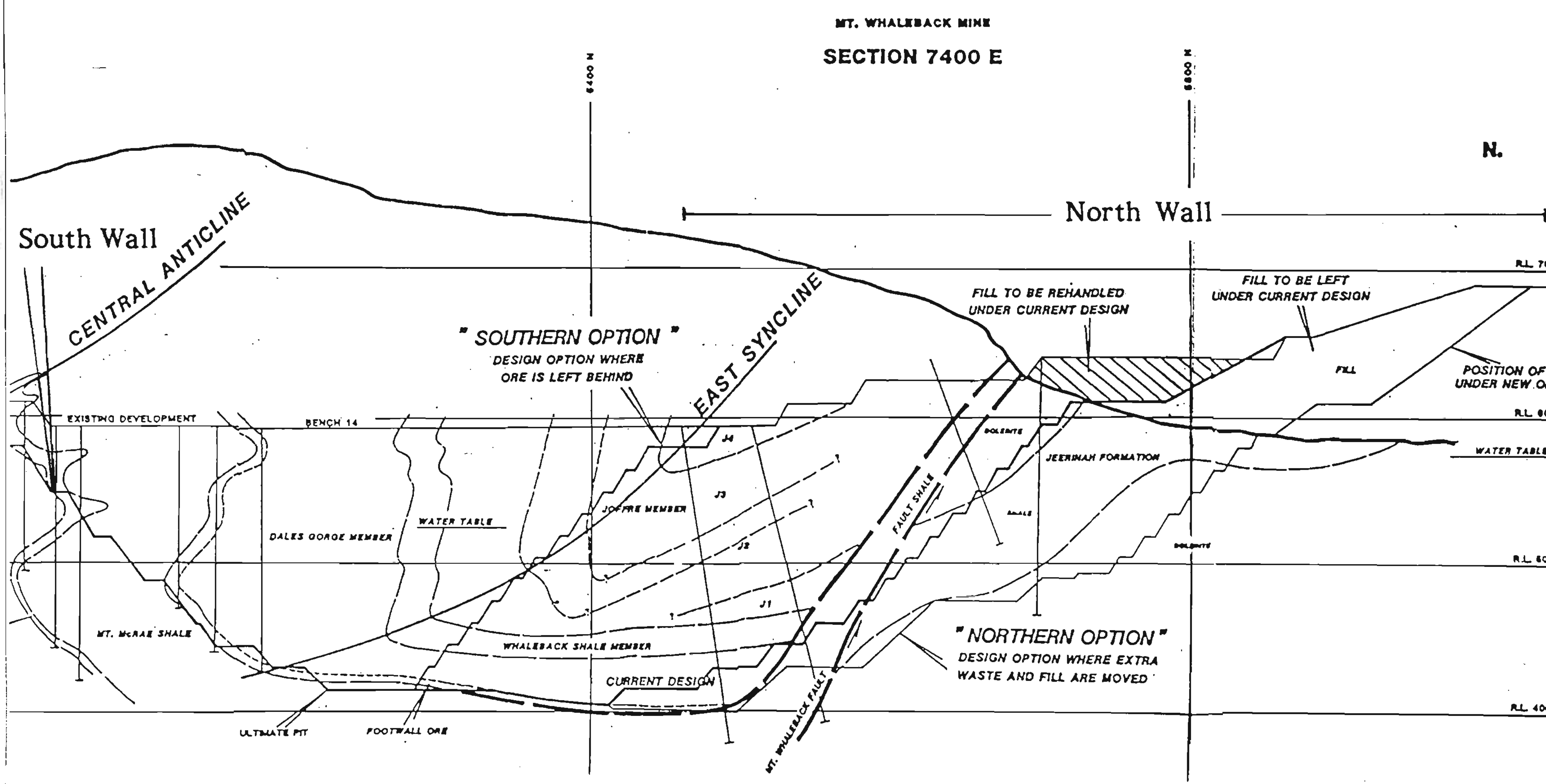
Figure 3.1 shows a typical section through the East Pit showing the South Wall and the North Wall, although more detailed plans and sections are available in the Appendices. This figure shows that the main orebody at Mt Whaleback is the Dales Gorge Member although considerable ore is also recovered from enriched banded iron formation (BIF) in the Joffre Member. The Dales Gorge Member forms a natural syncline which is truncated by the Whaleback Fault Zone (WFZ) on its northern limb. This synclinal structure basically determines the shape of the present and future pit at Mt Whaleback and also accounts for the fact that the South Wall has developed a greater extent of final pit slopes than the North Wall (see Figure 3.1).

The equipment employed by MNM Co is quite varied and is on a large scale. The ore and waste are mined by conventional truck and shovel operations and MNM Co not only have one of the largest truck and shovel fleets of any mine in the world but also have the largest capacity machines. MNM Co have also recently installed (in 1987) an in-pit crushing and conveying system for the waste on the North Wall of the East Pit in order to improve the efficiency of material movement. This crusher and conveyor system is located on an interim slope (as opposed to a final slope) on the North Wall.

Previous geotechnical investigations at Mt Whaleback have by necessity concentrated on the South Wall since final slopes were being rapidly developed on this wall. However, as discussed in Chapters 9, 10 and 11, it has been known for a long time that the North Wall of Mt Whaleback would present significant geotechnical problems. This is not only because of the presence of the large Mt Whaleback Fault in the North Wall, but also because of the southerly–dipping structures on either side of it. In addition, the North Wall is the location of all the major waste dumps and therefore surface mapping or drilling is either impossible or extremely difficult to carry out. Therefore, the structures within the North Wall have, until the completion of this research project, only been known in general terms.

Chapters 9 and 10 outline the stability analyses in detail. It is important to stress here that the major impetus for this research project was due to the assumed southerly–dipping structures north of the Whaleback Fault Zone which were considered to be very adverse to slope stability during the proposed development of the North Wall. Moreover, the final pit limits were being rapidly developed and a major crusher and conveyor facility was installed on the North Wall. All these factors necessitated a detailed research project into the pit wall design for the North Wall.

Given the tight financial position of the iron ore industry and the very large tonnages of waste and ore involved between different possible North Wall Options, a rational and scientific basis for pit wall design was required. The techniques used to pursue this goal were both detailed and innovative and have been applied in a unique way to arrive at preferred design options. These techniques are outlined in detail in the following Chapters.



Scale		
Drn	Typical section through the East Pit	FIGURE 3.1
Dwg No		

CHAPTER 4

THE RELEVANCE OF STRUCTURAL GEOLOGY TO SLOPE STABILITY ON MT WHALEBACK

4.1 SUMMARY

The structural geology of the North Wall of Mt Whaleback is one of the key elements in the overall geotechnical research for pit wall design. As such it has been accorded high priority and has involved a wide range of detailed investigations. The investigations have involved both percussion and diamond drilling, pit face mapping, mapping of costeans as well as collation and scrutiny of all existing geological information.

The investigations have been complicated by the fact that there are very few exposures of the Jeerinah Formation in the present pit and much of the crest of the North Wall is covered in waste dumps. Innovative techniques have been used to try and overcome these handicaps including the use of cross hole seismic tomography (see Chapter 5) and the use of sophisticated three dimensional computer graphics.

The main findings of this Chapter are that the Jeerinah Formation is not a simple 'layer cake' model, but is structurally complex being folded on a large scale. These folds generally plunge to the west although this plunge is variable. In addition to this variable fold style, the Jeerinah Formation consists of alternating 'shale' and 'dolerite' units which have been subjected to regional metamorphism resulting in slate/phyllite and amphibolites being present. This structural complexity has a major influence on pit wall design as discussed in Chapters 10 and 11.

One of the major structural units of the North Wall is the Whaleback Fault Zone (WFZ) and this zone can be conveniently used to separate other major structural units. The WFZ is a steep south dipping intense shear zone which is a normal fault with a displacement of at least 600m. This shear zone itself consists of a melange of altered rock types from both the south and the north of the WFZ and is approximately 10 - 15m wide. The altered rocks range from kaolinitic clays to only slightly altered dolerties and there are obviously numerous faults within the overall shear zone itself. The WFZ also cuts ore bearing horizons and acts as the northern boundary of the orebody.

South of the Whaleback Fault Zone. South of the WFZ the rocks are intensely folded with wavelengths measured in metres and comprise the Brockman Iron Formation. The orebody is truncated at depth by another intense shear zone, called the East Footwall Fault Zone (EFFZ), which therefore acts as the lower limit to the orebody. The rocks which constitute the EFFZ are sheared and folded versions of the rocks stratigraphically underlying those horizons which are mineralized.

North of the Whaleback Fault Zone. North of the WFZ are thick meta-dolerites and slates of the Jeerinah Formation. These rocks are the lowest stratigraphically in the area of interest. The wavelength of folds in these rocks is controlled by the thick and relatively massive meta-dolerites and hence is measured in hundreds of metres rather than in metres as on the south side of the WFZ. Otherwise the style of folding is identical north and south of the WFZ except that the axial planes of folds north of the WFZ trend more to the north-west rather than the west as is the case south of the WFZ. The Jeerinah slates are dominated by a strong south-dipping axial plane slaty cleavage although in conventional mining terminology these are referred to as shales.

4.2 COMMENTARY

The majority of the North Wall structure is hidden from view, unlike that of the South Wall, and therefore it has been necessary to fit together the pieces of the jigsaw puzzle in order to produce a logical and realistic structural model. Heavy reliance has been placed on drilling, and where exposures are available, face mapping.

In order to maximise the amount of information obtained from both the percussion and diamond drilling programmes, innovative engineering, geological and computer techniques have been employed. These techniques are worthy of mention and are as follows:

(a) The use of cross hole seismic tomography as described in the Chapter 5. This technique confirmed the presence of large scale fold structures in the Jeerinah Formation and negated the need for additional drilling in this area.

(b) The use of computer graphics to model three dimensional structural geological surfaces as well as to produce all the plans and sections. The modelling of these surfaces was always undertaken with a structural geologist reviewing the results and therefore the computer generated surfaces were never considered to be correct unless they were checked first. This computer modelling was partly responsible for the determination of the fold styles in the northern Jeerinah Formation. This computer modelling was also subsequently used to section the proposed North Wall design at different angles in order to produce cross sections for the stability analyses.

(c) The use and plotting of both geophysical (gamma) data and assay data down borehole traces on cross sections. Traditionally, the structural geology was interpreted from the lithological log drawn on the cross section with a manual reference to the geophysical and assay data if the interpretation was difficult. However in practice, the gamma log was at a different scale to the borehole trace on the cross section and therefore only point data could be compared by manually using a scale rule. Similarly, the assay data was kept as a computer print out which had to be manually compared with a position on the borehole trace.

Plotting this information on the borehole trace has enabled a much faster interpretation to be made, has highlighted structural anomalies and has for the first time enabled a direct visual representation of both Fe grade and structural geology on the same piece of paper.

(d) The use of a computer graphics plotting packaged called LOGGER to produce the graphical borelogs. This enabled the incorporation of both traditional histogram data (such as weathering, RQD, core loss, etc) with geophysical data (gamma logs) on the same bore log. This was not new in itself but use of digital bore log data was to prove invaluable for Point (c), as well as enabling bore logs to be produced at any scale, size, etc.

This bore log information is a fundamental source of data for this Chapter as well as for Chapter 6. It has been collated and presented to MNM Co on computer discs and has been formated such that it is simple to produce additional copies of the bore logs. A user guide was also presented in the report to MNM Co.

4.3 INTRODUCTION

4.3.1 North Wall Geology

The proposed North Wall of the Mt Whaleback East Pit will be excavated in lower Proterozoic rocks of the Fortescue and Hamersley Groups (see Figure 4.1 and Ref 4.1). These rocks have been strongly folded by westward plunging inclined to recumbent folds with axial planes dipping towards the south. This folded sequence has been truncated either late in the folding history or post folding by flat lying faults, the most important of which is the East Footwall Fault Zone (EFFZ) which forms the base of the orebody. This sequence of structures is, in turn, truncated on the northern side of the orebody by a steep, south dipping normal fault called the Whaleback Fault Zone (WFZ). Thus, the orebody is delineated on its base and on its northern side by fault zones.

To the south of the WFZ are rocks of the Hamersley Group; mineralised and unmineralised members of the Brockman Iron Formation (viz. the Dales Gorge, Whaleback Shale and Joffre Members) occur above the EFFZ and unmineralised parts of lower stratigraphic units, namely, the Wittenoom Dolomite, the Mt Sylvia Formation and the Mt McRae Shale, occur in and below the EFFZ (refer to Figure 4.2).

To the north of the WFZ are rocks which are much lower in the stratigraphy than those mentioned above. These are meta-dolerites and slates of the Jeerinah Formation belonging to the Fortescue Group (see Figure 4.1). These rocks are again folded by westward plunging folds with axial planes dipping at 30–45°SW. The style of folding is similar to that on the south side of the WFZ except that the amplitude and wavelength of folds on the north side is much greater (measured in 100's of metres) than on the south side (measured in metres).

The rocks comprising the WFZ are strongly deformed versions of the rocks within the EFFZ although occasionally parts of the Brockman Iron Formation and Jeerinah Formation are incorporated as well. Figure 4.2 shows a schematic picture of the main structural relationships for the North Wall.

4.4 STRATIGRAPHY

The description of the stratigraphy of the North Wall of Mt Whaleback is based both on the excellent reference work produced by the Resource Development Department of MNM Co (Ref 4.2) and on the detailed mapping undertaken as part of this investigation (Ref 4.3). Recent drilling and mapping associated with the other South Wall pit slope investigations have also enhanced descriptions of individual stratigraphic units (Ref 4.4). A general stratigraphic column for the Hamersley Basin is given in Figure 4.1 and the geology of the rock units exposed or to be exposed in the North Wall is described below. The abbreviations for each of the stratigraphic horizons is given after the formation name.

4.4.1 Fortescue Group

Jeerinah Formation (Pfj)

This is the principal stratigraphic sequence likely to influence pit slope behaviour and design in the North Wall area. It occurs immediately to the north of the WFZ, although drilling and mapping have indicated that sheared components of the Jeerinah Formation are located within the fault zone itself.

The formation consists of alternating grey-green, medium to coarse grained altered dolerites (amphibolites) and green, greenish black to black shales (now altered to slates and phyllites). The dolerites which show ophitic to sub-ophitic texture are considered to be intrusive sills and some contact features within specific dolerite units support this view. Approximately 4km south of Mt Whaleback itself a small exposure of the top of the formation just below the Marra Mamba Iron Formation shows the presence of a pillow basalt indicating a sub-aqueous extrusive phase at a late stage in the development of the sequence.

The sequence of the Jeerinah Formation can be summarised as follows from youngest to oldest (it should be noted that the rock unit names used here differ from former usage by MNM Co and an attempt has been made here to rationalise the usage and present the rock unit names in a more logical manner than had grown piecemeal at Mt Whaleback over a period of time):

UNITS OF THE JEERINAH FORMATION

Unit	Approximate Thickness (m)
Shale C	Indeterminable
Dolerite C	Indeterminable
Shale B	Variable but generally 70–130m
Dolerite B	120m
Shale A	Variable but generally 40–130m
Dolerite A	200m +

Some of these units are exposed in the pit with the Dolerite A being exposed in the extreme eastern end of the pit and being overlain by Shale A further west. Dolerite B overlays Shale A further west and so on. Shale thicknesses appear to be variable along and across strike. These variations are related in some degree to E-W trending fold closures where shales thicken considerably. Variations along strike may be due to any of a number of possible structural or stratigraphic complications, eg. cross-cutting intrusive contacts, primary stratigraphic thickness variations, en echelon folding or cross folding.

Specific petrological and geochemical characteristics for the various dolerite and shale horizons are discussed in more detail later in this Chapter. However, the following broad observations have been used for stratigraphic identification of dolerite and shale units:

- **Dolerite A** is a medium grained grey-green amphibolite with a distinctive top zone (at least 50m) containing black shale xenoliths (inclusions) and vugs with carbonate infilling.
- **Dolerite B** is a medium to coarse gained felspathic amphibolite with ophitic to sub-ophitic texture. The Dolerite B/Shale A contact is indistinct with gradational or transitional characteristics.
- **Shale A** tends to be chloritic and in places is dolomitic. In a few places, Shale A also changes from a predominantly chloritic shale to a sericitic shale and this can be picked up from the detailed assay logs.

Thin shaly and sheared zones have been identified within the dolerite units. The continuity of these zones is uncertain although there appears to be some correlation between boreholes on a few sections.

Shear zones are present within the Shale units. This is most frequent adjacent to the WFZ but others have been located wholly within shale units and remote from the WFZ and these consist of slickensided and disturbed shale.

4.4.2 Hamersley Group

° **Marra Mamba Iron Formation (Phmm)**

This formation consisting primarily of chert and BIF, has no direct influence on pit operations or pit stability at Mt Whaleback. A narrow block of Marra Mamba outcrops just north of the eastern tailings pond within the easterly extension of the WFZ. A dolomitic breccia intersected in one of the North Wall diamond holes within the WFZ contains clasts of folded banded iron formation BIF which may have originated from the Marra Mamba Iron Formation at depth.

° **Wittenoom Dolomite (Phd)**

This stratigraphic unit consists primarily of greyish brown dolomitic shales at the top grading downwards into grey crystalline dolomites. Massive grey crystalline dolomitic breccias have been intersected at depth in some holes drilled through the EFFZ and WFZ. Brecciation and shearing is related to the faulting. These tectonically disturbed components of the Wittenoom Dolomite may have some influence, at specific locations, on the stability of the toe of the North Wall.

° **Mt Sylvia Formation (Phs)**

The stratigraphy of this formation consists of a distinctive BIF unit (Bruno's Band) at the top underlain by chert/shales and siltstones. The lower section contains two thin but distinctive BIF units the lowest of which forms the base of the formation. This formation within the WFZ is complicated by folding, shearing and brecciation but two important sub-units within the formation have been identified in this fault zone and form very useful marker horizons. These are:

- Bruno's Band BIF (or hematite where mineralised), and a
- Siltstone sub-unit

These have been used to define the style of faulting as shown in Figure 4.2. A typical sequence of the Mt Sylvia Formation is given in Figure 4.3.

- **Mt McRae Shale (Phr)**

This formation is shown in Figure 4.4 and is an 'undisturbed' and complete stratigraphic column (Ref 4.4). Within the WFZ and also below the orebody shearing has produced 'thinning' (dismembering) of the unit. The undisturbed unit consists, in the upper part, of a distinctive black shale and pyritic black shale containing a marker horizon identifiable by the presence of pyrite nodules.

The upper limit of this nodular zone is marked by a distinctive massive pyrite band. The lower part of the unit contains chert beds separated by black shale grading downwards into a chert/shale sequence above Bruno's Band. In a weathered state the shales vary from brown to white and are kaolinitic.

- **Brockman Iron Formation**

This formation is the major location of high grade hematite orebodies in the Hamersley Range. Detailed studies over the last 20 years have identified four main stratigraphic units referred to as 'members' within the Brockman Iron Formation. Figure 4.5 illustrates in detail the Brockman Iron Formation Stratigraphy applicable to Mt Whaleback including a detailed gamma log trace which is used to locate horizons in percussion holes.

Dales Gorge Member (Phbd)

This is the main ore-bearing horizon comprising 24 hematite or BIF microbands alternating with 23 shale bands, the latter varying in thickness from a few centimetres up to a metre or more. The enriched Member has an average thickness of about 65m. Unenriched, the Dales Gorge is some 130m thick. The Dales Gorge Member can be divided into four alternating shaly and non-shaly units. The Basal Shaly section (also referred to as the Colonial Chert Member) occurs at the base of the Dales Gorge Member and is officially regarded at Mt Newman as part of the Mt McRae shale. The next unit contains five thin shale bands, is usually high grade and extends from CS6 – DS6 (see Figure 4.5). The Middle Shaly Zone contains seven closely spaced shale bands (DS6 – DS12) three of which are of the order of 1 – 2m in thickness. The upper non-shaly high grade zone contains six thin shale bands.

Whaleback Shale Member (Phbw)

Unweathered, these shales are green to greenish-black. A 'marker' chert band known as the Central Chert Band or CCB occurs just above the base of the unit. The shales weather to pink and brown kaolinitic shales. When tectonically superimposed on Mt McRae Shale, as for example along the Whaleback Fault, there are some problems in distinguishing the two shale units when both are in the weathered state.

Joffre Member (Phbj)

This comprises a BIF succession up to 240m thick with minor shale bands and some sporadic enrichment. Only the lower 100m of this member is thought to be exposed at Mt Whaleback. Using a variation in the number of shale bands present, the Joffre Member at Mt Whaleback has been sub-divided into four sub-units denoted J_1 to J_4 from the base upwards (see Figure 4.5).

4.5 STRUCTURAL GEOLOGY

4.5.1 Overview of Structural Geology

The structural geology of the North Wall of Mt Whaleback can be broadly divided into three structurally (and stratigraphically) distinct areas:

- South of the Whaleback Fault Zone
- The Whaleback Fault Zone
- North of the Whaleback Fault Zone

As discussed previously the area south of the WFZ comprises stratigraphy from the Brockman Iron Formation, Mt McRae Shale and Mt Sylvia Formation. Macroscopically, this area is folded into two synclines separated by an anticline. Two flat-lying fault zones occur on the flat limb areas of the north limbs of the two synclines. These are termed the Central Fault and the East Footwall Fault (EFFZ), the latter forming the lower boundary to the orebody.

The northern boundary to these lithologies is the WFZ. A possible, early generation of folds has been identified in an area north of the mine. The effect of these early folds in the mine is uncertain, however they are thought to be responsible for local variation of small scale fold plunges. The Central Fault and EFFZ are thought to be related to each other (and to numerous smaller scale low-angle normal faults) yet pre-date the development of the WFZ.

The WFZ also forms the northern boundary to the ore bearing horizons and brings stratigraphy of the Brockman Iron Formation into contact with that of the Jeerinah Formation and has a minimum displacement of 600m. The fault zone dips to the south and is normal in sense of displacement, that is, the southern block is downthrown.

The material within the WFZ is highly deformed material of predominantly the Mt McRae Shale and Mt Sylvia Formation. This material is thought to be derived largely from the EFFZ. The WFZ and the EFFZ system together are thought not to form a listric fault system but rather comprise the intersection of two faults with the WFZ post-dating and thus offsetting the EFFZ.

North of the WFZ are dolerites and shales of the Jeerinah Formation. The layered sequence of dolerites and shales is asymmetrically folded on a large scale. Large folds which intersect the WFZ have been identified by drilling. In the area of investigation three dolerite units and three shale units have been identified due to new interpretations. Cross-hole seismic tomography has been successful in confirming both the large scale folding in the Jeerinah Formation as well as the presence of small parasitic folds on larger scale fold limbs and this technique is described in Chapter 5. The folds in this area are of the same generation as the main phase of folding exposed in the mine.

4.5.2 Structure of the Area South of the Whaleback Fault Zone

The area south of the WFZ comprises a folded and faulted sequence of BIF, chert and shales. These lithologies form the Joffre Member, Whaleback Shale Member and Dales Gorge Member of the ore bearing Brockman Iron Formation.

Rocks below the orebody consist of mainly faulted, sheared and folded Mt McRae Shale Formation, Mt Sylvia Formation and part of the Wittenoom Dolomite. The rocks of the Brockman Iron Formation are folded but contain small faults and shear zones. The rocks of the Mt McRae and Mt Sylvia Formations in the area of interest constitute a large shear zone termed the East Footwall Fault Zone (EFFZ) which forms the lower boundary to the orebody.

° Folding

The main fold system in this area is characterised by large westerly plunging asymmetric folds (refer to Figures 4.6 and 4.7).

The axial planes and rarely developed axial plane cleavage dips at 20° to 60° to the south-southwest (Figure 4.8). The sense of overturn on the folds is to the north, that is, the folds verge to the north. Consequently, southern limbs of anticlinal folds dip at 20° to 40° to the south while the northern limbs dip steeply, becoming overturned locally. The southern limbs tend not to develop small scale parasitic folds while the steep to overturned limbs are intensely folded at this scale. Macroscopic folds tend to develop a divergent cleavage fan with small scale folds on steep limbs having shallowly dipping axial planes while those on shallow limbs of folds generally have more steeply dipping axial planes.

The hinges of these folds are classified as close to open and rounded although there is considerable variability (Plate 4.1). Some folds in fact are quite tight with angular hinges. The difference in fold styles does not always appear to be lithology controlled. However, folds in predominantly shale units tend to be tighter with steep limbs becoming more overturned than folds in predominantly iron formation units (Plate 4.2).

Fold plunge can vary by as much as 20° in a small area although large scale regular variation has not been recognised in the area under investigation (Figure 4.9). However, a systematic plunge reversal occurs at the far western part of the mine. In this area folds plunging west change plunge to east over a zone where small scale fold plunges are quite variable. Small scale fold plunges in the eastern part of the pit generally range in azimuth from 250–270°.

An axial plane cleavage is developed occasionally in shale bands interbedded with banded iron formation. In predominantly shale units with lesser banded iron formation this cleavage is better developed and of more consistent orientation. The cleavage developed in shale bands within predominantly banded iron formation units has a more variable orientation due to the large competence contrast between shale and layered chert and hematite. In banded iron formation rocks, planes of dissolution (pressure-solution cleavage) are sometimes developed.

A lineation is pervasively developed which often appears to be an intersection of bedding and cleavage. This suggests the possibility that a solution cleavage may be reasonably widespread in predominantly banded iron formation lithologies. Alternatively, and more commonly, this lineation exists as microfolds of bedding laminations. This lineation is broadly parallel to the observed orientation of mesoscopic fold hinges. It may therefore be assumed to parallel the large scale fold hinges in any particular area.

In mapping the distribution and orientations of particular fabric elements, the large scale structure can be determined. The distribution of asymmetries of small scale folds and cleavage-bedding relationships indicates the location of the large scale structures.

In this area a large syncline (locally termed the East Syncline) has been mapped out with numerous smaller scale structures developed on its limbs. The axial trace of this fold trends approximately west and is located near the centre of the East Pit. The flat lying northern limb of this fold is truncated by the WFZ to the north.

A late phase of brittle deformation is developed in the mine. This consists of kink-type folds sometimes becoming chevron folds (Plate 4.3). These folds refold (locally) the main fold system and its axial plane cleavage thus post-dating it. Many folds and irregularities in the surface of the Footwall Fault may be attributed to the kink-type folding. The amplitude of these folds varies from a few centimetres to several metres with steeply dipping axial planes. The plunge is variable since these folds are obliquely superimposed on the main fold system.

Faults with small displacements are sometimes developed along the boundary of a kink band. Development of kink folding is widespread but is concentrated in a zone near the centre of the pit. This is important for slope stability considerations since the orientation and shape of the EFFZ may be expected to change in this area.

° **An Early Fold Phase**

Differences in style and orientation of folding are common. An area where these differences are pronounced was mapped at approximately 5900E (locally known as 'Death Valley' see Ref 4.3). A consistent south-westerly oriented structure is clearly evident which is oblique to the main fold system present in the mine (Plate 4.4). On close inspection it was revealed that a large proportion of these early 'folds' contained chert bodies (oblate in cross-section, see Figure 4.10) whose three-dimensional shape could not be ascertained.

The chert bodies appear to be remnants of partially dissolved chert layers within the iron formation. The possibility exists that the chert bodies are elongate and actually form the core of the structures which are oblique to the main generation of folds. These chert bodies occur in another area where they are better exposed. Most commonly their shape is that of an oblate spheroid however irregular forms showing incomplete dissolution occur (Figure 4.10).

They have not been observed to have the geometry of a prolate spheroid which would be required to form an apparent fold. Folds also occur which do not have a core composed of chert and it seems probable that those folds which do have chert cores actually nucleated on pre-existing chert bodies.

The geometry of refolding observed in 'Death Valley' indicates that the formation of the folds which are oblique to the main phase of folds pre-dated the formation of the main phase of folds (Figure 4.11). That is, where two anticlines meet a 'dome' is formed while the intersection of two synclines produces a 'basin' structure.

Given that the main fold system becomes strongly overturned while the fold system oblique to it appears more upright, then to produce a series of 'domes' and 'basins' the more upright oblique fold system must have pre-dated the main fold system observed in the mine (Plate 4.5).

Evidence for these early folds in the mine is rare although some good examples exist. A faint lineation related to these early folds is rarely developed. This lineation consists of microfolds of bedding where it can be observed in section. No other fabric development related to these folds has been observed so that microstructural evidence for the existence of these folds cannot be found. Similarly microstructural overprinting criteria (more reliable than re-fold produced geometries) cannot be established.

The presence of the early fold generation has two important implications for the structural geology (and slope stability):

- roughness at the scale of a bedding discontinuity.
- unusual orientations of bedding on a larger scale.

If a particular bedding surface has developed on it two generations of folds rather than just one the rock masses above and below this surface will be 'keyed in'. That is, if only one fold system existed on a particular surface, slip could be accommodated in the direction of plunge of that fold system. The presence of two fold systems intersecting obliquely greatly diminishes the possibility of slip on the bedding.

Bedding surfaces showing both generations of folding at the mesoscopic scale have only been encountered in the Joffre Member. This phenomenon could, conceivably be present where thinly laminated Banded Iron Formation occurs. In the area of investigation, areas of unusual orientations of bedding cannot be predicted since large scale early folds have not been found.

° Faulting

The Brockman Iron Formation in the area under investigation is bound below and to the north by faults. The fault below the ore-bearing units is a shear zone of uncertain thickness termed the East Footwall Fault Zone (EFFZ). The fault to the north also consists of a zone of sheared rocks whose thickness is variable but known in most places. This zone is here termed the Whaleback Fault Zone (WFZ) (see Section 4.3).

Small scale faults are relatively common in the area mapped although the sense of displacement is often difficult to determine due to lack of marker horizons within many of the units. Based on style of faulting in better exposed areas, it is thought that most faults exposed have a normal sense of movement. However, some are clearly reverse in sense (Plate 4.6).

Based mainly on style, faults in the area may be broadly divided into two generations. Steep normal faults dipping to the south (generally) appear to post-date the main phase of folding and thus are probably related to the Whaleback Fault. One exposure at the east end of the East Pit (Bench 16, Plate 4.7) shows a steeply dipping normal fault which displaces the East Footwall Fault. Clearly this fault, and probably most of the steeply dipping normal faults, post-dates the formation of the East Footwall Fault. Another exposure in the same area shows a shallowly north dipping reverse fault which displaces the East Footwall Fault. It appears then that the East Footwall Fault formed early in the faulting history of the area.

A well developed system of steep south dipping normal faults extends for a considerable distance along the north side of the orebody just to the south of the WFZ. These faults are well displayed on sections 8000E westwards to 7480E.

Some other good examples of shallowly dipping normal faults are exposed on the South Wall. They occur on the steep limb area of a large scale fold in rocks of the Mt Sylvia Formation. These faults generally dip to the south and appear more pronounced where they displace an intensely folded competent layer such as Banded Iron Formation.

It appears that at some stage during folding, volume problems were created which caused the formation of these antithetic low-angle normal faults. The style of folding observed does not allow for infinite 'stacking' of folds through the stratigraphy. That is, volume problems are created when the folds are not of a similar (class 2) style thus the fold 'dies out' along its axial zone and the strain is taken up elsewhere. Combine this with rocks of vastly differing ductile properties and extensional regimes will be formed to accommodate areas of volume deficiency. This process appears to be responsible for the formation of low angle, antithetic normal faults.

Although the Central Fault is not part of this investigation it is thought to be related to the low-angle, antithetic normal faults observed in other areas. That is, the zone directly below the Central Fault consists of predominantly extensional structures developed similarly to the previously discussed faults on the steeply dipping to overturned limb of the Central Anticline.

The East Footwall Fault appears to be a similar structure with, probably, similar timing to the Central Fault. Internal structure of the zones directly below the upper surface of both faults is very similar with folding and evidence of extensional tectonics (Plate 4.8). An important aspect in common to both of these structures is the direction of younging directly above and below the upper surface of each fault. The Central Fault has upwards younging Dales Gorge Member above.

This is clear since the stratigraphy in this area can be traced up into the Whaleback Shale and Joffre Members respectively. Directly under the fault, bedding dips shallowly to moderately to the south with the Mt Sylvia Formation overlying Mt McRae Shale, that is, the sequence is younging downwards. The fold vergence in this zone is upwards and to the south.

The East Footwall Fault also has upwards younging Dales Gorge Member directly above which can be seen to underlie the Whaleback Shale and Joffre Members respectively. Directly below the upper surface of the East Footwall Fault the position within the stratigraphy is not clear. However, it is almost certain that under the East Footwall Fault at the east end of the East Pit is Mt McRae Shale. Younging cannot be determined by direct stratigraphic relationships, however a section in this area clearly shows that fold vergence is upwards to the south (Plate 4.9). This implies that the stratigraphy is overturned directly below the East Footwall Fault.

Based on the above evidence relating to similarity of geometry, style, position of faults in the stratigraphy and relation to fold structure it is apparent that these faults are of similar timing in the deformation history of the area.

Gently south-dipping, normal faults become an important aspect of the structure of rocks south of the WFZ in the western part of the area (see Sections 7120E through to 6880E in Appendix A). This is particularly important because such structures will be an integral part of any proposed buttresses composed of Brockman Formation rocks (see Chapter 10).

° Summary of Structure South of the Whaleback Fault Zone (WFZ)

South of the Whaleback Fault Zone is a complexly folded and faulted stratigraphy consisting of predominantly Brockman Iron Formation rocks. Large scale folds in the mine, from north to south, include the East Syncline, the Central Anticline and the South Syncline. Only the East Syncline and part of the Central Anticline are within the area of investigation.

These folds are typical of the main phase of folding observed in the mine. An early phase of folding appears to have a limited effect on the rocks although this is uncertain. The early structures would be oriented approximately southwest–northeast on flat–lying limbs of the main folds.

The area is affected by two generations of faulting one of which appears to be broadly coeval with the folding. The faulting which appears coeval with the folding includes small scale antithetic, low–angle, normal faults as well as the East Footwall and Central Faults probably being large scale examples. The other generation of faulting is probably related to the WFZ which clearly post–dates folding. These faults are generally steeply south dipping with a normal sense of movement.

4.5.3 The Structure of the Whaleback Fault Zone (WFZ)

The ore bearing horizons of the Brockman Iron Formation are fault–bounded to the north at Mt. Whaleback. This fault is termed the Whaleback Fault, however it consists of a zone of faulted material which is here termed the WFZ. The fault zone brings the Brockman Iron Formation into tectonic contact with the Jeerinah Formation.

The WFZ consists of two south dipping, east–west striking (generally) boundary faults, designated the North and South Whaleback Faults, which enclose a zone of sheared and faulted material. The boundary faults display a marked degree of curvature (north–south, steeply pitching). The fault zone is of variable thickness and, to the west, becomes a single fault surface. From drilling information it is apparent that where two boundary faults are present, these faults generally diverge with depth. It then seems likely that to the west there would be a fault zone at depth, with the intersection of the two faults plunging to the west moderately steeply.

The WFZ constitutes a normal fault with a minimum movement of 600m across the zone. The material within the fault zone is derived from below the orebody. This has been established by the recognition of marker units within the fault zone. In one area (Bench 17, East Pit) part of the nodule zone and triplets (chert) of the Mt McRae Shale Formation are recognisable within the fault zone. Silty material within the fault zone is highly oxidised but still recognisable as part of the Mt Sylvia Formation siltstone unit.

In most areas where the WFZ can be observed, the enclosed material is highly sheared with relic, small scale fold hinges preserved. The shear surfaces and axial surfaces of infrequently preserved folds (now parallel) dip less steeply than the boundary faults. In other areas numerous steeply dipping faults which broadly parallel the boundary faults are recognisable.

It is thought that most of the material within the WFZ is highly deformed material derived from the EFFZ which has subsequently been cut by fault planes related to the WFZ. Much of the movement across the WFZ would have been accommodated by these internal fault planes as well as by significant movement on the boundary faults themselves. It is not clear whether or not the two boundary faults were formed simultaneously although this seems likely.

Structure within the WFZ appears largely discontinuous although where sufficient drilling has been carried out certain marker horizons can be traced out (for example elements of Mt McRae Shale, Mt Sylvia Formation and Wittenoom Dolomite in sections 7880E, 7800E, 7720E and 7640E). The interpretation of structure in this area is very tentative due to the intensity of deformation of EFFZ material and largely unknown amounts of deformation and rotation of material within the WFZ.

Another problem is that, at depth, the position of the south boundary fault (South Whaleback Fault) of the WFZ is not clear below its intersection with the EFFZ. This problem arises because directly below this intersection rocks of similar lithology (Mt McRae Shale) are in tectonic contact. Extensive drilling would be required to intersect recognisable marker units and thus accurately determine the lower extent of the South Whaleback Fault.

A breccia zone composed of fragments of chert, shale and Banded Iron Formation within a (mainly) dolomitic matrix is present at depth in the northern part of the WFZ. This breccia zone is laterally quite extensive although not necessarily continuous. It has been intersected in drill holes on Sections 8360E, 7960E and 7920E. The breccia is thought to be developed within the Wittenoom Dolomite with fragments of Banded Iron Formation possibly derived from the underlying Marra Mamba Formation.

In many areas rocks (both shales and dolerites) of the Jeerinah Formation are included within the WFZ and these may occur either on the south side of the WFZ with stratigraphically higher rocks between faulted Jeerinah and Jeerinah Formation proper, or they may occur on the north side of the WFZ in close proximity to unfaulted Jeerinah. This latter situation is especially common in the east end of the area making the north side of the WFZ difficult to define.

° The South Whaleback Fault and the East Footwall Fault

The relationship of the WFZ to the EFFZ is a matter of considerable discussion. An exposure on Bench 19 (Plate 4.10) shows the intersection of the southern boundary fault of the WFZ and the East Footwall Fault (see also Figure 4.12). Despite a great deal of faulting and shearing, it is apparent that the South Whaleback Fault truncates the East Footwall Fault, thus demonstrating that the East Footwall Fault is an earlier structure than the WFZ.

This exposure also demonstrates the anastomosing nature of the South Whaleback Fault. In this area the Footwall Fault dips moderately steeply to the southwest but becomes flatter towards the intersection with the steeply dipping South Whaleback Fault. Where exposed, the large undulations and occasionally steep dip seen on the East Footwall Fault can be attributed to the effect of large kink type folds (Plate 4.11) which post-date the main phase of folding. This causes the intersection of the two faults to range from being at a high angle to being sub-parallel.

As discussed in Section 5.2, the EFFZ is thought to be related to the main phase of folding. The WFZ however, clearly post-dates the main phase of folding. The latter relationship is exposed on Bench 18 where a fold observed in oblique section has both limbs truncated by the Whaleback Fault Zone (Plate 4.12).

It has been postulated that the South Whaleback Fault and the East Footwall Fault are the same fault constituting a listric normal fault. Listric normal faulting is characterised by rotation of markers adjacent to the fault (Figure 4.13). The geometric constraint of rotation of markers adjacent to a curved fault surface is not observed in the area of investigation. Fold axial surfaces near the fault do not show any dextral rotation looking west nor does flat lying strata in this area. In some cases a sinistral rotation is observed where these markers are actually 'dragged' up against the fault.

The South Whaleback Fault and the East Footwall Fault are thought not to constitute a listric normal fault for the following reasons:

- the Whaleback Fault appears to truncate and displace the East Footwall Fault thus post-dating it.
- the East Footwall Fault is thought to be related to and coeval with the main phase of folding while the Whaleback Fault clearly post-dates this.
- expected rotation of strain markers adjacent to the fault is not observed.

Summary of the Whaleback Fault Zone Structure

The WFZ constitutes a normal fault bringing into tectonic contact the Brockman Iron Formation with the Jeerinah Formation. The fault zone itself consists of sheared rocks derived from the East Footwall Fault containing material from the Mt McRae Shale and the Mt Sylvia Formation. In many instances rocks of the Jeerinah Formation are also present within the WFZ.

4.5.4 Structure of the Jeerinah Formation

As a result of the work undertaken as part of these investigations, a new interpretation has been made for the Jeerinah Formation which is to the north of the WFZ. The Jeerinah Formation comprises deformed dolerites and shales and three dolerite units and three shale units have been recognised due to the new interpretation of the structure in this area. Surface mapping of exposures north of the waste dumps has revealed new aspects of the style and scale of deformation in the Jeerinah Formation. Results of this mapping, previous mapping, previous drilling and drilling from the current program have been incorporated to form these new structural interpretations.

These interpretations indicate that the folds plunge generally to the west with shallowly to moderately, south to south-west dipping axial planes. A penetrative axial plane slaty cleavage developed in the Jeerinah Shale units is defined by the parallel alignment of white mica (Plate 4.13). This cleavage is the predominant foliation in the shale units and bedding is often difficult to distinguish.

The dolerite units show little internal fabric except when they occur in sheared areas close to the WFZ. Dolerite A shows anisotropy when weathered, suggesting the presence of limited fabric development. Dolerite B appears coarser grained and more massive (from field observations) and no internal fabric caused by deformation has been observed.

A crenulation cleavage is somewhat sporadically developed in the Jeerinah Shale. The crenulation cleavage dips steeply and is most prevalent in the western part of the area mapped (ie. west of approximately 7400E). The crenulation cleavage is formed by deformation of the earlier formed slaty cleavage. It ranges from zonal to discrete and, in one sample, a second crenulation cleavage was observed.

The second crenulation cleavage clearly overprints the first crenulation cleavage and so the two sets observed are not conjugate. In all, four folding deformations occur in the Jeerinah rocks (including later-stage kink bands), however only the first deformation appears to have controlled the geometry of the folded surface.

A generation of late stage kink folds is also well developed north of the WFZ. These are generally smaller and have better defined kink bands than those south of the WFZ. These folds do not appear to affect the gross geometry of the area and may be related to the kink-type folding observed south of the fault.

The geometry of the folded surfaces has been determined using surface outcrop mapping and drilling information. Surface outcrop mapping has not only provided constraints on the position of shale/dolerite contacts at the surface but has delineated the presence of large scale folds.

By mapping cleavage/bedding relationships (Plate 4.14) and the orientations of other fabric elements, a large scale synclinal axial trace was found. This fold has a minimum wavelength of several hundred metres.

Using previous surface outcrop mapping (undertaken by Mr S Kale in the late 1970s (Ref 4.6)), of an area now concealed under waste dumps, and using drill hole information, a structural model based on the style of deformation in currently exposed areas has been developed. The previous outcrop mapping and rock type at the collars of shallow drill holes in this area has been used to construct a new outcrop map (see Ref 4.3). Both past and present deep drilling have provided constraints on the shale dolerite contacts at depth and have enabled a coherent structural model to be produced.

° **Folding**

The folds north of the WFZ are nearly identical in style and orientation to those mapped south of the WFZ. The difference between folds from the two areas is in amplitude and wavelength.

The folds to the north of the fault zone are larger in both amplitude and wavelength than those to the south, this being a function of the greater layer thickness in the Jeerinah shales/dolerites than that present in the more thinly layered banded iron formations and shales. This effect is probably also a function of homogeneity within the layers. That is, the shales and more so the dolerites of the Jeerinah Formation are more homogeneous than the finely layered banded iron formations and regularly interbedded chert and shale units.

Locally irregular fold plunges are observed in this area as they are to the south (Plate 4.15). These fold plunges are not always attributable to the deformation/folding which produced the crenulation cleavages. This phenomenon may be caused by the presence of an early, weaker deformation, as is thought to be the case regarding plunge variations in folds to the south of the fault.

The cross-sections, mapping and modelling broadly show two large scale folds which trend north-westwards and away from the WFZ in going from east to west and these are shown in detail in Appendix A. The eastern-most section (7680E) shows a small syncline containing Shale A. Progressing through the sections westwards, the Shale A within this syncline becomes larger in profile as the hinge of the fold deepens and moves to the north.

As a consequence of the angular relationship between the orientation of the WFZ and the plunge of the folds, an anticline appears containing Dolerite A in its core around 7560E and is well established by 7480E. The hinge of this anticline moves northwards on the sections for a westward progression of sections. The western part of the area lies on the upper flat limb of this anticline. The expected next syncline to the south has, at this stage, not been intersected.

Correlation of drilling constrained contacts with the shape and positions of those outlined by the cross-hole seismic tomography program is good (see for instance Section 6880E between drill holes G1463 and G1481 in Chapter 5). This information has been incorporated into the three-dimensional structural model.

◦ Summary of the Structure North of the Whaleback Fault Zone

The dolerites and shales north of the WFZ are interlayered and folded on a large scale. The axial planes of these folds dip to the south-southeast with plunges to the west. Detailed maps have been constructed from previous surface outcrop mapping and drilling and these maps are available elsewhere (Ref 4.3). Drilling and surface outcrop mapping have enabled the assessment of the style and scale of deformation in the Jeerinah Formation and been further confirmed by cross-hole seismic tomography in areas where this technique was applied. To the best knowledge of the writer it is the first time that such a technique has been used to reliably define geological structure.

4.5.5 Timing Relationships of Geological Events

A number of geological events post–dating deposition of sediments and emplacement of doleritic intrusions have contributed to the evolution of the area under investigation. These include deformation and fluid related processes such as mineralisation and dissolution of some rock layers. Lack of fabric development and hence lack of conclusive overprinting criteria has made the task of timing various events difficult. Rare cross–cutting relationships and folded surface geometries have been used to outline a geo–chronological framework for the area.

The interpretation of post–depositional geological events is as follows:

(i) the dissolution of some chert layers within banded iron formations. It is not clear whether this process is a product of tectonism or of earlier burial/diagenesis. The interpretation that dissolution of chert layers occurred early in the history of the area is dependent on the relative timing of the two fold phases. Since the chert bodies often occupy the core of the suspected early folds it is thought that these folds may have nucleated on them.

(ii) Folding in the area of investigation is the dominant post–depositional feature. Folding post–dates the formation of the chert bodies. The main phase of folding observed post–dates the occasionally developed oblique group of folds.

(iii) The second group of folds, that is the main phase of folding observed in the mine is associated with antithetic, low–angle, normal faulting. It is thought that these faults developed in response to volume problems associated with the large scale folding of competent lithologies.

The two prominent examples of this type of fault are the Central Fault and the East Footwall Fault. An axial plane cleavage of variable intensity was developed parallel to the main fold phase. The main phase of folding in the mine is also well developed in the Jeerinah Formation to the north of the Whaleback Fault.

(iv) Post-dating this main folding event in the Jeerinah Formation are two sporadically developed crenulation cleavages. No large folds associated with these crenulation cleavages were found.

(v) A late stage brittle deformation caused kink folding which locally refolds the main phase of folding and its associated faulting. The kink folds also appear to post-date the crenulation cleavages developed in the Jeerinah Formation although no direct overprinting relationships could be found.

(vi) Timing of mineralisation is uncertain with respect to the kink folding. Mineralisation does however post-date the main phase of folding and obscures fabric in banded iron formation lithologies. Zones of mineralisation also cross-cut large scale folds. Mineralisation probably pre-dates the development of the WFZ. Mineralised 'slices' of iron formation occur in the WFZ suggesting mineralisation occurred before faulting.

This, however could be lithology controlled process whereby a 'slice' of Banded Iron Formation becomes isolated within shales of the fault zone and becomes mineralised later by fluids selectively replacing components of only Banded Iron Formation.

(vii) Rarely developed, north-south, steeply pitching, open kink folds deform small faults and shear surfaces within the WFZ. This is the latest phase of deformation to affect the area.

4.6 COMPUTER GRAPHICAL MODELLING

The use of three dimensional computer graphics has been extensively used to aid with the interpretation of geological structure of the North Wall of Mt Whaleback. An Intergraph computer system was employed to take two–dimensional geological interpretations made by the study team and link them together to form a three–dimensional surface of each of the geological contacts. Where surface geological plans were also available, these were also used as input into the three–dimensional model.

It became apparent that the only way to fit the known data together in a logical way was to incorporate large scale folds in the Jeerinah Formation and this is a major finding of this research project. These folds are now consistent with surface outcrops and cross–sectional information. Considerable effort was made to ensure the fold styles postulated fitted the known data and this is sensitive to the order of spline fit chosen. Nevertheless, the three–dimensional surfaces produced enabled cross–sections to be taken at any orientation, a feature which was subsequently used in the stability assessment.

The software used in this work was not a geological modelling package, but a mechanical engineering design package. However, it was sufficiently powerful and flexible that the skilled operators employed on this work were able to produce the results required. All the detailed plans and sections produced for the detailed mining report have also been made using an Intergraph CAD system.

4.6.1 Modelling of the Jeerinah Formation

The detailed structure of the Jeerinah Formation was one of the major unknown factors prior to this geotechnical research. The reasons for this are that the Jeerinah Formation has relatively few exposures in the present pit and that the natural ground surface north of the WFZ is covered in waste dumps. Therefore detailed face mapping or drilling investigations are difficult to undertake.

Part of the solution to this problem was to use cross hole seismic tomography between boreholes and also to use sophisticated computer graphics as described above. The steps involved in this computer graphics modelling have been described in Ref 4.8 and were as follows:

1 Obtain all the previous borehole information including stratigraphic contacts and assay data and put this onto the CAD system. This involved a total of about 2500 boreholes for the North Wall.

2 Put all the known stratigraphic contacts available from bench plans onto the three dimensional model.

3 Digitise existing cross-sectional structural interpretations and keep in a separate file.

4 Fit data from Steps 1 and 2 into three dimensional surfaces using MEDS modelling software. This step involved complex surfaces being created in order to fit known data points from 1 and 2. It also became apparent that there were many ways to fit a three dimensional surface to known data points depending on the splice fit chosen (ie. the degree of curvature).

5 The three dimensional surfaces from Step 4 were then sectioned and compared to the original cross sections obtained from Step 3.

6 Data from Steps 3, 4 and 5 were then reviewed by a structural geologist in order to ensure that fold styles were admissable. A new three dimensional model was then created based on all previously known information. The purpose of this model was to identify areas which were lacking in information and which could therefore be used to optimise drill hole locations and face mapping programs.

7 As additional drill hole and mapping information became available it was input into the computer model and an updated model produced. This was continually reviewed by a structural geologist to ensure that the model was not 'computer driven'. This process was repeated until all of the information was incorporated into the computer model and a final interpretation was made.

8 The three dimensional surfaces were then sectioned in the conventional north-south direction, in a direction normal to the pit wall and in the maximum down-plunge direction. These three cross sections were then used in the stability analyses in order to optimise pit wall design and to ensure that the actual minimum factors of safety were obtained. These aspects are discussed further in Chapter 10.

4.6.2 Three Dimensional Surface Modelling

Some examples of the three dimensional surfaces produced by the method described above are shown in Plates 4.16 to 4.21 and the detailed north-south cross sections also showing these surfaces are given in Appendix A. The surfaces modelled include all of the Jeerinah dolerite/shale contacts, the southern and the northern extent of the WFZ as well as the original topographic surface and the pit wall designs. It should be pointed out that the accuracy of all the three dimensional surfaces is dependent upon the amount of information available and in particular the WFZ at depth and the Jeerinah contacts in the far western area, are based on limited information. Also the shapes of the southern edges of the Jeerinah contacts are unusual, since these folded surfaces are truncated by the WFZ which is itself an irregular surface.

The surfaces produced from the computer modelling confirmed much of the previously suspected structural geology, ie. folding on a large scale in the Jeerinah Formation and drag folding close to the WFZ.

The steps to achieve the three dimensional surfaces have been outlined in the previous section. Previous work indicated that there was a general plunge in the stratigraphy to the west, but three dimensional modelling revealed that this plunge was also variable.

Plate 4.16 shows a cross section through the North Wall from the west (left side) to the east (right side) with all of the Jeerinah Formation units in their correct spatial position. The colour coding of these surfaces represents the following horizons:

Colour of Surface	Strat Horizon Beneath
Blue	Dolerite A
Green	Shale A
Orange	Dolerite B
Red	Shale B
Above Red	Dolerite C

It can be clearly seen that the plunge decreases towards the west. The blue and green three dimensional surfaces are truncated at depth because they are below the final proposed pit floor and there is no information at this depth. It should also be noted that the scale of Plate 4.16 is over 3km long and 300m high. On a closer inspection, these three dimensional surfaces are revealed as quite complex.

Plate 4.17 shows four different views of the Shale A/Dolerite A contact and it is quite clear that there is folding in both an east–west direction and in a north–south direction and that there are parasitic folds on large scale fold structures.

Plate 4.18 shows four different views of the Dolerite B/Shale A contact and Plate 4.19 shows the same contact after some additional borehole information had become available. Plates 4.18 and 4.19 show that the fold styles are dependent upon the amount of information available and therefore it is essential that the fold styles produced by computer modelling are verified by a structural geologist.

Plate 4.20 also shows how apparent steep folds can be caused by truncation, in this case by the WFZ. The straight red lines shown on Plate 4.20 represent the north–south section lines from which the three dimensional surface was generated.

In the stability analyses these three dimensional surfaces were sectioned in the maximum dip direction and normal to the proposed pit wall in order to check rock slope stability. The conventional north–south cross sections were also used in the stability analyses.

Consideration was also given to three dimensional slope stability, and there is obviously considerable potential for 3–D interlocking of large blocks on these surfaces. However, there is presently no method available to realistically model the slope stability considering such complex 3–D surfaces as potential slip surfaces.

Finally Plate 4.21 shows all of the 2–D structural, geological and geophysical data that were used to compile the 3–D surfaces. This information was then linked section by section in order to create the 3–D model. It is interesting to note that the tomograms themselves do show some anomalies and need to be considered with this other information in order to produce a realistic interpretation of the structure of the North Wall. This structural modelling proved to be a crucial step which had to be taken before a reliable wall design could be developed.

4.7 GEOCHEMISTRY OF THE JEERINAH FORMATION

Core specimens were taken from diamond holes in order to determine the petrography and geochemistry of rocks in the Jeerinah Formation. These rock samples were forwarded to AMDEL Laboratories and the results are presented in their Report (Ref 4.5). Details of the location and nature of each sample are given in Figures 4.14 to 4.16.

The main purpose of the geomechanical work was to determine if recognisable petrographic and/or geochemical differences exist between the various shale units and between the dolerite units. These differences if present could be used for correlation and hence better structural interpretation from borehole intersections. Another objective was to determine if distinctive marker horizons are present within the shale or dolerite horizons themselves.

4.7.1 Petrography of Jeerinah Shale Units

Both Shale A and Shale B units were sampled with fourteen (14) Shale A specimens and seven (7) Shale B specimens being analysed. A review of the petrographical results indicates the following distinguishing features:

SHALE A

- generally distinctly 'chloritic', although 'sericitic' in places
- occasionally 'dolomitic' (two examples)
- occurrence of chloritic siltstones (three examples)

SHALE B

- generally distinctly 'sericitic'

Figure 4.17 is a triangular plot of the shales using sericite – chlorite – quartz as the end members. The above brief comments are clearly illustrated by the constituent mineralogy plot.

4.7.2 Geochemistry of Jeerinah Shale Units

Table 4.1 gives the averages and standard deviations of the major 'oxide' constituents determined by AMDEL (Ref 4.5). Relatively speaking, Shale B has a higher Al_2O_3 and K_2O content and a lower Fe_2O_3 content. Shale A shows a wider scatter of values of these particular constituents as indicated by the standard deviations.

The higher Fe_2O_3 in Shale A reflects its chloritic nature. The higher K_2O and Al_2O_3 in Shale B reflect its sericitic nature. Shale A appears to have a distinct 'dolomitic' component. This is indicated by higher MgO and CaO percentages and a higher Loss on Ignition (LOI).

Plate 4.21 shows the gamma logs, plotted along the borehole traces. Chemical analyses are also shown by coloured histograms. In such plots, potassium peaks correspond to sericitic shales and are accompanied by correspondingly high gamma intensities; magnesium peaks correspond to chloritic shales. The plots indicate that the results shown in Figure 4.17 are perhaps an over-generalisation and that chlorite rich areas or sericite rich areas are characteristic of both Shales A and B. More detailed work would be necessary to bring out useful correlations although some sections show broad correlation in mineralogy of the shales from one drill hole to the next.

The significance of the tomograms to the Structural Geology is discussed further in Chapter 5.

4.7.3 Petrography of Jeerinah Dolerite Units

Table 4.2 compares the proportions of the major mineral constituents of the two dolerite units sampled; Dolerite A (9 specimens) and Dolerite B (13 specimens). The following distinctive features are recognised:

DOLERITE A

- Contains a noticeably higher percentage of chlorite compared with Dolerite B. This is probably due to the assimilation of the presence of Shale A xenoliths.
- Calcite percentage is also higher. As observed in the core, 'vugs' are usually filled with carbonate. This is a distinctive Dolerite A feature.

It should be noted that Dolerite A was drilled and hence sampled mainly in the top 50m, ie. in the xenolithic and vuggy top. Hence the characteristics observed may not be completely representative of the dolerite unit as a whole.

DOLERITE B

- Generally low chlorite content.
- Increase in amphibole and epidote content with depth, ie. towards contact with Shale A. There is corresponding decrease in plagioclase and pyroxene proportions.

4.7.4 Geochemistry of Jeerinah Dolerite Units

Recognisable geochemical differences between Dolerite A and Dolerite B reflect the mineralogical differences discussed previously. The higher chlorite content in Dolerite A is reflected in a generally higher Fe_2O_3 percentage and lower Al_2O_3 percentage. The higher Loss on Ignition (LOI) in Dolerite A is thought to be due to a higher carbonate content. Other significant but less explicable differences are higher values of TiO_2, K_2O and P_2O_5 in Dolerite A as compared with B (see Table 4.3). The gamma log traces and assay histograms shown in Plates 5.1 to 5.7 in Chapter 5 indicate a sharp change in value between shale and dolerite.

4.7.5 Summary of Geochemical Data

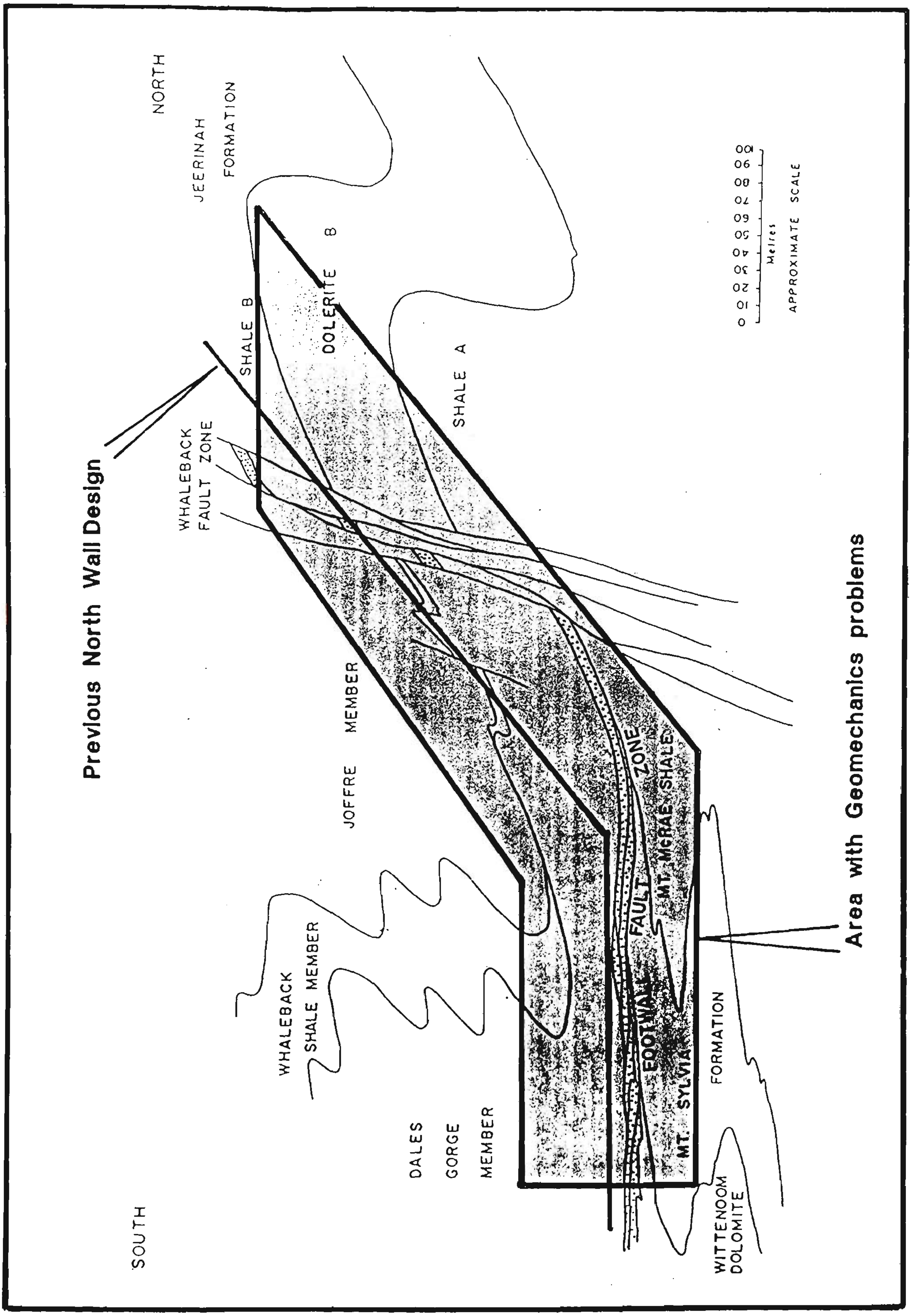
From the limited sampling and analysis carried out as part of this investigation it would appear that there are some real, albeit general, petrographical and geochemical differences between the major shale and dolerite units within the Jeerinah Formation. However distinctive petrographical and/or geochemical 'marker horizons' <u>within</u> individual shale or dolerite units have been observed, although Shale A does contain 'dolomitic' and 'siltstone' components. Continuity of these is unknown at this stage.

STRATIGRAPHY

PERIOD	GROUP	CODE	FORMATION	MEMBER	THICKNESS (m)	DOMINANT LITHOLOGIES
RECENT			VALLEY FILL			
RECENT		Cal	CALCRETE			
TERTIARY		Ts	TERTIARY SEDIMENTS			
LOWER PROTEROZOIC	HAMERSLEY GROUP	Pho	BOOLGEEDA IRON FORMATION		220	BIF and shale
		Phw	WOONGARRA VOLCANICS		120-600	acid lavas, tuffs
		Phj	WEELI WOLLI FORMATION		150-500	BIF, dolerite, shale
		Phbj	BROCKMAN IRON FORMATION	– Joffre Member	370	BIF, minor shale
		Phbw		– Whaleback Shale Member	30	shale, chert
		Phbd		– Dales Gorge Member	120	BIF, minor shale
		Phr	MT. McRAE SHALE		30	shale, chert
		Pha	MT. SYLVIA FORMATION		45	shale, chert, dolomite, BIF
		Phd	WITTENOOM DOLOMITE		150	dolomite, chert, shale
		Phda		– West Angelas Member	35	shale
		Phmn	MARRA MAMBA IRON FORMATION	– Mount Newman Member	45	BIF, minor shale
		Phmm		– McLeod Member	30	BIF, shale
		Phmu		– Nammuldi Member	70	BIF, chert, shale
	FORTESCUE GROUP	Pfj	JEERINAH FORMATION		750	shale, dolerite, basalt, chert
		Pfb	MT. JOPE VOLCANICS		120	vesicular basalt, pyroclastics

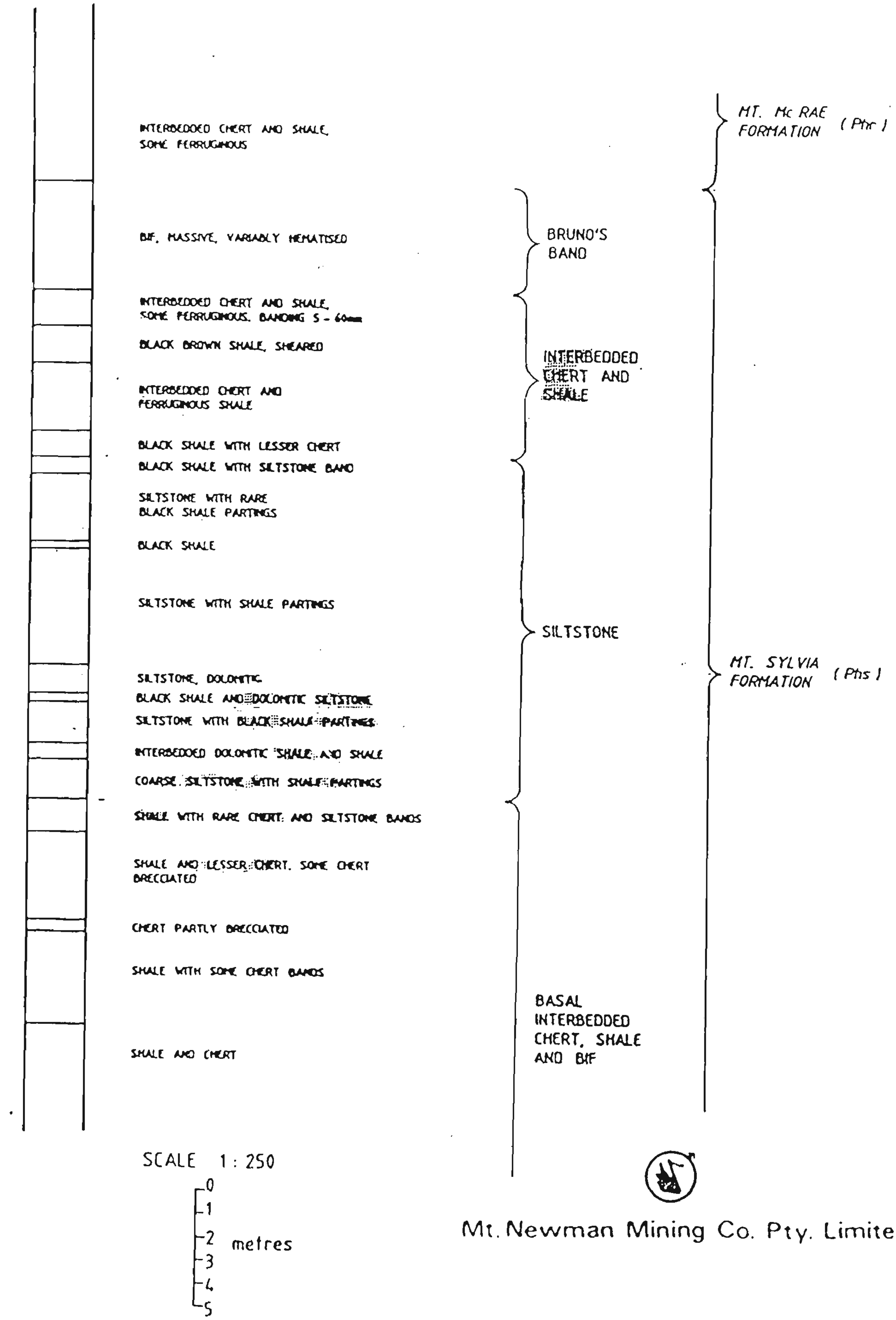
BIF Banded Iron-Formation

Scale	REGIONAL STRATIGRAPHY	FIGURE 4.1
Drn		
Dwg No.		



Scale	CARTOON SHOWING STRUCTURAL ELEMENTS AT MT. WHALEBACK EAST PIT	FIGURE 4.2
Drn		
Dwg No.		

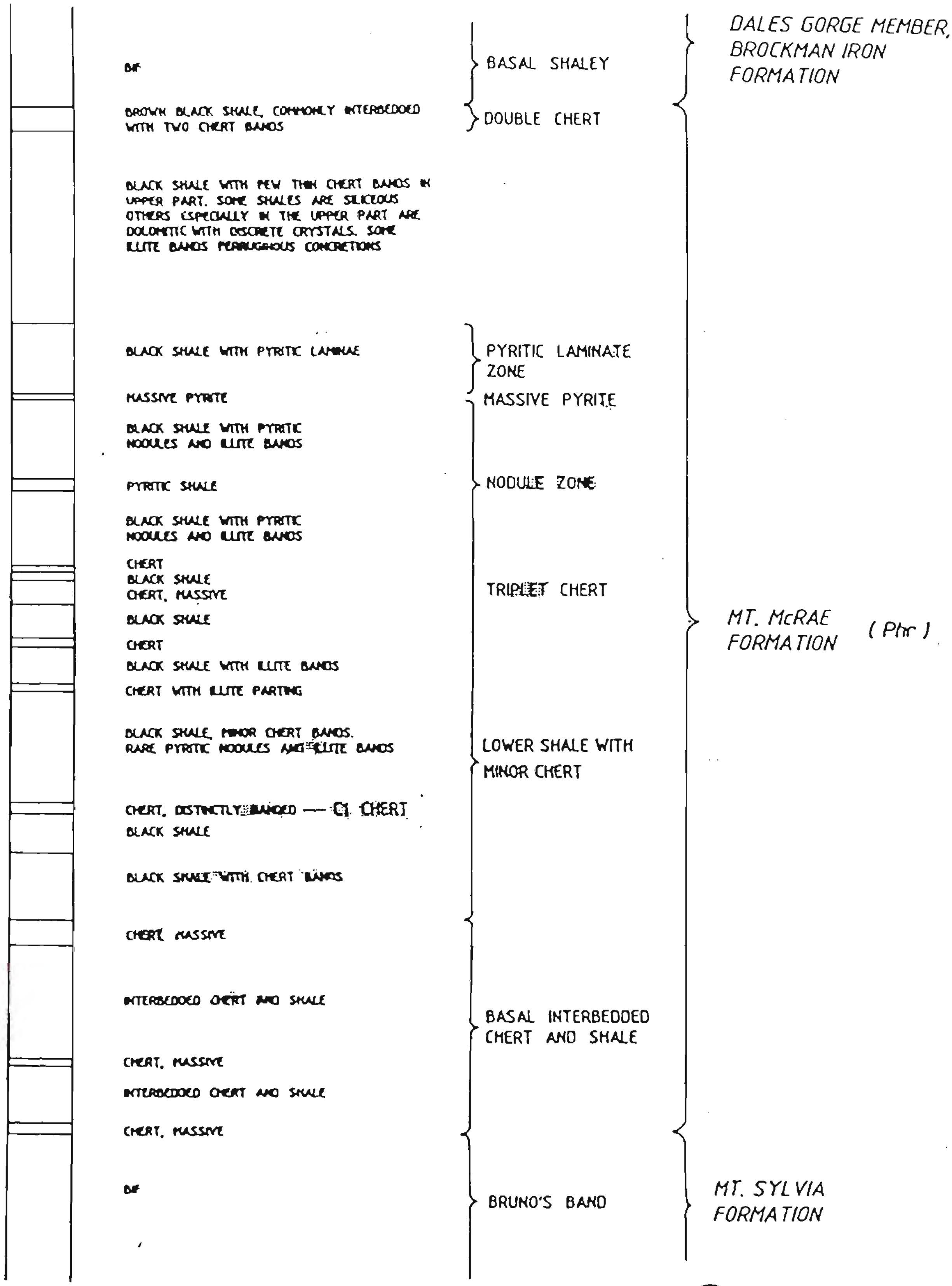
Typical sequence in the upper part of the formation
at Mt. Whaleback (from drillhole D238)



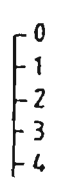
Mt. Newman Mining Co. Pty. Limited

Scale	MT SYLVIA FORMATION: TYPICAL SEQUENCE	FIGURE 4.3
Drn		
Dwg. No.		

Typical sequence at Mt. Whaleback
(compiled from drillhole D227 and mapping information)

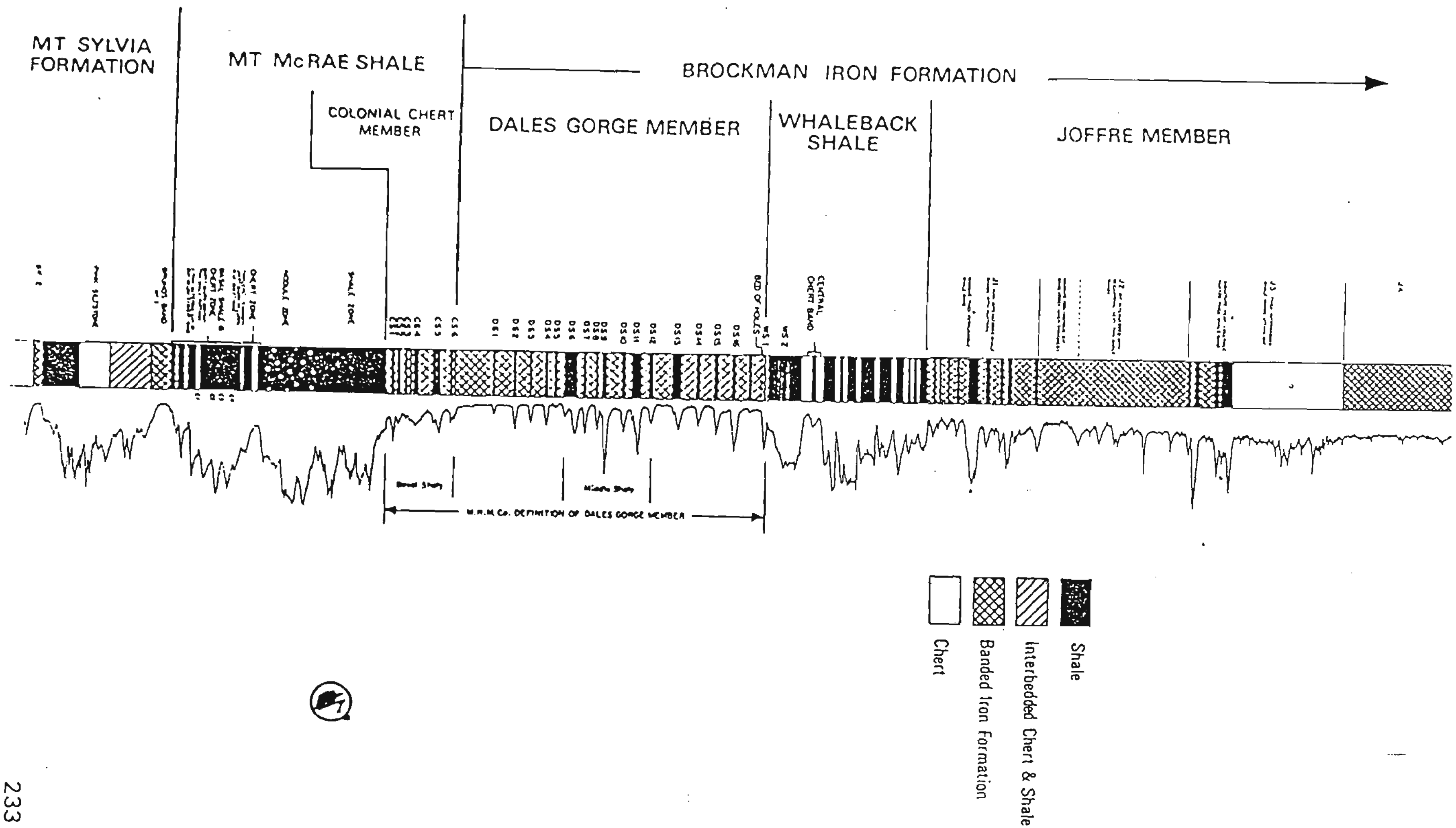


SCALE 1 : 250



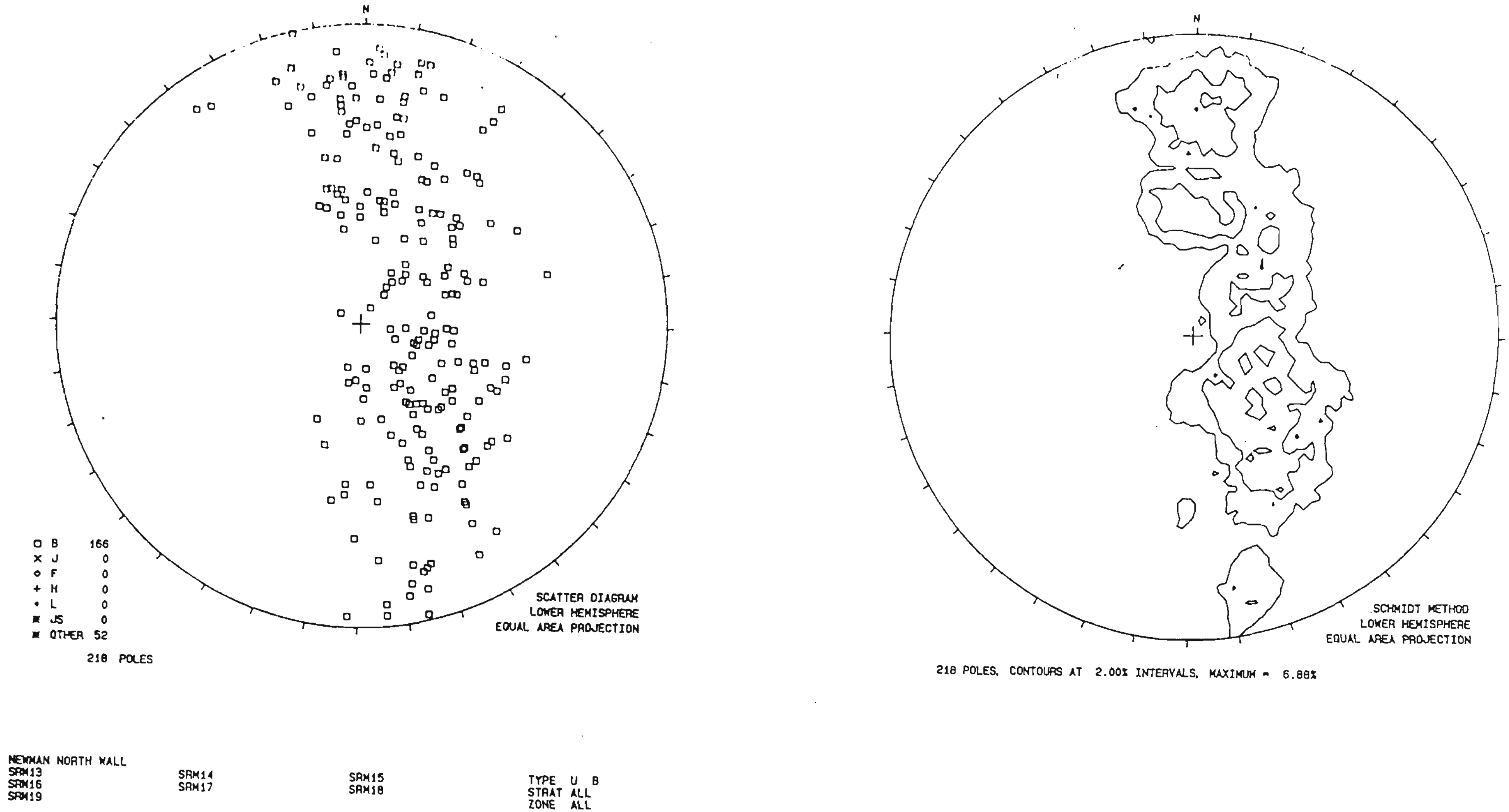
Mt. Newman Mining Co. Pty. Limited

	Scale	MT MCRAE FORMATION: TYPICAL SEQUENCE	FIGURE 4.4
	Drn		
	Dwg. No.		



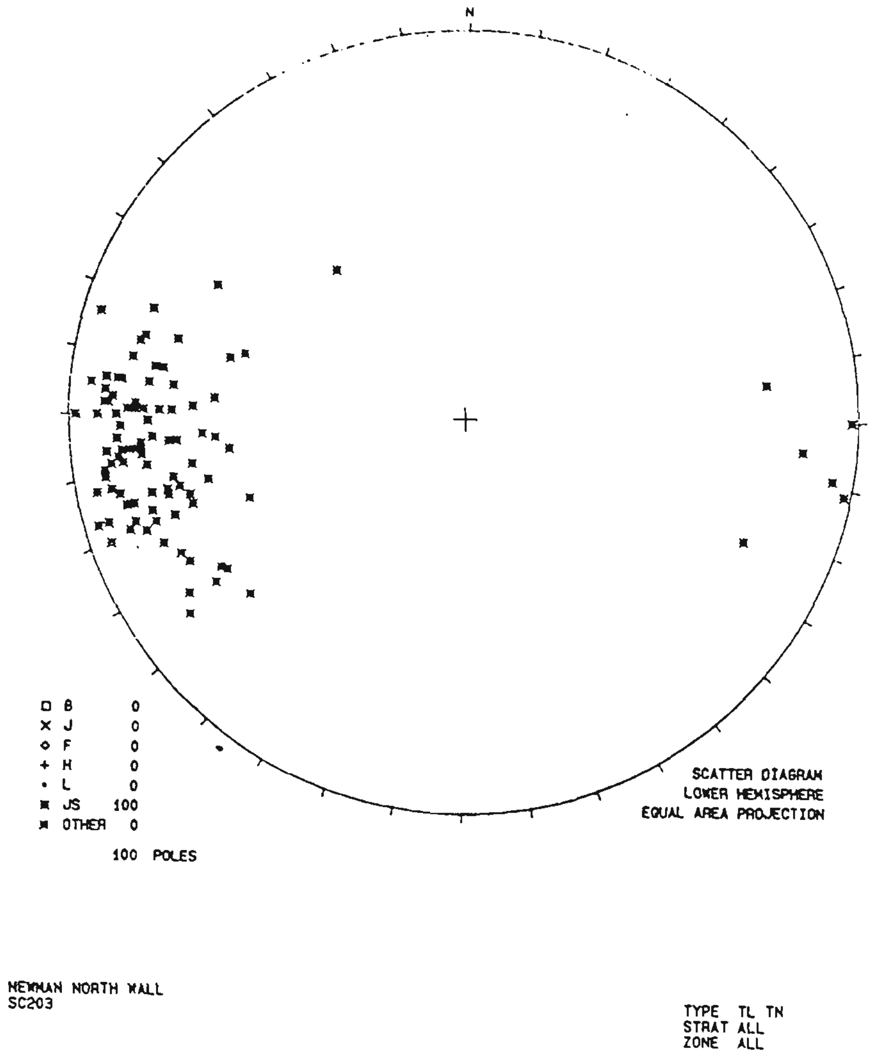
233 E

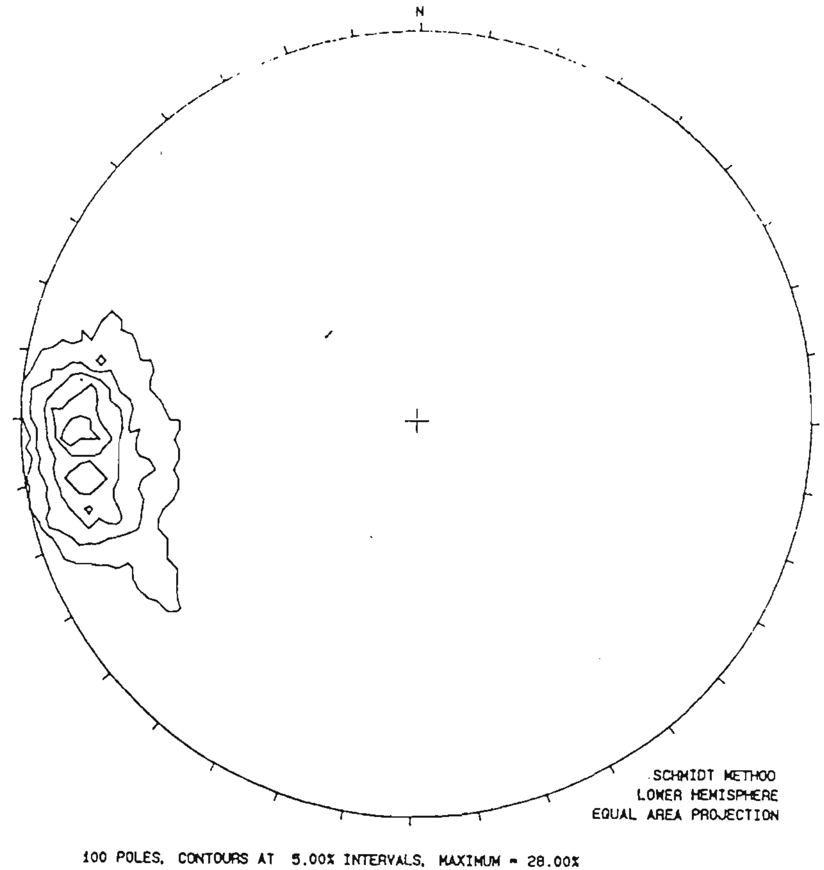
Scale	COMPOSITE GAMMA LOG REFERENCE SECTION FOR MT. WHALEBACK.	FIGURE 4.5
Drn		
Dwg No.		



Well defined great circle of redistributed poles to bedding caused by main phase of folding about an approximately E-W axis.

Scale	STEREOGRAPHIC PROJECTION OF POLES TO BEDDING SOUTH OF THE WHALEBACK FAULT ZONE	FIGURE 4.6
Drn		
Dwg No.		

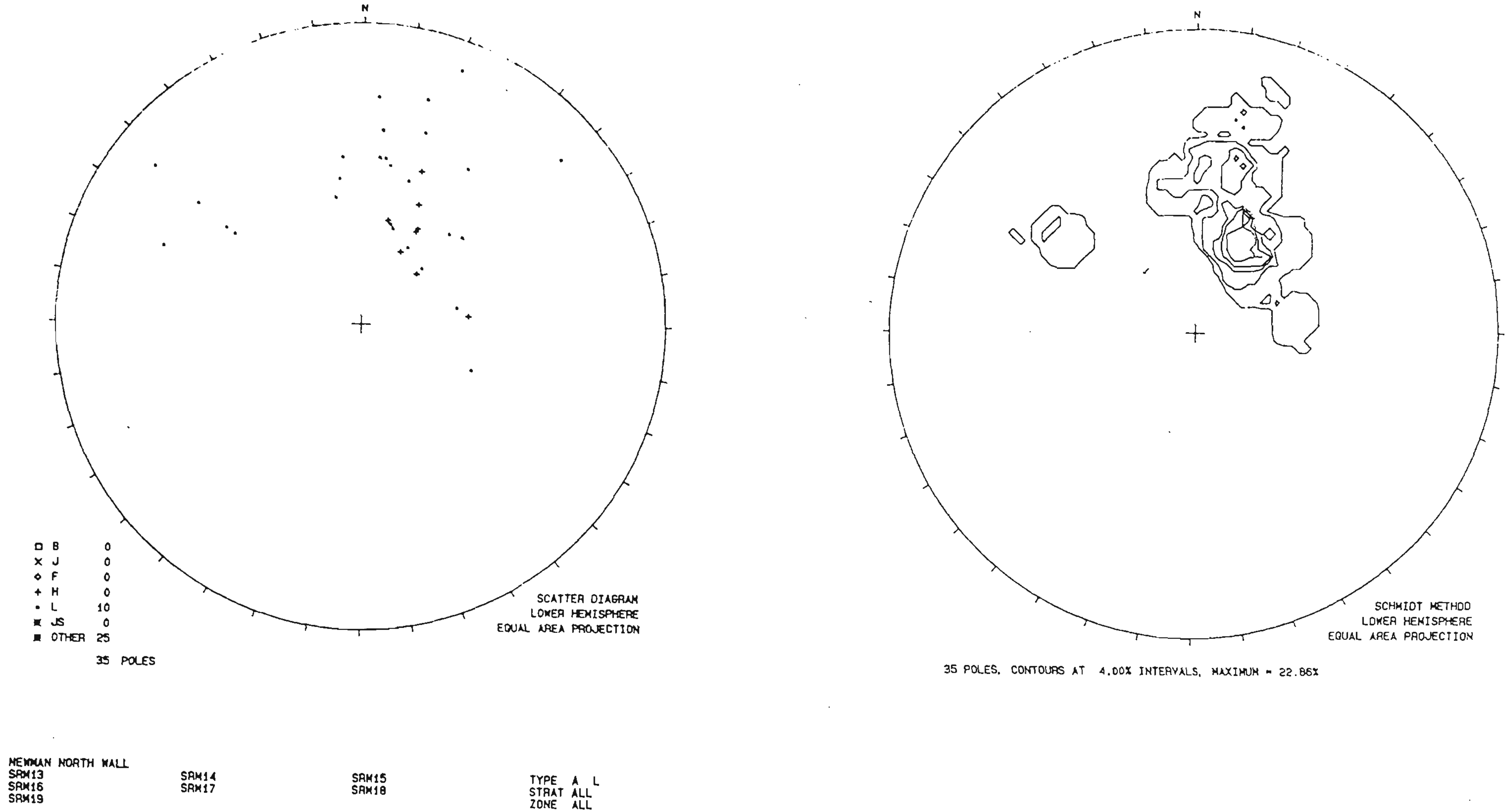




Predominant shallow westerly plunge of main fold system well defined.

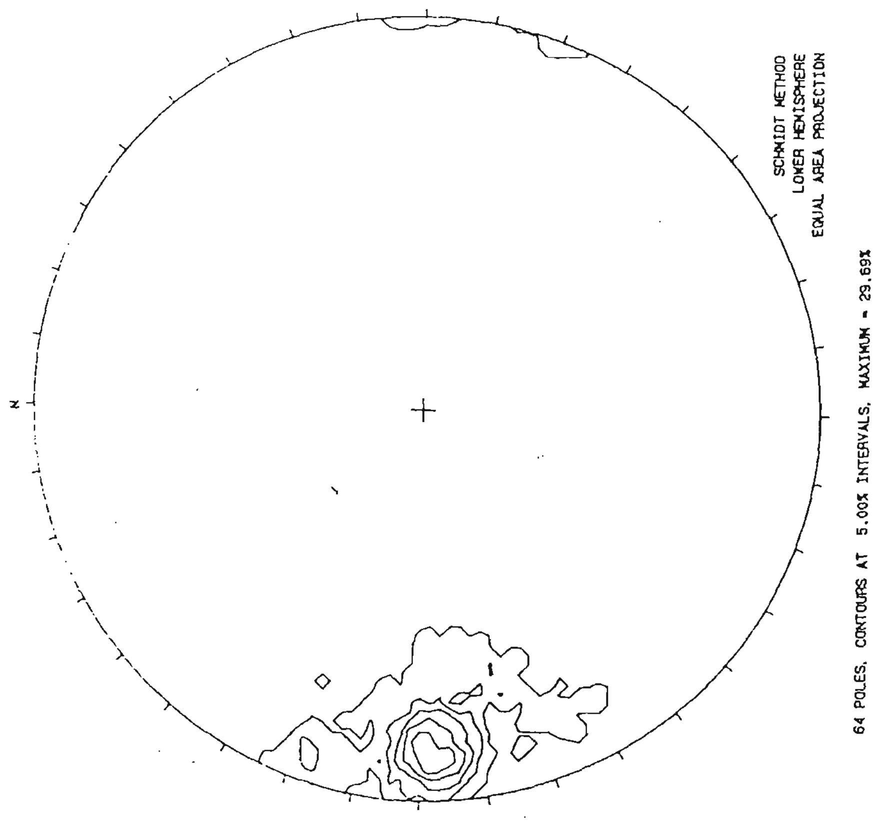
Scale	STEREOGRAPHIC PROJECTION OF LINEATIONS AND MINOR FOLD AXES WITHIN THE EAST PIT
Drn	
Dwg No.	

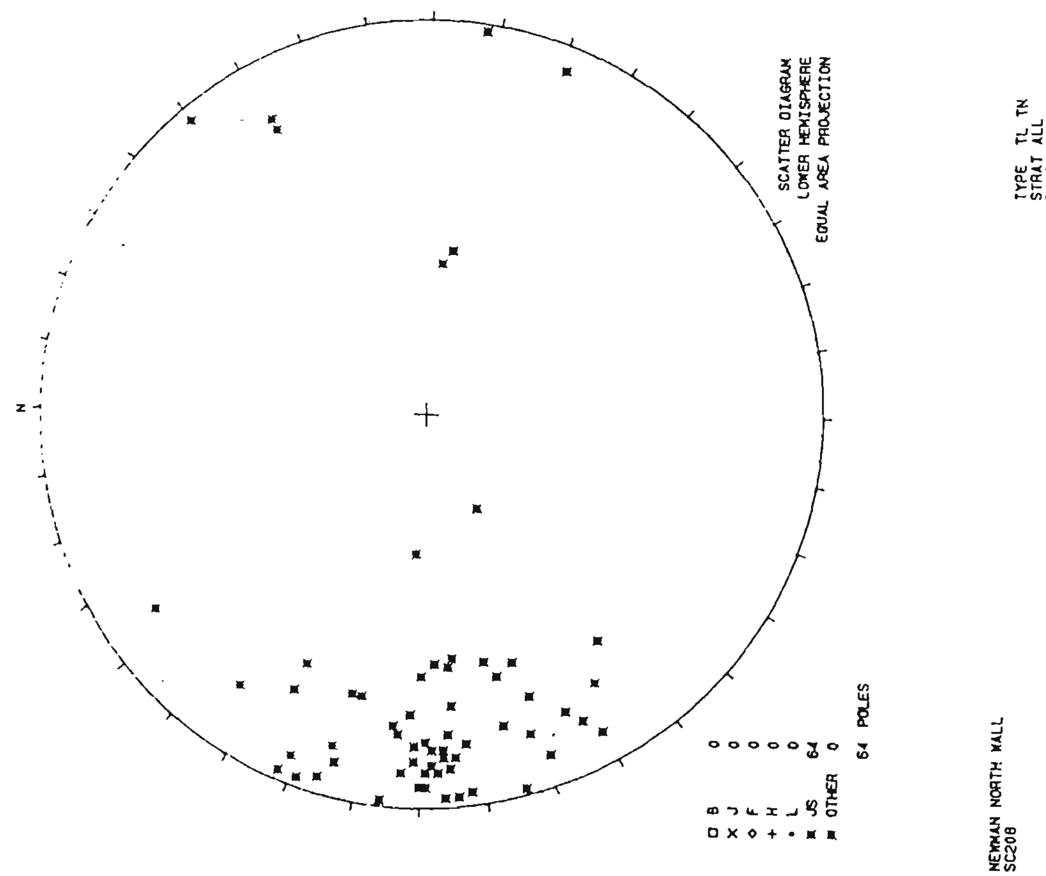
FIGURE 4.7



South-southwest dip is evident and hence north sense of overturn on main fold systems.

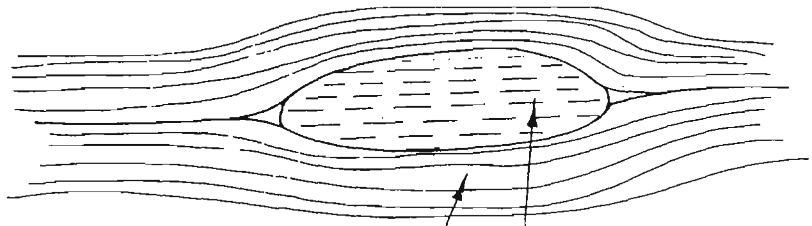
Scale	STEREOGRAPHIC PROJECTION OF POLES TO AXIAL PLANES AND CLEAVAGE	FIGURE 4.8
Drn		
Dwg No.		



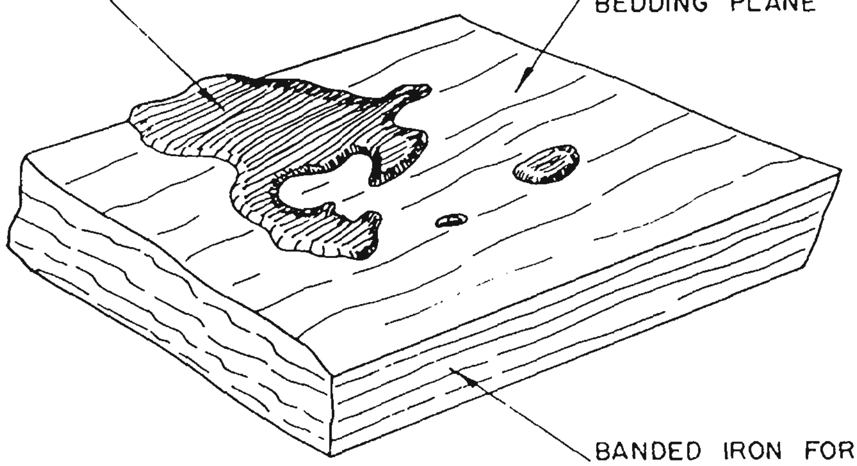


Large variation in fold plunges can occur in relatively small areas.

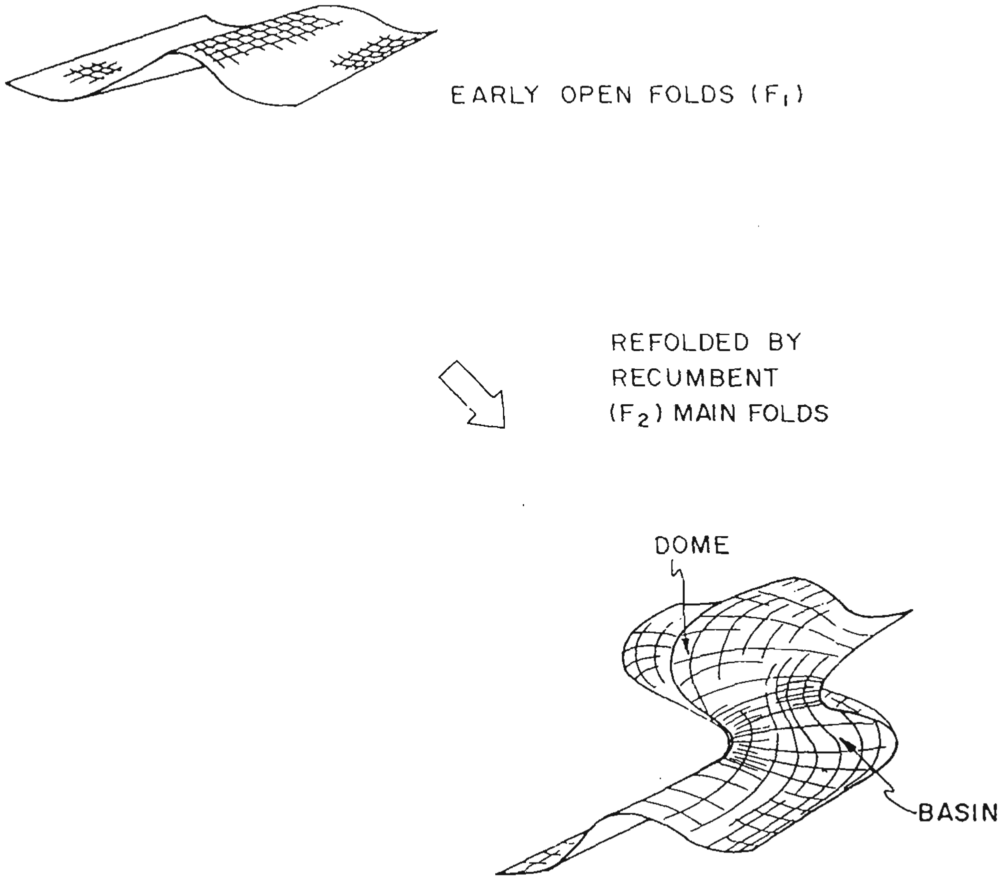
Scale	STEREOGRAPHIC PROJECTION OF LINEATIONS AND SMALL SCALE FOLD AXES IN "DEATH VALLEY"	FIGURE 4.9
Drn		
Dwg No.		



Field sketch of chert pod. Bedding is visible and clearly truncated without apparent deformation hence probably due to dissolution.

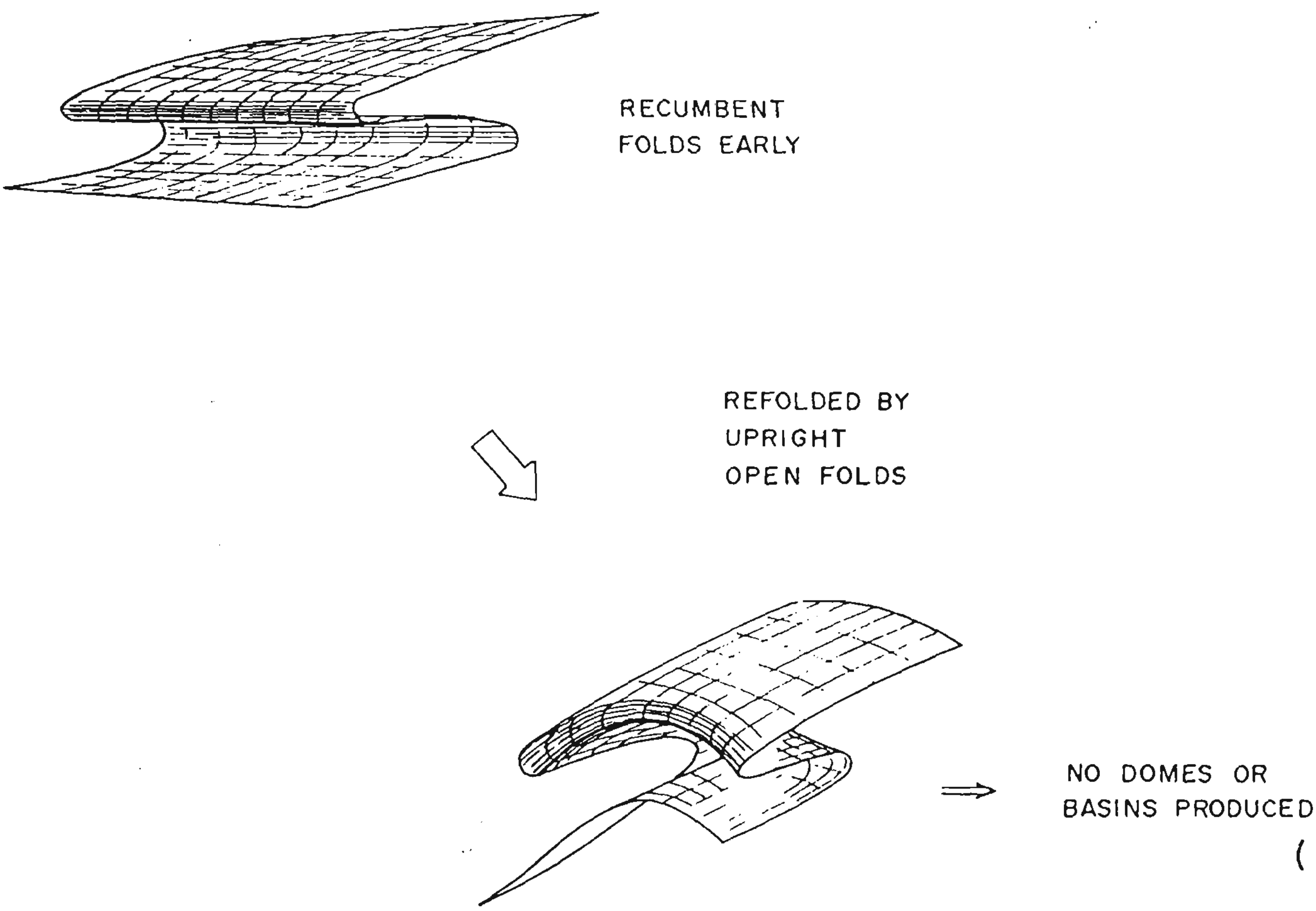


	Scale	SCHEMATIC SECTION THROUGH CHERT POD AND SKETCH OF DISSOLVED CHERT LAYER	FIGURE 4.10
	Drn		
	Dwg No.		



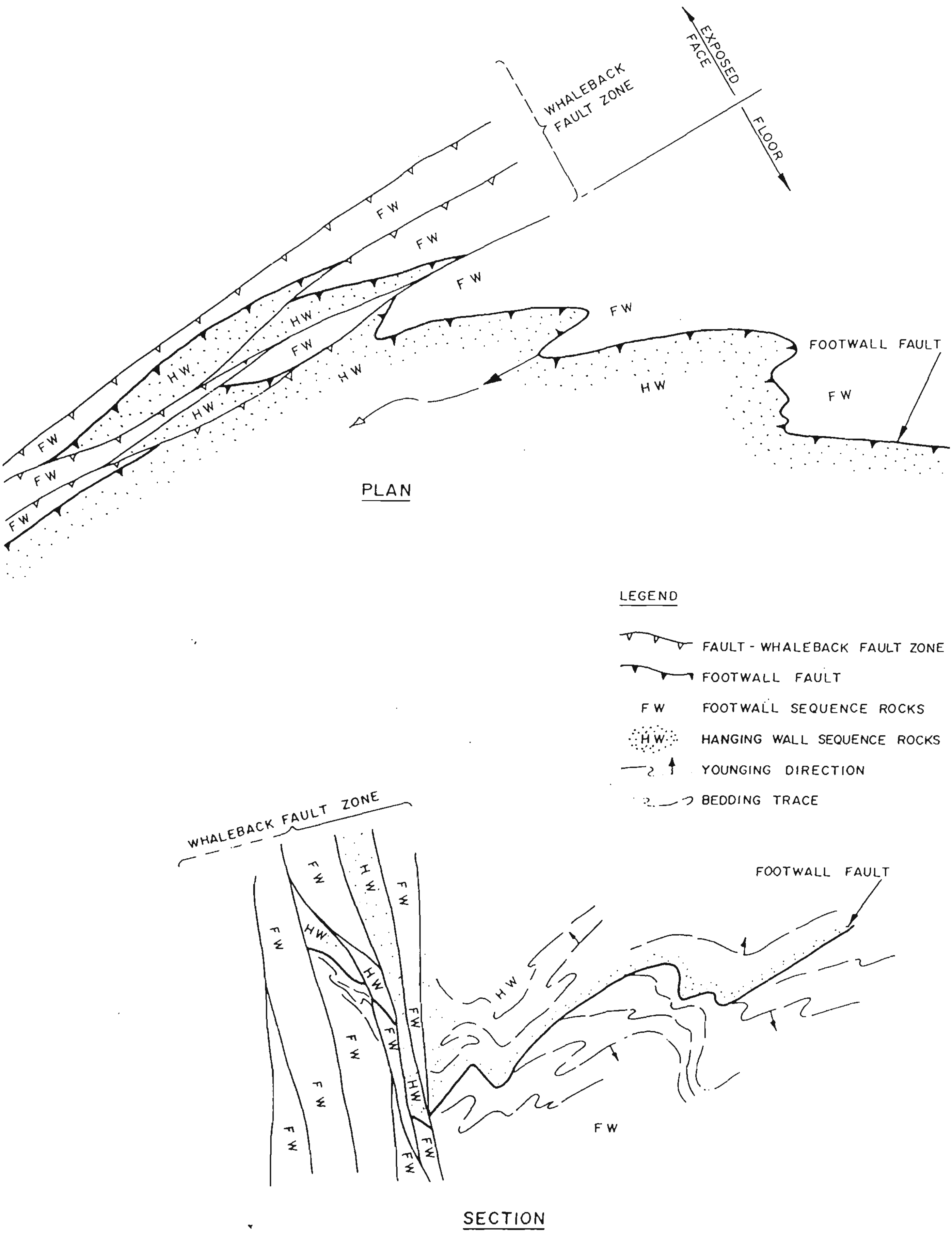
(a)

Two sequences of refoling. The first produces domes and basins as exposed in " Death Valley " the second does not.

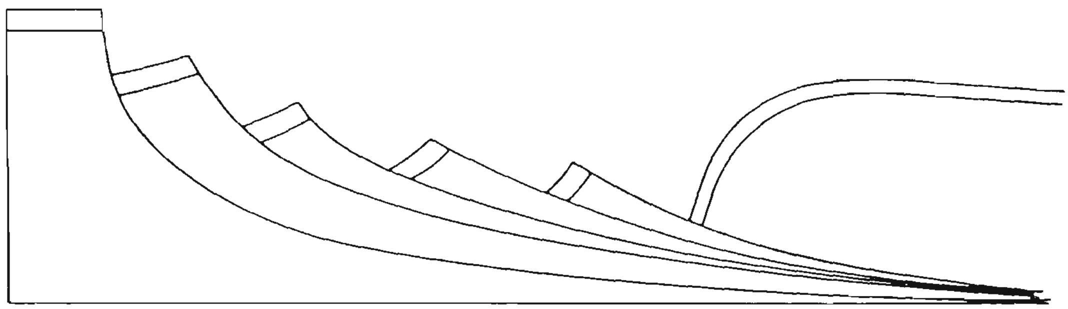


(b)

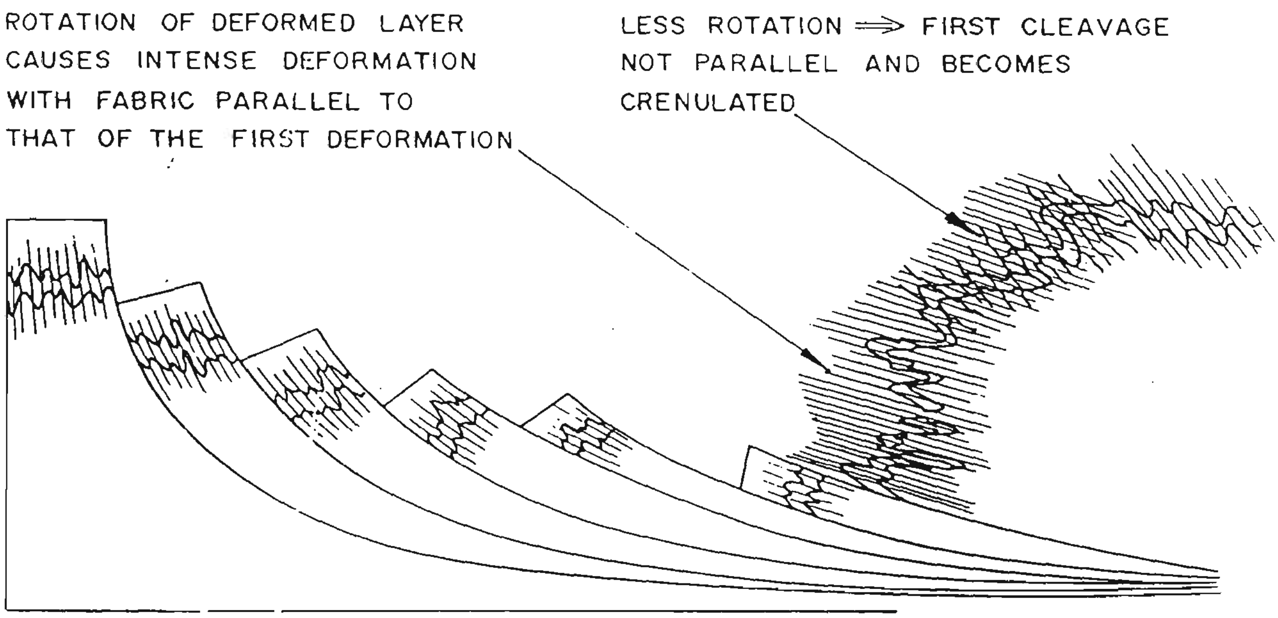
	Scale	DIAGRAM OF REFOLDING GEOMETRIES	FIGURE 4.11
	Drn		
	Dwg No.		



Scale	SCHEMATIC MAP AND CROSS-SECTION SHOWING THE TYPES OF RELATIONSHIPS OBSERVED IN THE AREA WHERE THE EAST FOOTWALL FAULT AND WFZ INTERSECT	FIGURE 4.12
Drn		
Dwg No.		

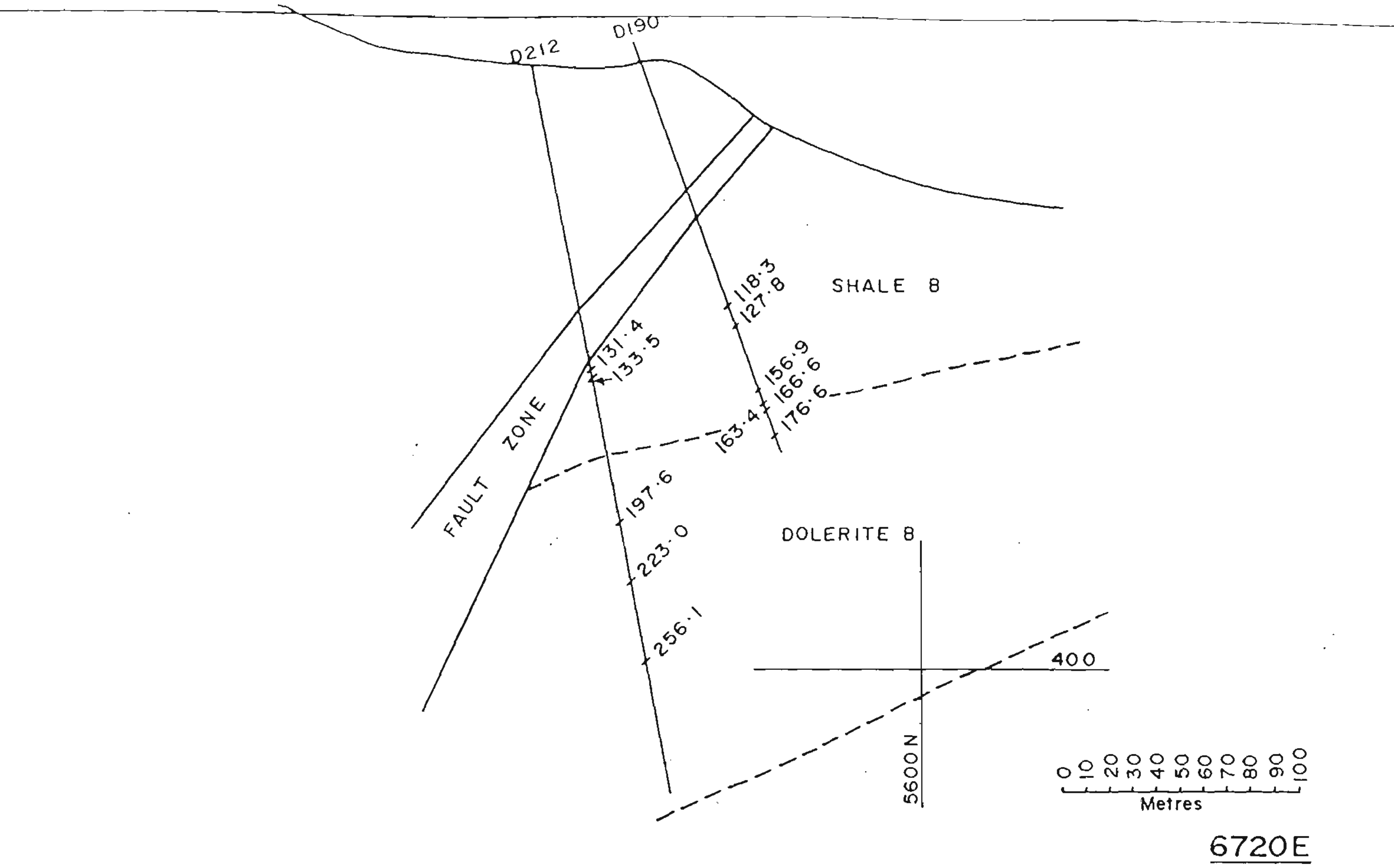


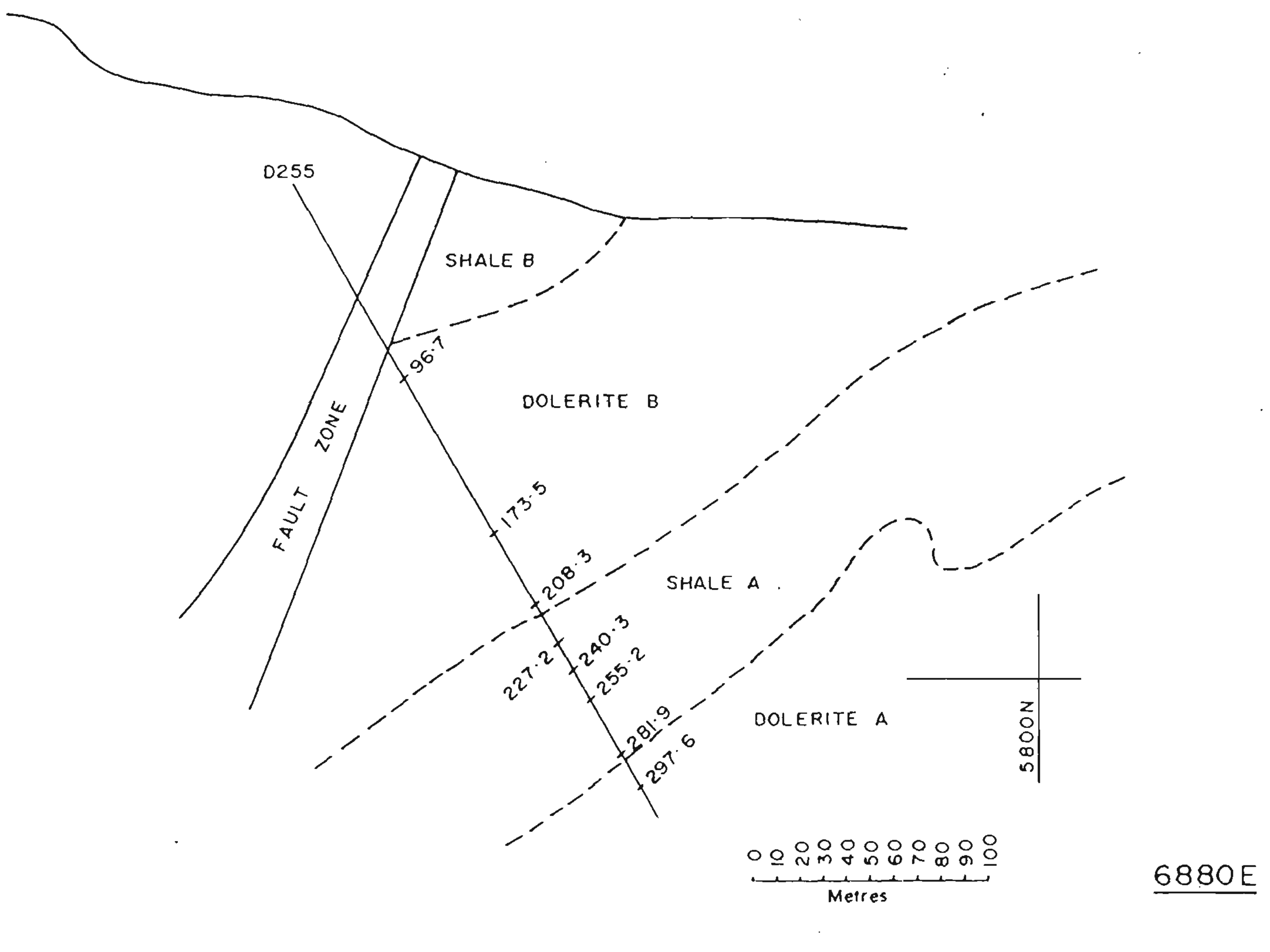
a) EXPECTED ROTATION OF HORIZONTAL BEDDING ASSOCIATED WITH LISTRIC FAULTING



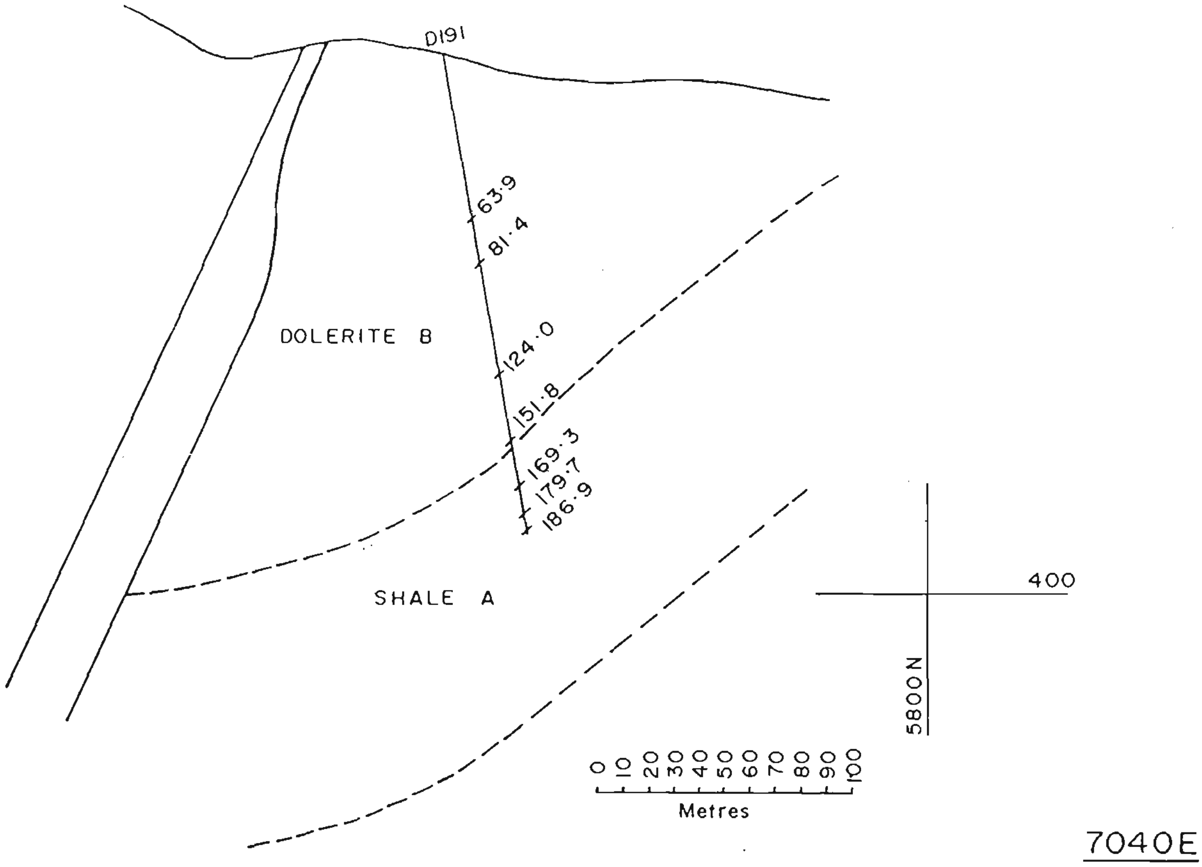
b) EXPECTED ROTATION OF AXIAL PLANE CLEAVAGE ASSOCIATED WITH LISTRIC FAULTING

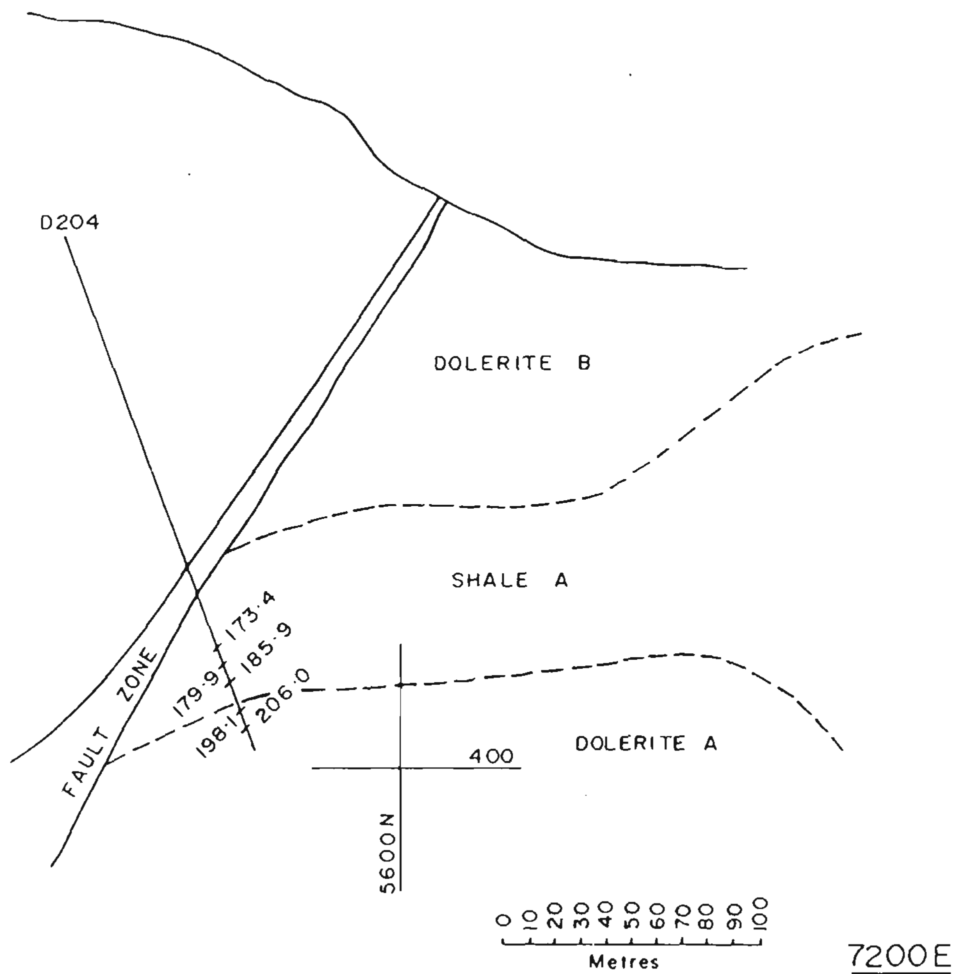
	Scale	ROTATION OF STRAIN MARKERS ADJACENT TO A LISTRIC FAULT	FIGURE 4.13
	Drn		
	Dwg No.		



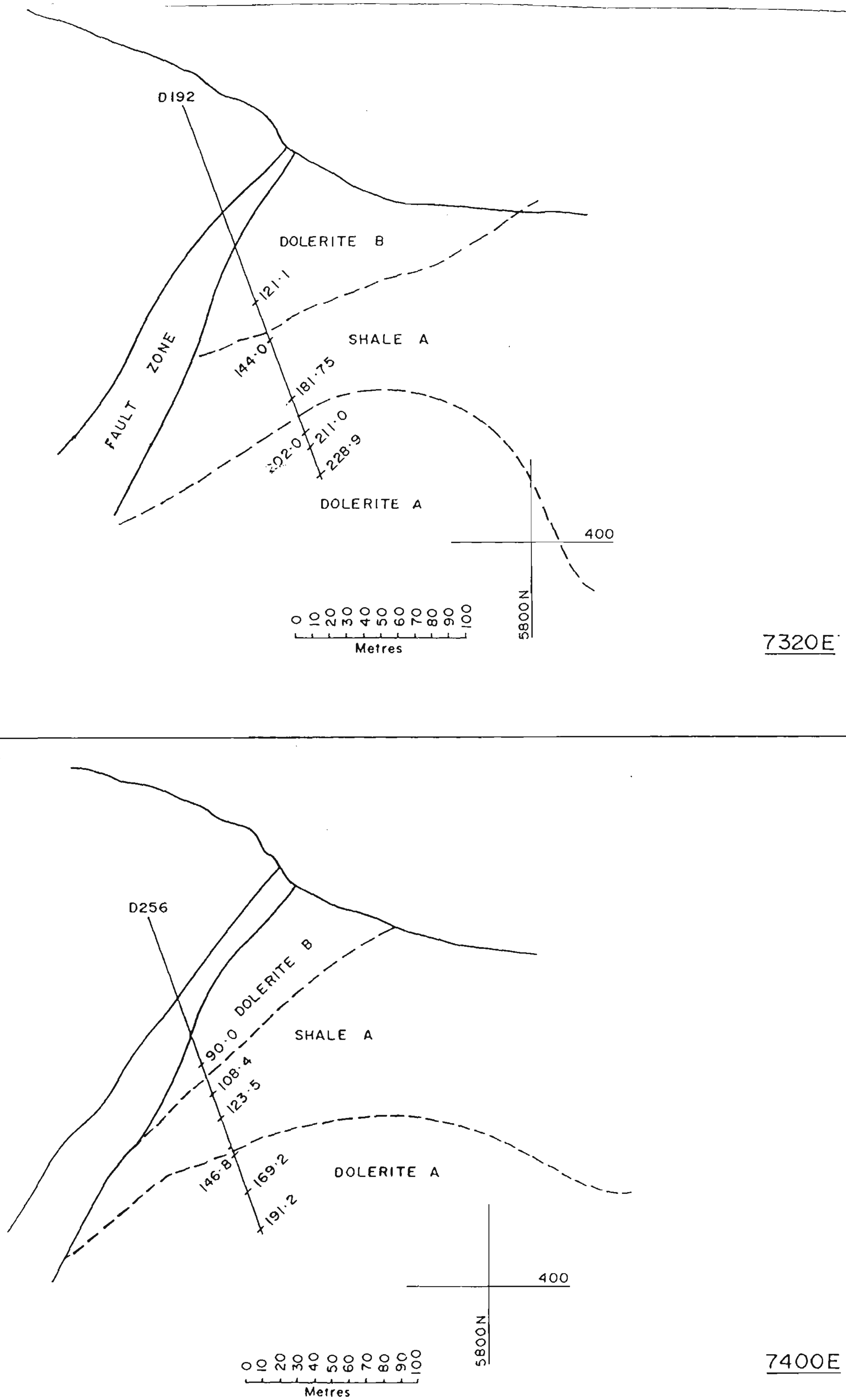


Scale	SECTIONS 6720E AND 6880E SHOWING POSITIONS OF ROCK SAMPLES FOR GEOCHEMICAL ANALYSIS.	FIGURE 4.14
Drn		
Dwg No.		

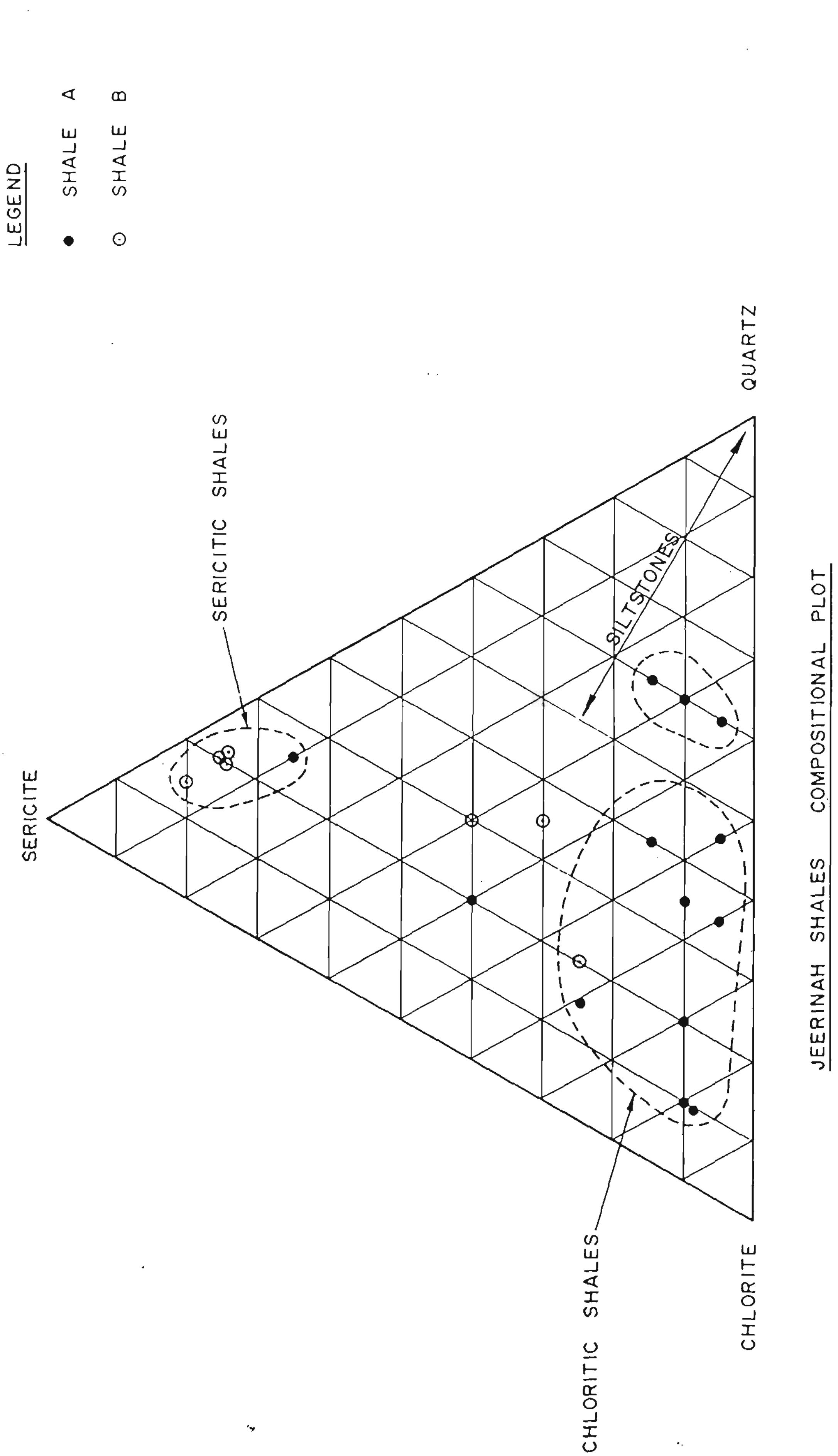




Scale	SECTIONS 7040E AND 7200E SHOWING POSITIONS OF ROCK SAMPLES TAKEN FOR GEOCHEMICAL ANALYSIS.	FIGURE 4.15
Drn		
Dwg No.		



Scale	SECTIONS 7320E AND 7400E SHOWING POSITIONS OF ROCK SAMPLES	FIGURE 4.16
Drn		
Dwg No.		



	Scale	TRIANGULAR PLOT SHOWING MINERALOGICAL COMPOSITION OF THE JEERINAH SHALES	FIGURE 4.17
	Drn		
	Dwg No.		

TABLE 4.1

AVERAGE OXIDE GEOCHEMISTRY: JEERINAH SHALES

OXIDE	SHALE A		SHALE B	
	%	S D	%	S D
SiO	58.2	9.9	59.8	3.0
TiO	0.82	0.32	0.95	0.06
Al O	12.1	3.5	16.5	1.1
Fe O	15.7	5.6	9.9	2.0
MnO	0.14	0.11	0.05	0.01
MgO	4.2	1.8	3.8	0.3
CaO	1.14	1.8	0.15	0.04
Na O	0.06	0.03	0.77	1.18
K O	1.84	2.18	3.15	1.02
P O	0.13	0.04	0.11	0.02
LOI	5.4	2.4	4.2	0.7

TABLE 4.2

MINERALOGICAL PERCENTAGES FOR JEERINAH DOLERITES

DOLERITE A

MINERAL	HOLE NUMBER								
	D192 inc depth			D204		D255	D256		
Plagioclase	35	40	25	45	30	25	40	45	40
Amphibole	10	20	30	-	45	40	20	25	25
Epidote	20	5	20	-	7	25	15	10	10
C. Pyroxene	20	-	-	-	-	-	-	-	-
Calcite	3	5	3	-	-	-	4	5	3
Pot. Felspar	5	5	2	-	-	2	-	1	-
Quartz	-	-	-	15	5	-	1	-	3
Opaques	2	-	-	3	5	1	5	4	3
Chlorite	5	20	15	30	5	7	15	10	15
Sericite	-	-	-	7	-	-	-	-	-
Carbonate	-	-	-	-	3	-	-	-	-
Leuco-Sphene	-	4	3	-	-	-	-	-	-

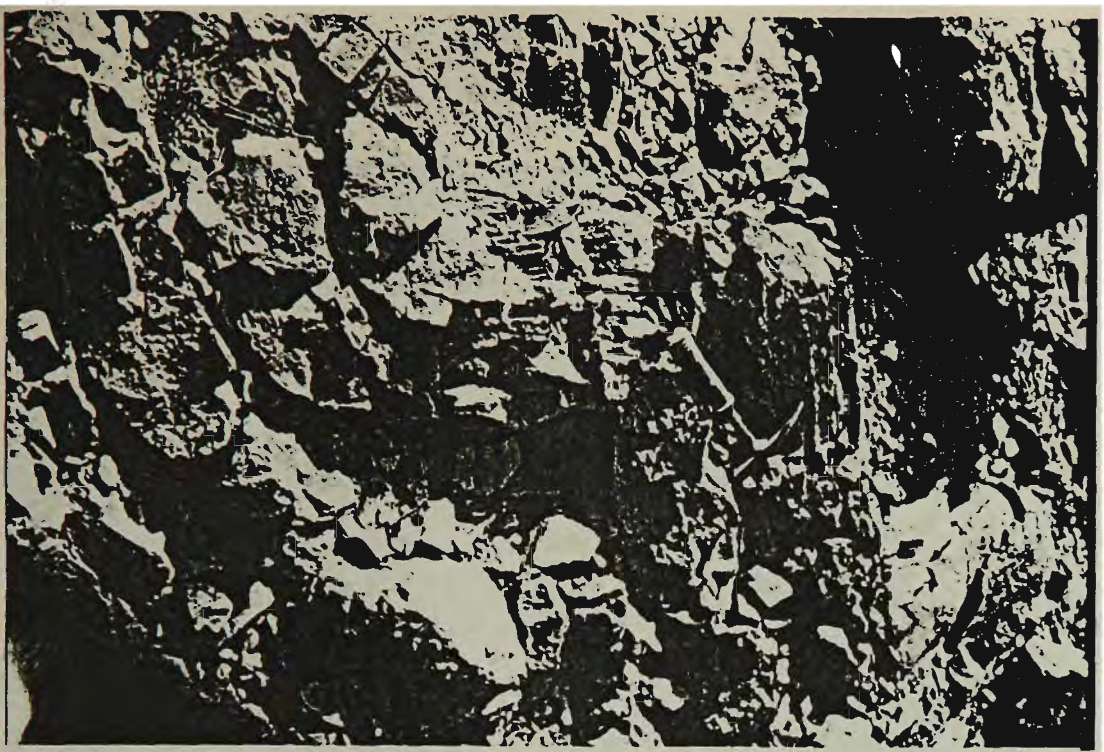
DOLERITE B

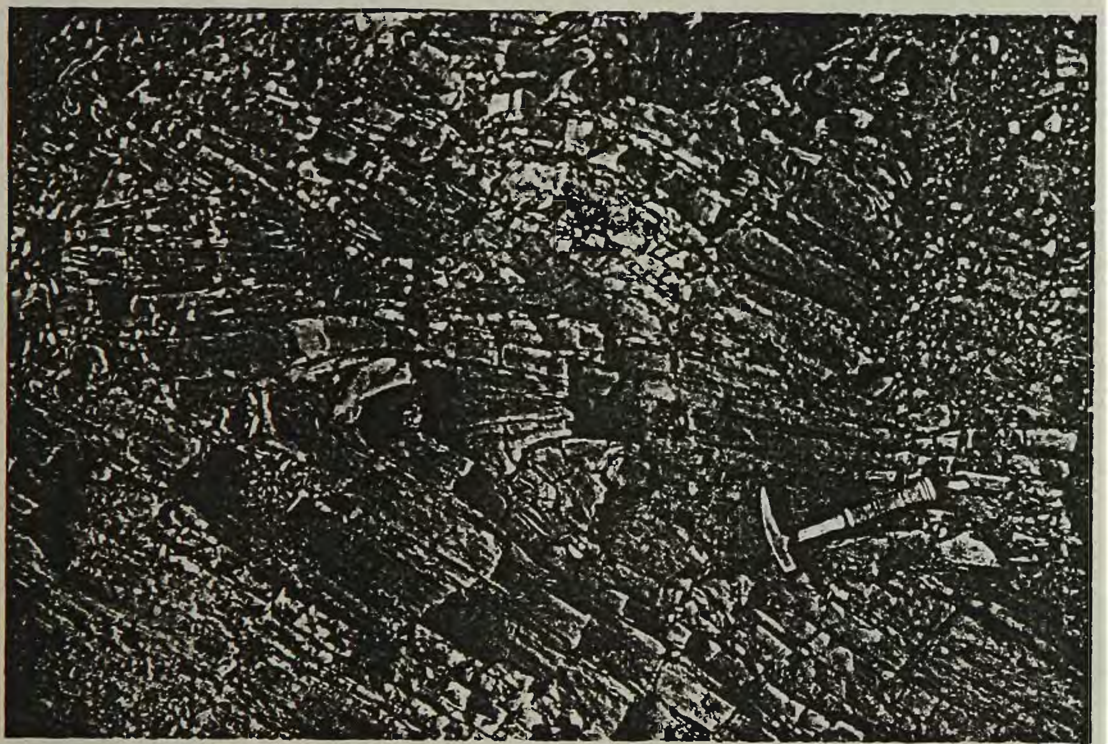
MINERAL	HOLE NUMBER												
	D190	D191 inc depth				D192	D212 inc depth			D255 inc depth			D256
Plagioclase	35	45	40	1	5	10	45	45	-	25	40	8	13
Amphibole	30	20	20	50	55	55	10	15	45	40	25	50	50
Epidote	25	15	25	40	35	30	10	15	50	25	20	30	30
C. Pyroxene	3	20	10	-	-	-	20	15	-	-	10	-	-
Calcite	3	-	-	-	-	-	2	2	-	-	1	-	-
Pot. Felspar	1	-	-	1	3	1	3	-	3	3	2	5	1
Quartz	1	-	1	2	-	-	-	-	-	-	-	-	-
Opaques	1	1	-	2	1	2	-	2	1	1	1	1	1
Chlorite	-	-	2	2	-	1	10	5	-	1	-	5	5
Sericite	-	-	-	1	-	1	-	-	-	2	-	1	-
Carbonate	-	-	-	-	-	-	-	-	-	3	-	-	-

TABLE 4.3

AVERAGE OXIDE GEOCHEMISTRY: JEERINAH DOLERITES

OXIDE	DOLERITE A		DOLERITE B	
	%	S D	%	S D
SiO	48.9	1.4	47.6	1.2
TiO	1.04	0.14	0.64	0.11
Al O	14.3	0.6	15.5	0.9
Fe O	11.0	1.6	9.9	0.9
MnO	0.15	0.03	0.16	0.02
MgO	8.1	1.5	9.5	1.2
CaO	8.1	1.9	10.5	2.3
Na O	2.6	1.3	2.3	0.7
K O	1.4	0.9	0.8	0.5
P O	0.16	0.04	0.06	0.02
LOI	4.5	1.0	2.9	0.8





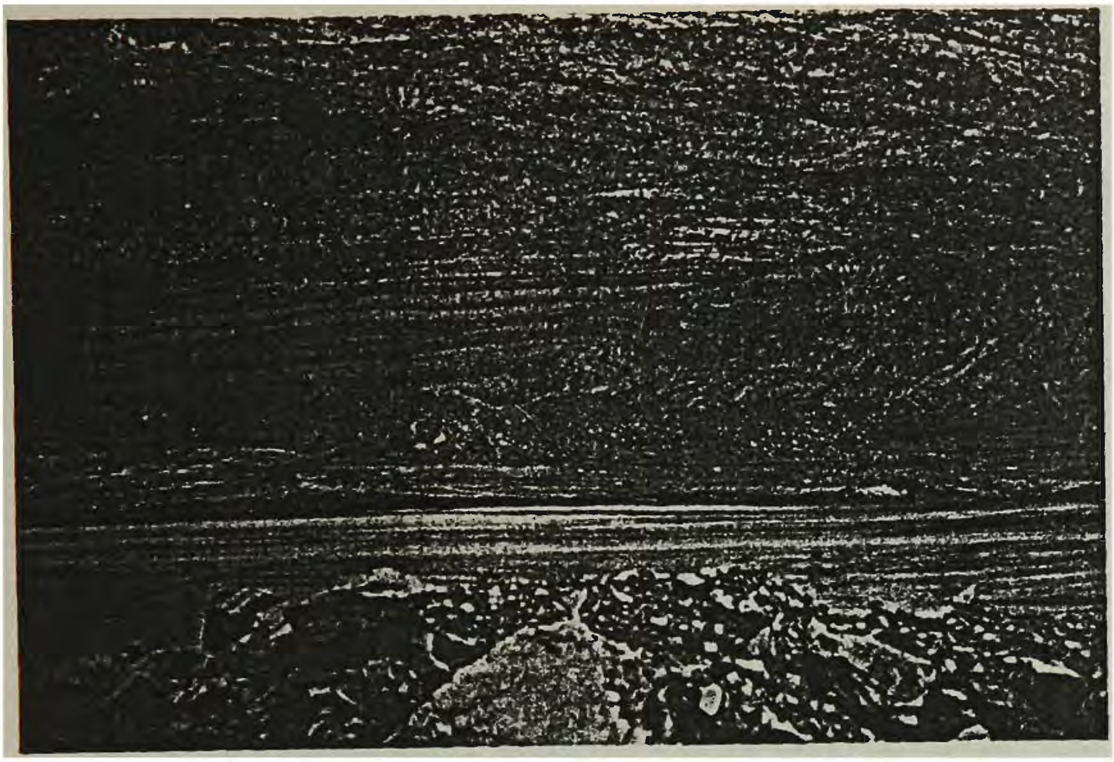


Plate.4.2 Strongly overturned fold limb in the current breakthrough developed in predominantly shales of the Mt. Sylvia Formation.

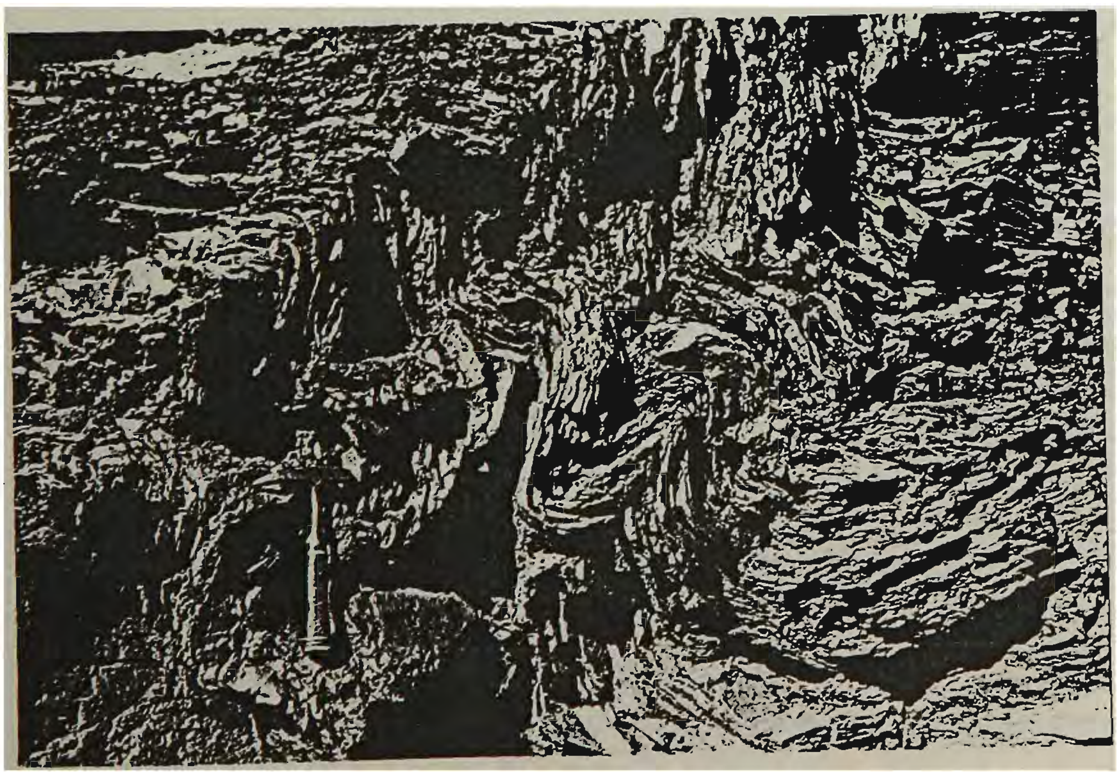


Plate.4.3 Well developed chevron folds in the Mt. McRae Shale.

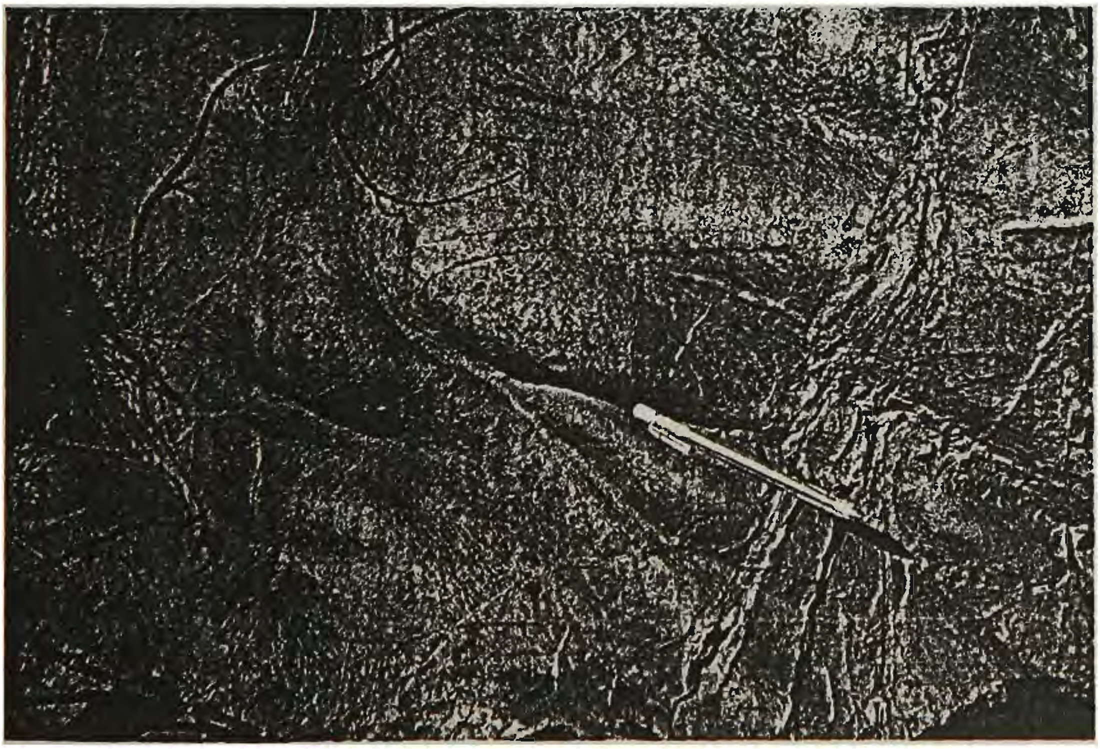


Plate.4.4 "Early" oblique structures developed in "Death Valley".

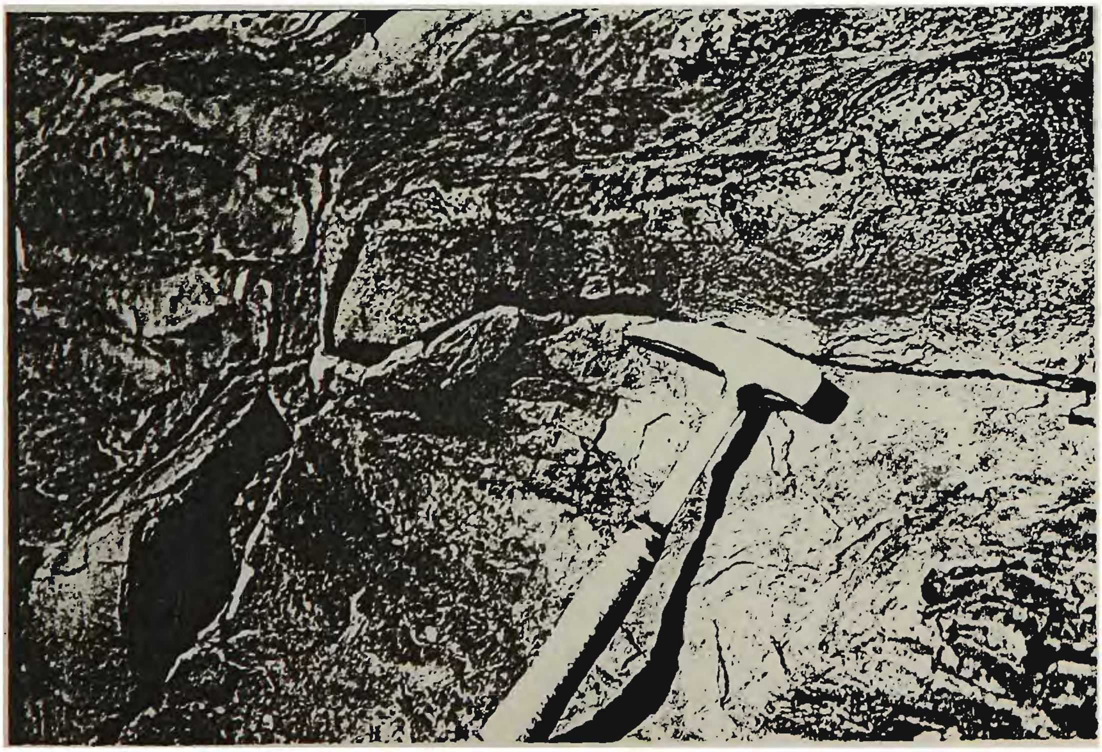


Plate.4.5 Well developed domes and basin structures developed in "Death Valley".

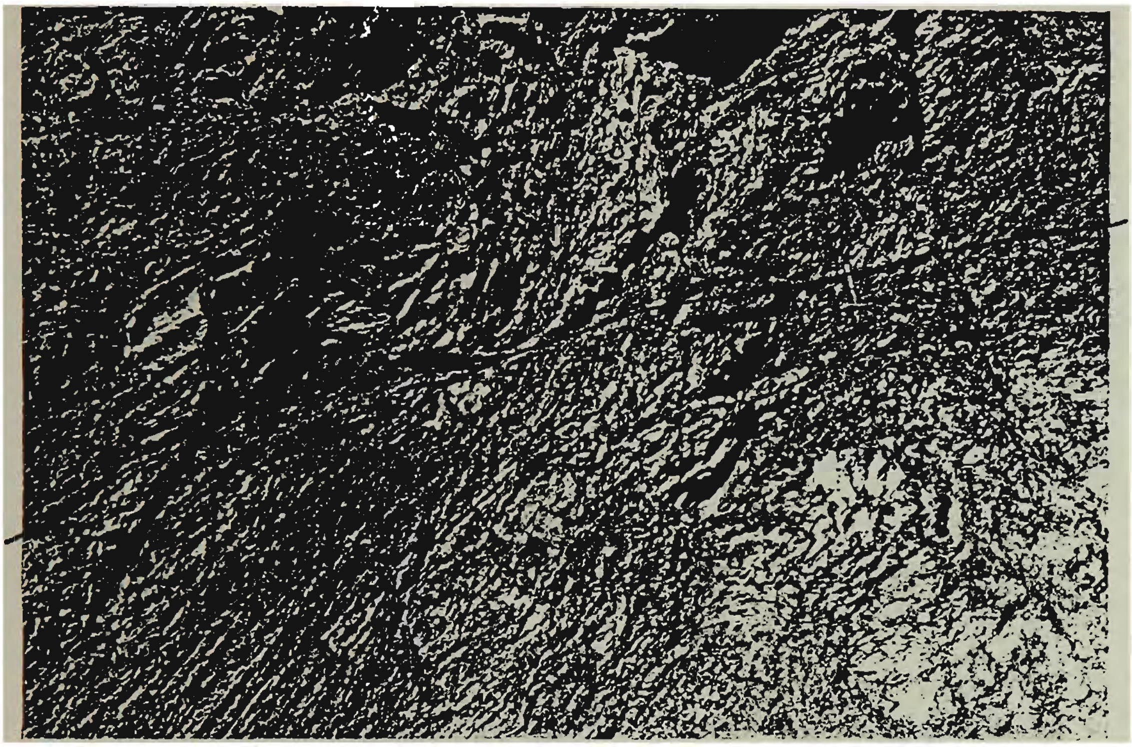


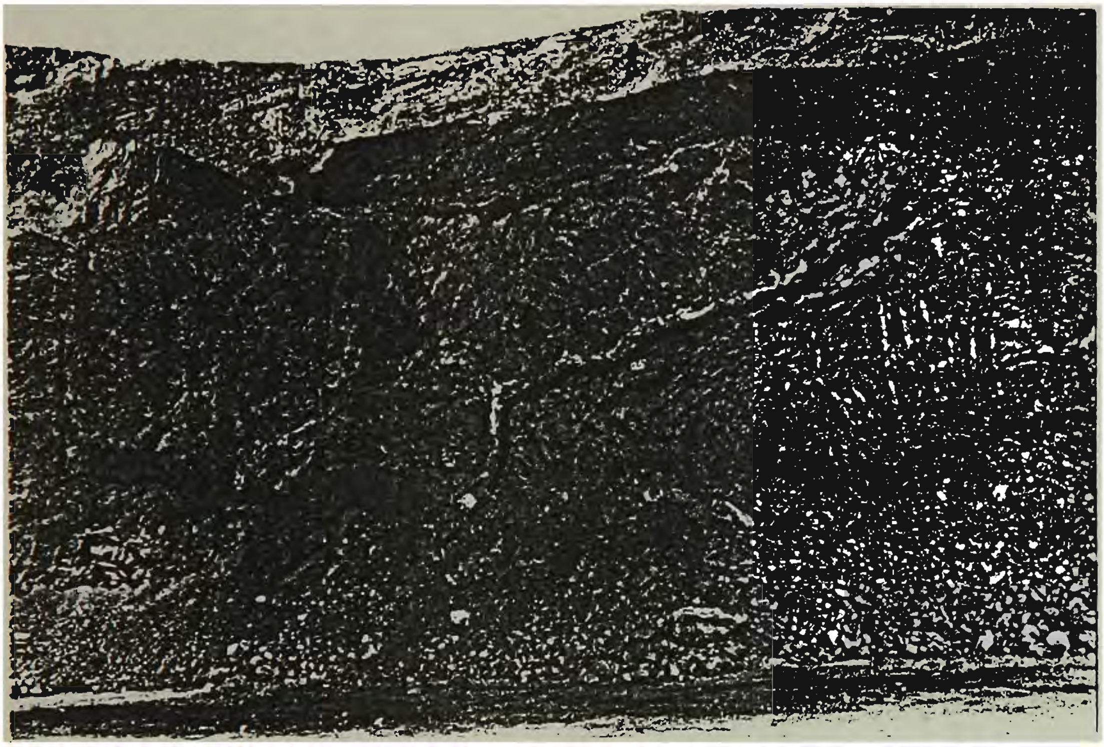
Plate.4.6 Low angle reverse fault offsetting the East Footwall Fault. Dextral sense looking east.

Plate. 4.7 Steeply dipping normal fault offsetting the East Footwall Fault. Dextral sense looking east.

Plate.4.8 Photograph showing intensely folded chert and shale sequence (Mt. Sylvia Formation?) below the East Footwall Fault.

Plate.4.9 Folds in the Mt. McRae Shale immediately below the East Footwall Fault. Looking east, vergence is upwards to the south and stratigraphy is overturned.

Plate.4.10 Intersection of the Whaleback Fault Zone (steep, to the left of photo) and the East Footwall Fault (shallow, to the right of photo).



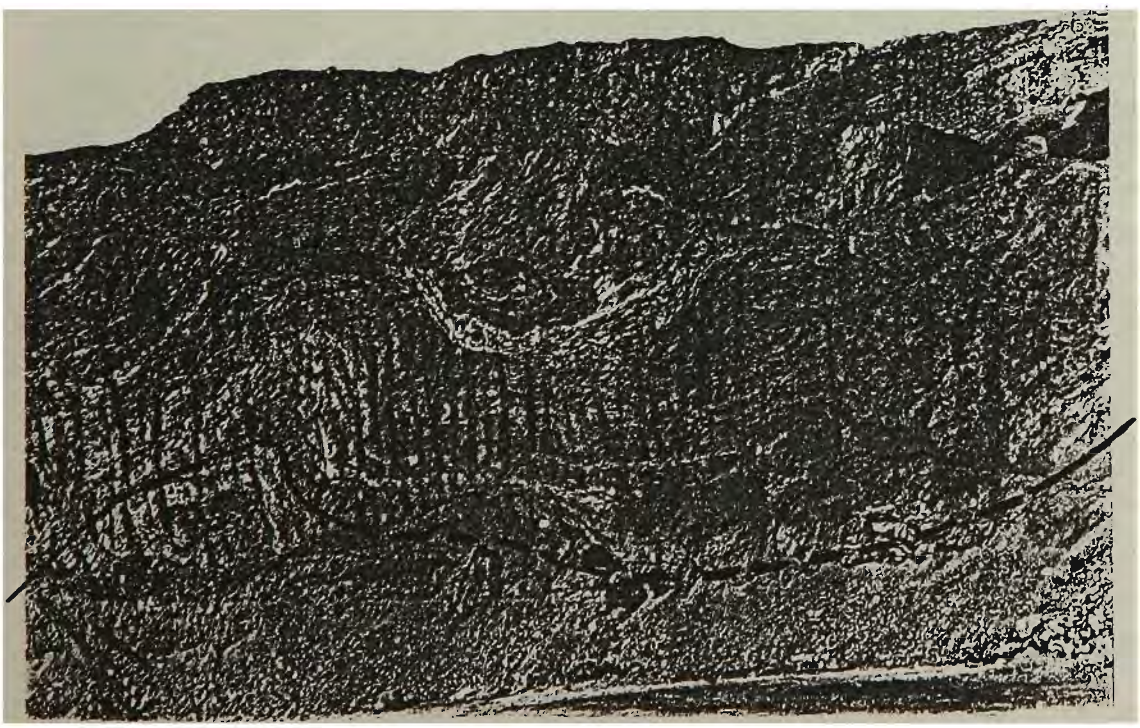


Plate.4.11 Late phase kink folds which fold the surface of the Footwall Fault.

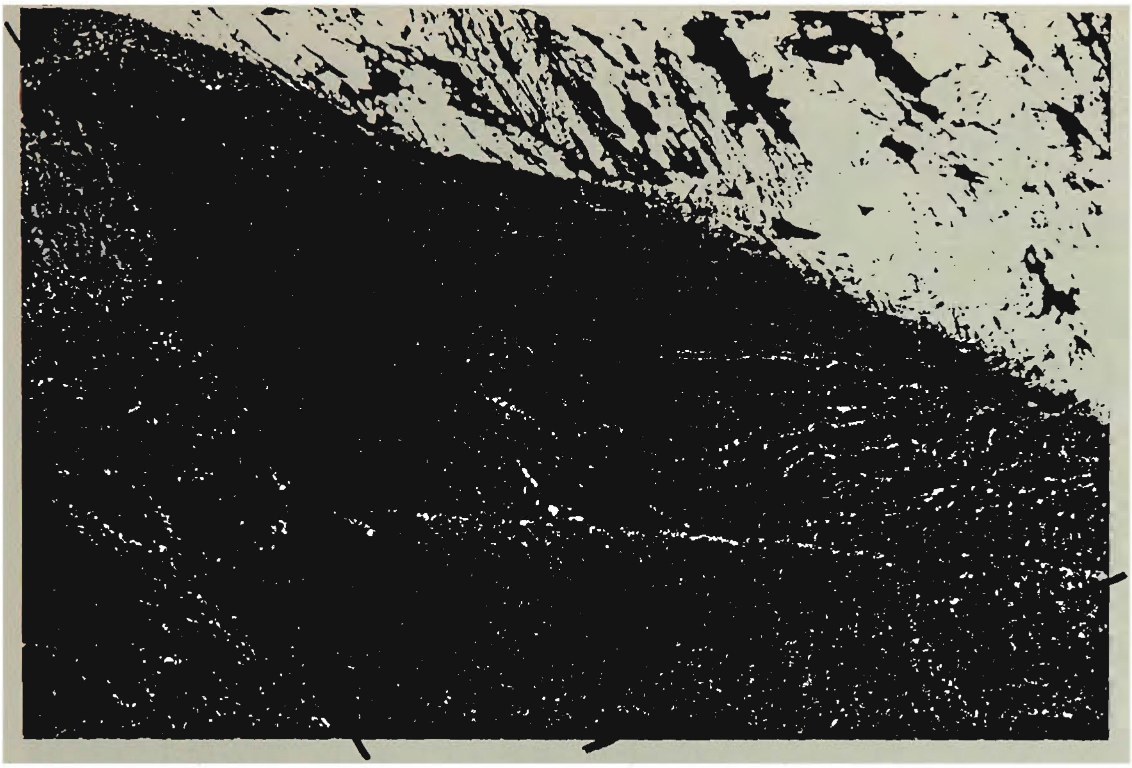


Plate.4.12 The Whaleback Fault Zone clearly truncating a fold (main phase of folding) exposed in oblique section.

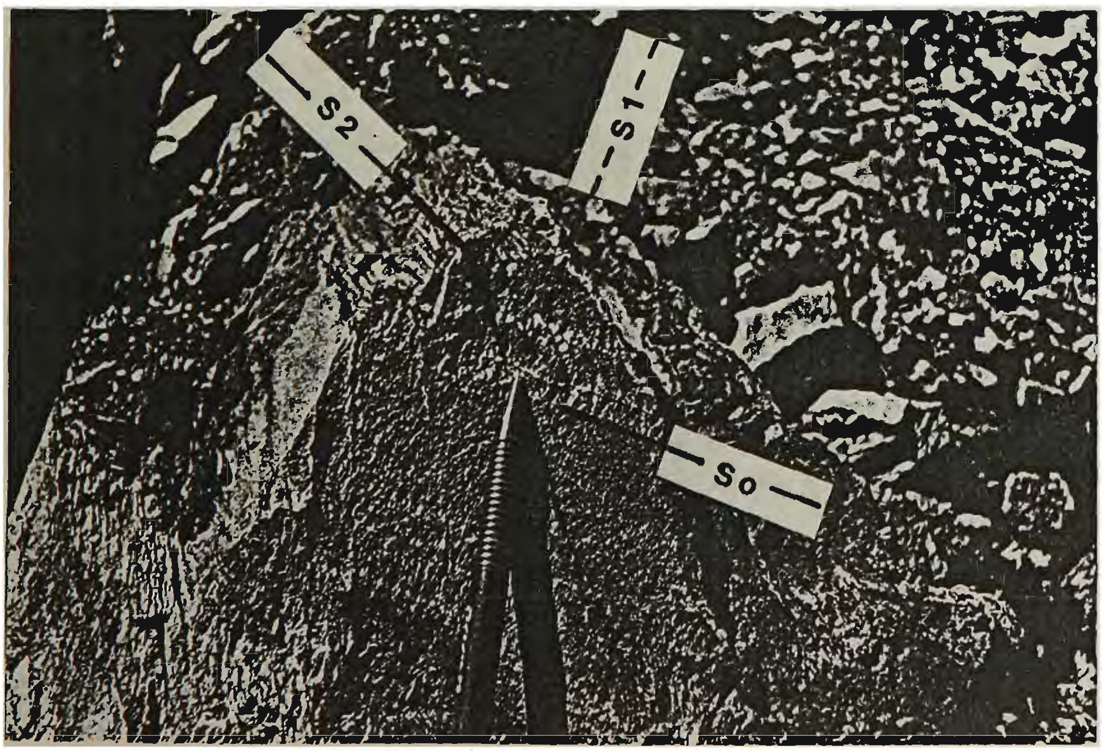


Plate.4.13 Bedding (S_o), slaty cleavage (S_1) and crenulation cleavage (S_2) relationships in the Jeerinah Shale (steeply dipping fold limb).

Plate.4.14 Bedding (S_o)/Cleavage (S_1) relationships in the Jeerinah Shale (shallow fold limb). Photograph taken looking west (down plunge) vergence downwards to the north.

Plate.4.15 Doubly plunging fold in a chert layer within the Jeerinah Shale.

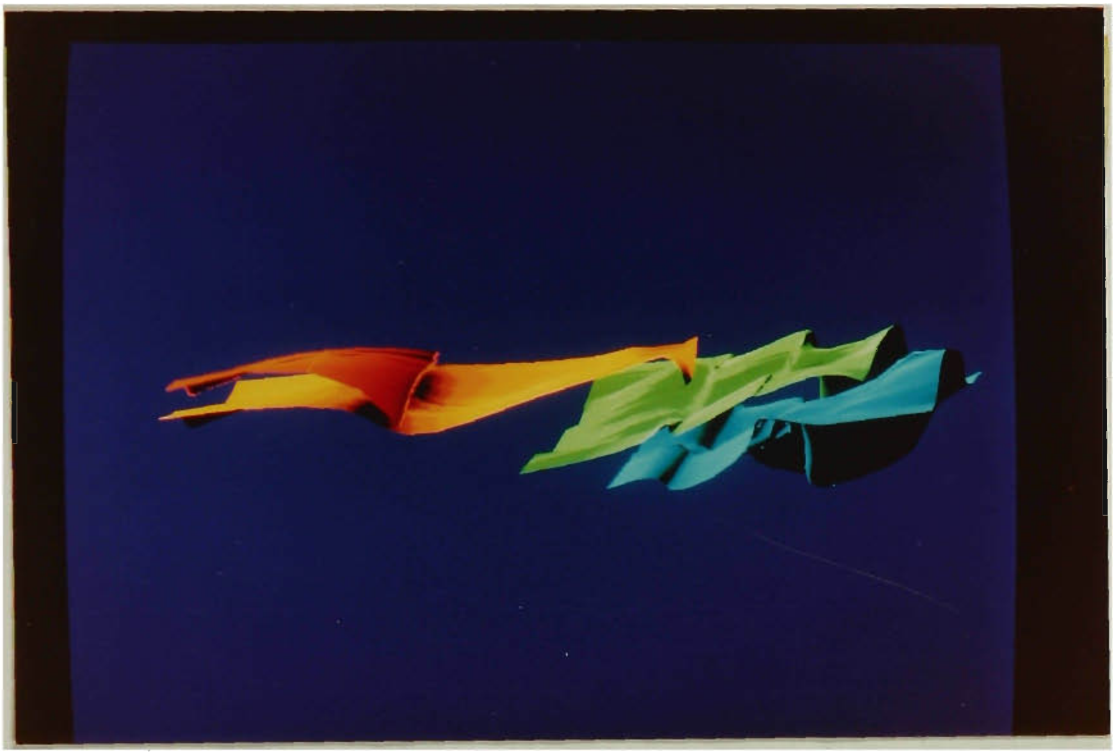


Plate 4.16 Cross section through Jeerinah Formation looking north.
Blue – Dolerite A; Green – Shale A; Orange – Dolerite B;
Red – Shale B; Above Red – Dolerite C
(Note: Scale is about 3km from left to right)

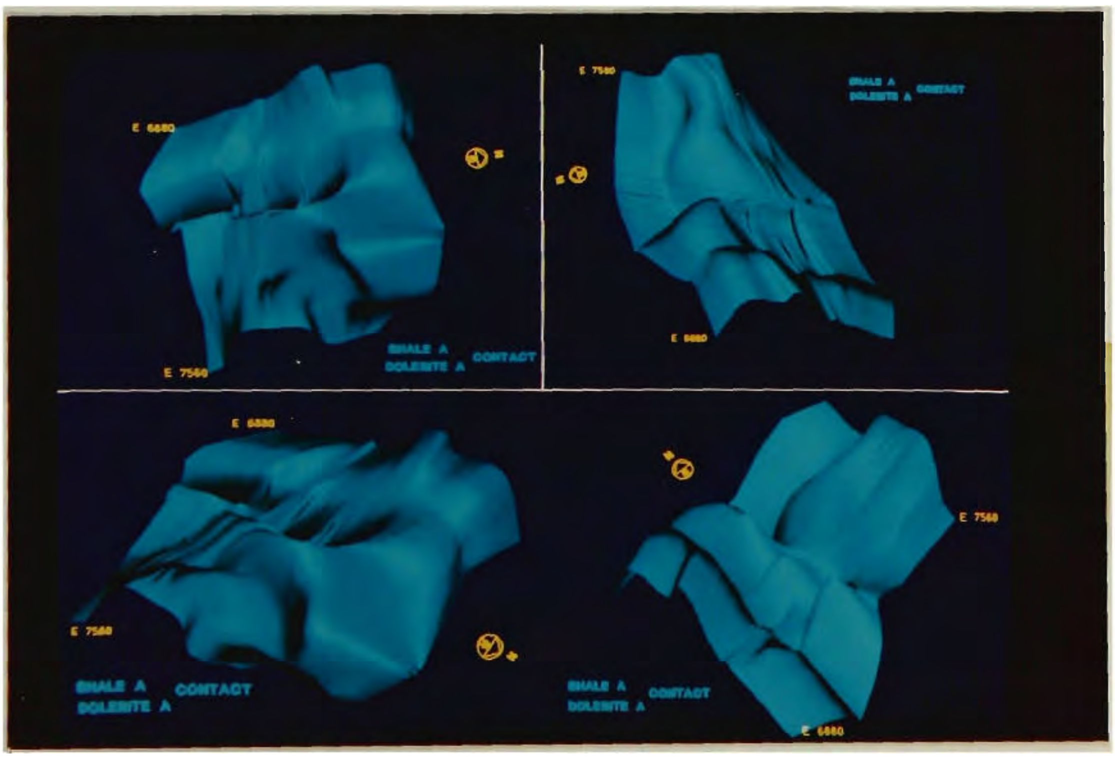


Plate 4.17 Contact between Shale A (above) and Dolerite A (below)

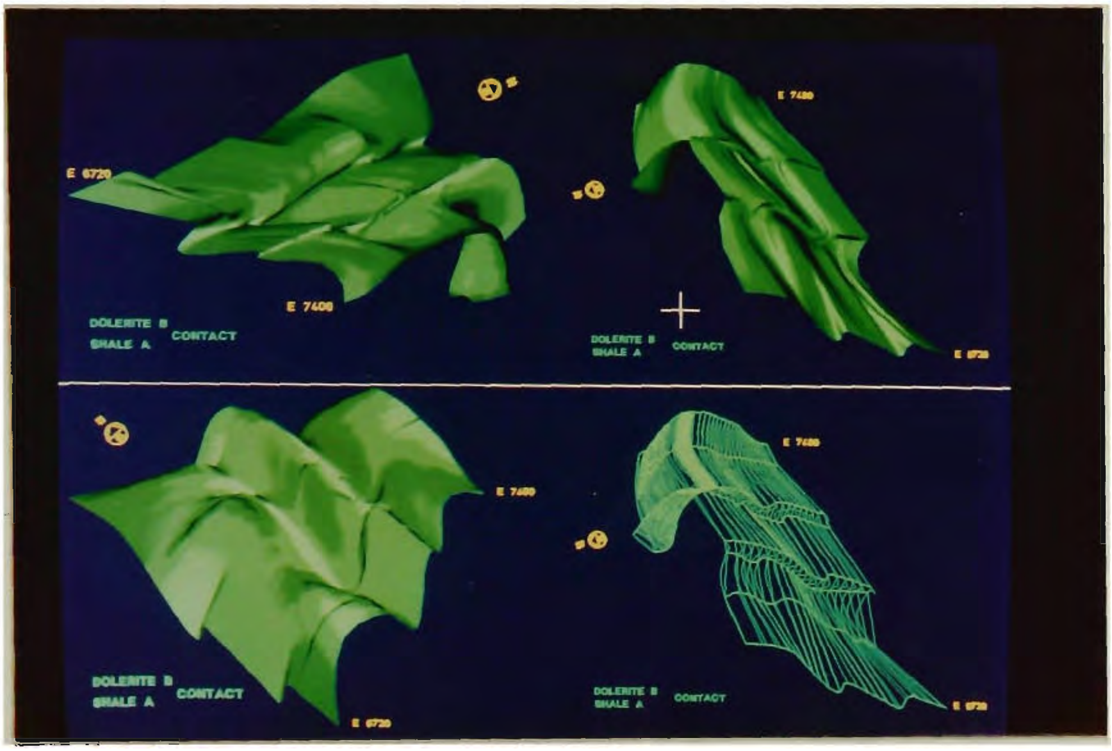


Plate 4.18 Contact between Dolerite B (above) and Shale A (below)

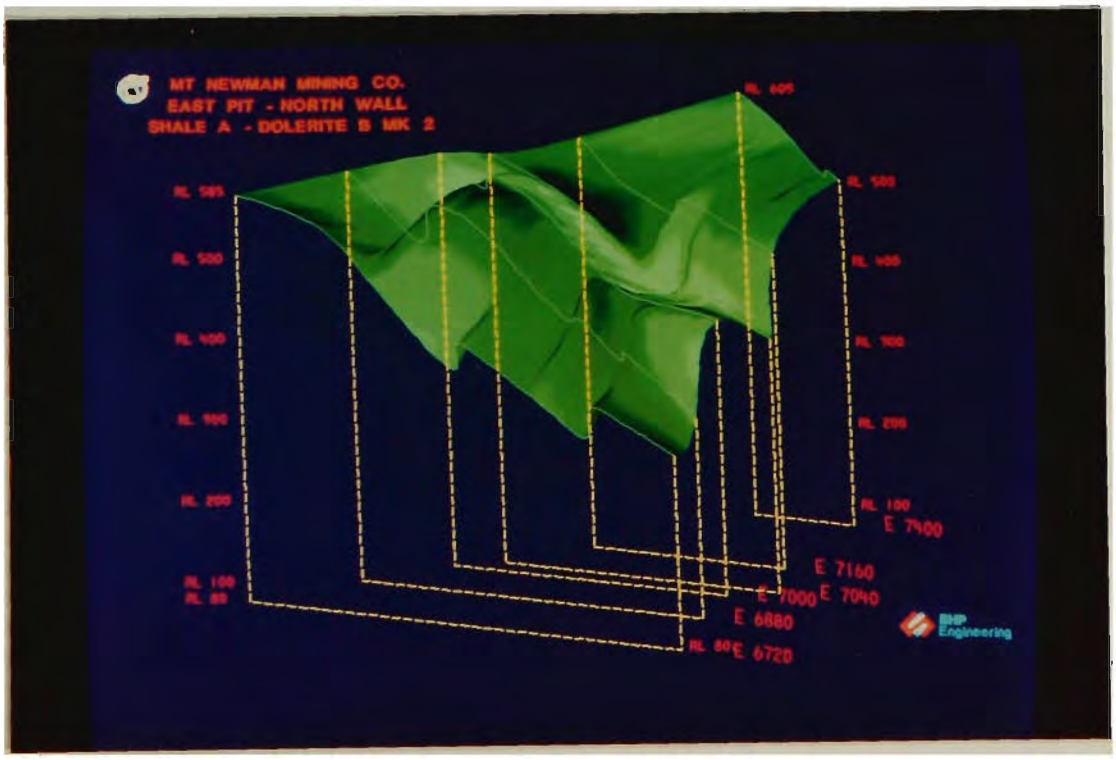


Plate 4.19 Contact between Dolerite B/Shale A (revised from Plate 4.18 after additional borehole information available)

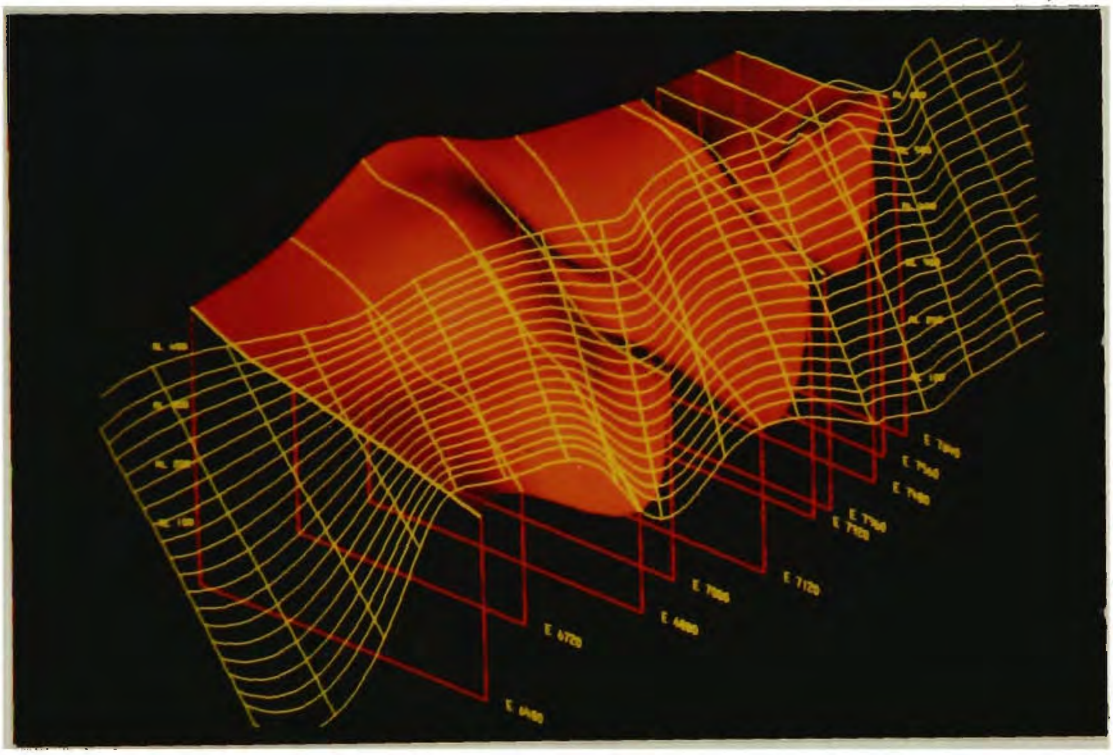


Plate 4.20 Contact of Shale B (above) and Dolerite B (below) which has been truncated by the northern extent of the WFZ (yellow mesh)

CHAPTER 5

CROSS HOLE SEISMIC INVESTIGATIONS

5.1 INTRODUCTION

The Structural Geology of the North Wall of Mt Whaleback has been described in detail in Chapter 4. However, the conclusions reached in that Chapter would not have been possible without the information obtained from the Cross Hole Seismic tomography investigations.

It has been emphasised in Chapters 3 and 4 that most of the structure of the North Wall is hidden from view, and therefore there were only two sources of information available. These were:

- information available from face mapping which was very limited, or
- information available from drill holes which was obtained at a high cost. There were difficulties associated with obtaining more of this information due to waste dumps located on the crest of the North Wall. Drilling through these waste dumps is not feasible at reasonable cost.

Information from these two sources indicated that there might be large scale folding in the Jeerinah Formation north of the Whaleback Fault zone. This large scale folding could not be confirmed by drilling without a significant increase in the size of the drilling program and of the budget which was unacceptable. This folding could not be confirmed by other techniques such as surface seismics because of the large waste dumps located on top of the Jeerinah Formation. Nevertheless the presence or absence of large scale folding in the Jeerinah Formation was of fundamental importance to the overall stability of the North Wall. Hence there was strong incentive to find an alternative method to confirm the geological structure.

BHP Central Research Laboratories had been refining seismic techniques for some years and had already conducted cross hole seismic tomography investigations at a prospective lead-zinc mine at Blendervale (Ref 5.3) in the Kimberley region of Western Australia and had developed software and expertise over a number of years in seismic tomographic techniques (Refs 5.1, 5.2, 5.4). Therefore it was logical that cross hole seismic tomography be used on the North Wall of Mt Newman in order to confirm the structural geological interpretations. This was the first time that this technique had been used in a large scale operating pit in Australia. It may have been used in underground mines in some other countries, but to the writer's knowledge it has not been applied in surface mining situations elsewhere.

5.2 METHOD

Cross hole seismic tomography involves generating a signal in an exploration borehole, and receiving the signals with a detector in another borehole. This is repeated many times so that a tomgraphic image can be created to provide a 2-dimensional image of the structures between the boreholes. Tomography is the method whereby the area between the two boreholes is divided into a large number of cells and a seismic velocity is calculated for each cell by means of iterative techniques. The size of the cells determines the resolution and this in turn is dependent upon the number of shots and detectors used.

In principle, cross hole tomography can be applied to a wide range of minerals, provided that there is sufficient contrast of properties, such as velocity and attenuation, in the structures.

In general, the application of cross hole seismic surveys should enable mineralisation and/or overburden boundaries to be determined with fewer boreholes, thus reducing the costs of exploration and mine planning. Other information, such as the location of fractured zones and highly stressed areas, may also be obtained.

5.3 FIELD PROCEDURE

The area under investigation is shown in Figure 5.1. Eight boreholes in the vicinity of the proposed northern pit limit were selected, as listed in Table 5.1. These boreholes were used to form eight hole pairs, as shown on the Figure 5.1.

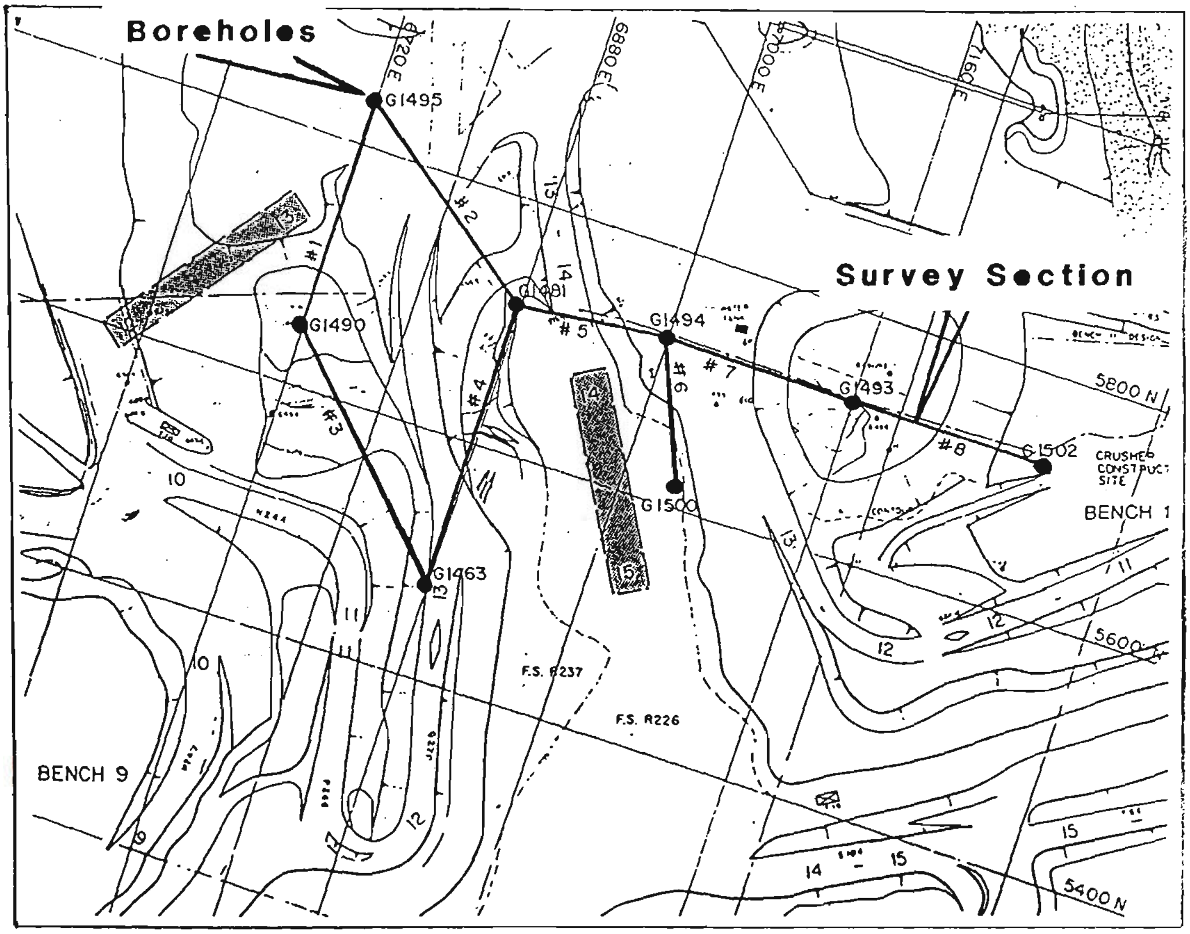


Figure 5.1 Location of hole pairs used in the tomographic surveys

The layout of the cross hole seismic survey is shown in Figure 5.2. The source consisted of two small explosive boosters and a seismic detonator, attached to the end of a small steel pipe. These were prepared on the surface and lowered to various depths down one borehole.

A detector string was lowered into the other borehole utilising ground water for coupling. A data acquisition system developed at CRL was used for recording.

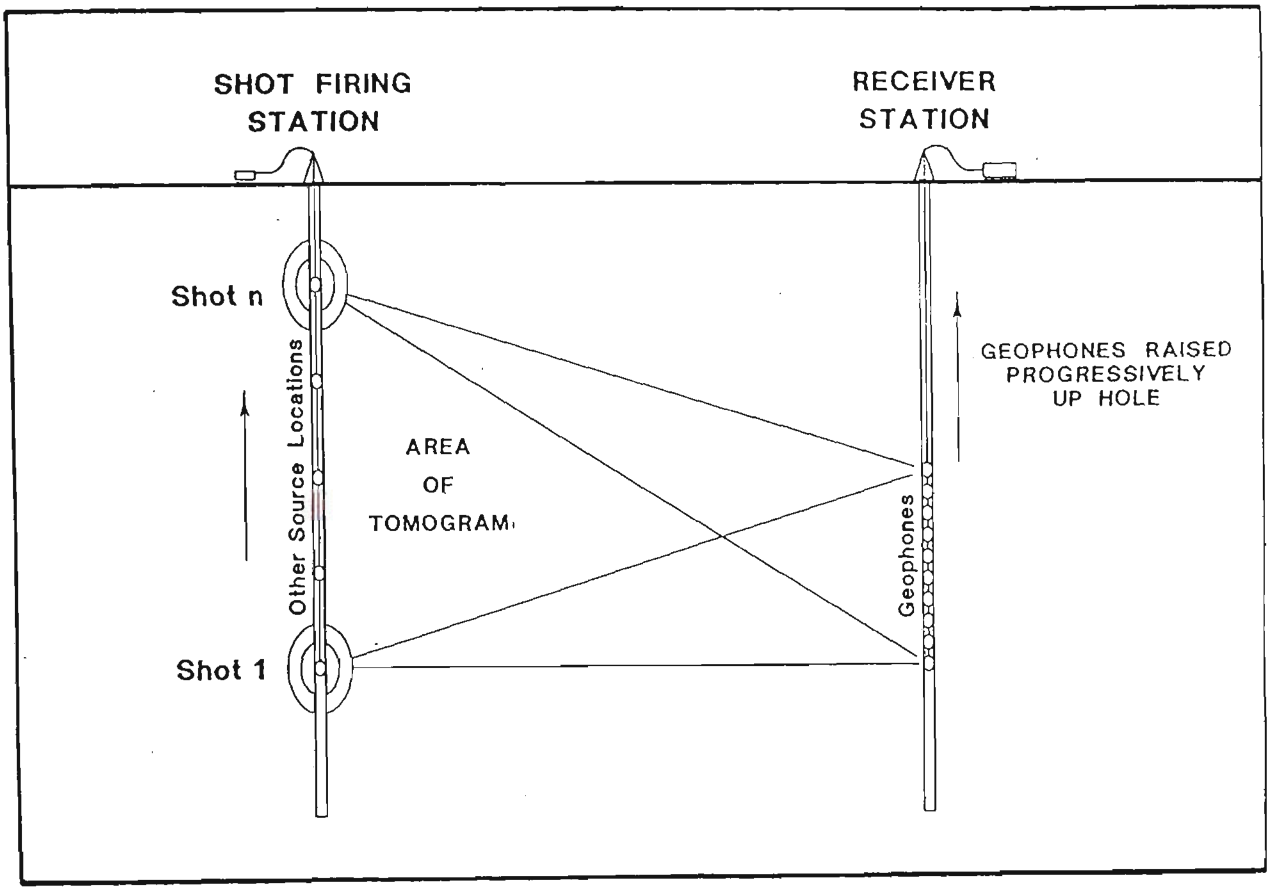


Figure 5.2 General field layout for tomographic surveys

Personnel requirements included two technical personnel and a shot-firer provided by Mt Newman Mining Co.

During the two week survey period a total of 400 explosive shots were fired, as shown in Table 5.2. Turn-around time (time between shots) ranged from four minutes for shallow shots (around 80m) to ten minutes for deep (300m) shots.

All shots were fired below the water table. Attempts to raise the water level (and hence increase the coverage of the survey) in the boreholes by filling with water were not successful as the holes drained within minutes of filling. All holes remained open at the completion of the survey.

5.4 DATA PROCESSING TECHNIQUES

Data processing for the cross hole seismic survey involved mainly tomographic imaging of transmission data in a cross-section bounded by the boreholes.

Tomography is the process of determining the extent of anomalies within a body from projections made through the body. For geophysical applications, tomographic imaging enables anomalies of physical properties, such as velocity and attenuation of seismic wave propagation, to be mapped onto a 2–dimensional section. P–wave velocity has been used in this present tomography work.

The mathematical algorithms for tomographic imaging have been described in Reference 5.4. In brief, the section of interest is represented by a 2–dimensional grid of small cells, as shown in Figure 5.3. Each cell has a uniform velocity distribution within its boundaries, and changes between cells are stepwise. The time taken for the seismic wave to travel from the source to the detector is equal to the sum of the time taken for the wave to travel through each cell. The time taken in each cell is obtained from the known distance divided by the velocity of that cell. By iteration, the velocity in each cell is adjusted such that the difference between the measured time and the calculated time is minimal.

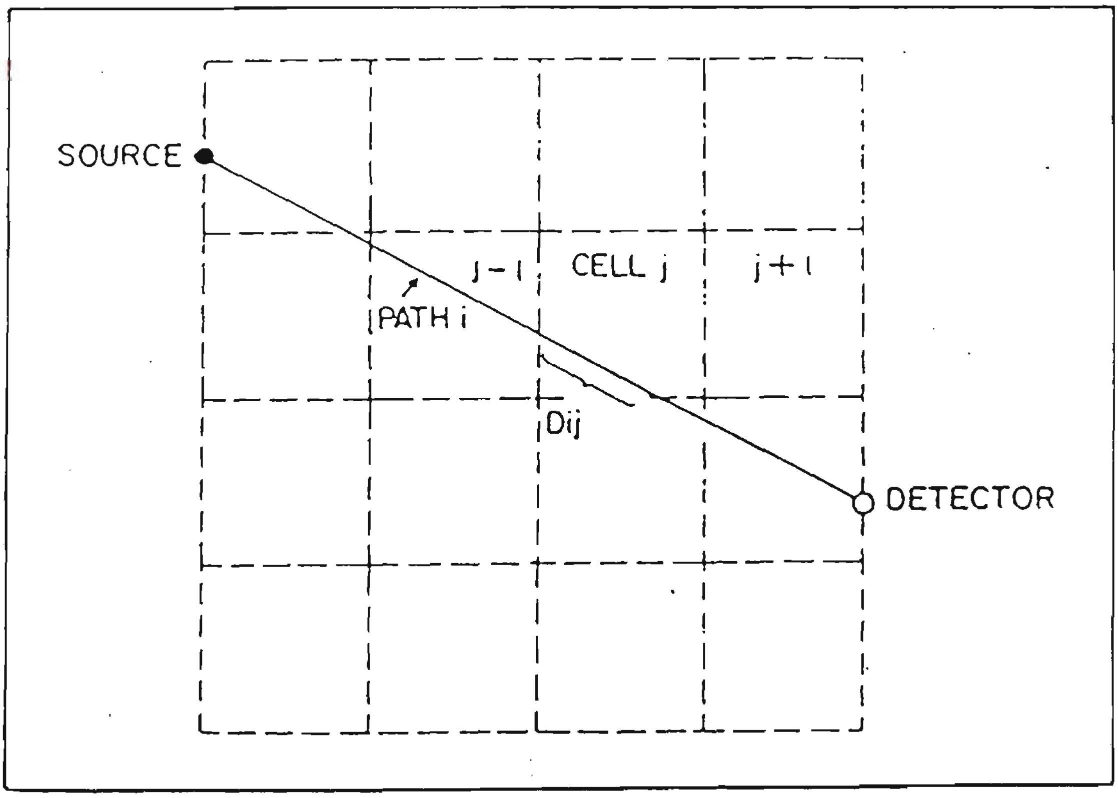


Figure 5.3 Cells and transmission paths for tomographic imaging

An important aspect of tomographic imaging with velocity is the determination of the arrival time of the seismic wave. This was achieved firstly by computer techniques (Ref 5.5) followed by visual verification and editing on a graphics terminal.

5.5 PROCESSING RESULTS

The raw data of a typical shot and the frequency spectrum of one of the traces are shown in Figures 5.4 and 5.5 respectively. The frequency content of the data ranges from approximately 50 hz to 1000 Hz.

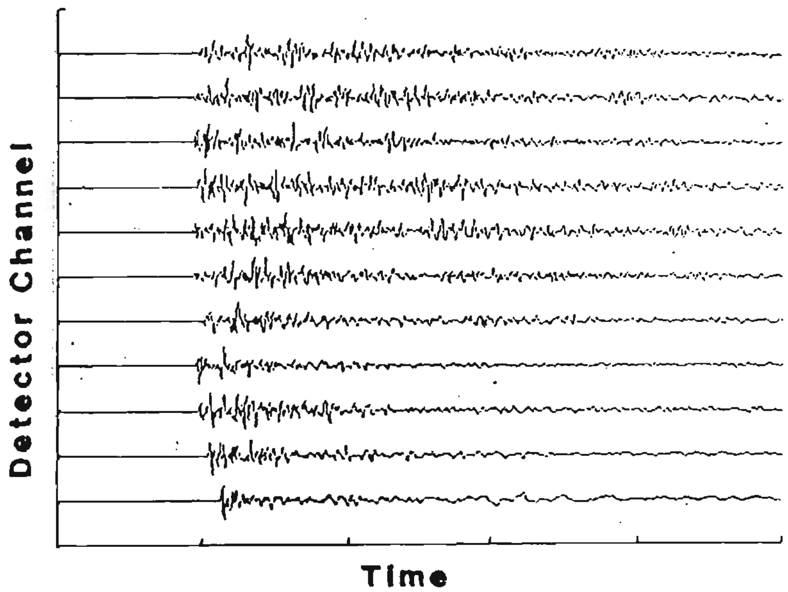


Figure 5.4 Typical raw data

The overall dynamic range of the data is approximately 70 dB, indicating that the data are of only moderate quality. This has been due to large background noise generated from mining activities in the area. This has caused some difficulties in determining the arrival time of the seismic signals using the automatic picking software.

The tomographic imaging results (tomograms) for two adjacent hole pairs, Nos 1 and 3, are shown in Figures 5.6 and 5.8, respectively. Interpreted shale/dolerite contacts are also superimposed, with low velocity regions corresponding to shale, and high velocity regions corresponding to dolerite.

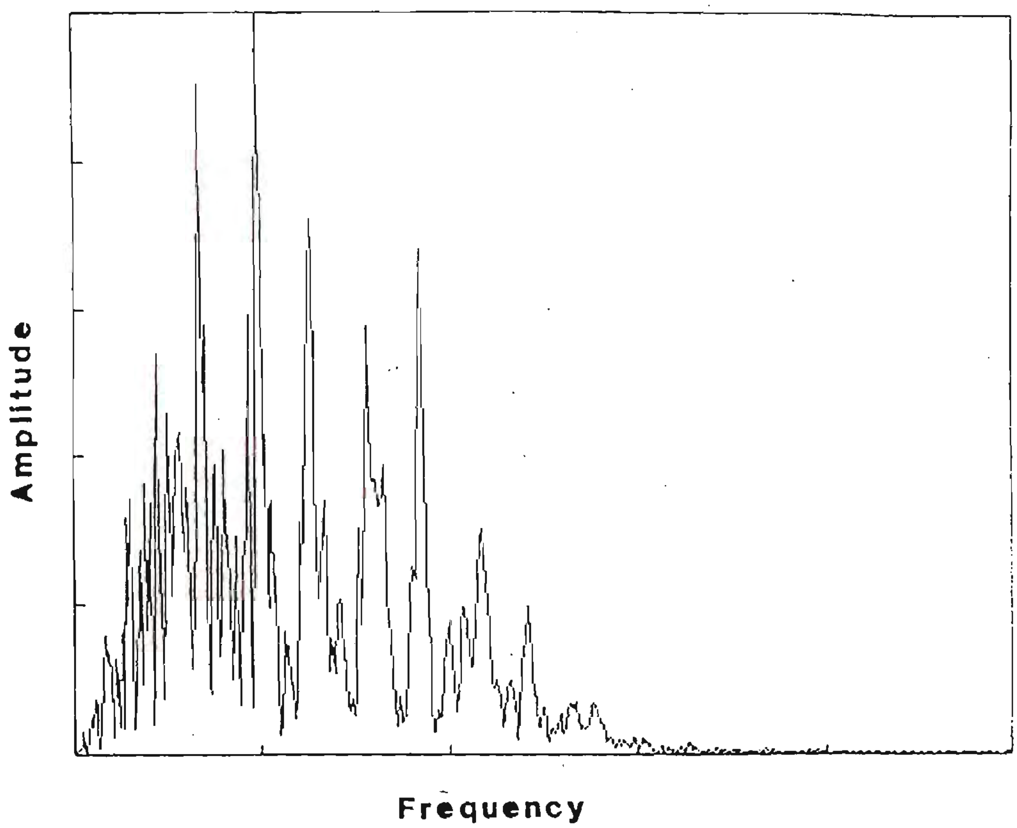


Figure 5.5 Typical frequency spectrum

It should be noted that the velocity values shown in Figures 5.6 and 5.7 are relative only. Absolute velocity was not considered due to information bias necessary to reduce image noise, possible depth measurements errors, anisotropy and ray path refraction.

Overall results for the eight hole pairs are given in Figures 5.6 to 5.13, inclusive. In these figures the section considered is a vertical plane containing the collars of the two boreholes. Interpreted dolerite/shale contacts are drawn in dotted lines. It should be noted that, as shots and detectors were located below the water table, contacts very close to the water table could not be interpreted with confidence, and were thus not shown in some cases.

A high velocity region is noted within the shale region in Hole G1481 on all the sections utilising this borehole (Figures 5.7, 5.9 and 5.10), which was subsequently shown to be indicative of an area of varying mineralogy. This specific high velocity zone was caused by a change from a chloritic shale to a sericitic shale (see Chapter 4).

A high velocity region is also noted within the shale region in Hole G1493 (Figures 5.12 and 5.13) which is again due to sericitic shale. However, compared with that in Hole G1481 this area is relatively small.

5.6 DISCUSSION AND CONCLUSIONS

Cross hole seismic tomography survey undertaken at the North Wall of Mt Whaleback was the first tomography survey conducted for an open pit mine in Australia. Inevitably there were some minor technical problems but the survey was conducted efficiently, with no disturbance to mining activities. An average of one hole pair per day was surveyed and a total of eight pairs were surveyed in the complete survey.

The following conclusions can be made about the data and the results obtained from this survey.

- The field data was only of moderate quality due to the large background noise associated with adjacent mining activity. This noise is mainly from haulpaks in the immediate area although shovels and conveyors must also contribute to this noise.

- Despite this noise, tomographic imaging was possible from the majority of the data.

- The shale/dolerite contacts can be defined by the tomographic images. Where the nature of the dolerite intrusion is gradational, the tomographic images produced are also gradational.

- Detailed structural and lithological interpretations can be significantly enhanced by combining all the available tomographic, borehole, gamma and assay data on each cross-section.

- This present tomography work was limited to areas below the water table (approximately RL575) since water is used as a coupling fluid to the rock. However this did not prevent information being obtained from the area of interest.

- The cross hole seismic tomography work confirmed that there was large scale folding in the Jeerinah Formation at depth (down to 300m) and that ths folding was complex in three dimensions (see modelling section at the end of Chapter 4).

- A summary plot of the tomography results for a typical section plotted in its true spatial position with borehole gamma log data and assay data, is shown in Plate 5.14. The folded structures in the Jeerinah Formation are evident from the tomogram which shows the Shale A between the underlying Dolerite A and the overlying Dolerite B. This picture graphically demonstrates the power of this technique in locating geological structures.

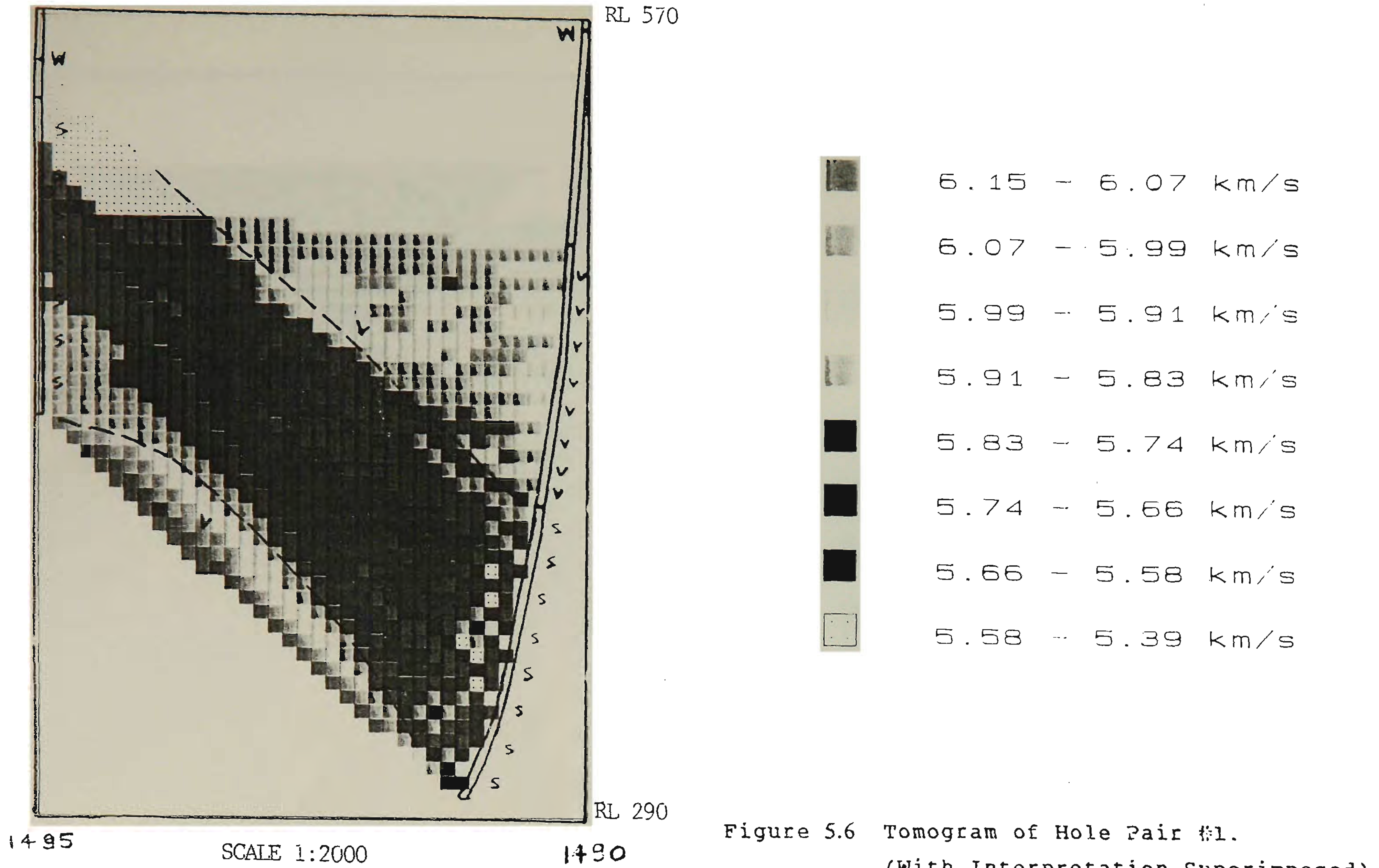


Figure 5.6 Tomogram of Hole Pair #1.
(With Interpretation Superimposed)

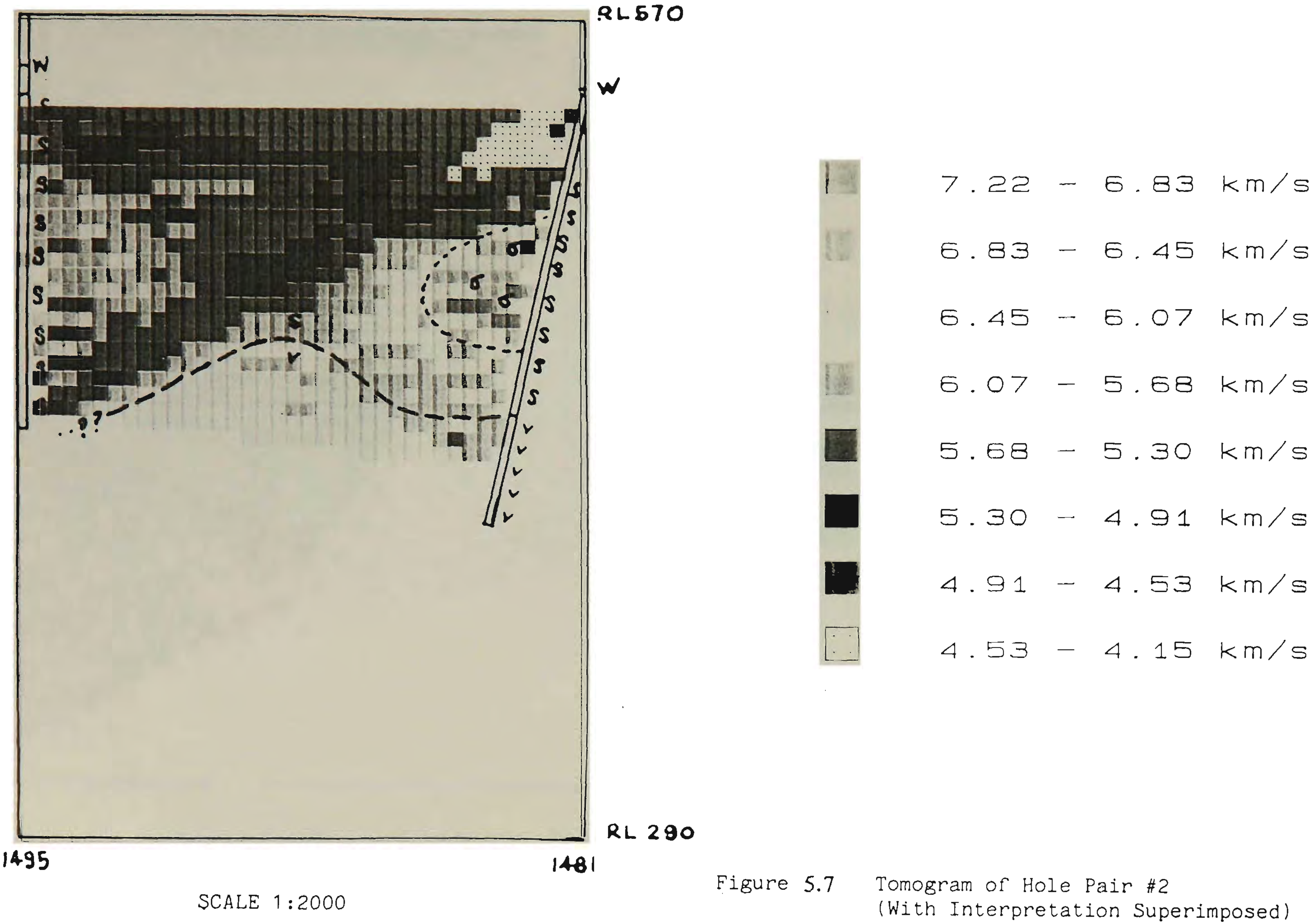


Figure 5.7 Tomogram of Hole Pair #2 (With Interpretation Superimposed)

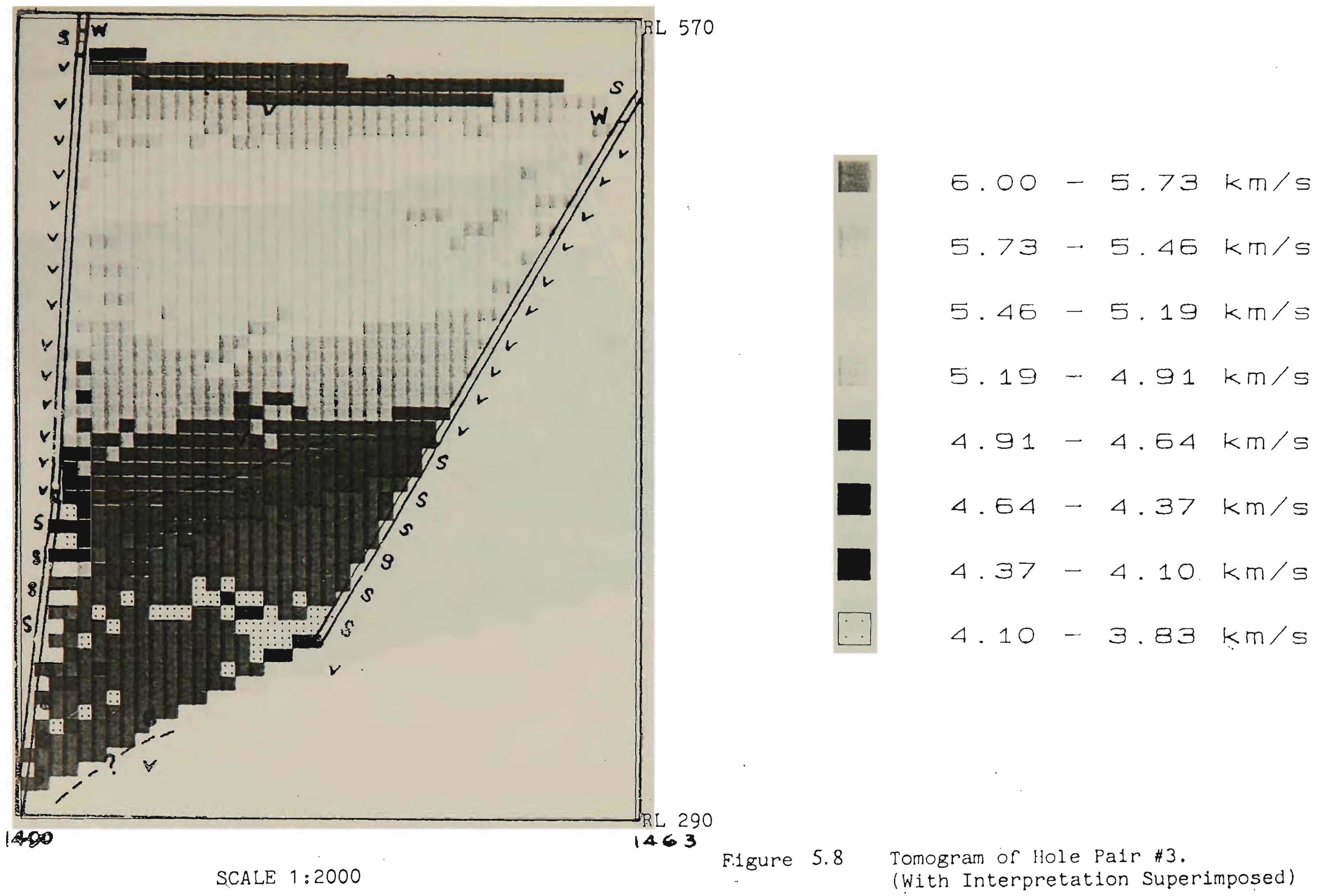
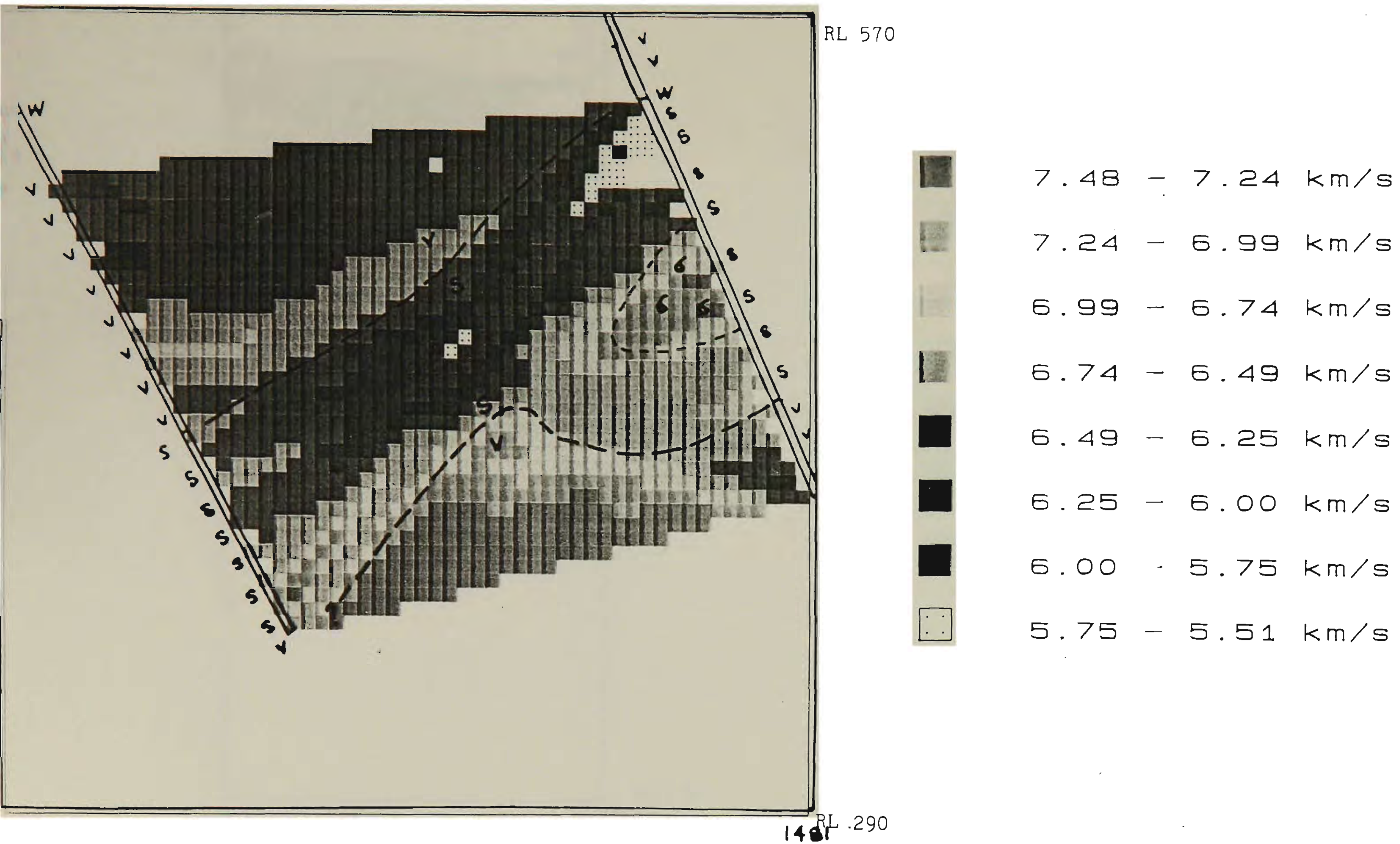
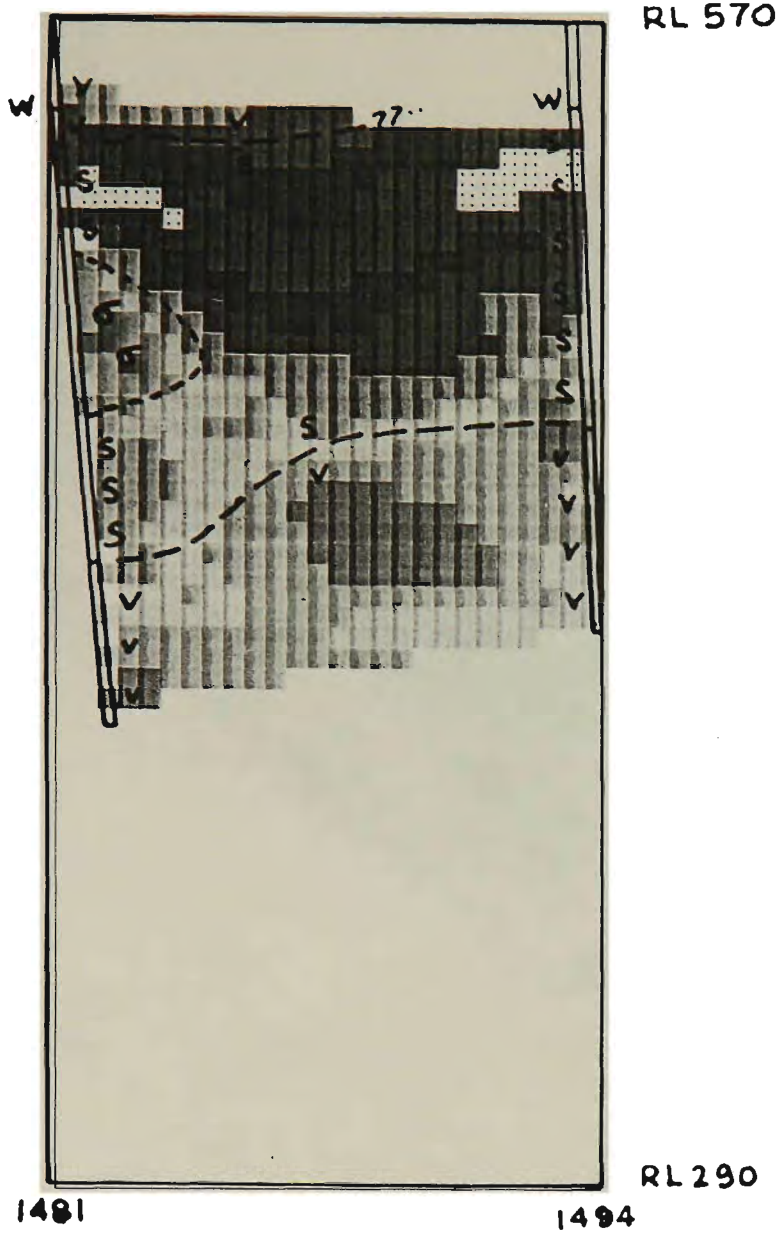


Figure 5.8 Tomogram of Hole Pair #3. (With Interpretation Superimposed)



SCALE 1:2000

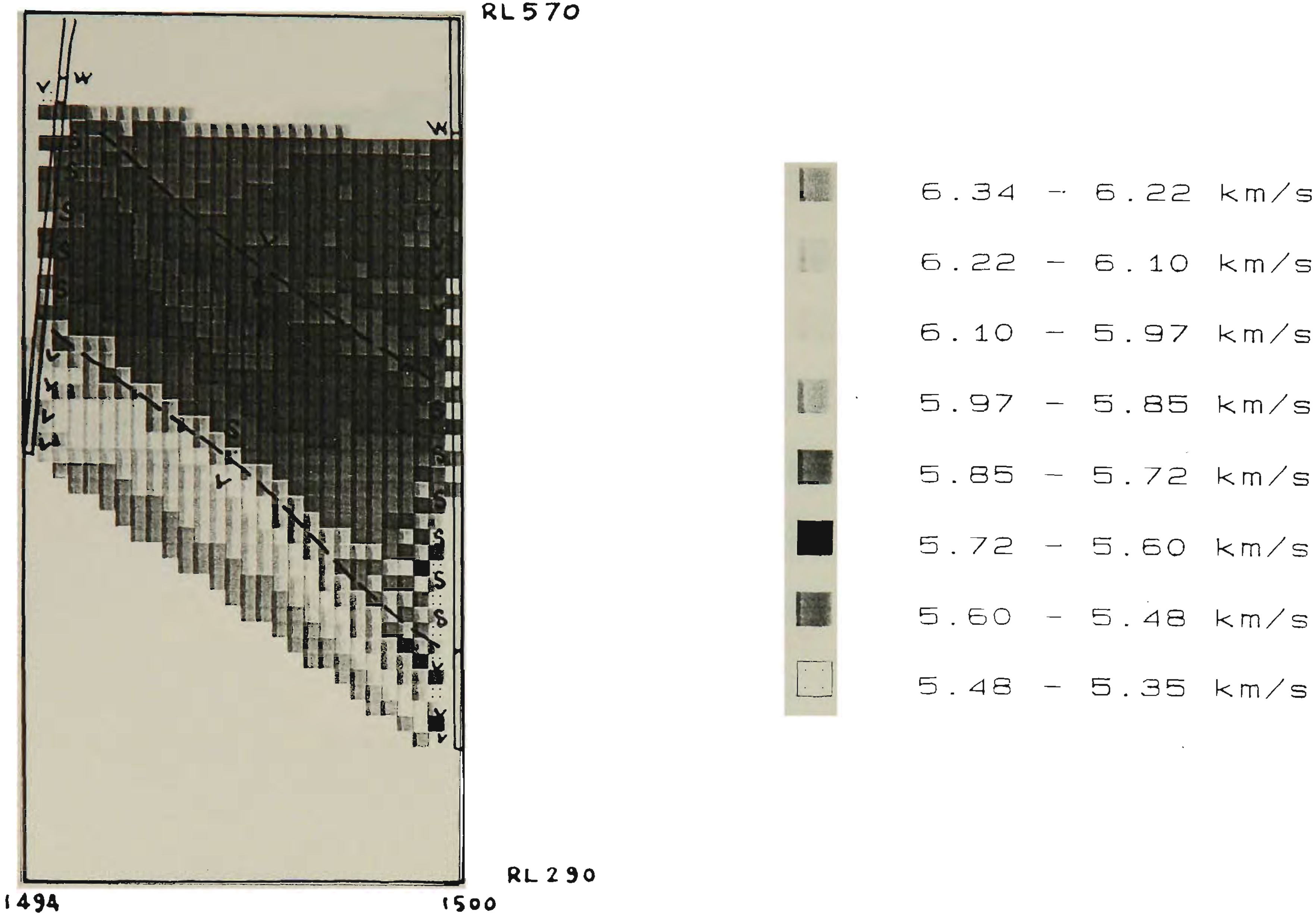
Figure 5.9 Tomogram of Hole Pair #4 (With Interpretation Superimposed



SCALE 1:2000

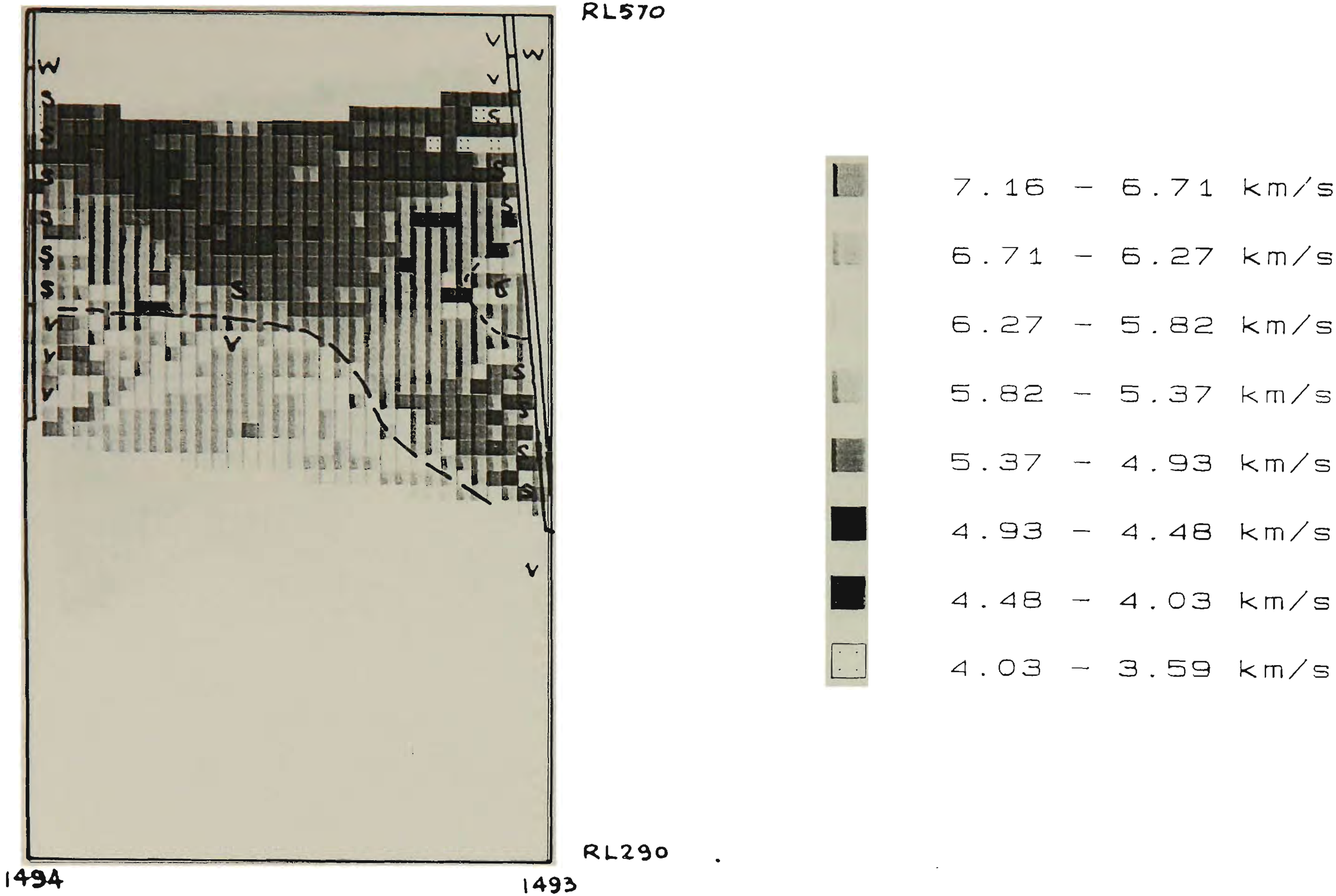
6.69 – 6.34 km/s
6.34 – 5.99 km/s
5.99 – 5.65 km/s
5.65 – 5.30 km/s
5.30 – 4.96 km/s
4.96 – 4.61 km/s
4.61 – 4.26 km/s
4.26 – 3.92 km/s

Figure 5.10 Tomogram of Hole Pair #5 (With Interpretation Superimposed)



SCALE 1:2000

Figure 5.11 Tomogram of Hole Pair #6 (With Interpretation Superimposed)



.SCALE 1:2000

Figure 5.12 Tomogram of Hole Pair #7
(With Interpretation Superimposed)

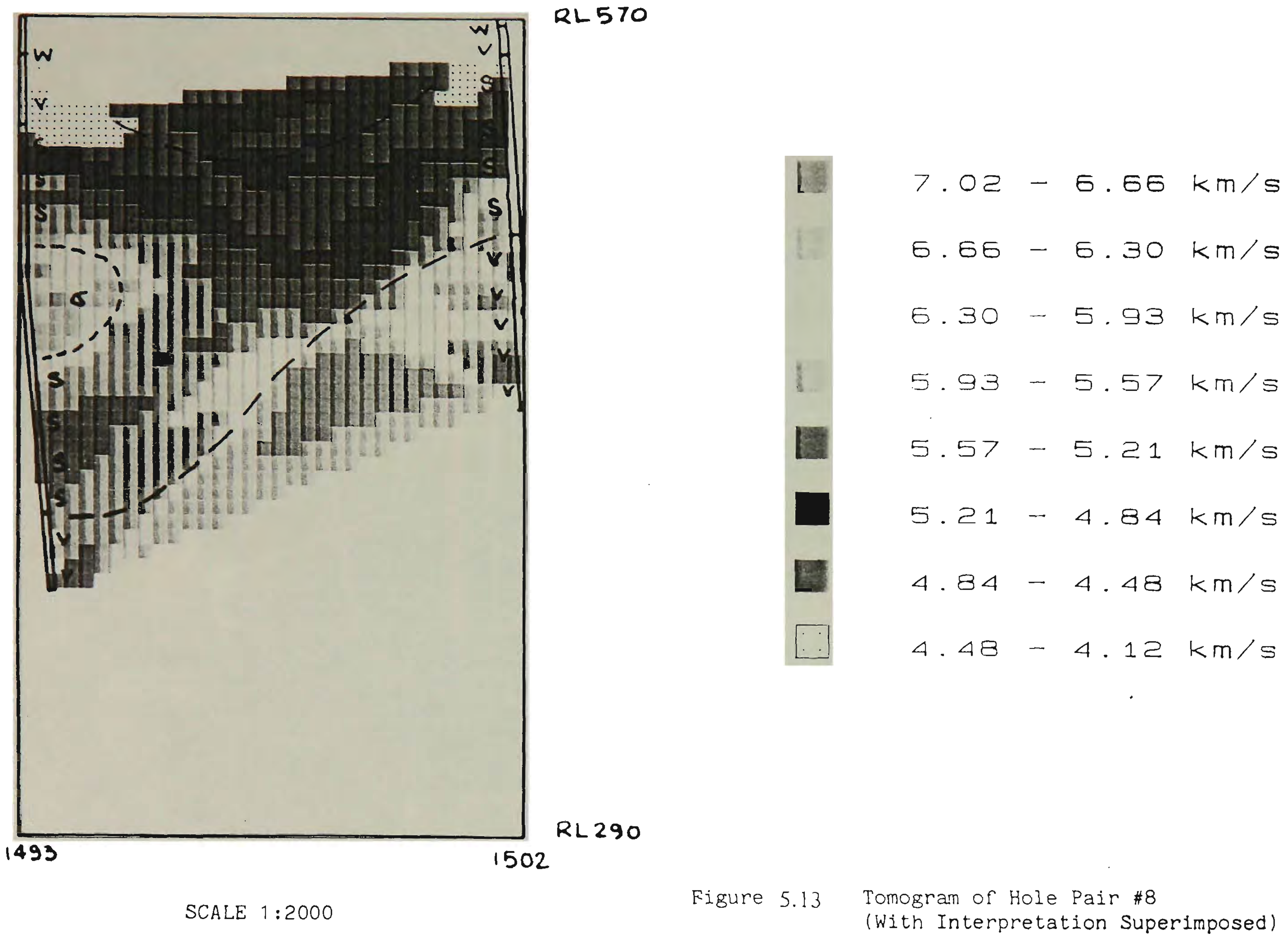


Figure 5.13 Tomogram of Hole Pair #8
(With Interpretation Superimposed)

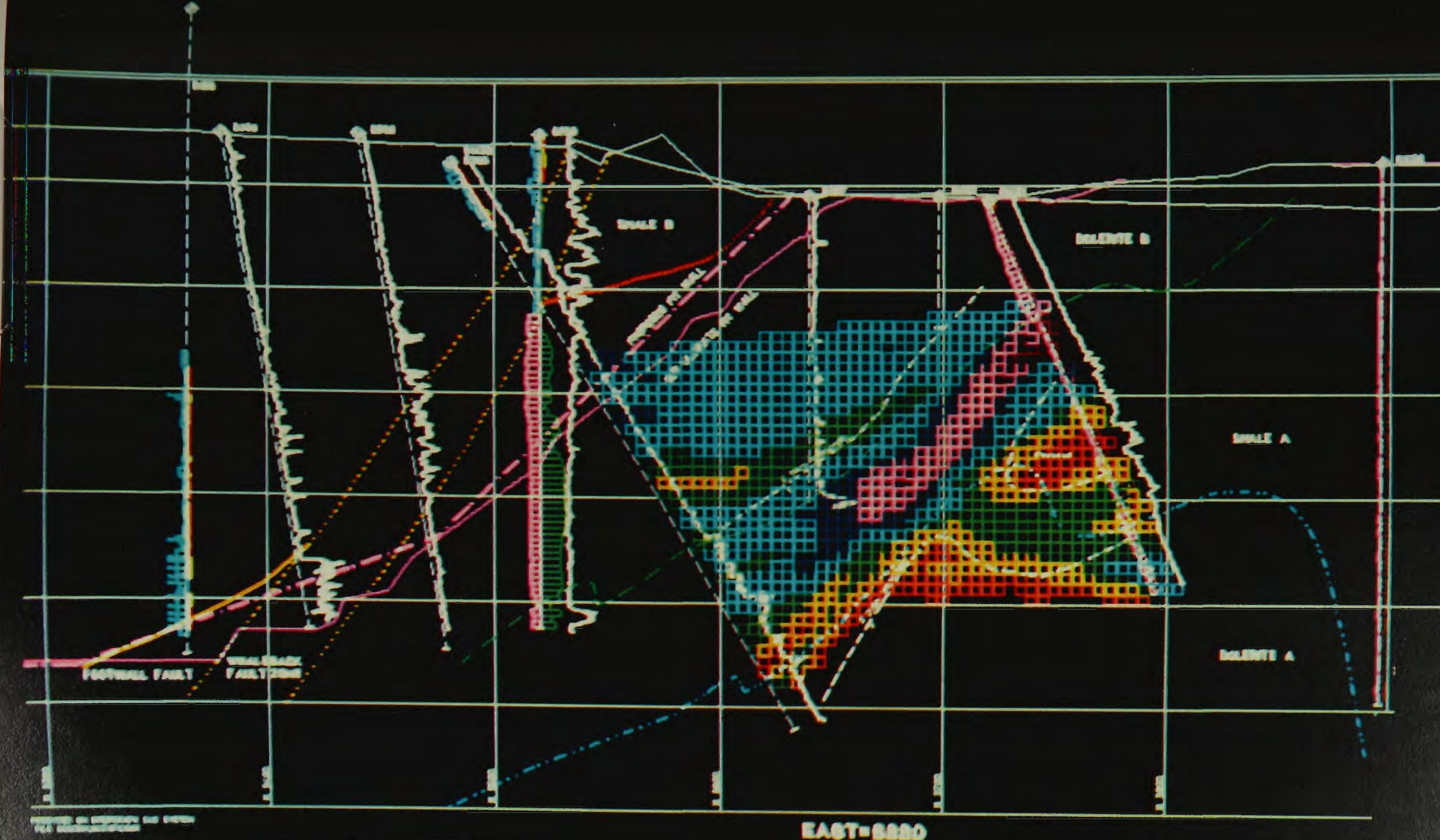


Plate 5.14. A summary plot of the tomography results for a typical section with borehole gamma log data and assay data

CHAPTER 6

THE RELEVANCE OF ENGINEERING GEOLOGY TO SLOPE STABILITY ON MT WHALEBACK

6.1 INTRODUCTION

The research into the geotechnical parameters which influence the design of a major pit slope has necessitated the detailed examination of the engineering geology of the North Wall of the Mt Whaleback pit. The relevant engineering geology parameters have been studied in detail and have been determined by field mapping of all pit face exposures and costeans as well as an examination and re-logging of borecore samples.

This Chapter forms a crucial link in the thesis and progresses naturally on from the Structural Geology (Chapter 4) which in turn naturally progresses onto the Physical Properties (Chapter 8) and Stability Analysis (Chapter 10) and Economic Assessment (Chapter 11).

Much innovative work has also been undertaken as part of the work presented in this Chapter. The analysis of discontinuities have been undertaken in detail using a computer package called DCONB and a summary of the results is presented in the figures. The investigations have also been widespread and have not solely been restricted to the North Wall of Mt Whaleback. The Jeerinah Formation has been examined in the extreme north-east of the study area (at least 1km north of the pit and beyond the area of waste dumping) where costeans were excavated specifically for this purpose. Also a flow unit in dolerite was examined in the Marra Mamba area 4km to the south-west of Mt Whaleback in order to determine discontinuity patterns away from the Whaleback Fault Zone. It should be pointed out that suitable exposures of the Jeerinah Formation are extremely limited on the North Wall itself and hence the need for this comprehensive investigation.

The objective of this Chapter was to determine the rock mass characteristics of the North Wall rocks both adjacent to and remote from the WFZ in order to predict rock behaviour and rock mass shear strength.

6.2 DATA COLLECTION AND ANALYSIS

The stratigraphy has been described in detail in Chapter 2 and the discontinuity measurements have been undertaken not only on the Jeerinah Formation north of the WFZ, but also on the Hamersley Group (ie. the Mt Sylvia Formation, the Mt McRae Shale and the Brockman Formation) to the south of the WFZ.

As these discontinuity measurements were being undertaken, it was evident that the rocks close to the WFZ were disturbed and so the discontinuity measurements have also been designed to locate the extent of this disturbed zone.

6.2.1 Location of Exposures for Discontinuity Measurements

Exposures of Jeerinah Formation rocks within the East Pit suitable for discontinuity surveys are very limited and mainly occur to the east of 7600E and are largely located in the Jeerinah Dolerite A (refer to Plates 6.1 and 6.2). Exposures of the Jeerinah Shale are restricted to small outcrops within the 'disturbed' zone, adjacent to the WFZ (Plate 6.1), and are of limited value for assessing the general rock mass characteristics. Surface exposures of Jeerinah Shale A occur to the north of existing waste dumps (approximately 900m north of current pit limits) and costeans have been used for discontinuity surveys. More extensive exposures of the Jeerinah Dolerite A occur within the existing East Pit but these are all located within approximately 50m of the WFZ, and therefore will be influenced by it.

A small exposure of Jeerinah Dolerite is located to the south of Mt Whaleback within a flow unit close to the Jeerinah/Marra Mamba Iron Formation contact (Plate 6.3). Planar jointing is well developed at this location and discontinuity surveys have been conducted for control purposes in assessing the influence of the WFZ upon disturbed exposures within the East Pit.

Discontinuity surveys have also been conducted within the Joffre Member of the Brockman Iron Formation to allow a comparison with the joint systems of the Jeerinah Formation rocks. These surveys have utilised outcrops at varying distances from the WFZ and have enabled its influence upon jointing to be assessed (see Plates 6.5 to 6.8).

The width of the 'disturbed' zone adjacent to the WFZ has been estimated by:

- discontinuity surveys conducted within the East Pit,
- an assessment of diamond drill core for fracture frequency, nature of the discontinuity surface and degree of weathering,
- by the variation in the uniaxial compressive strength of diamond drill core.

During the data collection phase it became clear that variations existed in the discontinuity systems associated with the different fold limbs of large amplitude folds in the Jeerinah Shale. An example of the fold style developed with shallow-dipping, southern limbs and steeply-inclined (locally overturned), northern limbs is shown in Plate 6.9 for a Whaleback Shale sequence to the south of the WFZ. The variations in the discontinuity systems are described in detail in the following sections because of the influence upon the stability of any North Wall design where the Jeerinah Shale is exposed.

6.2.2 Features of Data Surveys

These discontinuity measurements have been made in accordance with guidelines recommended by the International Society of Rock Mechanics (Ref 6.1) and the practices of Mt Newman Mining Company (MNM). Discontinuity surveys of surface exposures using scanlines and window mapping includes a description of the following features:

- discontinuity type
- discontinuity orientation
- discontinuity shape/roughness (small scale roughness less than 100mm)
- infilling type and material
- infill width
- discontinuity termination
- discontinuity trace length or persistence
- discontinuity wavelength and amplitude (medium scale roughness 0.1 - 10m)
- physical condition including weathering.

Similar information is recorded on the structure logs of diamond drill cores with the exception of discontinuity termination, trace length and only small scale roughness. Values for discontinuity spacing have been calculated for surface scanline surveys, whereas fracture frequency/rock quality designation (RQD) have been calculated for drill core structural logs.

The detailed discontinuity surveys have been conducted by using relatively long base lines (eg 20 - 50m) at selected locations with the discontinuity orientation data being assessed in detail for each location.

This method of data measurement has been chosen because of the limited number of suitable exposures of the Jeerinah Formation, particularly the Jeerinah Shale units, away from the WFZ and the requirement of assessing the influence of the WFZ upon the discontinuity systems of different geological formations with significant lithological and rock strength variation.

It was also important to check for the presence of disturbed, weathered zones of varying width particularly within the Jeerinah Formation rocks and the above method was ideally suited to this.

The data collected as part of this work has resulted in approximately 700 data sets being collected from five discontinuity surveys in the Joffre Member, 750 data sets from six surveys in the Jeerinah Dolerite A and 500 data sets from three surveys in the Jeerinah Shale A. There were no exposures of the other Jeerinah Formation units and, therefore, no discontinuity measurements were taken on them. The location of the individual surveys within the East Pit is shown in the maps presented elsewhere (Ref 4.3).

This discontinuity data has then been processed using a computer package called DCONB which produces output in the following forms:

- stereoplots (both pole plots and contoured plots) of discontinuity orientation allowing 'clusters' or discontinuity sets to be identified.

- discontinuity cluster analysis in terms of a mean pole orientation, Fisher's Constant (a measure of the clustering) and the variation about the mean pole determined by Eigen vector analysis.

- frequency histograms of discontinuity spacing, persistence, shape/roughness and termination for each cluster or set identified.

The discontinuity parameters that have a significant influence upon slope stability and wall design considerations are discussed in the following sections.

6.3 DISCONTINUITY ORIENTATION

A summary of the discontinuity data for each of the main stratigraphic units (Jeerinah Dolerite, Jeerinah Shale and Joffre Member) is presented on Figures 6.1 to 6.3 respectively. A summary of the discontinuity sets present on individual surveys is included in Table 6.1.

A comparison of joint set orientation in Table 6.1 reveals that only one set (Set D), with an approximate north-south orientation, is common to all stratigraphic units surveyed (Jeerinah Dolerite A, Jeerinah Shale A and Joffre Member).

Each stratigraphic unit appears to be characterised by a distinct joint system such that each of the three units can be considered as a major structural domain with a strong lithological, stratigraphical and structural influence evident. The discontinuity surveys conducted in the vicinity of the WFZ have identified a modifying influence upon the joint systems in both the Joffre Member and Jeerinah Dolerite such that further sub-domains could be defined, if necessary, as more exposures become available. (For example, the underlined discontinuity measurements in Table 6.1 have been influenced by the WFZ.)

The mean pole orientation values for the sets identified within each geological unit, are summarised in Table 6.2 together with the calculated values of Fisher's Constant K, which provides a measure of the clustering of discontinuity sets.

If the discontinuity sets due to the WFZ are omitted (Set H), three dominant joint sets (Sets B, C and D) can be defined in the Joffre Member. Only two dominant sets (D and I) are well defined in the Jeerinah Dolerite A within the East Pit, although two others (Sets E and F) may be developed beyond the influence of the WFZ. The limited discontinuity surveys conducted in the Jeerinah Shale A suggest that two structural domains (fold related) can be distinguished. These are described in more detail in Section 6.3.2

The joint systems for the three stratigraphic units are presented graphically on Figures 6.4 to 6.6 respectively, together with relevant bedding and cleavage information. A comparison of these figures indicates that the joint systems for the Jeerinah Dolerite and Shale, in particular, are strongly related to the principal stress direction. This is discussed further in the following sections.

6.3.1 Jeerinah Dolerite A

A summary of discontinuities in the Jeerinah Dolerite A is given in Tables 6.1 and 6.2 and indicates that joints are characterised by well ordered jointing with well developed sets trending north–west to south–east (Set I) and north–south (Set D).

The relationship between these sets and the less well developed Sets E and F, suggests that Set I formed parallel to the fold axis direction, Set E normal to the fold axes, with Sets D and F representing a conjugate set of shear joints (refer to Figures 6.4 and 6.7). This joint system indicates a fold axis direction of 315° which is discussed further in Section 6.3.4.

All four joint Sets (D, E, F and I) are present in a flow unit at the top of the Jeerinah Formation which is exposed close to the Marra Mamba contact, south of Mt Whaleback (Figure 6.8 and Plate 6.3). In the North Wall pit exposures Sets E and F are generally poorly defined due to the influence of the Whaleback Fault. The steep dip associated with all of the joint sets identified in the Jeerinah Dolerite is consistent with a gently dipping limb present in the North Wall to the east of approximately 7000E.

6.3.2 Jeerinah Shale A

One of the major features of the Jeerinah Shale A is that it is characterised by an axial plane slaty cleavage having two structural domains defined by folding. Shallow, south–dipping limbs are characterised by up to four dominant joint sets (A, D, G, I) and a minor joint set (B). In addition bedding is generally partially preserved although the axial–plane slaty cleavage is well developed and dominant.

Steeply dipping northern limbs, however, are characterised by only two well defined joint sets (A and G). The north–south joint set (D) is poorly represented in contrast to all other survey locations. Bedding is generally poorly preserved and both an axial–plane slaty cleavage (S1) and a later developed crenulation cleavage (S2) are present. The S1 cleavage dips to the WSW at approximately 30° and the S2 cleavage dips at 50–60° to the SW. These relationships are demonstrated further by means of schematic drawings (Figures 6.9 and 6.10 and Plate 6.11) with a summary stereoplot of cleavage data presented on Figure 6.11.

6.3.3 Joffre Member

Table 6.1 shows that the discontinuity system in the Joffre Member differs markedly from that in the Jeerinah Formation with only the dominant Joint Set D common to both units. A joint set parallel to the fold axis direction of 250–270° is absent with a normal set (Set B) of limited occurrence. An axial–plane cleavage was only found to be well developed on one scanline situated in a synclinal keel. The joint system is generally not well defined and the relationship to folding is indistinct (refer to Figures 6.6 and 6.12). This is probably due to the variation in minor fold axes described in Chapter 4 with a range of 230–300°. A range of 253–280° was measured over the survey areas contained in Table 6.1 and this is shown on Figure 6.6.

Low angle faulting was found to occur in several of the mapping areas and is clearly related to the fold axial–planes with a dip of 40° (see Plate 6.7). Faults with a flatter dip are evident on the steeply–dipping fold limbs (see Plates 6.5 and 6.6) where quartz veins up to 10mm thick occur. These faults appear to be quite extensive in Plate 6.6 but with dip angles of less than 25° they have had little effect upon the stability of the interim pit walls. The formation of these flat–dipping faults is discussed in Chapter 4.

6.3.4 Summary of Fold–Related Discontinuities

The relationship between systematic jointing and folding is well documented (Ref 6.2, 6.6) although different joint genesis models have been proposed. No single hypothesis can account for the formation of joints in different geological environments, and therefore different joint formation mechanisms probably apply in different cases. The most common of these hypotheses consider that joints are shear or tensional fractures resulting from local or regional compression, tension, torsion or some combination of these forces (Ref 6.6). Other papers were also examined but found to have no direct relevance to the subject matter of this section (Ref 6.7, 6.8, 6.9, 6.10 and 6.11).

The forces associated with folding are often considered to be responsible for joint formation. Where these forces are compressional, two sets of joints are often formed which intersect at an acute angle and which face the direction of the compressive force. However, if the forces are tensile, then two sets of joints at right angles to each other are generally formed, one set oriented down dip and the other along the strike of the fold (Ref 6.6).

This joint genesis model applies to many folding related joints, but again the model considers only two joint sets and exceptions to this model can be found. Therefore, it cannot be considered to be representative of all field situations.

The theories for joint formation include the following:

- **SHEAR THEORY** which states that failure will occur at some point where the maximum shear stress exceeds the shear strength of the material and the fractures formed will form an acute angle facing the direction of the maximum principal compressive stress. However this assumes that only two joint sets would form in the rock mass.

- **TENSION THEORY** which states that failure will occur along a plane at right angles to the direction of the maximum tensile stress. This theory has the disadvantage that it assumes that the direction of tensile stress must vary in order to accommodate different joint sets in the same rock mass.

- **TORSION THEORY** this assumes that two sets of fractures would form at right angles to each other as a result of a torsional stress system. Such orientations of joint sets are difficult to choose in practice.

There are other hypotheses for joint formation including fatigue and stress relief but these are also related to a change in the stresses acting on the rock mass. In practice, most rock masses exhibit joints formed as a result of one or more of the above mechanisms.

For the Jeerinah Formation at Mt Whaleback, the joint sets do correspond to the conventional theories of joint genesis. These joints are formed either by shear or tensile forces.

The Jeerinah Dolerite exposures, both within the East Pit and to the south of Mt Whaleback exhibit a joint system developed in response to folding as described above (refer to Figure 6.4). Tension joints (Sets E and I) form approximately parallel to the two principal stress directions and hence should be characterised by rough, undulating surfaces. A conjugate set of shear joints (Sets D and F) form obliquely to the principal stress direction and frequently have planar surfaces. The term 'shear joints' does not necessarily imply that any movement has occurred across the joint surface but simply describes the relationship to the principal stress direction. Planar jointing within the Dolerite A unit in the East Pit is rarely found to be developed. However, when such jointing has been found, it has been associated with Sets D and F on surveys SW014 and SW016 (refer to Plate 6.4).

The joint system developed within the Jeerinah Dolerite indicates a fold axis direction of 310–315° which is demonstrated by the folds to the north-east of the study area. This suggests that the joint system developed in the Jeerinah Dolerite is related to an early fold phase.

The joint system within the Jeerinah Shale is not as well defined (refer to Figure 6.5) with the shear joint sets only poorly developed. However, the fold axis direction indicated by the joint system of 290–305° compares favourably with the S1 cleavage developed on both fold limbs within the Jeerinah Shale. This suggests that the joint system developed in the Jeerinah Shale has been influenced by a later fold phase. Alternatively, it could be a consequence of the rotation of the major fold limbs in relation to the principal stress direction.

The relationships discussed above are summarised in Table 6.4 and compared with those for the Jeerinah Dolerite and Joffre Member. These aspects have also been described in Chapter 4.

The joint system present in the Joffre Member is not as well defined as those in the Jeerinah Dolerite and Jeerinah Shale and its relationship to folding is inferred from the presence of an axial-plane cleavage.

The indicated fold axis direction of 250–280° is confirmed by more extensive mapping summarised in Chapter 4 and suggests a further fold phase or rotation of major fold limbs. The major fold axes in the Jeerinah Formation to the north-west of the study area also have an east-west trend. There is a fold limb rotation, from north-west to south-east in the eastern part of the study area, to an orientation east-west in the western part of the study area (west of 7500E). The effect of this on the discontinuity system of the Jeerinah Dolerite and Shale, needs to be further investigated as suitable exposures become available.

6.3.5 Whaleback Fault Zone (WFZ)

The Whaleback Fault is a major shear zone with a WSW–ENE strike and a dip to the south of about 60–75° which is intersected by the eastern section of the present North Wall between approximately 7000E and 8500E. Nearly all of the discontinuity mapping in the Jeerinah Dolerite A and to a lesser extent the Joffre Member, has been influenced by the Whaleback Fault in the following ways:

- a modification of the orientation of existing discontinuity sets (ie. bedding, jointing);
- the formation of additional discontinuities, both with and without associated displacement.

6.4 THE DISTURBED ZONES NEXT TO THE WFZ

6.4.1 Disturbed Zone in the Jeerinah Dolerite A

The effect of the WFZ on the Jeerinah Dolerite A is shown by the discontinuity mapping on Bench 16, located within approximately 15m from the North Whaleback Fault (Figure 6.13 and Plate 6.13). This mapping indicates a strong effect of the WFZ with a predominance of steep, southerly–dipping jointing. The joint sets that are recognisable to the south of Mt Whaleback are generally poorly defined with the exception of Set D which has a decrease in mean dip of around 25° to 61°. This is consistent with an increased southerly dip of the Dolerite A unit adjacent to the Whaleback Fault produced by drag effects.

The results of discontinuity mapping on Bench 15, which is approximately 25m from the North Whaleback Fault, is shown on Figure 6.14. This reveals a reduced influence due to faulting with a decrease in the importance of southerly–dipping discontinuities relative to northerly–dipping jointing. These discontinuity surveys suggest that the 'disturbed' zone adjacent to the WFZ is approximately 15–20m wide within the Jeerinah Dolerite A unit. Jointing within the 'disturbed' zone is shown schematically on Figure 6.15 and can be compared with Figure 6.7 which is not in the disturbed zone.

A summary of all faulting within the Jeerinah Dolerite A is shown on Figure 6.16. This reveals the presence of southerly–dipping normal faults and a series of northerly–dipping shears, faults and shear joints forming in response to drag folding on the Whaleback Fault (refer to Plate 6.14).

This northerly–dipping faulting should also be characterised by a normal sense of displacement (a normal fault situation). North–south orientated faulting is generally limited, being indicated by a series of shear joints (Figure 6.16). The relationships between jointing, shearing and the WFZ are shown schematically on Figure 6.17.

6.4.2 Disturbed Zone in the Jeerinah Shale A

The effect of the WFZ upon the Jeerinah Shale A has been difficult to assess because of the limited exposures available. The discontinuity sets identified within the 'disturbed' zone of the Shale A are shown on Figure 6.18 and include an axial–plane cleavage and joint set G, both of which dip to the south–southwest to south–southeast at approximately 35–60°.

Bedding may also be present and be co–planar with cleavage in some areas. The dip of these discontinuity sets can be expected to increase, however, wherever drag folding adjacent to the WFZ occurs. The width of any 'disturbed' zone within the Jeerinah Shale and the mean dip of any south–dipping discontinuities has important implications for the stability of individual benches in any pit wall design. Further data collection will therefore be required to assess the variation of discontinuity dip, due to drag folding in the various Jeerinah Shale units as suitable exposures become available.

6.4.3 Disturbed Zone in the Joffre Member

The effect of the WFZ upon the Joffre Member to the south is indicated by Table 6.1. Discontinuity surveys S0005–7 are located between 150–500m from the WFZ and can be considered to be unaffected by it. Survey S0008 (Plate 6.8) is within approximately 60m of the WFZ with joint set C absent and joint set D rotated by 10–15°. Survey S0009 (Plate 6.8) is about 30m from the WFZ with only joint set D remaining of those developed to the west.

In addition, joint set H is present with a similar strike to that of the WFZ and its origin is certainly fault related. Although these data are limited, the width of the 'disturbed' zone adjacent to the WFZ appears to be greater than that indicated for the Jeerinah Dolerite units and is at least 30m.

6.5 DISCONTINUITY SPACING, PERSISTENCE AND TERMINATION

The analyses of joint spacing, persistence and termination, for the Jeerinah Dolerite A, the Jeerinah Shale A and the Joffre Member are summarised in Table 6.3 and are described in detail below.

6.5.1 Spacing Data

The spacing data for joint sets of all units is variable. However it generally has a negative exponential distribution with similar range of mean values of 0.16–0.35m for the Joffre Member and 0.10–0.43m for the Jeerinah Shale A. The increased mean spacing values of 0.40–0.90m for the Jeerinah Dolerite A is consistent with a high strength, relatively isotropic igneous intrusion (Plate 6.12). The spacing of the axial-plane cleavage in the Joffre Member was found to be 0.26m for Survey S0006.

6.5.2 Persistence Data

The persistence data was generally found to have a uniform or log normal distribution over the range of values chosen. Mean values for each joint set analysed are shown in Table 6.3 with large standard deviation values for the Joffre Member and Jeerinah Dolerite A indicative of a large range of individual values. A range of mean values of 0.42–0.82m for the Jeerinah Shale A may be due in part to the size of exposures available for mapping with costeans being limited to a vertical height of 2m. The termination data indicate that only Sets A and G could be affected, however, because of the proportion of individual joints extending beyond both exposure limits is limited to 24% for Set A and 17% for Set G.

The mean persistence values for the Jeerinah Dolerite A have a range of 1.51–3.92m but these are considered to be lower bound values. The termination data indicates that between 50–80% of individual joints extend beyond one or both limits of the exposure formed by a single 15m bench and so the persistence values should be weighted accordingly.

The mean persistence values for jointing in the Joffre Member also indicate a range of 1.00–3.05m. However joints often extend beyond one or both limits of the exposure (refer to Table 6.3) and hence these values should also be considered to be lower bound values. This is clearly demonstrated by Plate 6.5 which also reveals the localised persistence of the axial–plane cleavage.

6.5.3 **Features of the Disturbed Zone Next to the WFZ**

The borecore data for diamond drill holes has also been analysed in detail to determine the width of a 'disturbed' zone in the Jeerinah Formation rocks adjacent to the WFZ. The results are summarised in Table 6.5 for holes D248–D263 together with data available from previous drilling holes (D190–D212). Data for the Dolerite A, Shale A and Dolerite B units is generally good, but only limited information is available for the Shale B and Dolerite C units to the west.

This disturbed zone has been identified by using the fracture frequency and RQD data from borehole data.

The northern limit of this 'disturbed' zone has therefore been defined as occurring at the point at which the fracture frequency falls to a base level. In the Jeerinah Dolerites this was less than 5 (frequently 2–3) and 5–15 for the Jeerinah Shales. The corresponding RQD values are generally 90–100% for the Jeerinah Dolerites and 30–80% for the Jeerinah Shales. On this basis the best estimate of the width of the 'disturbed' zone indicated by the data in Table 6.5 is as follows:

Stratigraphic Unit	Width (m)
Dolerite C	10
Shale B	15–28
Dolerite B	3–12
Shale A	10–17
Dolerite A	4–8

The width of the 'disturbed' zone is reduced in the Dolerite units as would be expected for stronger, more competent material but it also appears to be reduced in the Dolerite A and Shale A units to the west. The data available suggest that the 'disturbed' zone in the Dolerite B and Shale B units to the west is also more highly fractured than the Dolerite A and Shale A units to the east (see Table 6.5).

6.6 DISCONTINUITY ROUGHNESS

For unfilled discontinuities surface roughness forms an important component of shear strength. Three scales of 'roughness' are generally used for slope stability studies as follows:

a Large scale roughness (wavelength greater than 10m) which is usually interpreted by marking stratigraphic contacts in a number of drill holes. It influences large-scale slope stability. However, the data are not usually sufficient to accurately define this roughness.

b Medium scale roughness (wavelength 0.1–10m) which can be estimated in field surveys and influences the stability of individual benches.

c Small scale roughness (wavelength less than 0.1m) which can be measured from single drill cores and field exposures and influences the shear strength that is determined in laboratory direct shear tests.

Medium scale roughness has been assessed where possible by means of amplitude and wavelength measurements and used to calculate a roughness angle i which contributes to the overall discontinuity shear strength as follows:

$$\tau = \sigma_n' \tan(\phi + i)$$

where τ = shear strength of discontinuity
σ_n' = effective normal stress
ϕ = base friction angle
i = roughness or dilation angle

Small scale roughness has been described in accordance with both the ISRM suggested methods (Ref 6.1) for discontinuity field surveys and MNM Company practice for HQ size diamond drill core. Both systems utilise four categories of roughness (rough, smooth, polished and slickensided) with three shape categories in the ISRM method (planar, undulating and stepped) and eight shape categories (designated A to H) in the MNM Company system.

However the ISRM method was adopted for field surveys because it facilitated the reduction in total number of classes from 32 to only 12 and this was considered to be quite adequate for the information required for this thesis.

6.6.1 Medium Scale Roughness

Mean 'i' values are included in Table 6.3 for joint sets within the Joffre Member and Jeerinah Dolerite A and also the axial-plane, slaty cleavage developed in the Joffre Member. No data has been collected for the Jeerinah Shale A because of limited exposures of undisturbed material within the present pit limits.

The 'i' values for the Joffre Member are reasonably consistent with a mean value of 4.5 ± 2.5° for both cleavage and jointing. In contrast the jointing in the Jeerinah Dolerite is more irregular and consistent with relatively thick igneous intrusions and the south-dipping jointing adjacent to the WFZ has a roughness value of 8 ± 3°.

These results are discussed further in Chapter 10 in relation to the stability of individual benches.

6.6.2 Small Scale Roughness

Data from both the field discontinuity surveys and the structural drill logs have been used to assess small scale roughness and the influence of the WFZ on such roughness. For the discontinuity surveys histograms of combined shape and roughness for each of the main joint sets have been analysed and are summarised on Figures 6.19 and 6.20 for holes D248 - D263. Earlier drill log information with the exception of D202, D204 and D211 for the 'disturbed' Jeerinah Shale A has not been used because of the different logging systems employed.

Jointing in the Jeerinah Dolerite is generally characterised by undulating and rough surfaces on a small scale whereas the planar jointing that is present in a dolerite flow unit to the south of Mt Whaleback (refer to Plate 6.3) is only sporadically developed in the intrusive Dolerite A unit (Plate 6.4). This planar jointing is mainly associated with the shear joint set D, as expected and also with joint set I which has a parallel trend to the fold system in the Jeerinah Dolerite. The structural drill hole information presented in Figure 6.19 confirms that undulating/rough surfaces of categories G and H dominate both joint and fault surfaces in the 'undisturbed' Jeerinah Dolerite units.

Joint surfaces in the Jeerinah Shale (Figure 6.20) are more variable but the following comments are made in relation to the field surveys:

- Joint sets A and I are normally the least planar joint surfaces. They are characteristic of sets which have developed parallel to the fold axes, ie. Set 1 which generally has undulating and rough surfaces, and set A which forms a conjugate set with the S2 cleavage and consequently a stepped surface dominates.

- Joint Set D has the most planar joint surfaces. They are characteristic of sets which have developed normal to fold axes but as shear jointing in an earlier fold phase.

- The shear joint sets (B and G) are characterised by rough surfaces which could be either planar or undulating.

The structural drill hole information (Figure 6.19) for the 'undisturbed' Shale A unit indicates that undulating (Category G) joint surfaces dominate but that the surface roughness may be smooth, polished or slickensided. This suggests that the 'disturbed' zone due to the WFZ may extend beyond the limit of in-pit diamond drilling with differential movement occurring on most discontinuity surfaces.

The field discontinuity surveys were conducted at distances up to 900m from current pit limits and can be considered to be beyond the influence of the WFZ. The nature of the S2 cleavage surfaces is discussed in detail in relation to the WFZ (Section 6.5.3).

Discontinuity surfaces in the Joffre Member are also variable, as follows:

- The least planar surfaces are characteristic of joint set H which is developed adjacent to the WFZ.

- The most planar surfaces are characteristic of the axial-plane cleavage which has developed normal to fold axes.

- Joint sets B, C and D all exhibit variable surfaces (planar, undulating and stepped) with stepped surfaces forming up to 40% of the total on individual surveys.

The small scale roughness characteristics for each of the main stratigraphic systems are summarised in Table 6.6.

6.6.3 **Discontinuity Roughness Next to the WFZ**

The WFZ has produced pronounced drag folding in some areas of the upthrown Jeerinah Formation rocks as well as incorporating Jeerinah, Mt McRae and Mt Sylvia units within the WFZ itself.

This drag folding should be accompanied by differential movement particularly on south–dipping discontinuities which will increase in dip in the 'disturbed' zone adjacent to the WFZ.

In the Jeerinah Shale units, south–dipping discontinuities in the form of joint sets G and I, bedding and the axial–plane cleavage (southern fold limbs), were formed prior to the WFZ (refer to Section 6.3.5). The differential movement would therefore be expected to occur on these discontinuities particularly the axial–plane cleavage which is the dominant foliation on the southern fold limbs.

With reference to Figure 6.20 this appears to be true with the axial–plane cleavage being the dominant discontinuity set in the 'disturbed' zones of the Shale A unit (42%) and particularly the Shale B unit (81%), although this relates to only one drill hole, D263.

In the Jeerinah Dolerite units no inclined south–dipping discontinuities were present before the WFZ formation and hence, steeply–dipping jointing and faulting developed in the 'disturbed' zone adjacent to the WFZ (refer to Section 6.3.5).

The discontinuities associated with shearing within the 'disturbed' zone should be characterised by a reduced roughness when compared with discontinuity roughness from areas to the north of the WFZ. This appears to be confirmed by Figure 6.19 for the Jeerinah Dolerite units with the 'disturbed' zones being characterised by faulting with slickensided, polished or smooth surfaces. The histograms for the 'undisturbed' material reveal that undulating (Category G) and irregular (Category H) jointing and faulting with a rough surface dominates.

A comparison of discontinuity roughness for the 'disturbed' and 'undisturbed' Jeerinah Shale A material indicates a similar trend. The percentage of slickensided and polished discontinuities is 77% in comparison to 62% for Jeerinah Dolerite units. The dominance of slickensided and polished faulting in the 'undisturbed' plot is largely due to extensive shearing on the Shale A/Dolerite B contact identified by drill hole D255 and hence the difference should be greater than indicated.

The histograms in Figure 6.20 for Jeerinah Shale A also indicate an increasing proportion of planar discontinuities (Category A) in the 'disturbed' zone (24% compared to 12% for the 'undisturbed' zone).

The occurrence of the steeply-dipping, axial-plane cleavage in the 'disturbed' zone is particularly important for bench-scale stability where drag-folded, Jeerinah Shale outcrops in upper part of the North Wall. This occurs between 6280E and 6880E in the Shale B unit and 7400E to 7560E in the Shale A unit. The data for the Shale B and Dolerite C units is limited to only two reliable drill holes and hence further drilling should be considered in this area to confirm the dominance of the axial-plane cleavage in the 'disturbed' zone and the associated weathering effects which are discussed in Section 6.8.

6.7 DISCONTINUITY INFILLING

The width of an infilled discontinuity and the composition of the infilling are both important factors in modifying the shear strength of a discontinuity due to surface roughness. If the infill thickness is greater than the amplitude of the surface asperities, then the shear strength properties will be largely those of the infilling. Filled discontinuities that have originated as a result of weathering along them may have fillings composed of decomposed or disintegrated rock. This process is particularly important in 'mudrocks', including shales, where residual clay minerals are deposited on persistent discontinuities, particularly bedding and cleavage planes. Although some discontinuity infillings in the 'disturbed' zone adjacent to the WFZ can be related to weathering processes, all infillings are described in this section because of the modifying influence upon the shear strength of discontinuities due surface roughness described in Section 6.6.

Structural borehole log data has been used to assess the nature of discontinuity infillings with summary histograms for the Jeerinah Formation presented in Figure 6.22 and for the Joffre Member in Figure 6.23. These histograms include all of the data available for boreholes D190 onwards with no differentiation possible for 'disturbed' material adjacent to the WFZ.

The following comments are made in relation to the Jeerinah Formation (Figure 6.22):

- discontinuity infillings are present as surface films in the Shale units but coatings with a mean thickness of 1.7mm form up to 33% for the Dolerite A unit and 19% for the Dolerite B unit.

- chloritic mineral infillings dominate in all units either individually in the case of the Dolerite A unit or together with iron oxides (found in Shale A), carbonate (found in Dolerite B) or significant clay (found in Shale B).

- infillings with a high friction angle occur in up to 30% of the discontinuities in the Dolerite units in 45% of the Shale A unit but in only 9% of the Shale B unit.

The discontinuity infillings in the Dolerite units can be considered to have minimal influence upon the discontinuity shear strength because of the limited thickness when compared to the surface roughness (both small and medium scale). The Shale discontinuity surfaces however, are characteristically smooth or polished, particularly in the 'disturbed' zone adjacent to the WFZ (refer to Section 6.6.3) and hence, any infillings with a low friction angle will have an important bearing upon bench-scale stability.

The limited data available for the Shale B unit indicates that chlorite and clay minerals are present on a high percentage of discontinuity surfaces. This is particularly significant for the area to the west of 6880E where the Shale B unit will be exposed. This area is also associated with increased fracturing and weathering, and with a reduced rock mass strength. The presence of discontinuity surfaces with a low friction angle can be expected to extend beyond the 'disturbed' zone adjacent to the WFZ because of the presence of dark grey, graphitic phyllites in both Shale units.

The majority of discontinuity infillings (90%) in the Joffre Member are present as a film with iron oxide being the dominant type, forming 68% of the overall total. The remaining discontinuities are mainly infilled with clay (9%) or both clay and iron oxides (14%).

Clay coatings comprise 6% of the total discontinuity infillings and these are predominantly associated with bedding planes with a mean thickness of 2.7mm. The presence of clay-coated bedding planes is relevant to the interim pit wall designs that utilise a toe buttress within the Joffre Member. This includes the slope below the in-pit crusher station at 7800E where in-dipping Joffre beds occur at dips of approximately 15°.

6.8 WEATHERING EFFECTS

Weathering is important in assessing the engineering characteristics of a rock mass because of its profound effect on the physical and mechanical properties of rock material eg. porosity and permeability, friability, strength, deformability, etc. Of particular importance in relation to the North Wall is the effect upon the weathering profile of penetrative weathering down the WFZ and laterally into Jeerinah Formation rocks.

Little weathering data was available prior to the commencement of the field work of this thesis and so all available field exposures within the East Pit have been mapped for an assessment of both weathering and intact rock strength. All diamond drill core for D248–D263 has also been assessed for weathering with the grade described according to the recommendations of the ISRM (Ref 6.1).

Data from the diamond drill holes is summarised in Table 6.7 together with the depth of intercept with the North Whaleback Fault. The limited data available suggests that three zones of weathering can be defined according to the depth of weathering adjacent to the WFZ as follows:

a 5920E – 7000E approximately where weathering to a vertical depth of 300m has occurred affecting the Dolerite C, Shale B and Dolerite B units.

b 7000E – 7800E approximately where weathering down to 150m has affected the Dolerite B, Shale A and Dolerite A units.

c 7800E – 8400E approximately where weathering to a depth in excess of 200m has occurred in the Dolerite A unit.

The northward penetration of weathering into the Jeerinah Formation also appears to be quite variable but there are insufficient data to establish any definite trends. A zone of moderate weathering occurs adjacent to the North Whaleback Fault up to 3m wide in the Dolerite A and B units and possibly even wider in the Shale B and Dolerite C units to the west. Slight weathering indicated by discolouring has been detected at distances up to 50m from the WFZ in the Dolerite A unit and 85m in the Dolerite B unit.

Existing exposures of the Dolerite A unit to the east of 7800E indicate that the weathered zone increases in width upwards and towards the original topographic surface. This is one aspect that should be investigated further in the area to the west of approximately 6900E where the Shale B and Dolerite C units will be exposed.

6.8.1 Strength Reduction

The strength reduction associated with increasing grade of weathering is well documented. In order to assess the strength reduction adjacent to the WFZ the uniaxial compressive strength (UCS) values for Jeerinah Formation rocks are summarised in Table 6.8. This shows that UCS results for weathered material adjacent to the WFZ are available for the Dolerite B and Shale B units.

A strength reduction in a zone approximately 10m wide is evident in the Dolerite B unit with D192 exhibiting decreasing strength with increased weathering grade towards the North Whaleback Fault contact. The UCS range for the weathered and 'disturbed' zone of 19–66 MPa is significantly lower than the mean UCS value of 96 MPa for the Dolerite B unit. A zone of similar width in the Shale B unit is evident from Table 6.8 with a strength reduction of about 75% occurring based on the limited UCS values available.

6.9 SUMMARY AND IMPLICATIONS FOR SLOPE STABILITY

Analysis of the engineering geology data has enabled an assessment to be made of discontinuity characteristics of the Jeerinah Formation including a comparison with the discontinuity characteristics of the Joffre Member to the south of the WFZ.

The width of a 'disturbed' zone of Jeerinah Formation rocks adjacent to the WFZ has been estimated by analysing discontinuity data (for orientation, fracture frequency and surface roughness), rock mass weathering and the strength reduction produced by weathering. A summary of this engineering geology data for both 'undisturbed' and 'disturbed' material adjacent to the WFZ is presented in Table 6.9. Only those engineering geology aspects which have an important bearing on slope stability are discussed in further detail.

6.9.1 Jeerinah Dolerite A

Jeerinah Dolerite A is a strong to very strong, medium grained amphibolite which is characterised by steeply–inclined joint sets in a well defined and regular pattern. The 'disturbed' zone adjacent to the WFZ has been defined by both surface mapping and diamond drilling and has the following characteristics:

- increased 'fracturing' (mainly as faulting) in a zone up to 8m wide with relatively smooth surfaces (slickensided, polished or smooth) on a small scale (less than 100mm) forming 40% of the total.

- fault-related, south-dipping discontinuities dominate in a zone up to 15m wide with the dip direction having a large angular range between 100° and 240°, which increases the potential for wedge formation.

The potential for localised wedge failures, within the 'disturbed' zone in particular, is assessed further in Chapter 10.

6.9.2 Jeerinah Shale A

The Jeerinah Shale A is a medium strong, slate/phyllite with the dominant foliation being a well-developed axial-plane cleavage which generally dips to the south-west at 40° to 60°. Bedding is occasionally preserved on shallow-dipping southern fold limbs and may be co-planar with the axial-plane cleavage locally.

Jointing is generally well defined and related to a fold axis direction in the range 290° - 305° and to the axial-plane cleavage. With the exception of jointing normal to the fold axis direction, all joint sets are inclined at approximately 40° to 70° with two sets dipping south-southeast to southwest. Four sets of discontinuities therefore, have the potential to influence the stability of individual benches in any North Wall design, with the axial-plane cleavage and east-west shear jointing dominating.

The influence of these discontinuities upon bench-scale stability will vary locally because of the effects of the WFZ upon individual discontinuity parameters including orientation, surface roughness, infilling material and extent of weathering. The width of this 'disturbed' zone adjacent to the WFZ has been defined by diamond drilling with limited exposure available in the existing East Pit. The 'disturbed' zone should have the following characteristics:

- local variations in discontinuity orientation (both strike and increased southerly-dip) due to drag folding.

- increased 'fracturing' in a zone up to 17m wide with relatively smooth surfaces (slickensides or polished) on a small scale (less than 100mm).

Shearing at the Dolerite A/Shale A contact has been detected in boreholes 160m apart with an associated strength reduction within the Jeerinah Shale material. This contact will be exposed to the west of approximately 7400E in proposed North Wall designs and therefore the implications of an increased dip within the 'disturbed' zone of the WFZ are discussed in Chapter 10. A detailed analysis of bench stability is also presented in Chapter 10.

6.9.3 Jeerinah Dolerite B

The Jeerinah Dolerite B is a strong to very strong, medium to coarse grained amphibolite that will be exposed in the proposed North Wall designs to the west of 7400E. No suitable exposures occur within or to the north of the East Pit and so all data are interpreted from drill hole logs.

The joint system developed in the Jeerinah Dolerite A unit has been found to be consistent with that developed in a flow unit to the south of Mt Whaleback and so the same system has been assumed to be present in the Dolerite B unit. As local variations in the development of any particular discontinuity set (including jointing, faulting, etc) can occur, particularly adjacent to the WFZ, it is recommended that further field mapping be undertaken, as exposures become available, to confirm the nature of the discontinuity system.

The drill hole data suggests that the width of the 'disturbed' zone is greater in the Dolerite B (up to 12m wide) than the Dolerite A unit (up to 8m wide). The 'disturbed' zone in the Dolerite B unit also appears to be more highly fractured with a significant strength reduction due to weathering. A strength reduction is also associated with shearing at the Shale A/Dolerite B contact which is significant for slope stability wherever this contact is exposed, and locally steepened, adjacent to the WFZ.

6.9.4 Jeerinah Shale B

The Jeerinah Shale B is a medium strong, slate/phyllite that will be exposed in the proposed North Wall designs to the west of approximately 6880E. Engineering geology data are available from only four drill holes which indicate that the 'disturbed' zone adjacent to the WFZ is extensive with increased fracturing up to 28m away, associated with significant weathering and strength reduction.

Revised structural interpretations indicate that the trend of the major fold axes affecting the Jeerinah Formation have rotated from northwest – southeast to approximately east-west within the Shale B unit. The discontinuity analysis described in Section 6.3 indicates that the discontinuity system developed is related to the direction of the major fold axes. Therefore, it may be inappropriate to adopt the Shale A discontinuity system for the Shale B material. However until additional data are collected, evidence in support of this assessment cannot be produced.

6.9.5 Joffre Member

The Joffre Member forms an important aspect of any North Wall slope design and structural and engineering geological mapping have indicated that the following structures could influence wall designs:

- shallow-dipping, normal faults occurring on the steep, northern fold limbs which may be locally steepened adjacent to the WFZ.
- axial-plane shears occurring in the hinge zones of minor folds.
- persistent, axial-plane cleavage which develops in the hinge zones of large folds.
- bedding which may be locally steepened (due to drag folding) and associated with clay infilling, adjacent to the WFZ.

Data concerning these structures is generally limited because of the emphasis placed on the structure north of the WFZ by MNM Company. However, the significance of these structures to both the large-scale stability of toe buttress designs as well as to small-scale bench stability will be discussed further in Chapter 10.

6.9.6 Jeerinah Shale/Dolerite Contacts

The Jeerinah Dolerites are major intrusive igneous bodies with sheared contacts which have been encountered in some of the diamond drill holes. The cross section on 7320E is a good example since it shows sheared zones within Shale A as well as within Dolerite A. These sheared zones were encountered in diamond holes D202 and D192. Sheared material is also present between the WFZ and the dolerite, with sheared dolerite often incorporated within the WFZ itself.

These sheared contacts are often associated with the presence of graphite and clay minerals which obviously reduce the shear strength along the contacts.

6.10 CONCLUSIONS

The main conclusions of the engineering geology investigations are as follows:

- An extensive engineering geology investigation of the Jeerinah Formation, north of the WFZ, indicates that the alternating sequence of regionally metamorphosed shale (slate) and dolerite (meta-dolerite) units are competent and/or medium to high strength (greater than 40 MPa for shale units and greater than 70 MPa for dolerite units). However, these rock mass strengths are reduced adjacent to the WFZ.

- The four main geological units that influence the North Wall design of the East Pit (ie. the Jeerinah Dolerite, Jeerinah Shale, WFZ material and the Joffre Member) are characterised by different lithologies, discontinuity systems and rock mass strengths.

- The WFZ material has been identified as the weakest geological unit to be exposed in the North Wall with a significant kaolinitic clay content. The rock mass strength properties are described in more detail in Chapter 8.

- The WFZ material that will be exposed in the North Wall is extensively sheared, altered and weathered with previously formed discontinuity systems largely destroyed.

- The Jeerinah Formation (both shale and dolerite units) and the Joffre Member are characterised by regular discontinuity systems. This work has enabled a detailed assessment of bench stability to be conducted (refer to Chapter 10).

- The discontinuity systems, developed in the various stratigraphic units are influenced mainly by the varying lithology, style of folding and the WFZ. It is also considered that the discontinuity system of the Jeerinah Shale is influenced by a rotation of the fold axis direction from northwest-southeast in the extreme east of the area under investigation to approximately east-west in the western area.

- A 'disturbed' zone adjacent to the WFZ is developed in all stratigraphic units particularly the Joffre Member (at least 30m wide) and Jeerinah Shale (up to 20m wide in the Shale A and 30m in the Shale B unit).

- The 'disturbed' zone is characterised by increased fracture frequencies, modified discontinuity orientation, variations in discontinuity surface roughness and weathering effects, ie. reduction in rock mass strength and discontinuity shear strength due to the formation of clay minerals.

- The extent of penetrative weathering associated with the WFZ appears to be greater to the west of 7000E where it can be detailed to a depth of approximately 300m. The northern extent of this weathering also appears to be greater in this area affecting the Jeerinah Dolerite B, Shale B and Dolerite C units.

- Drag folding adjacent to the WFZ occurs both to the north in the Jeerinah Formation and to the south in the Joffre Member resulting in an increased dip of south dipping stratigraphic contacts and discontinuities. This influence of the fault has a major bearing upon the bench stability of all stratigraphic units. The main south-dipping discontinuities include:

  (a) **Jeerinah Dolerite** - normal faulting and fault related jointing sub-parallel to the WFZ.

  (b) **Jeerinah Shale** - an axial-plane, slaty cleavage dominates but jointing and bedding may also be present.

  (c) **Joffre Member** - bedding dominates but low angle normal faulting, axial-plane shears and an axial-plane cleavage may be locally persistent.

- In addition to the discontinuities described above, stratigraphic contacts associated with shearing occur at shale/dolerite boundaries in the Jeerinah Formation, eg. Shale A/Dolerite B contact between 7160E and 7400E.

6.11 RECOMMENDATIONS

The discontinuity systems outlined for the Jeerinah Formation are based largely upon the mapping of existing exposures to the east of 7600E where the Dolerite A and Shale A units outcrop. Further data collection in areas to the west is strongly recommended as exposures become available and the following specific recommendations are made:

° Determine the effect of the fold axis rotation upon the discontinuity system in the Jeerinah Shale units by additional field mapping.

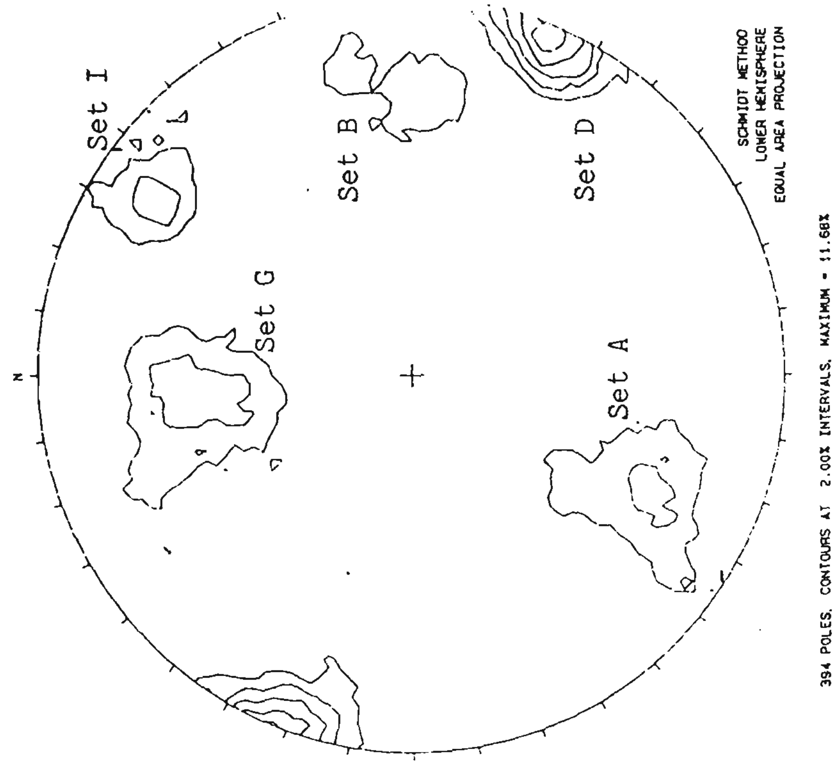
° Measure the variation in the dip of south-dipping discontinuities, especially the axial-plane cleavage produced by drag folding on the WFZ by field mapping as suitable exposures become available.

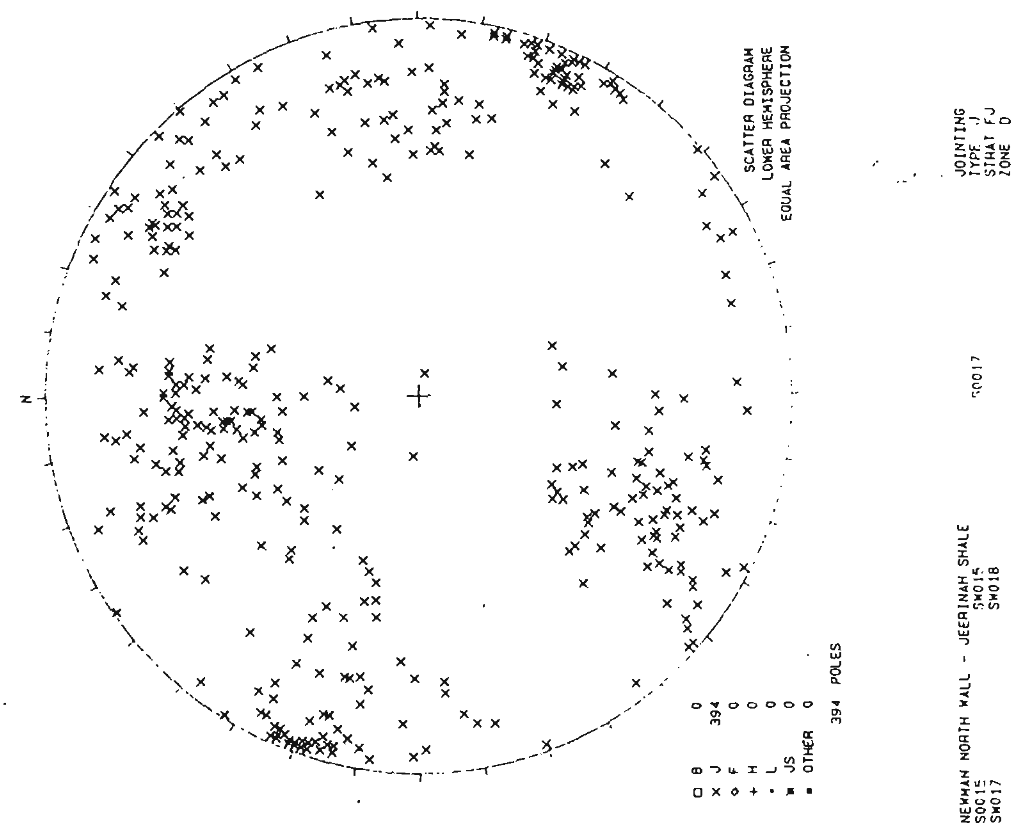
° Ascertain the nature and extent of the sheared shale/dolerite contacts in particular the Shale A/Dolerite B boundary between 7160E and 7400E by diamond drilling.

° Locate the extent of the 'disturbed' zone and discontinuity characteristics adjacent to the WFZ in the Dolerite B, Shale B and dolerite units further west than has previously been measured by diamond drilling and field mapping.

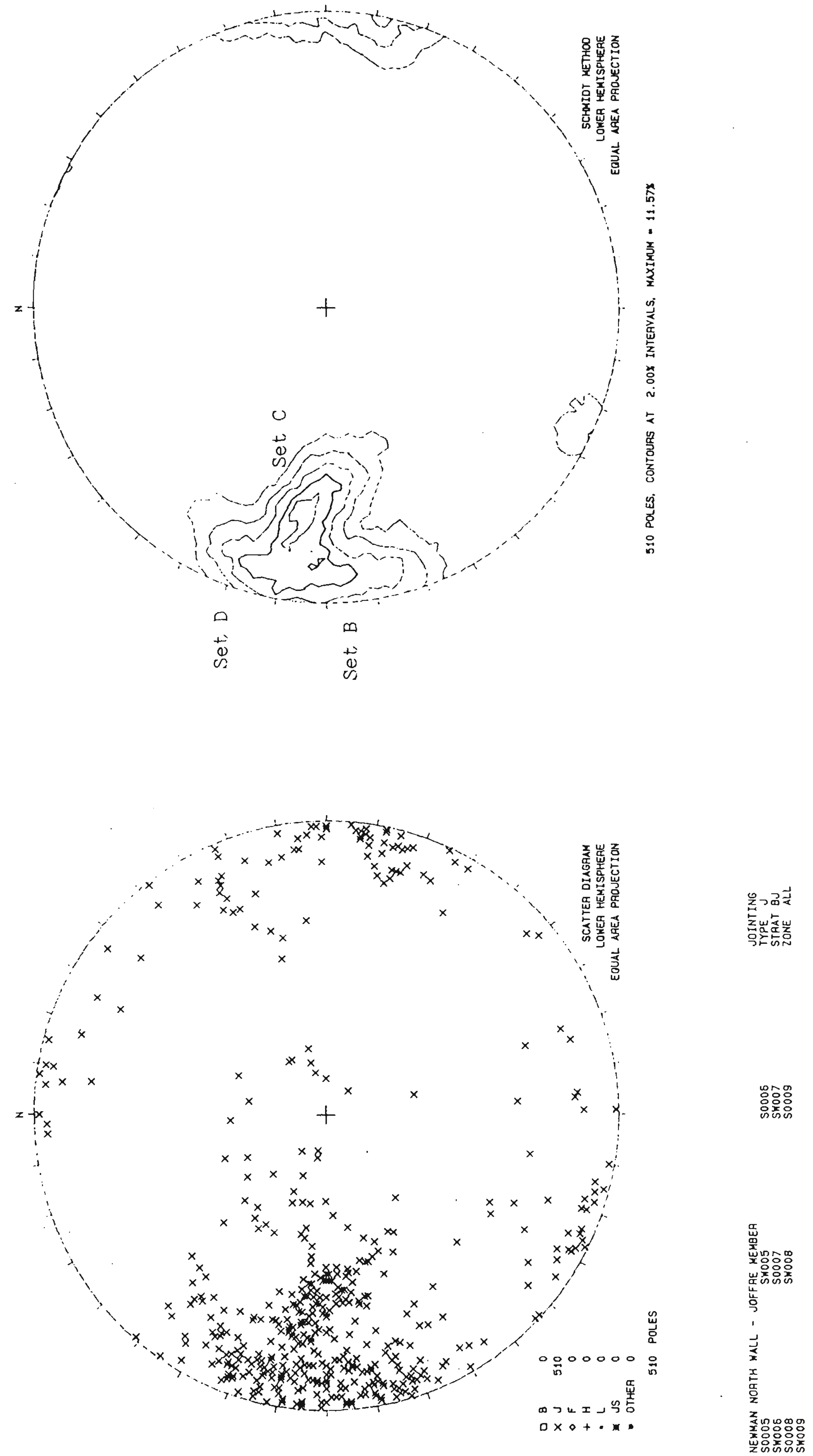
° Quantify the northern extent of penetrative weathering adjacent to the WFZ and the associated reduction in rock mass strength and occurrence of clay-coated discontinuity surfaces by diamond drilling. This applies particularly to the Jeerinah Shale B unit.

° If a North Wall design encompasses a toe buttress in the Joffre Member rocks, there will be a requirement to confirm the nature of south dipping discontinuities adjacent to the WFZ.

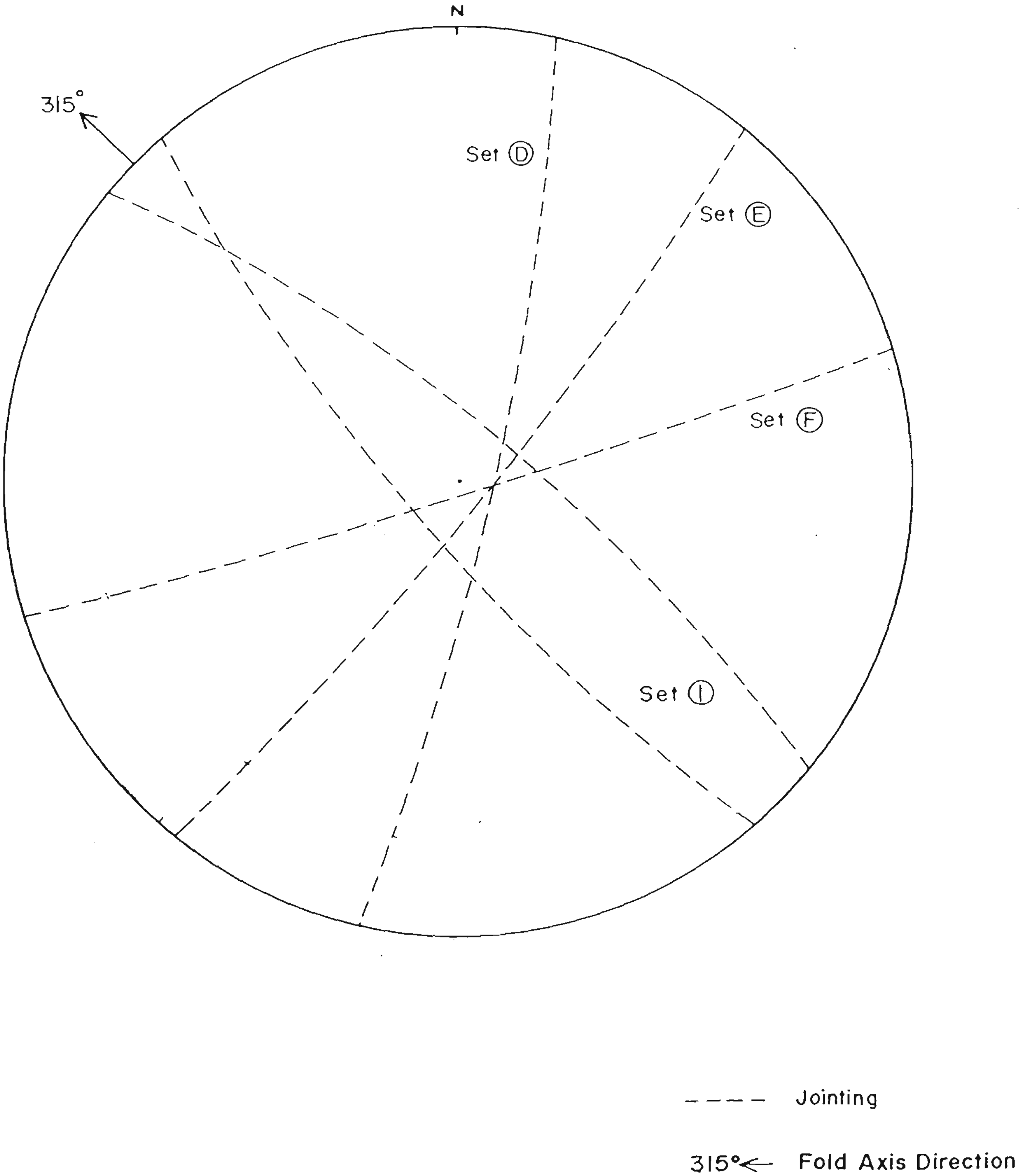




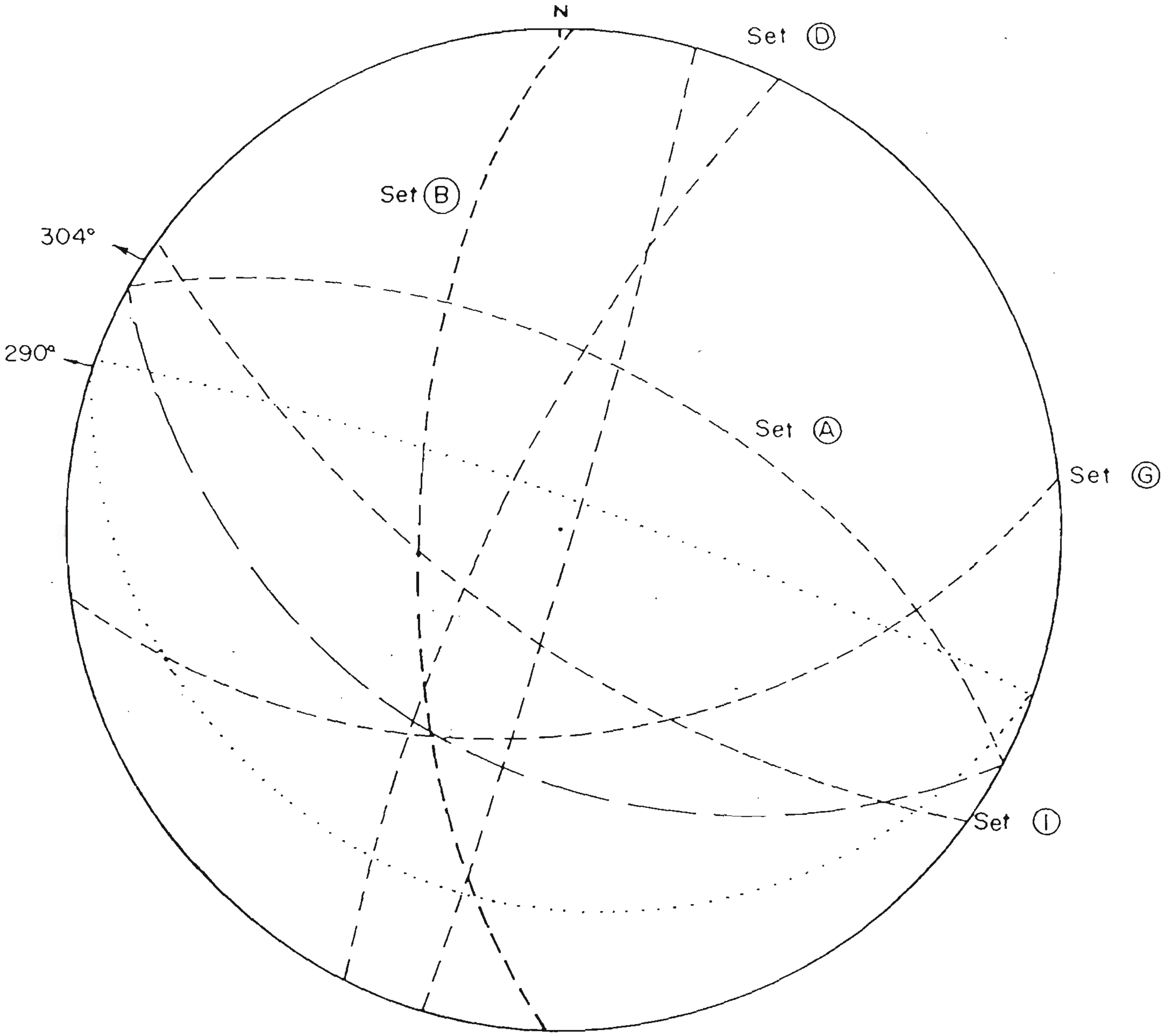
Scale	SUMMARY OF JOINTING IN THE JEERINAH SHALE A	FIGURE 6.2
Drn		



Scale	SUMMARY OF JOINTING IN THE JOFFRE MEMBER	FIGURE 6.3
Drn		

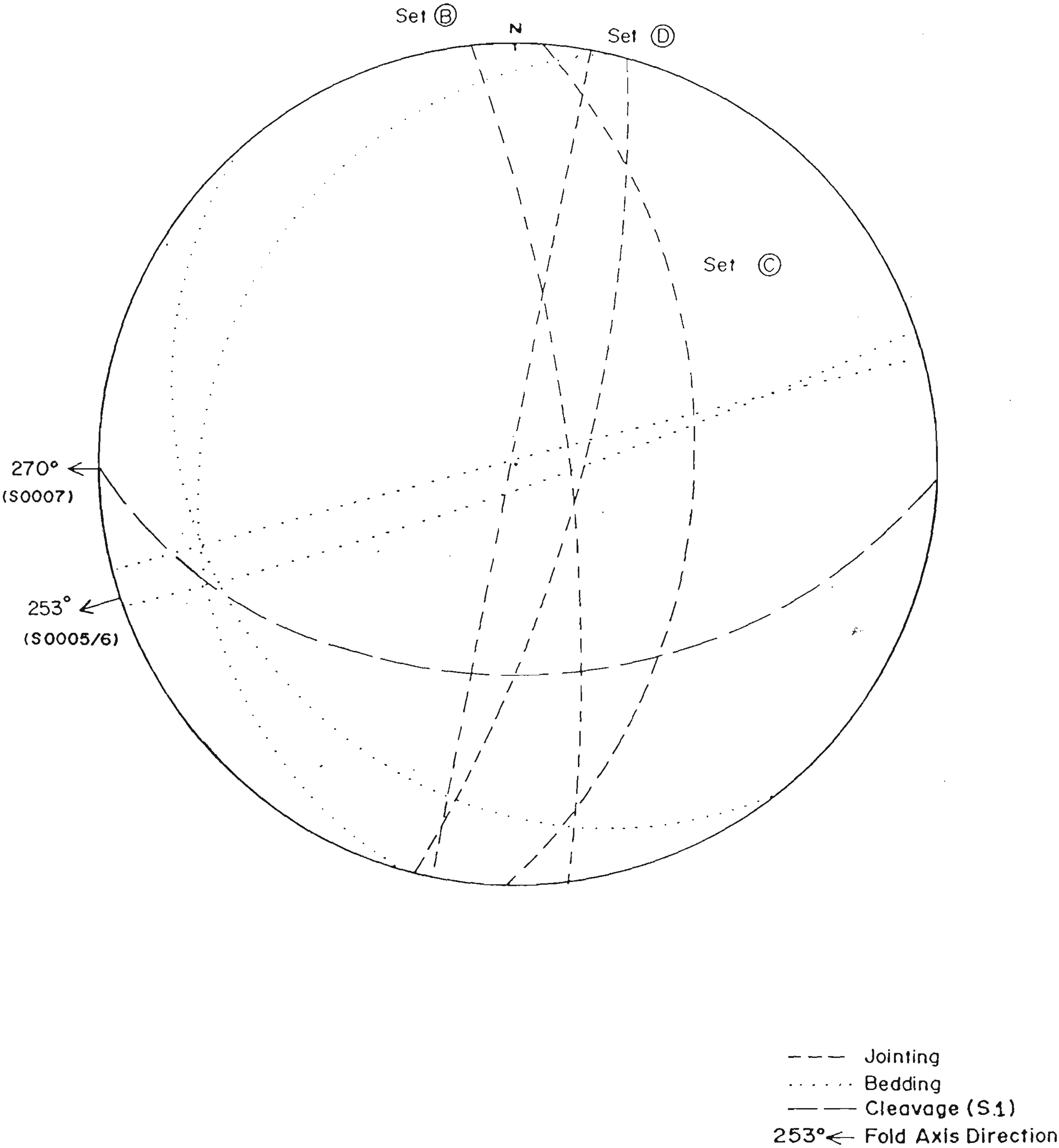


	Scale	JEERINAH DOLERITE A : JOINT SYSTEM	FIGURE 6.4
	Drn		
	Dwg No.		

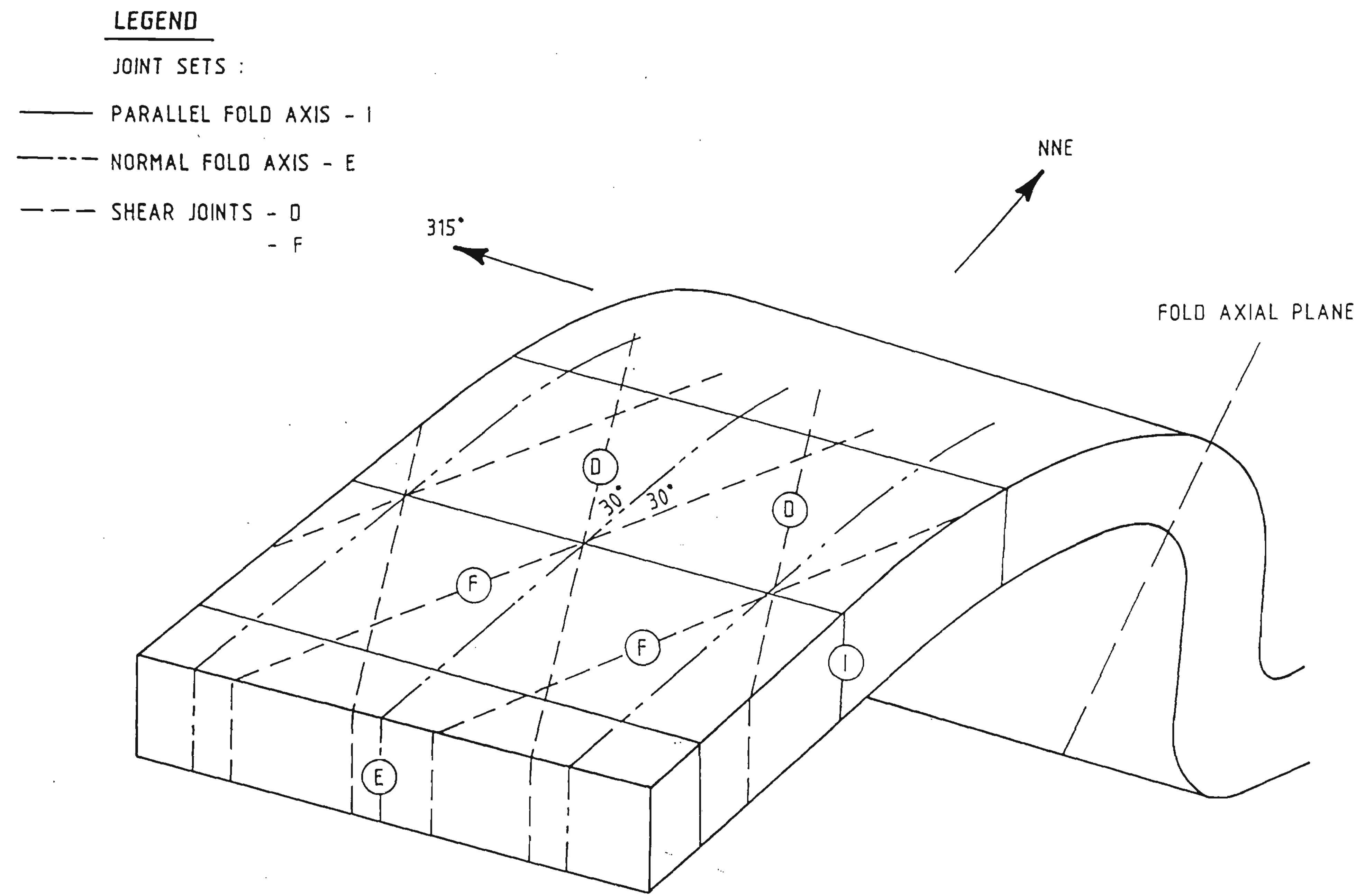


— — — Jointing
· · · · · Bedding
——— ———Cleavage (S 1)
290°← Fold Axis Direction

Scale	JEERINAH SHALE A : JOINT SYSTEM	FIGURE 6.5
Drn		
Dwg No.		



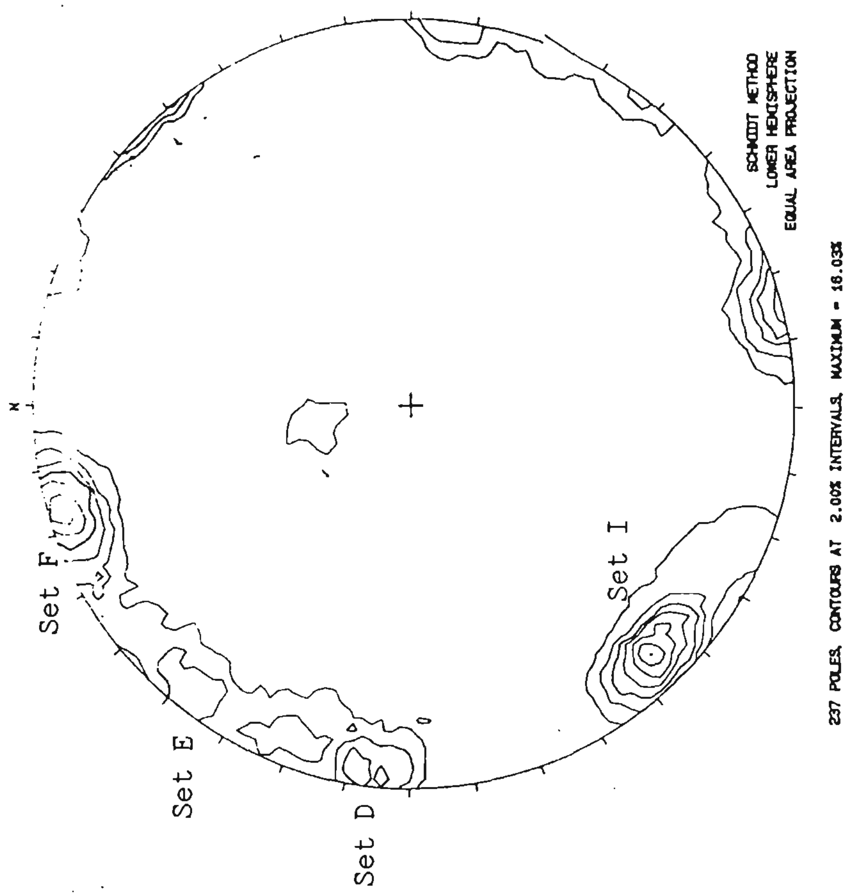
Scale	JOFFRE MEMBER : JOINT SYSTEM	FIGURE 6.6
Drn		
Dwg No.		

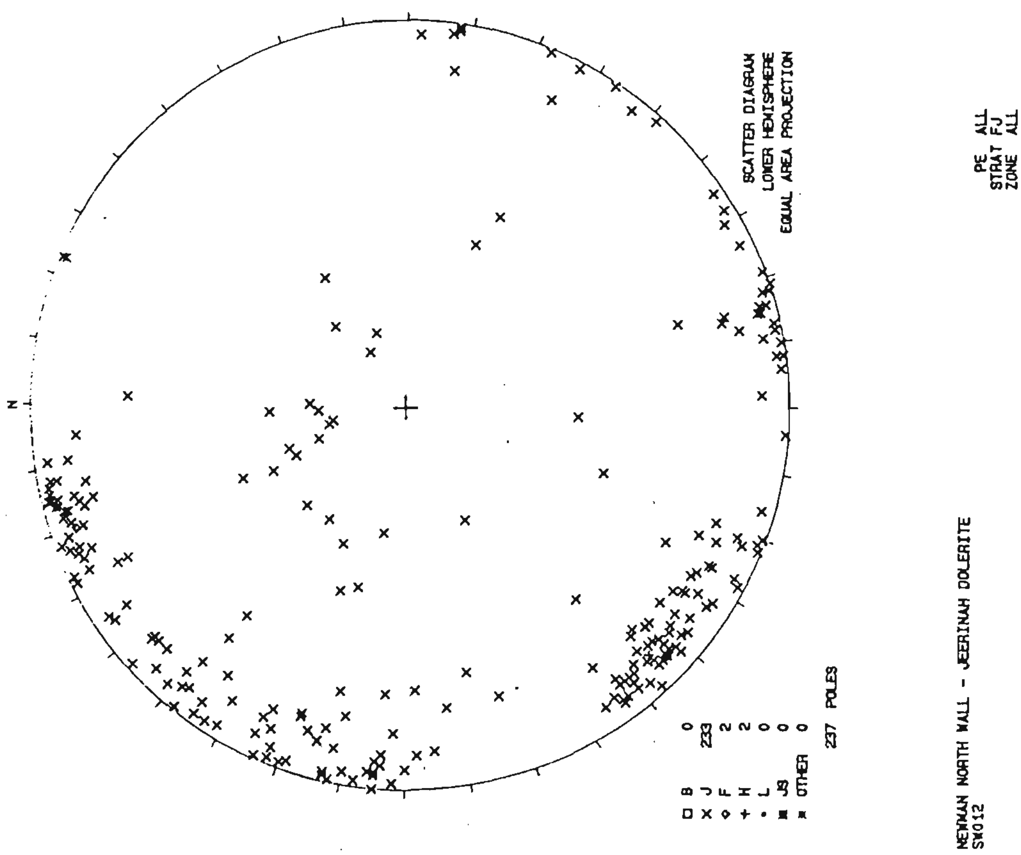


Scale	
Drn	
Dwg No.	

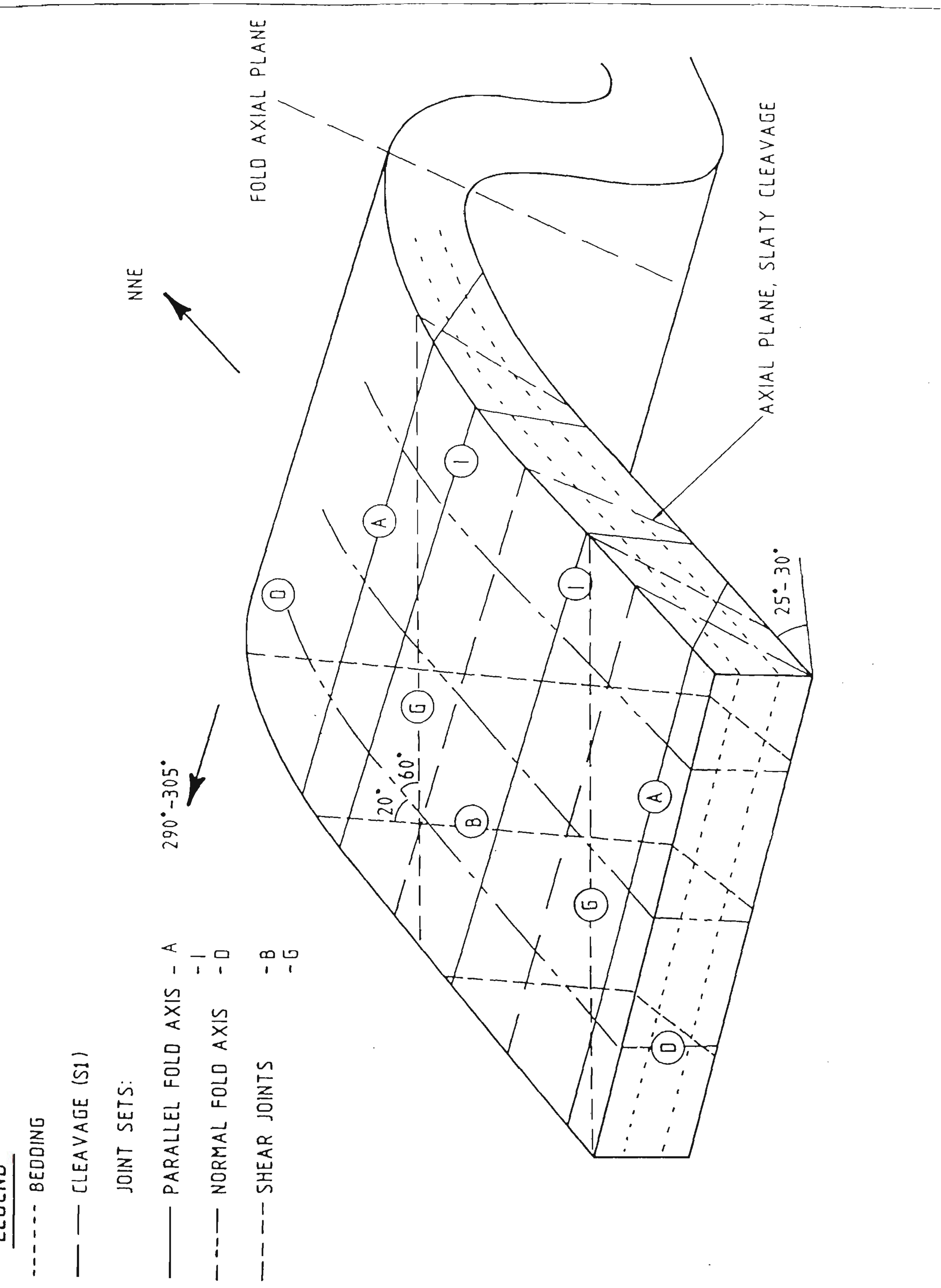
RELATIONSHIP BETWEEN JOINTING AND FOLDING IN THE JEERINAH DOLERITE

FIGURE 6.7

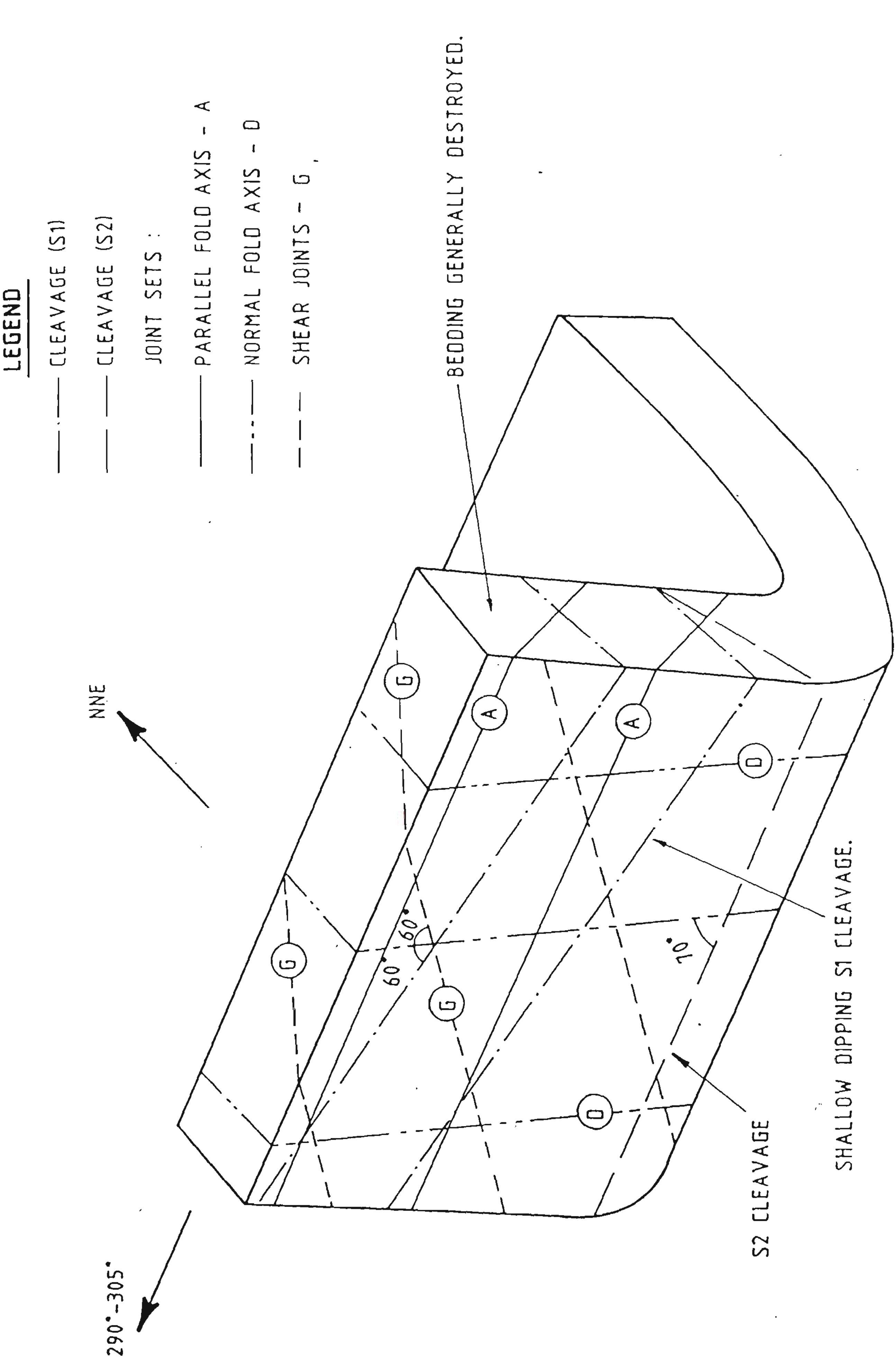




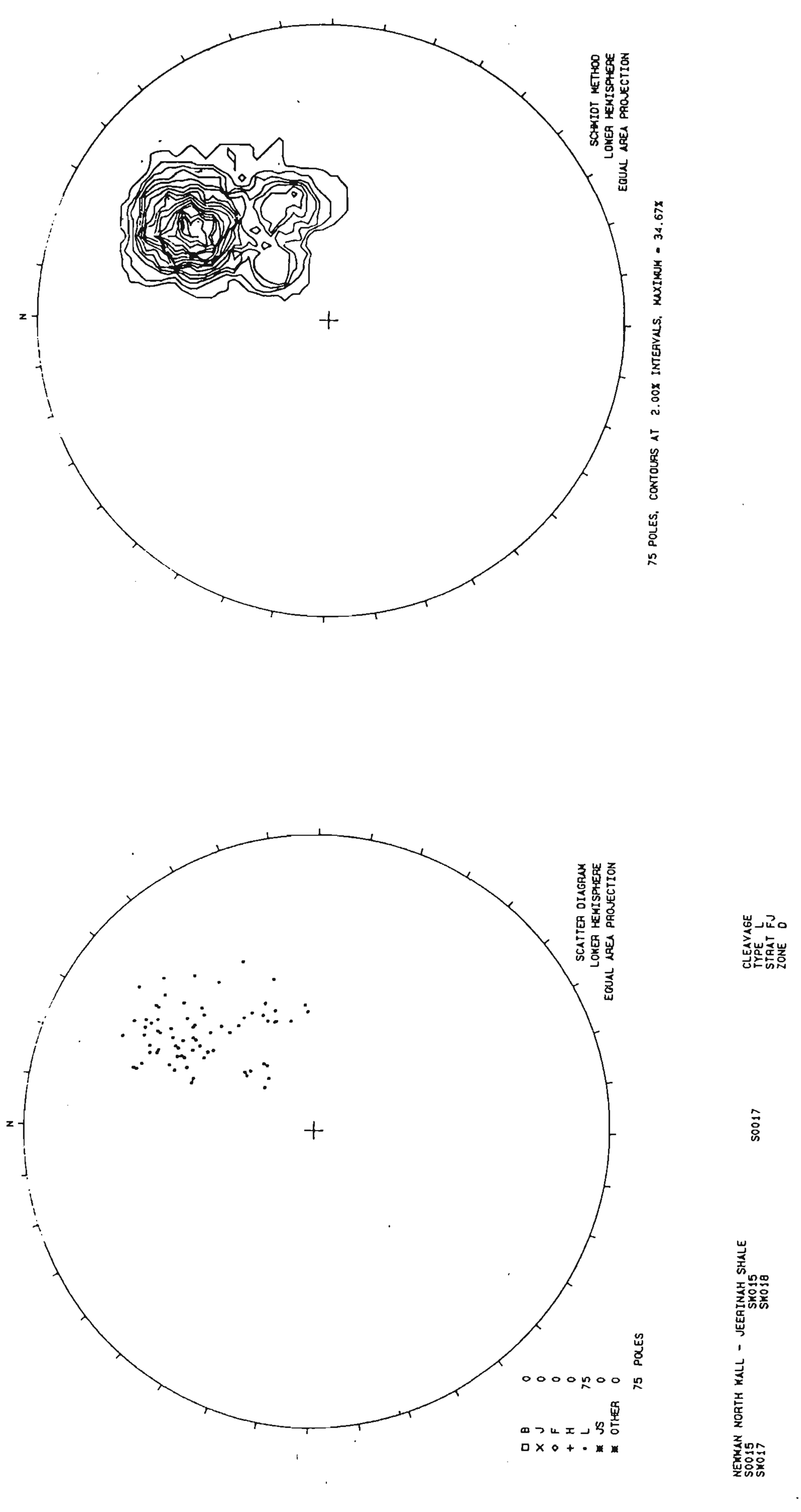
	Scale	PLANAR JOINTING DEVELOPED IN THE JEERINAH DOLERITE SOUTH OF MT. WHALEBACK	FIGURE 6.8
	Drn		
	Dwg No		



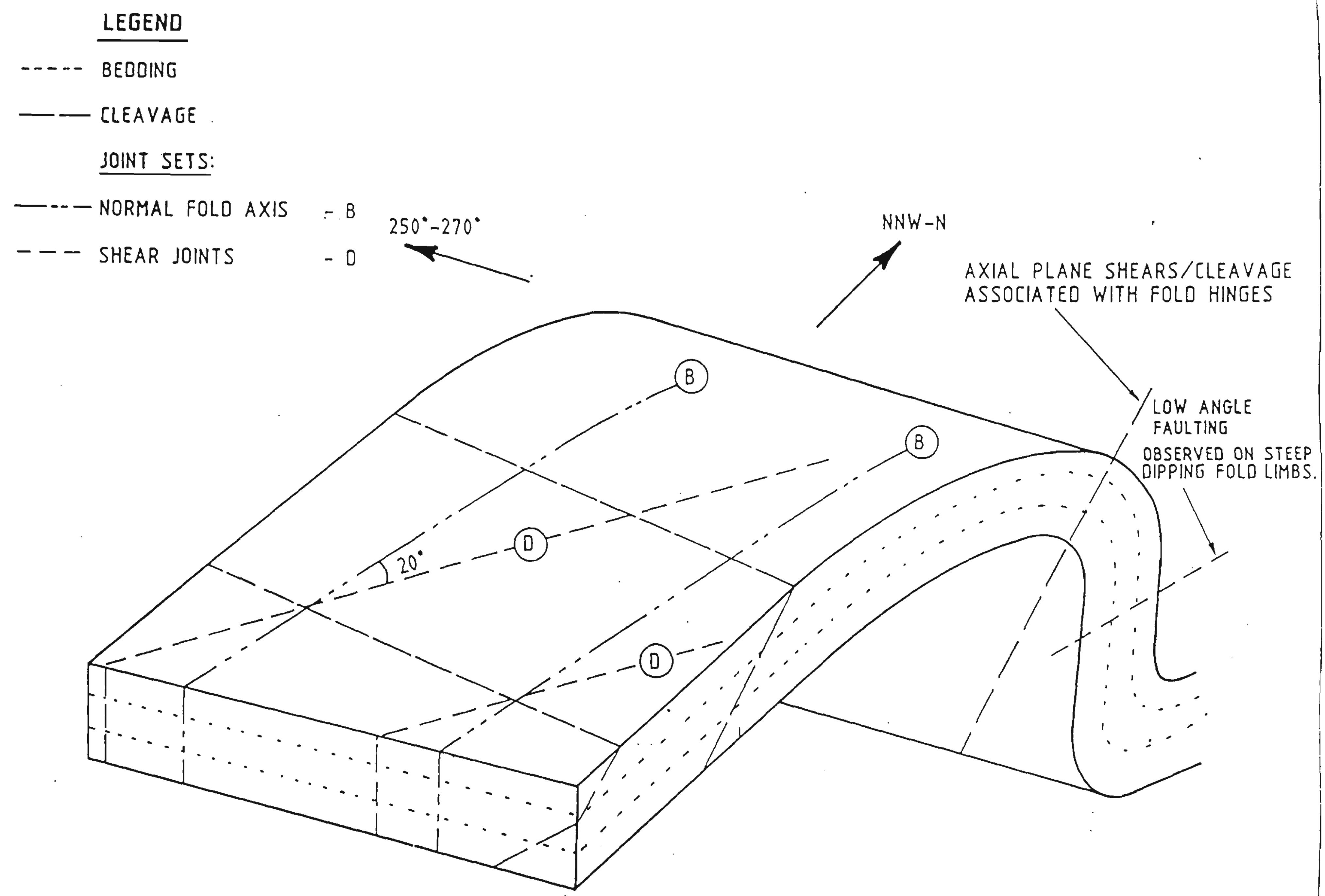
Scale	DISCONTINUITY TYPES PRESENT ON SOUTHERN FOLD LIMBS IN THE JEERINAH SHALE	FIGURE 6.9
Drn		
Dwg No.		



Scale	DISCONTINUITY TYPES PRESENT ON NORTHERN FOLD LIMBS IN THE JEERINAH SHALE	FIGURE 6.10
Drn		
Dwg No.		



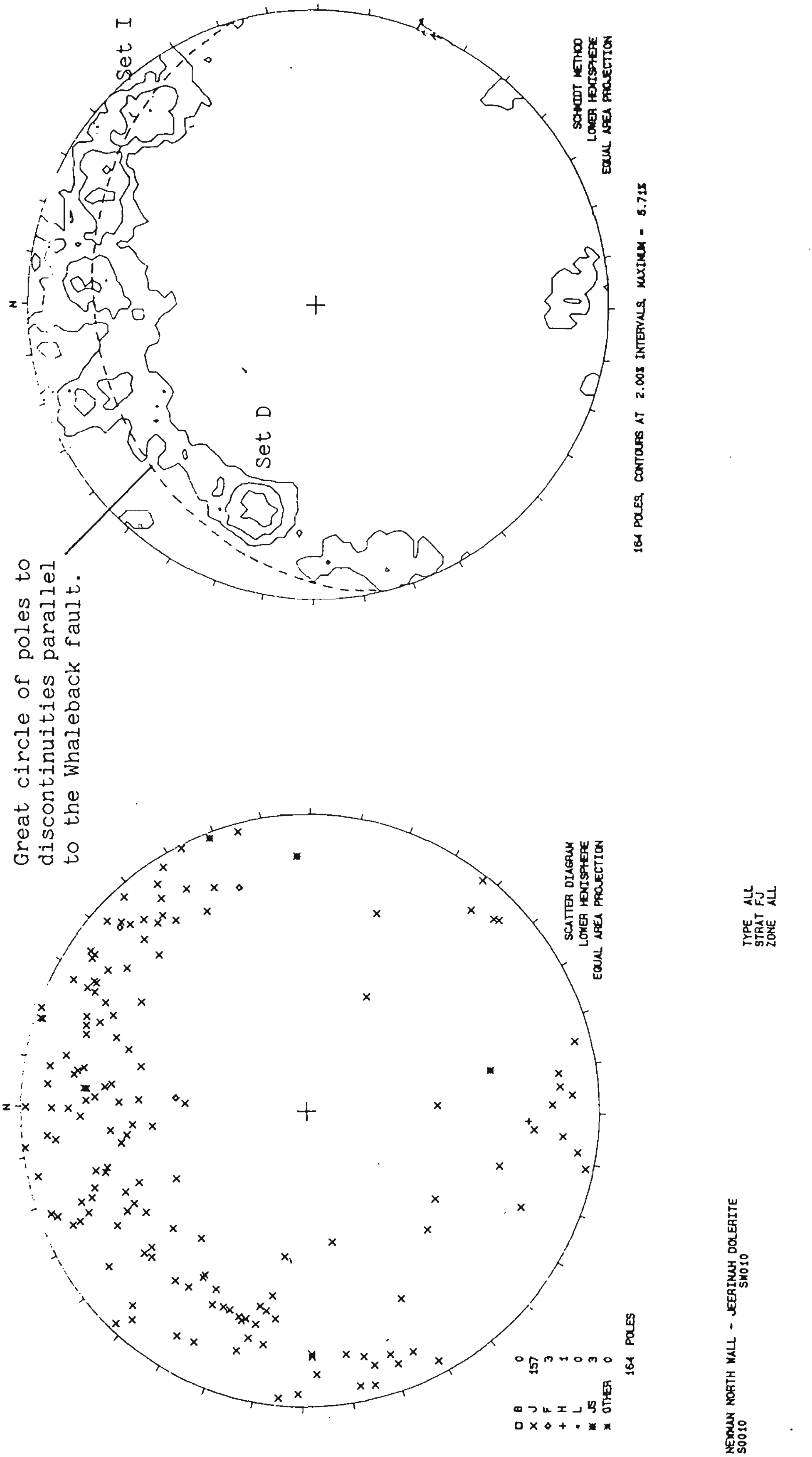
Scale	SUMMARY OF CLEAVAGE IN THE JEERINAH SHALE A	FIGURE 6.11
Drn		
Dwg No.		



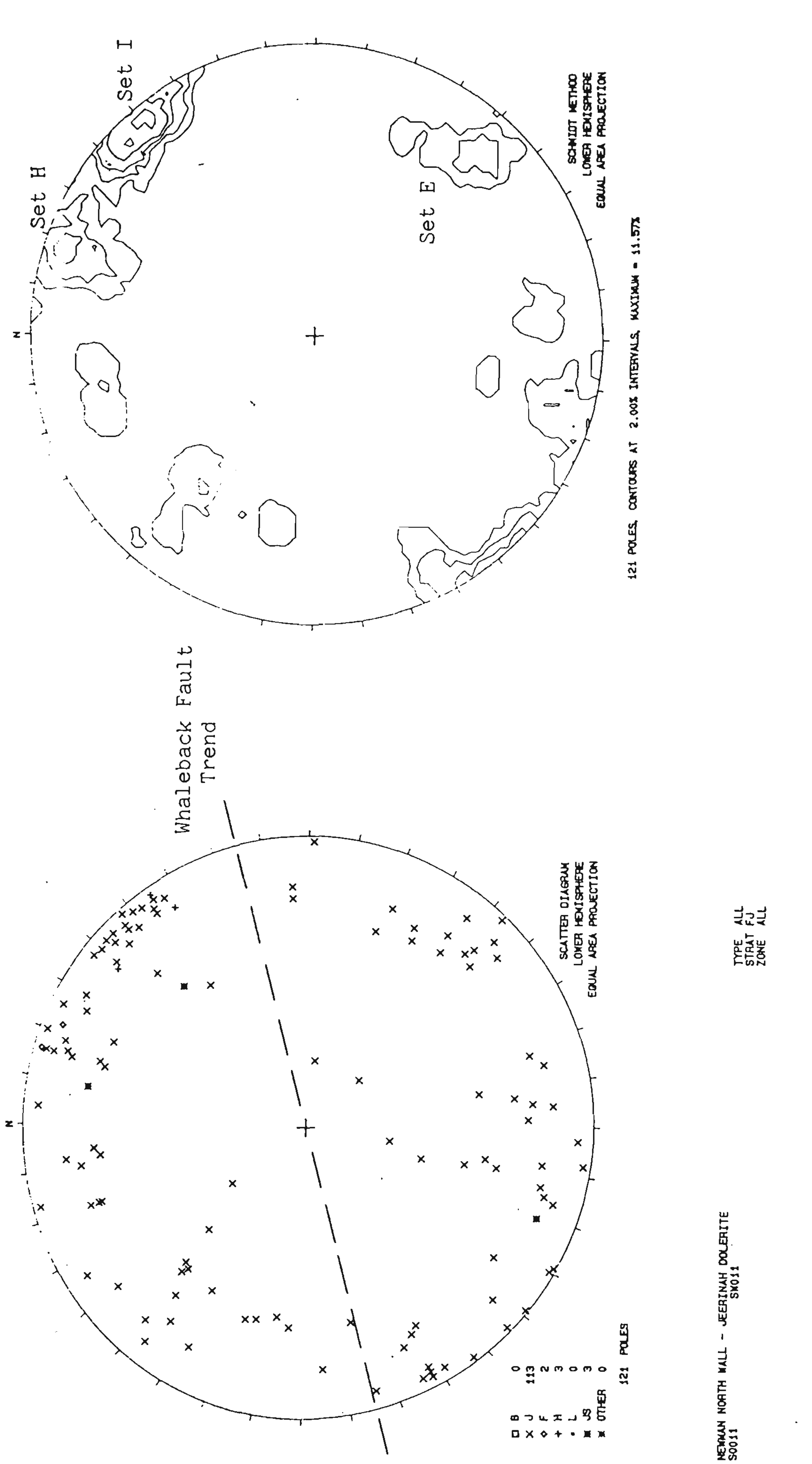
Scale	
Drn	
Dwg No.	

RELATIONSHIP BETWEEN DISCONTINUITIES AND FOLDING IN THE JOFFRE MEMBER

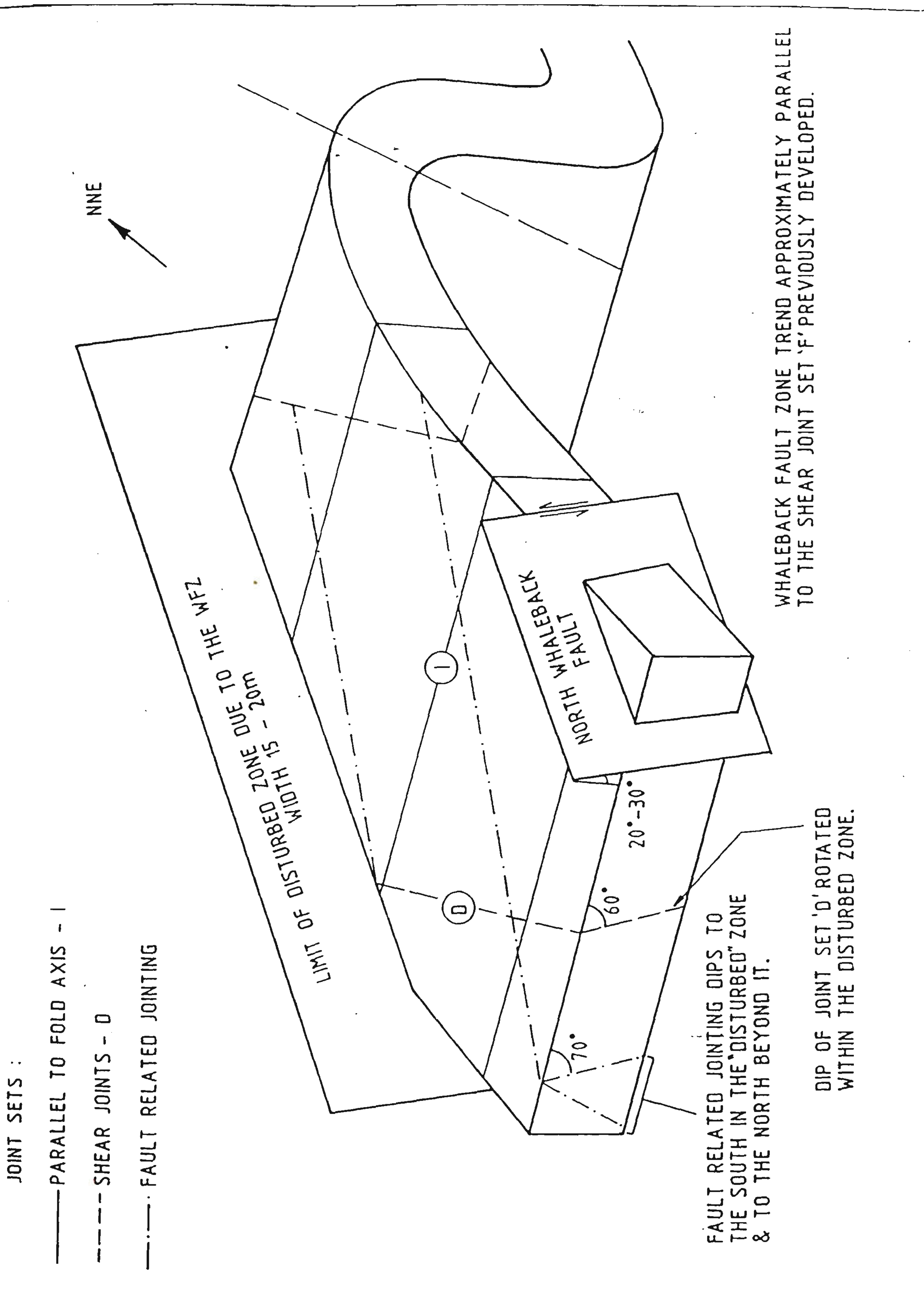
FIGURE 6.12



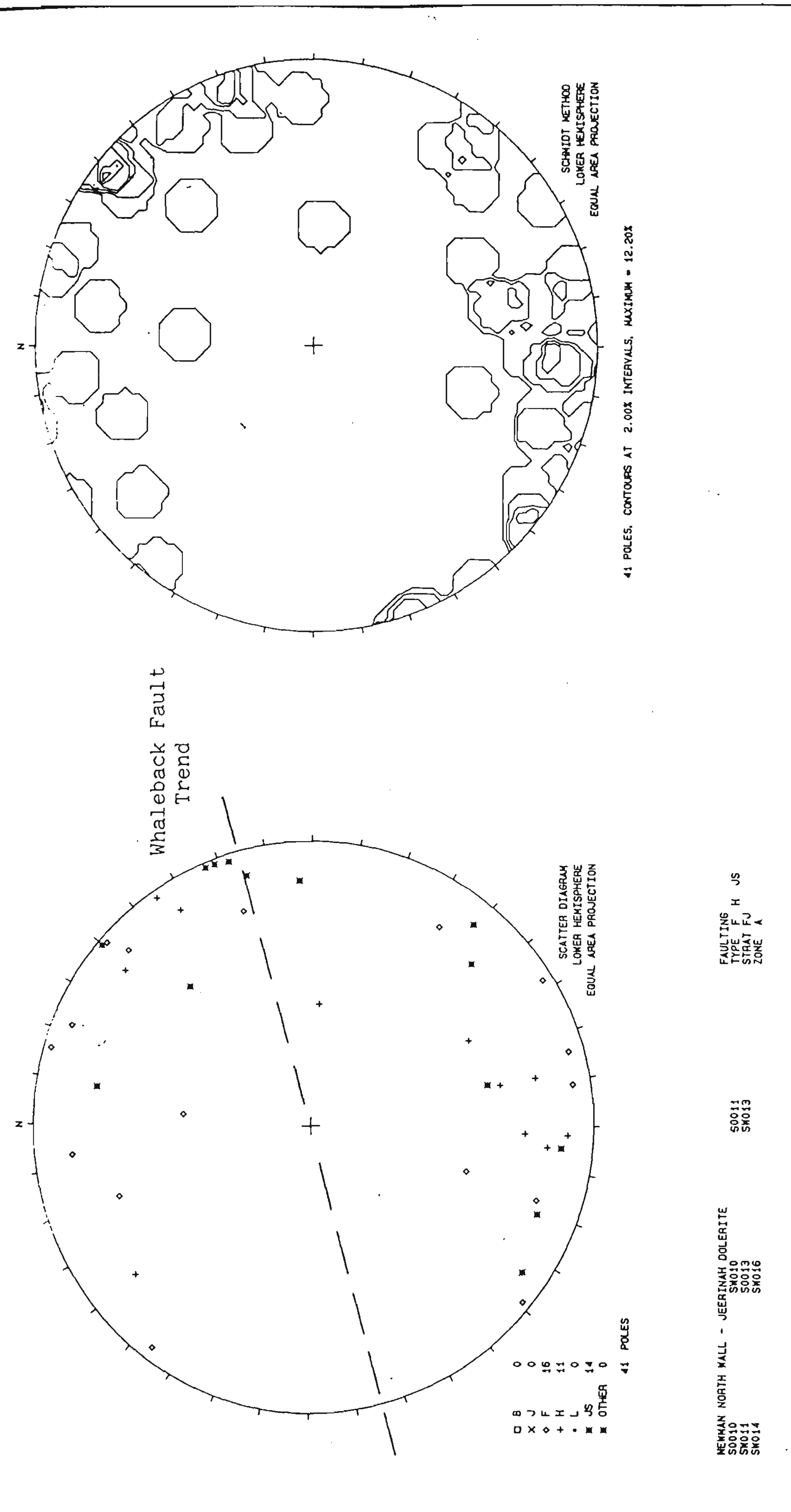
Scale	DISCONTINUITY MAPPING IN THE JEERINAH DOLERITE A - BENCH 16	FIGURE 6.13
Drn		
Dwg No.		



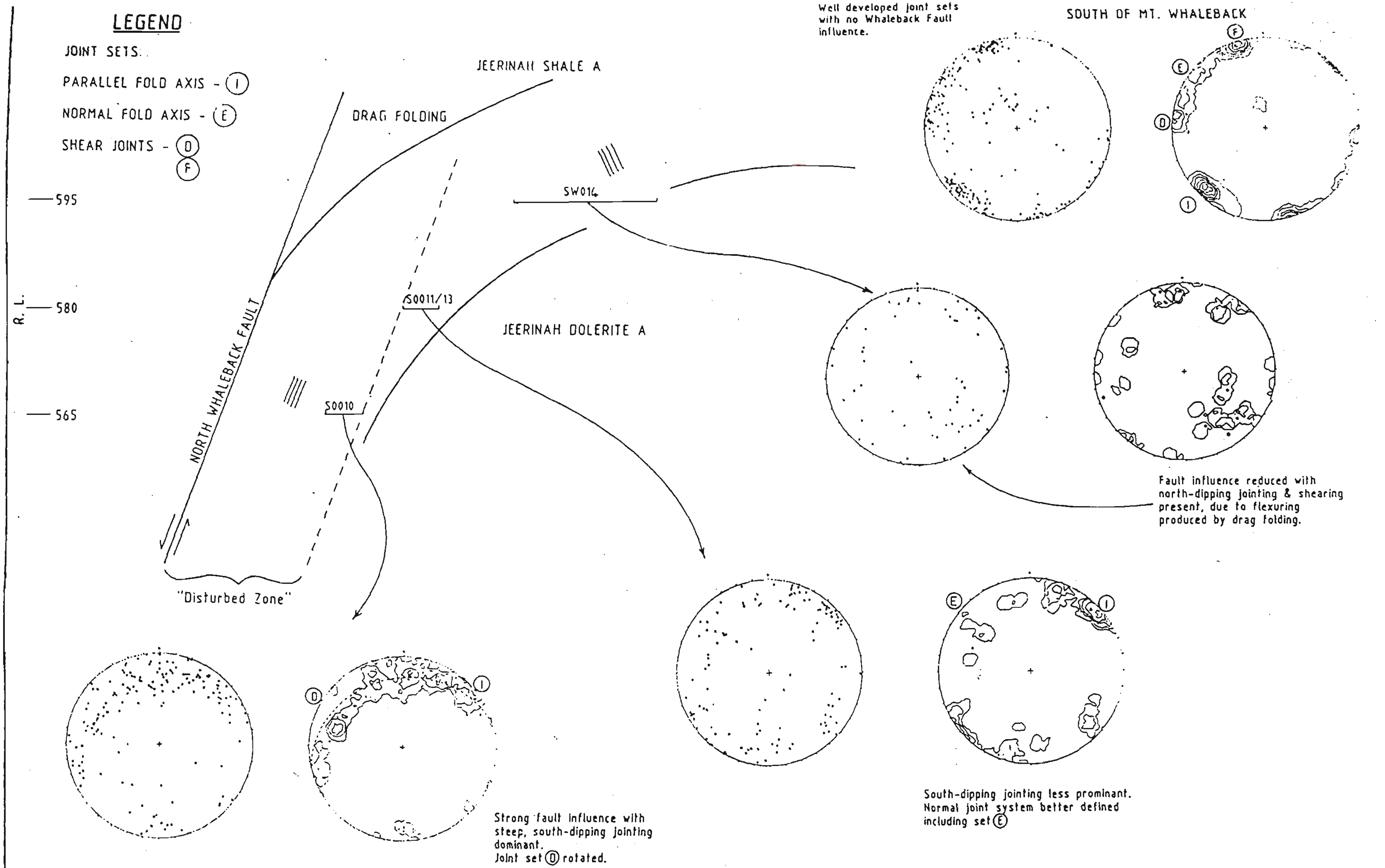
Scale	DISCONTINUITY MAPPING IN THE JEERINAH DOLERITE A - BENCH 15	FIGURE 6.14
Drn		
Dwg No.		



Scale	RELATIONSHIP BETWEEN JOINTING IN THE JEERINAH DOLERITE AND THE WHALEBACK FAULT ZONE	FIGURE 6 15
Drn		
Dwg No.		



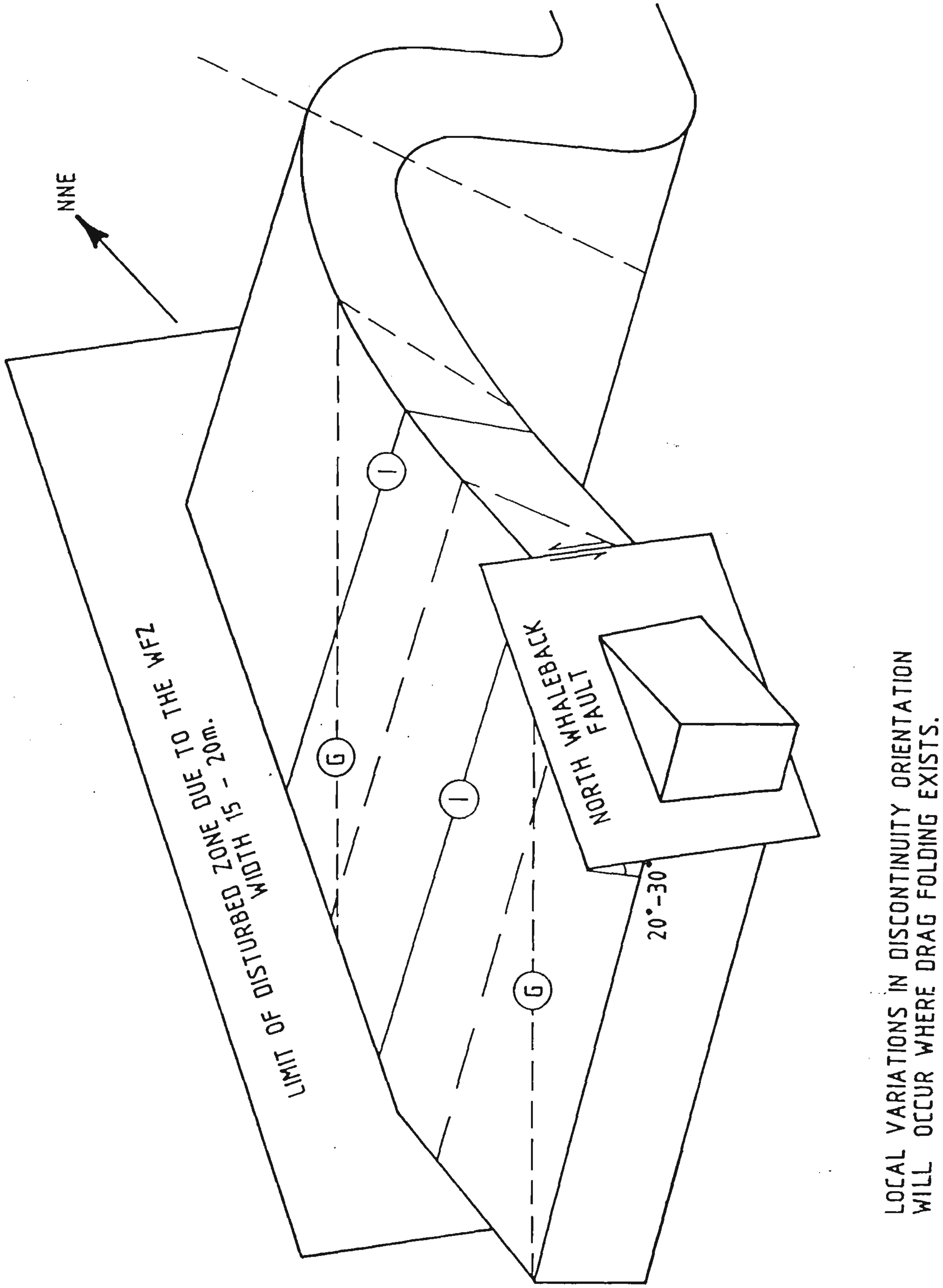
Scale	SUMMARY OF FAULTING IN THE JEERINAH DOLERITE A - (SCANLINES ONLY)	FIGURE 6.16
Drn		
Dwg No.		



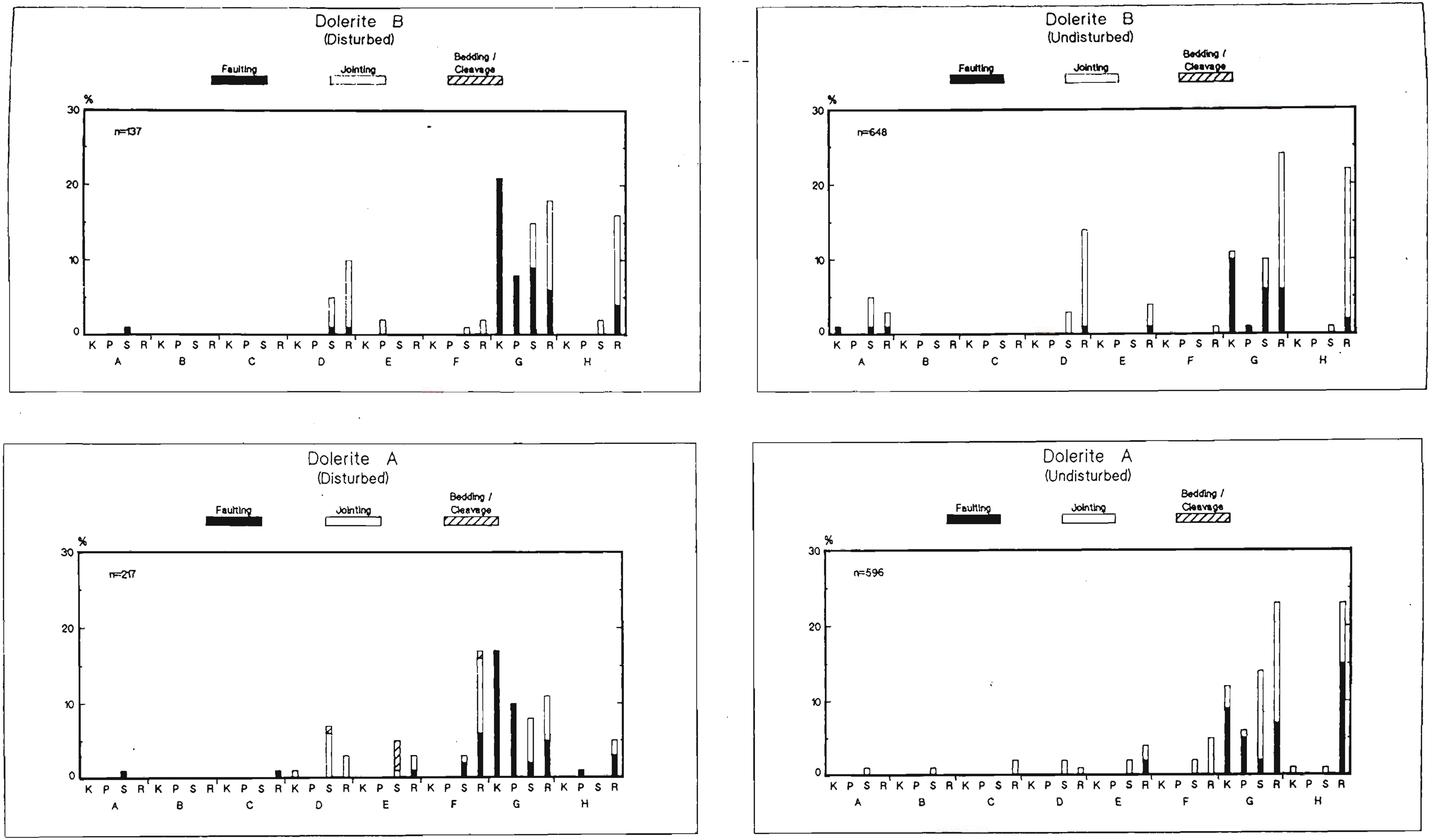
Scale	1 : 500
Drn	
Dwg No.	

INFLUENCE OF THE WHALEBACK FAULT ZONE ON THE DISCONTINUITY SYSTEM OF THE JEERINAH DOLERITE.

FIGURE 6.17

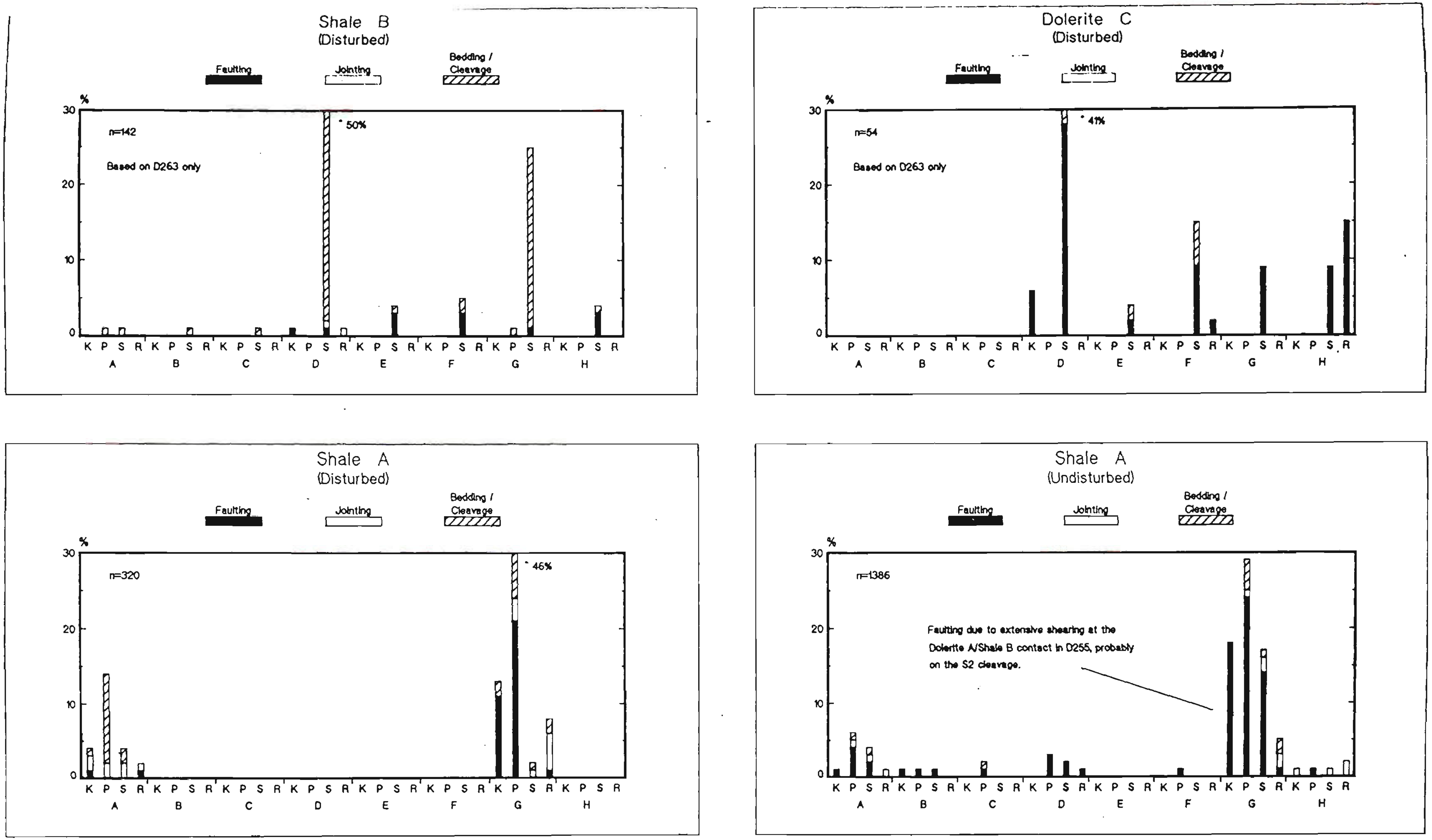


	Scale	RELATIONSHIP BETWEEN DISCONTINUITY TYPES IN THE JEERINAH SHALE AND THE WHALEBACK FAULT ZONE	FIGURE 6.18
	Drn		
	Dwg No.		



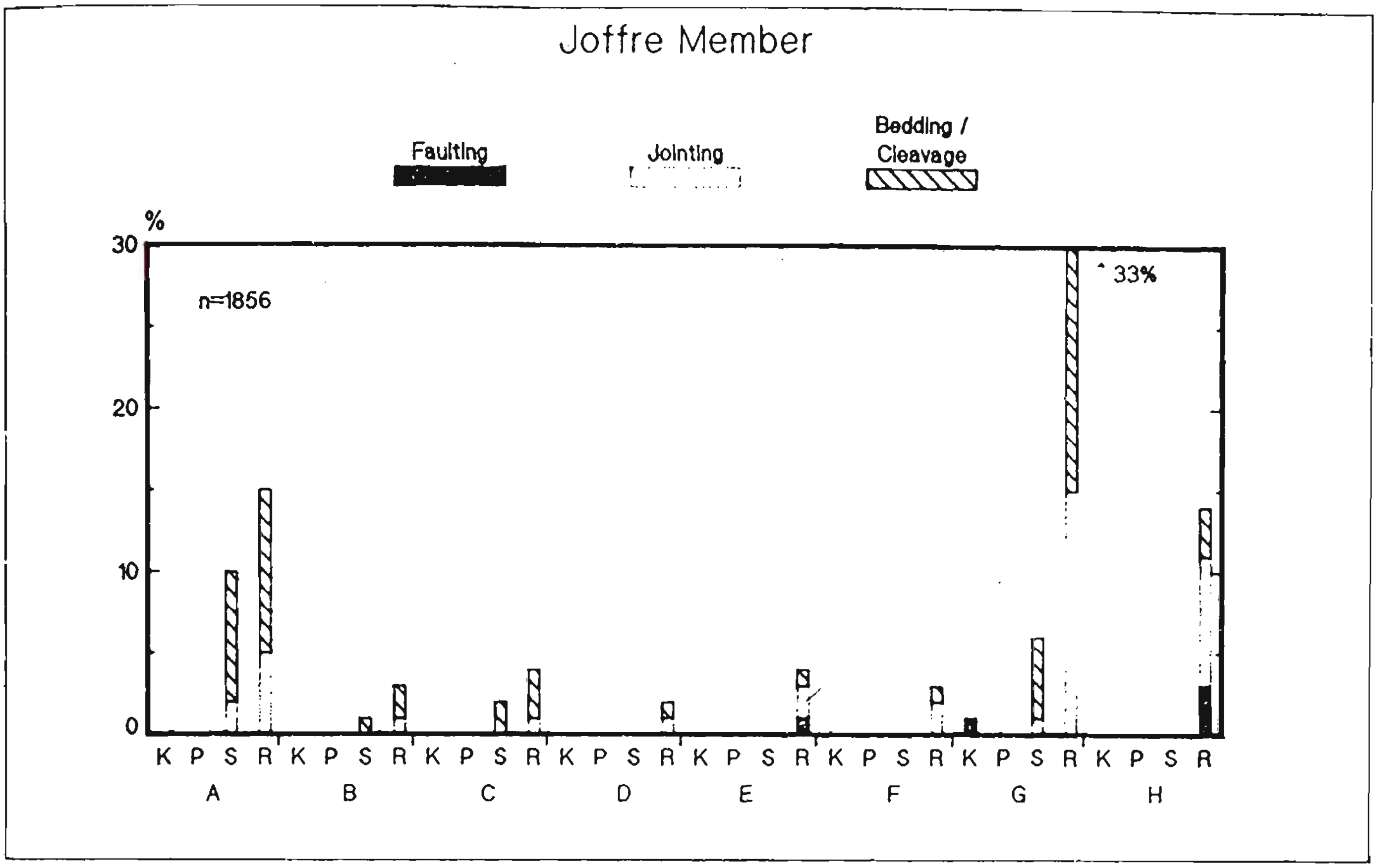
Refer to Figure 6.22 for explanatary Legend

Scale	SUMMARY OF SMALL SCALE ROUGHNESS FOR DISCONTINUITIES IN THE JEERINAH DOLERITE A AND B UNITS	FIGURE 6.19
Drn		
Dwg No.		



Refer to Figure 6.22 for explanatory Legend

Scale	SUMMARY OF SMALL SCALE ROUGHNESS FOR DISCONTINUITIES IN THE JEERINAH SHALE A AND B, DOLERITE C UNITS	FIGURE 6.20
Drn		
Dwg No.		



LEGEND

Shape Category

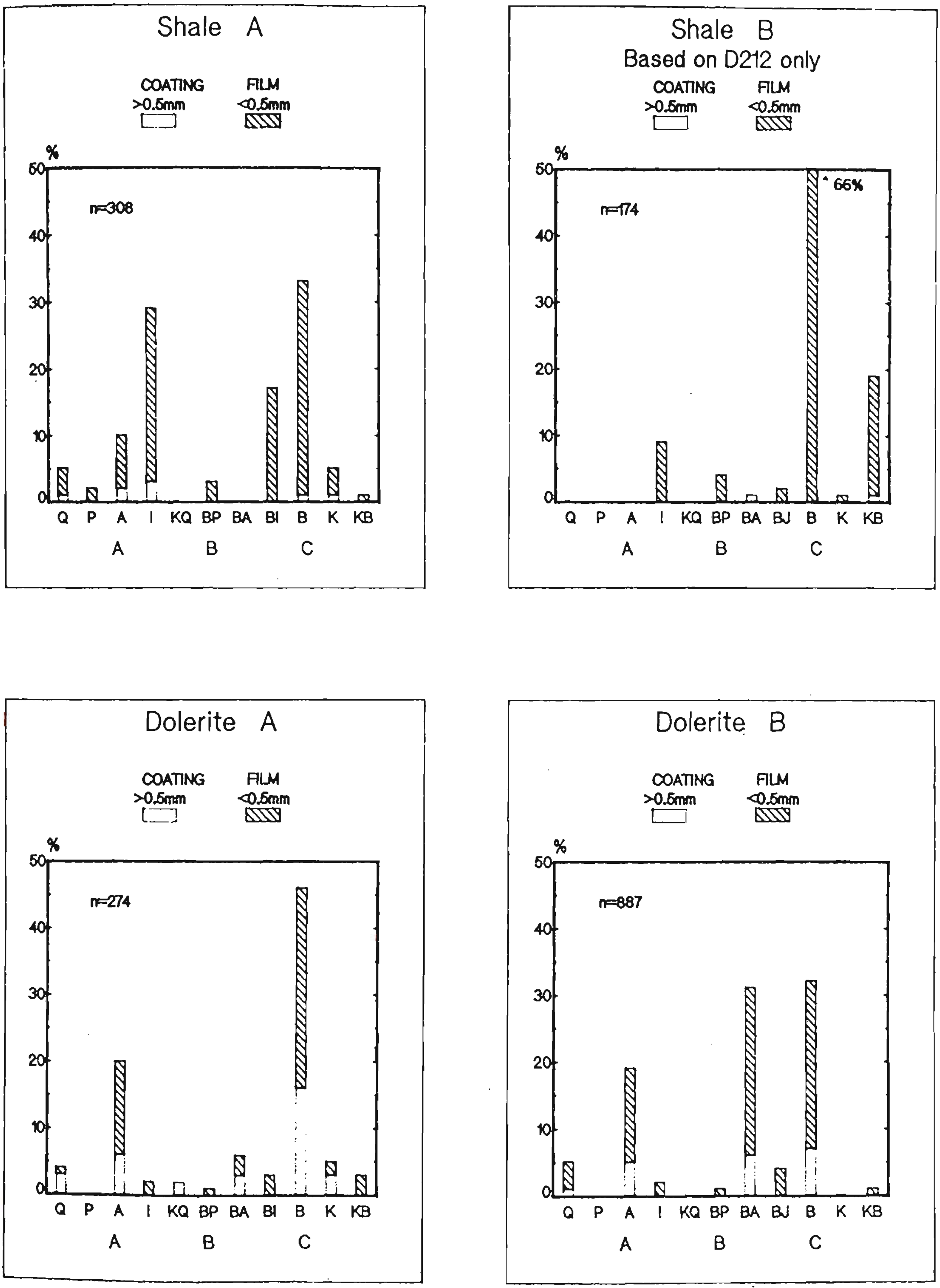
A Planar
B Stepped
C Large Stepped
D Undulating
E Rough Undulating
F Wavy
G Undulating/Curved/Conchadal
H Irregular/Undulating

Profile

K Slickensided
P Polished
S Smooth
R Rough

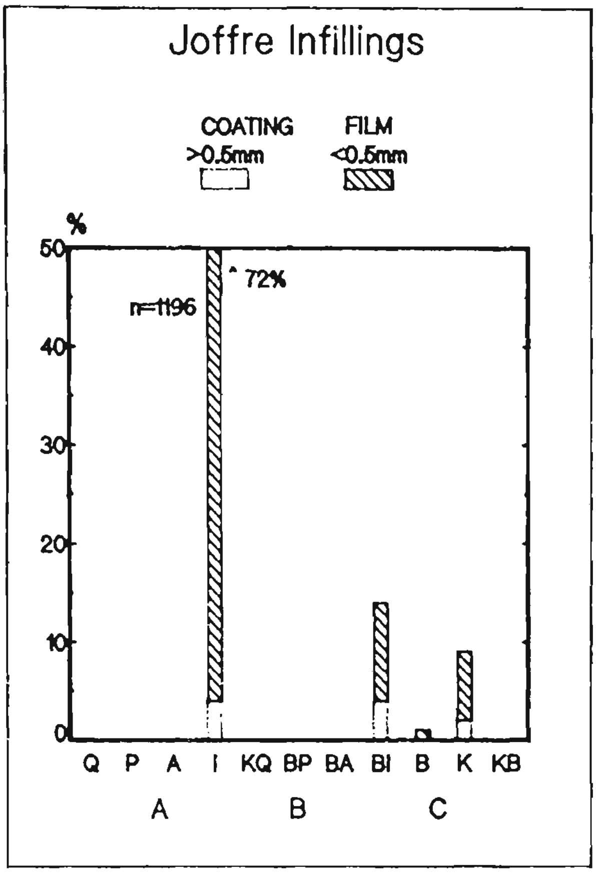
Refer to Fig. for Discontinuity Shape/Roughness Classification

Scale	SUMMARY OF SMALL SCALE ROUGHNESS FOR DISCONTINUITIES IN THE JOFFRE MEMBER	FIGURE 6.21
Drn		



Refer to Fig. 23 for description of infill materials

Scale	SUMMARY OF DISCONTINUITY INFILLING IN THE JEERINAH FORMATION	FIGURE 6.22
Drn		



LEGEND

Q - Quartz (including quartz & iron oxide)
P - Pyrite (including pyrite & iron oxide)
A - Carbonate (including carbonate & iron oxide)
I - Iron Oxide (including manganese oxide)
A

KQ - Clay & quartz
BP - Chlorite & pyrite (including clay & pyrite)
BA - Chlorite & carbonate
BI - Chlorite & iron oxide (including clay & iron oxide)
B

B - Chlorite
K - Clay
KB - Clay & chlorite
C

A - High friction angle infillings
B - High/low friction angle combinations
C - Low friction angle infillings

Scale	SUMMARY OF DISCONTINUITY INFILLING IN THE JOFFRE MEMBER	FIGURE 6.23
Drn		

TABLE 6.1

SUMMARY OF DISCONTINUITY SET ORIENTATION

DISCONTINUITY SET	JOFFRE MEMBER					JEERINAH DOLERITE A				JEERINAH SHALE A		
	S0005 SW005	S0006 SW006	S0007 SW007	S0008 SW008	S0009 SW009	S0010 SW010	S0011/3 SW011/3	SW014/6	SW012	S0015 SW015	S0017 SW017	SW018
Jointing A	-	-	-	-	-	-	-	-	-	030/48*	028/62	013/62
B	075/88	084/78	-	089/79	-	-	-	-	-	267/62*	273/78*	-
C	073/47*	094/57	094/56	-	-	-	-	-	-	-	-	-
D	281/89	284/89	108/78	120/65	101/77	109/61	-	090/88	102/84	296/80	106/88	103/60*
E	-	-	-	-	-	-	306/70*	-	129/85	-	-	-
F	-	-	-	-	-	183/70	-	349/78	163/88	-	-	-
G	-	-	-	-	-	-	-	-	-	173/56	-	165/37
H	-	-	-	-	023/86	-	200/90*	-	-	-	-	-
I	-	-	-	_	-	229/77	230/86	-	040/80	216/73	231/84*	-
Bedding	161/86	345/88	Various	285/25	265/22	-	-	-	-	200/25	205/30	020/85
Cleavage S1	-	179/49*	-	-	-	-	-	-	-	210/47	205/53	222/23
S2	-	-	-	-	-	-	-	-	-	-	-	215/48
	Synclinal Keel			Proximity to Whaleback Fault					Marra Mamba Contact	Shallow limb		Steep limb

Notes:
S - Scanline
SW - Window mapping
* - Minor set
—— - Whaleback Fault Influence

TABLE 6.2

SUMMARY OF MEAN POLE ORIENTATION VALUES FOR JOINT SETS

Joint Set	Stratigraphy					
	Joffre Member		Jeerinah Dolerite A		Jeerinah Shale A	
	Fisher's Constant(1) K	Mean Pole	Fisher's Constant K	Mean Pole	Fisher's Constant K	Mean Pole
A	-	-	-	-	99	019/60
B	51	077/86	-	-	36	272/66
C	28	091/63	-	-	-	-
D	38	105/86	69	101/86	71	292/88
E	-	-	-	-	-	-
F *	-	-	24	169/88	-	-
G	-	-	-	-	48	168/42
H *	48	023/86	28	200/90	-	-
I	-	-	42	229/72	56	216/73

NOTES:

* Data strongly influenced by the Whaleback Fault Zone

(1) Fisher's Constant, K, provides a measure of the clustering of discontinuity sets and is estimated by:

$$K = \frac{N - 1}{N - R}$$

(N = number of observations, R = resultant vector)
If there is no preferred orientation, K is close to unity and if all discontinuities are parallel, K approaches infinity. High K values thus indicate a small scatter.

TABLE 6.3

SUMMARY OF DISCONTINUITY SET ANALYSIS

Stratigraphy	Discontinuity Set	No Poles	Spacing(m) Mean	Spacing(m) S.D.	Persistence(m) Mean	Persistence(m) S.D.	Shape/Roughness (%) (1)	Termination (%) (2)	Waviness i (deg) Mean	Waviness i (deg) S.D.
Joffre Member	Jointing B	78	0.32	1.29	1.85	2.50	PR(58) UR(24) SR(18)	DD(56) XX(21) DX(23)	4.6	2.6
	C	156	0.16	0.31	3.05	3.57	PR(55) UR(28) SR(16)	DX(58) DD(28) XX(15)	3.6	2.5
	D	67	0.22	0.67	1.51	1.61	PR(49) UR(25) SR(22)	XX(39) DX(30) DD(27)	5.6	1.4
	H *	30	0.35	0.68	1.00	0.83	UR(77) PR(23)	DD(63) DX(37)	-	-
	Cleavage	35	0.26	0.31	0.44	0.71	PR(66) UR(26)	DX(71) DD(26)	4.0	1.7
Jeerinah Dolerite A	Jointing D	25	0.52	1.98	2.60	3.59	UR(44) PR(32) SR(24)	DD(40) DX(32) XX(16)	-	-
	F *	27	0.40	1.11	1.51	0.95	UR(74) PR(19)	DX(41) DD(30) XX(11)	-	-
	H *	47	0.72	1.43	3.58	3.45	UR(83) PR(17)	DX(47) XX(36)	8.3	2.8
	I	39	0.90	2.16	3.92	4.67	UR(56) PR(41)	DX(51) XX(21)	5.3	3.9
Jeerinah Shale A	Jointing A	17	0.12	0.61	0.70	0.26	SR(47) PR(29) UR(24)	DD(35) XD(35) XX(24)	-	-
	B	19	0.43	1.20	0.49	0.51	PR(58) UR(42)	DD(90) DX(10)	-	-
	D	51	0.34	0.61	0.55	0.45	PR(90) UR(10)	DD(82) DX(10) XD(8)	-	-
	G	30	0.16	0.29	0.82	0.61	PR(70) UR(30)	DD(66) DX(17) XX(17)	-	-
	I	12	0.10	0.24	0.42	0.27	UR(67) PR(33)	DD(100)	-	-

Notes:
* Data strongly influenced by the Whaleback Fault Zone

1 The shape/roughness term describing medium scale roughness are combined as follows:
   PR, PS, PP, PK - Planer, rough/smooth/polished or slickensided
   SR, SS, SP, SK - Stepped, rough/smooth/polished or slickensided
   UR, US, UP, UK - Undulating, rough/smooth/polished or slickensided

2 Termination describes thes the nature of the discontinuity ends as follows:
   X - Discontinuity extends beyond the exposure
   R - Discontinuity terminates in rock
   D - Discontinuity terminates against other discontinuities

TABLE 6.4

COMPARISON OF JOINTING AND CLEAVAGE WITH FOLDING

Discontinuity Type	Stratigraphy		
	Jeerinah Dolerite A	Jeerinah Shale A	Joffre Member
Jointing:			
Parallel Fold Axis	I	I + A	G
Normal Fold Axis	E	D	B
Shear Joints	D + F	B + G	D
S1 Cleavage	-	290-305 deg generally	250-280 deg
S2 Cleavage	-	290-310 deg	-
Fold Axis Direction Indicated by Jointing	310-315 deg	290-305 deg	Unclear

Table 6.5

SUMMARY OF FRACTURE DATA FOR THE JOFFRE MEMBER AND JEERINAH FORMATION

SECTION (E)	BOREHOLE NUMBER	DISTURBED ZONE WIDTH(m)	JOFFRE FORMATION				JEERINAH FORMATION																			
							DOLERITE A				SHALE A				DOLERITE B				SHALE B				DOLERITE C			
			RQD		FF		RQD		FF		RQD		FF		RQD		FF		RQD		FF		RQD		FF	
			DIST	UNDIST	DIST	UNDIST	DIST	UNDIST	DIST	UNDIST	DIST	UNDIST	DIST	UNDIST	DIST	UNDIST	DIST	UNDIST	DIST	UNDIST	DIST	UNDIST	DIST	UNDIST	DIST	UNDIST
5920	D248	DB4.5	-	40-80	-	5-10	-	-	-	-	-	-	-	-	-	100	-	< 5	-	-	-	-	-			
	D249	DB4.5	-	-	-	-	-	-	-	-	-	-	-	-	60-80	100	5-10	< 5	-	-	-	-	-	-	-	-
6280	D261	DB4	-	-	-	-	-	-	-	-	-	-	-	-	10-80	95-100	5-10	< 5	-	-	-	-	-	-	-	-
6480	D263	SB15	-	0-40	-	5-10	-	-	-	-	-	-	-	-	-	-	-	-	0-40	-	5-20	-	30-65	-	5-10	-
		DC10	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
6720	D190	SB28	15-30	40-80	-	5-20	-	-	-	-	-	-	-	-	-	-	-	-	0-40	40-80	5-15	5-10	-	-	-	-
	D212	SB3, J9	< 5	0-70	< 5	< 10	-	-	-	-	-	-	-	-	-	90-100	-	< 5	25-60	45-95	10-15	5-10	-	-	-	-
6870	D255	DB12	-	40-60	-	5	-	100	-	5	-	40-70	-		30-95	100	5-15	< 5	-	-	-	-	-	-	-	-
	D206	DB3, J2	35	0-99	5	< 10	-	-	-	-	-	-	-	-	90	80-100	< 5	< 3	-	-	-	-	-	-	-	-
7000	D205		-	30-90	-	2-5	-	-	-	-	-	-	-	-	-	100	-	< 3	-	50-100	-	5	-	-	-	-
7040	D191		-	-	-	-	-	95-100	-	< 5	-	30-80	-	5-25	-	-	-	-	-	-	-	-	-	-	-	-
7200	D204	SA17, J7	0-55	0-80	< 8	2-15	-	80-100	-	5	20-70	35-75	5-15	5-10	-	-	-	-	-	-	-	-	-	-	-	-
7320	D192	DB7	-	-	-	-	-	100	-	< 5	-	30-70	-	5-15	60	90-100	5-10	< 5	-	-	-	-	-	-	-	-
	D202	SA12, J2	0-5	25-75	< 5	5-20	-	-	-	-	30-70	70-100	10-15	5-10	-	-	-	-	-	-	-	-	-	-	-	-
7400	D256	DB4.5	-	0-40	-	5-10	-	100	-	< 5	-	40-70	-	7-20	40-70	100	5-15	< 5	-	-	-	-	-	-	-	-
7480	D211	SA10	-	20-85	-	3-13	-	95-100	-	< 5	60-90	-	5-10	-	-	-	-	-	-	-	-	-	-	-	-	-
7560	D251	DA4	-	-	-	-	80	100	5-10	< 5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	D258	DA4	-	0-10	-	0-10	70-80	80-100	5-10	< 5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
7640	D250	DA8	-	20-50	-	5-10	80	100	5-10	< 5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
7720	D257	DA4	-	0-80	-	5-10	80	100	5	< 5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
7800	D252		-	40-50	-	5-10	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
8360	D260	DA7	-	-	-	-	80	95	< 5	< 5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

NOTES;
Disturbed zone width values are estimated from borecore data : DC : Dolerite C; SB : Shale B; DB : Dolerite B; SA : Shale A; DA : Dolerite A; J : Joffre Member.

RQD : Rock Quality Designation
FF : Fracture Frequency (fractures/m)
Dist : Disturbed material adjacent to the Whaleback Fault Zone
South Whaleback Fault for Joffre Formation
North Whaleback Fault for Jeerinah Formation

TABLE 6.6

SUMMARY OF SMALL SCALE DISCONTINUITY ROUGHNESS

Discontinuity Type	Jeerlnah Formation						Joffre Member
	Dolerite A and B		Shale A		Shale B		
	Disturbed	Undisturbed	Disturbed	Undisturbed	Disturbed	Undisturbed	
Bedding	-	-	-	-	-	-	Undulating/rough
Bedding/S2 Cleavage	-	-	Planer or undulating/ polished	Undulating/ rough	Undulating/ smooth	?	
Jointing:							
- parallel fold axis	↑	↑	↑	A + I Undulating/ rough			B Planar, undulating/ rough
- normal fold axis	Undulating/ slickensided to rough	Undulating/ rough, minor smooth	Undulating/ smooth, polished or slickensided	D Planar/rough	?	?	G Planar/rough
- shear joints	↓	↓	↓	B + G Variable			D Planar, undulating/ rough
Faulting		Undulating, Irregular/ Slickensided to rough		Undulating/ slickensided	Undulating/ smooth	?	
	Drill hole data	Drill hole data	Drill hole data	Field data	Drill hole data	Drill hole data	Field data

TABLE 6.7

SUMMARY OF WEATHERING ADJACENT TO THE NORTH WHALEBACK FAULT

Section (E)	Borehole Number	Stratigraphy	Weathering Details	Depth (m) Down FZ	Depth (m) Vertical
5920	D248	Dolerite B	MW/HW - 0.7m SW - 30m (EOH)	180	165
	D249	Dolerite B	None	325	300
6280	D261	Dolerite B	SW - 8m (EOH)	310	290
6480	D263	Dolerite C	MW - 10m	170	155
		Shale B	MW - 22m SW - 24m (EOH)		
6870	D255	Dolerite B	MW - 3m SW - 85m	80	85
7400	D256	Dolerite B	SW - 13m	110	140
		Shale A	SW - 45m		
		Dolerite A	SW - 48m		
7560	258	Dolerite A	MW - 3m SW - 25m (EOH)	110	140
	251	Dolerite A	None	205	230
7640	250	Dolerite A	None	165	180
7720	257	Dolerite A	None	265	275
7800	252	Dolerite A	None	190	225
7920	254	Dolerite A	SW - 5m (EOH)	215	250
8360	260	Dolerite A	MW - 3.5m SW - 10m (EOH)	185	180

Notes:
FZ - Fault Zone
Weathering Grade - F = Fresh
SW = Slightly weathered
MW = Moderately weathered
HW = Highly weathered
EOH - End of hole

All distances quoted are calculated normal to the WFZ

TABLE 6.8

SUMMARY OF JEERINAH FORMATION UNIAXIAL COMPRESSIVE STRENGTH (MPa)

Jeerinah Formation Unit	Section/Borehole Number						
	6480E/D263	D190 6720E/D212	6880E/D255	7000E/D205	7040E/D191	7340E/D192	7400E/D256
Shale B	10 (NWF 7) 15 (NWF 10)						
		42 (NWF 50)		Whaleback Fault Zone			
		58,59 (Contact)					
Dolerite B			34 (NWF 10)	Disturbed Zone		19 (NWF Contact) 50 (NWF 3) 66 (NWF 8)	35 (NWF 5)
		144 (↑10) 127 (↑18)	69,131,140 (NWF 20)		130 60 56 77 94		
			140 (↓15)		126 (↓10)	114 (↓10) 56 (↓ 5)	
Shale A			74,105 (↑20)	67 (10)	37 (↑15) 50 (↑20) 41 75 55	41 (↑10) 76 (↑25) 49 28 (↓ 10)	77,50 (↑5) 19 42
			40 (Contact)			10 (Contact)	
Dolerite A			85,129,196 (Contact) 67 (↑15)			83 (Contact) 92 (↑ 5) 173 (↑ 25) 124 (↑ 35)	37 (Contact) 94,100,117(↑5) 177,119 (↑20)

Note:
Figures in brackets refer to the distance away from the North Whaleback Fault (NWF) or the geological boundary indicated by the arrow

TABLE 6.9

SUMMARY OF ENGINEERING GEOLOGY DATA AND WHALEBACK FAULT INFLUENCE

STRATIGRAPHY	DISCONTINUITY PARAMETER	UNDISTURBED	WHALEBACK FAULT ZONE INFLUENCE
Jeerinah Dolerite	General	Discontinuities as jointing in a well defined and regular system	Significant modification of discontinuity orientation, spacing, roughness, weathering characteristics
	Type	Four steeply-inclined sets identified (parallel fold axis, two shear joint sets and a less well developed set normal fold axis).	Fault-related south-dipping discontinuities dominate, joint set parallel fold axis and north-south shear jointing with decreased dip. Zone width = 15-20m.
	Spacing	Moderately to widely spaced (mean 0.4-0.9m), FF <5, RQD 90-100	Reduced spacing: Dolerite A FF = 5-10, RQD 70-80, Zone Width = 4-8m, Dolerite B FF = 5-15, RQD 10-95, Zone Width = 3-12m
	Persistence	Jointing persistent (mean 1.5-3.9m) with 50-80% extending beyond one or both bench limits	
	Roughness	Jointing irregular/undulating shape and surface roughness small scale always, medium scale on several sets)	Faulting with slickensided, polished or smooth surfaces on a small scale dominates, jointing with undulating shape and small scale surface roughness remains
	Infilling	Coatings form 33% - Dolerite A and 19% - Dolerite B, mean thickness 1.0mm Type C, 1.7mm Types A and B Main types: carbonate (20%), chlorite (Dolerite A - 46%, Dolerite B - 32%) or carbonate and chlorite (Dolerite B - 31%).	
	Weathering	Localised to surface topography Intact rock strength: Dolerite A 118 MPa, Dolerite B 96 MPa	Weathering in the WFZ to a depth of 300m in the Dolerite B and 200m in the Dolerite A, moderate weathering laterally to 3m, slight weathering to 50-80m Dolerite B intact rock strength <65 MPa within 8m of WFZ

TABLE 6.9

SUMMARY OF ENGINEERING GEOLOGY DATA AND WHALEBACK FAULT INFLUENCE

STRATIGRAPHY	DISCONTINUITY PARAMETER	UNDISTURBED	WHALEBACK FAULT ZONE INFLUENCE
Jeerinah Shale	General	Discontinuity system indicates strong structural (folding) influence.	Significant modification of discontinuity orientation, spacing, roughness, weathering characteristics.
	Type	Southern, shallow-dipping fold limbs - bedding, S1 axial plane cleavage and up to five joint sets: parallel fold axis (both north and south dipping), steeply inclined normal fold axis and two shear joint sets (east-west set better developed). Northern, steeply-inclined fold limbs - S1 cleavage, S2 crenulation cleavage and three joint sets: parallel fold axis (north-dipping), normal fold axis (less well developed) and east-west shear jointing.	South-dipping discontinuities dominate, especially S2 cleavage, steeply-inclined normal jointing and east-west shear jointing. Dip expected to increase where drag folding pronounced.
	Spacing	S1 cleavage variable, jointing close to moderately spaced (mean 0.1-0.4m)	Cleavage close to very closely spaced (mean 0.08m). Shale A FF = 5-15, RQD = 20-90, Zone Width = 10-17m, Shale B FF = 5-20, RQD = 0-60, Zone Width = 15-28m
	Persistence	S1 cleavage persistent, occasionally co-planar with bedding on southern limbs, jointing not persistent (mean 0.4-0.8m) with 66-100% terminating against other discontinuites.	
	Roughness	S1 cleavage and jointing variable shape (planar, undulating or stepped) and small scale roughness (jointing rough, cleavage smooth or rough).	Cleavage and faulting with undulating shape and slickensided or polished surfaces. Cleavage becoming increasingly planar.
	Infilling	Mainly as a surface film: iron oxides (Shale A - 29%, Shale B - 9%), chlorite (Shale A - 33%, Shale B - 66%), iron oxides and chlorite (Shale A - 17%) clay and chlorite (Shale B - 19%).	
	Weathering	Localised to surface topography, weathered zone thicker to the west in Shale B. Intact rock strength: Shale A - 52 MPa.	Data limited but moderate weathering to 22m recorded in the Shale B at 6480E, oxidation to 30m at 6720E. Shale A generally unoxidised. Shale B intact rock strength <15 MPa within 10m of WFZ.

TABLE 6.9

SUMMARY OF ENGINEERING GEOLOGY DATA AND WHALEBACK FAULT INFLUENCE

STRATIGRAPHY	DISCONTINUITY PARAMETER	UNDISTURBED	WHALEBACK FAULT ZONE INFLUENCE
Joffre Member	General	Discontinuity system poorly defined and fold relationship indistinct.	Significant modification of discontinuity orientation.
	Type	Only bedding and two joint sets universally present: normal to fold axis and north-south shear jointing. Additional north-south set developed on steep, northern limbs. Axial plane shearing and cleavage occasionally present.	Bedding with localised increased dip due to drag folding. North-south shear jointing present with fault induced jointing in addition. Zone width 30m+.
	Spacing	Jointing close to moderately spaced (mean 0.2-0.4m), bedding moderately spaced (mean 0.28m), cleavage moderately spaced (mean 0.26m)	
	Persistence	Normal and shear jointing persistent (mean 1.5-3.0m) with 44-75% extending beyond one or both bench limits. Axial plane cleavage locally persistent.	
	Roughness	Planar jointing dominates (50-66%) with small scale surface roughness. Cleavage similar. Bedding variable with undulating or planar shape and rough/smooth surfaces.	Fault-induced jointing with undulating shape and small scale surface roughness.
	Infilling	Mainly as surface film: iron oxides dominate (72%), clay and iron oxides (14%), clay (9%). Clay coatings (6%) associated with bedding, mean thickness 3mm, iron oxide coatings (4%) with jointing, mean thickness 1.5mm.	
	Weathering	Variable.	Increased weathering grade in a zone up to 5m wide detected in places. Extensive limonitisation in some areas.

Notes:
FF - Fracture frequency
RQD - Rock Quality Designation
Infilling - Type A includes quartz, pyrite, carbonate, iron oxide
Type B includes combination of Types A and C
Type C includes clay, chlorite

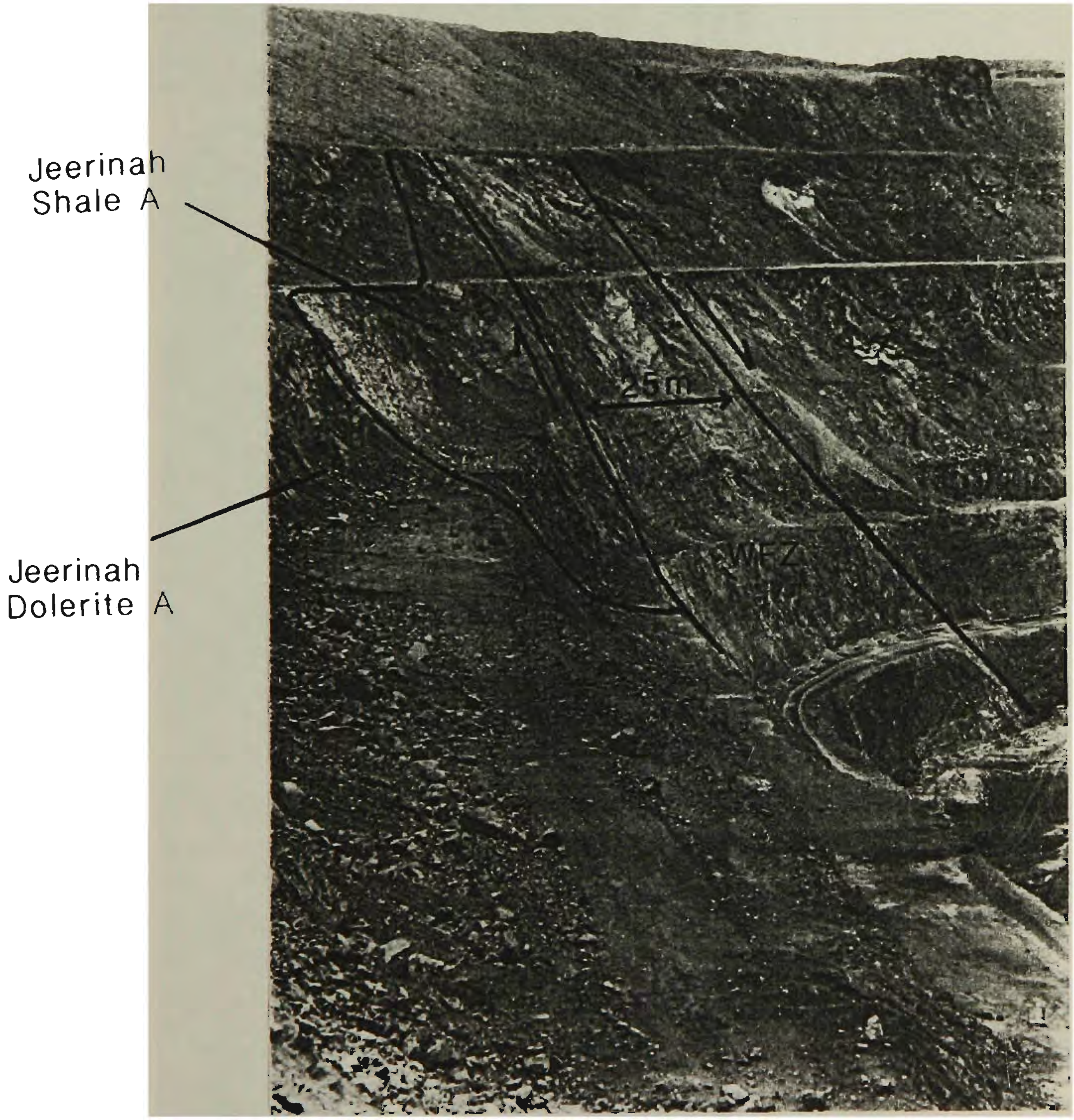


Plate 6.1 Whaleback Fault Zone exposed in the North Wall of the East Pit at approximately 8300E.

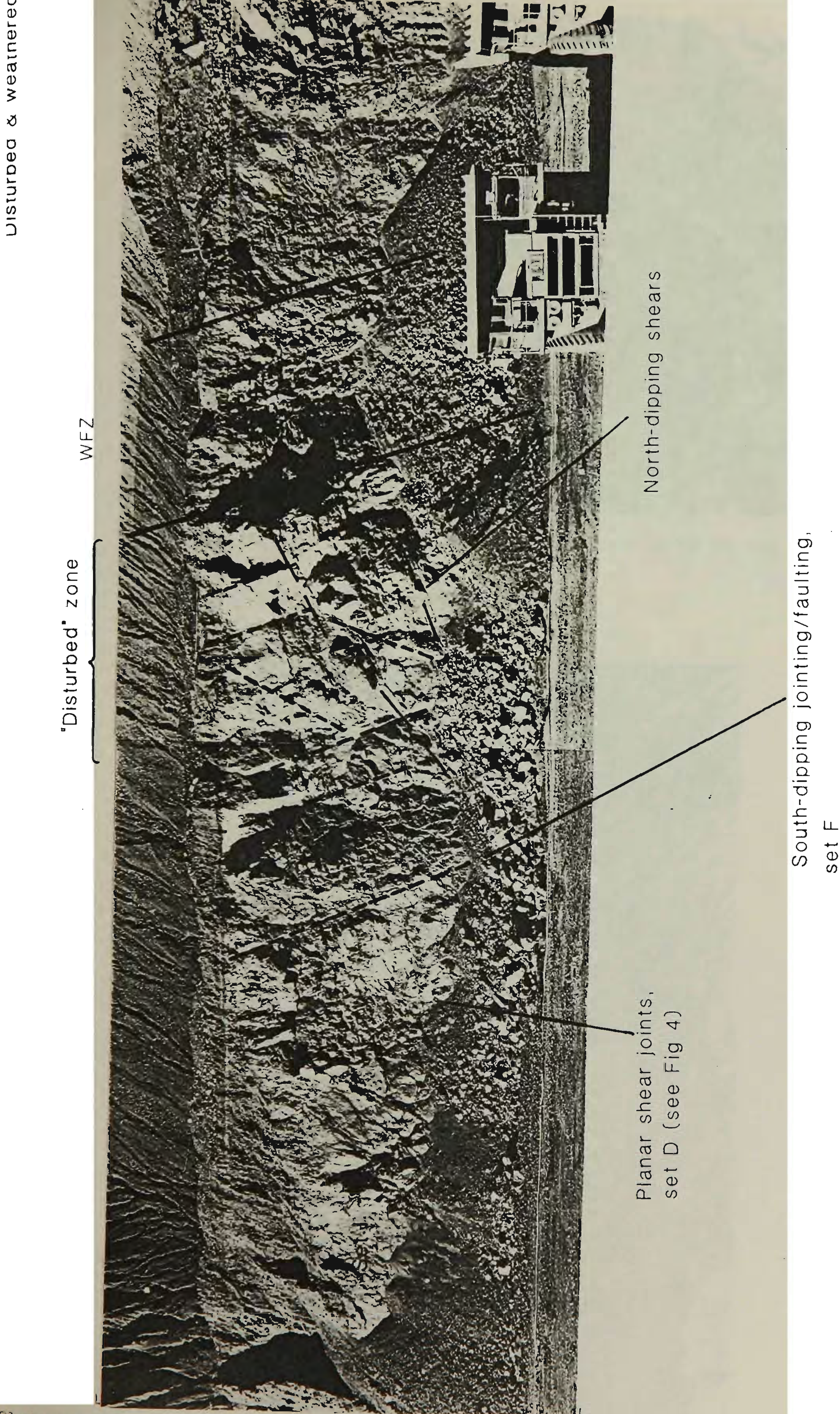


Plate 6.2 Section through the Whaleback Fault Zone and Jeerinah Dolerite A at approximately 7050E (Survey SW016)

Plate 6.3 Planar jointing developed in a flow unit to the south of Mt Whaleback.

Plate 6.4 Planar shear jointing (set D) developed in the ~~Jeerinah~~ Dolerite A at approximately 7050E

Plate 6.5 Widely-spaced jointing developed in the Jeerinah Dolerite A in the East Pit.

Plate 6.6 Fault-related, south-dipping jointing adjacent to the Whaleback Fault Zone.

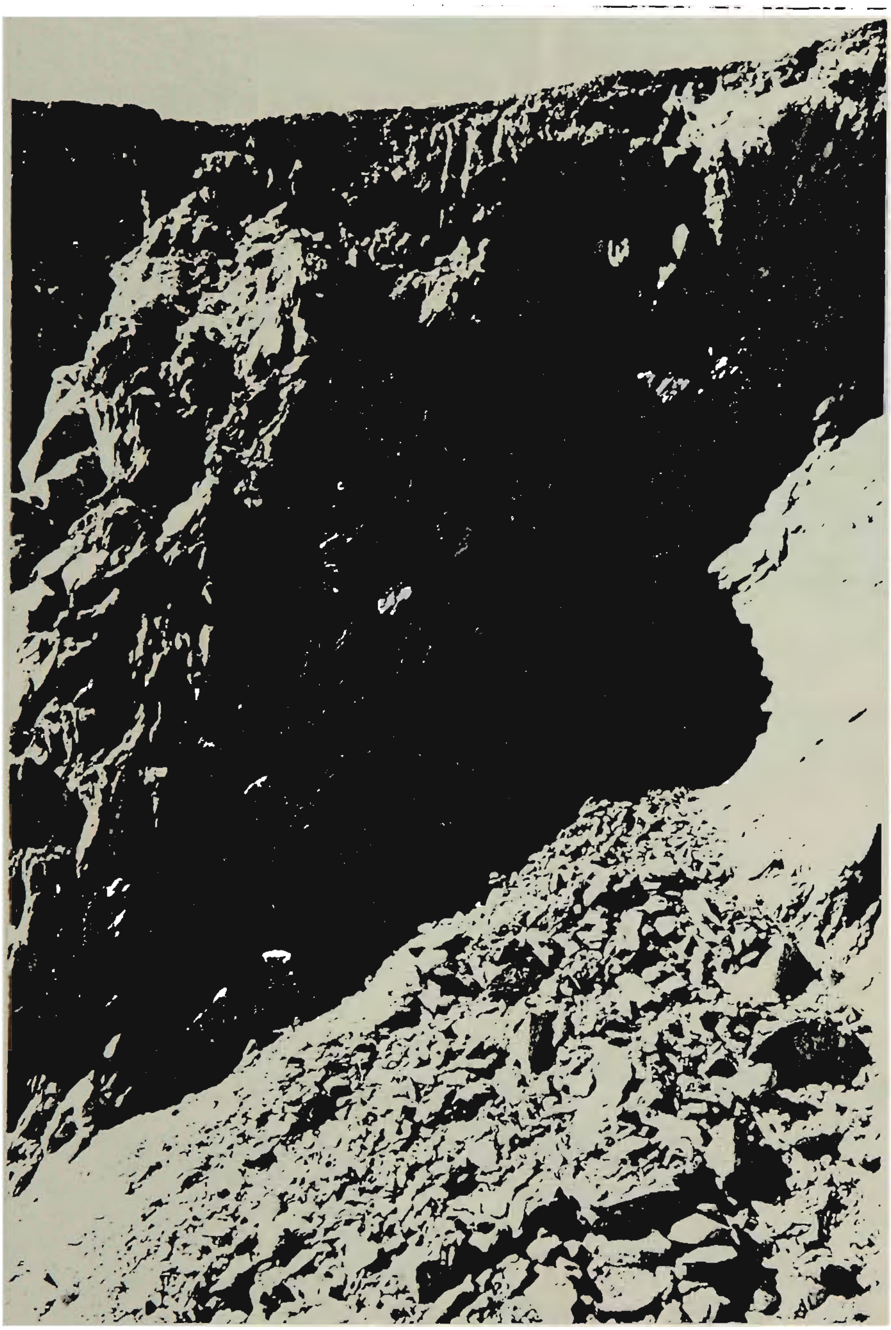


Plate 6.7 North-dipping shearing present in the Jeerinah Dolerite A at approximately 7800E (Survey SW014).

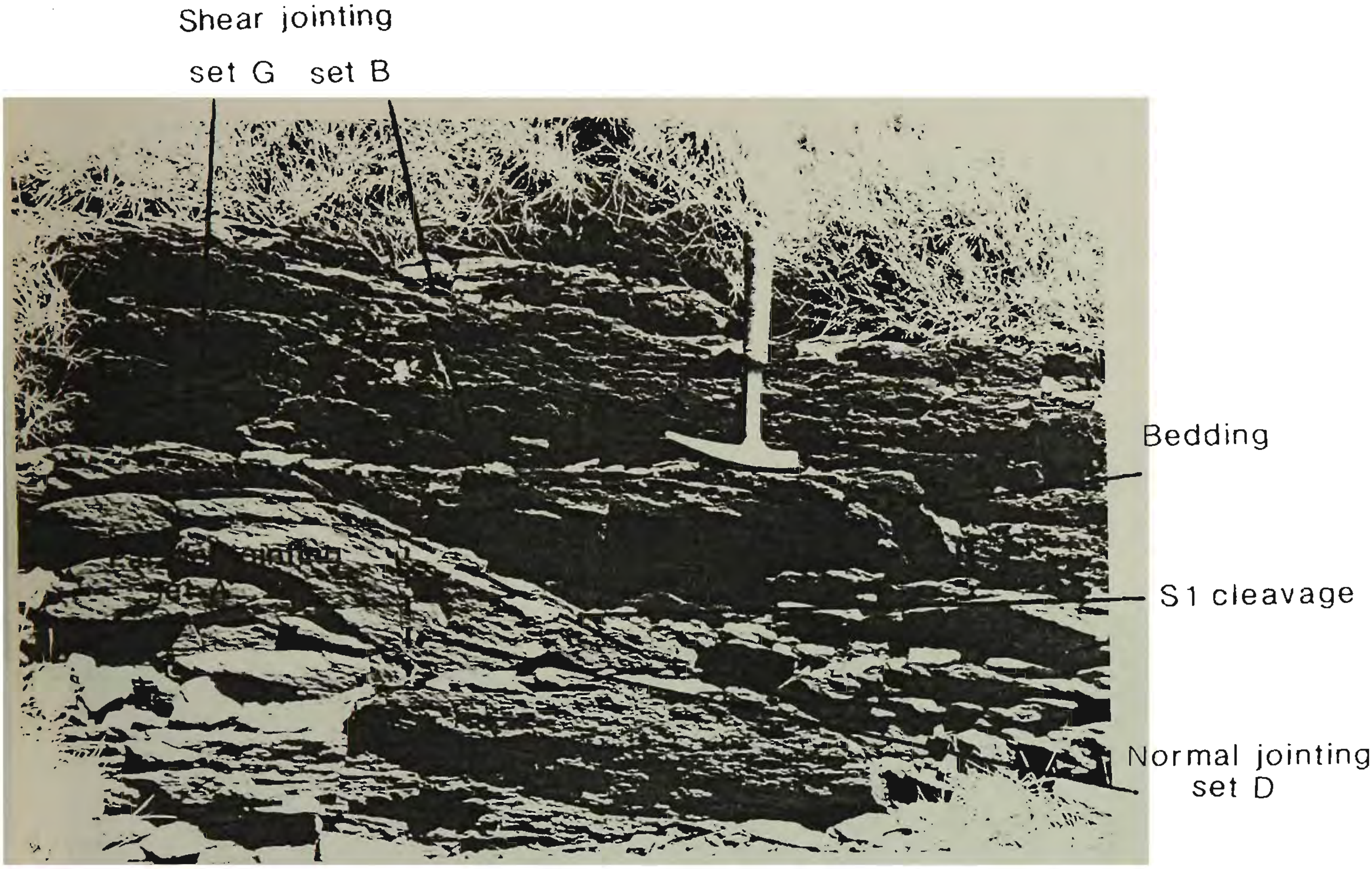


Plate 6.8 Oxidised Jeerinah Shale A on a shallow, south-dipping fold limb to the north of Mt Whaleback (Survey S0015). View to the SSW with joint sets described relative to the fold axis direction.

Plate 6.9 Unoxidised Jeerinah Shale A on a steeply-inclined, north-dipping fold limb (Survey SW018). View to the WSE, parallel to fold axis direction.

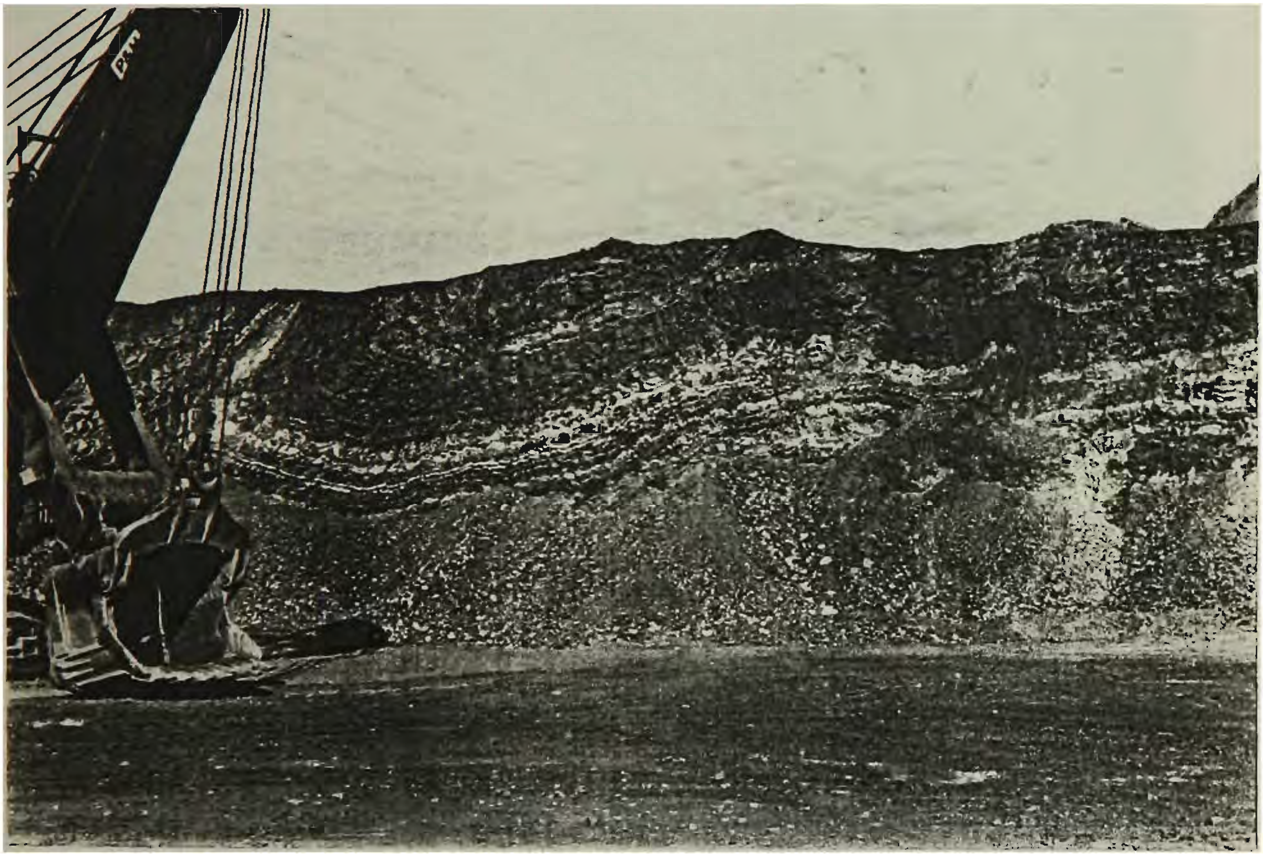
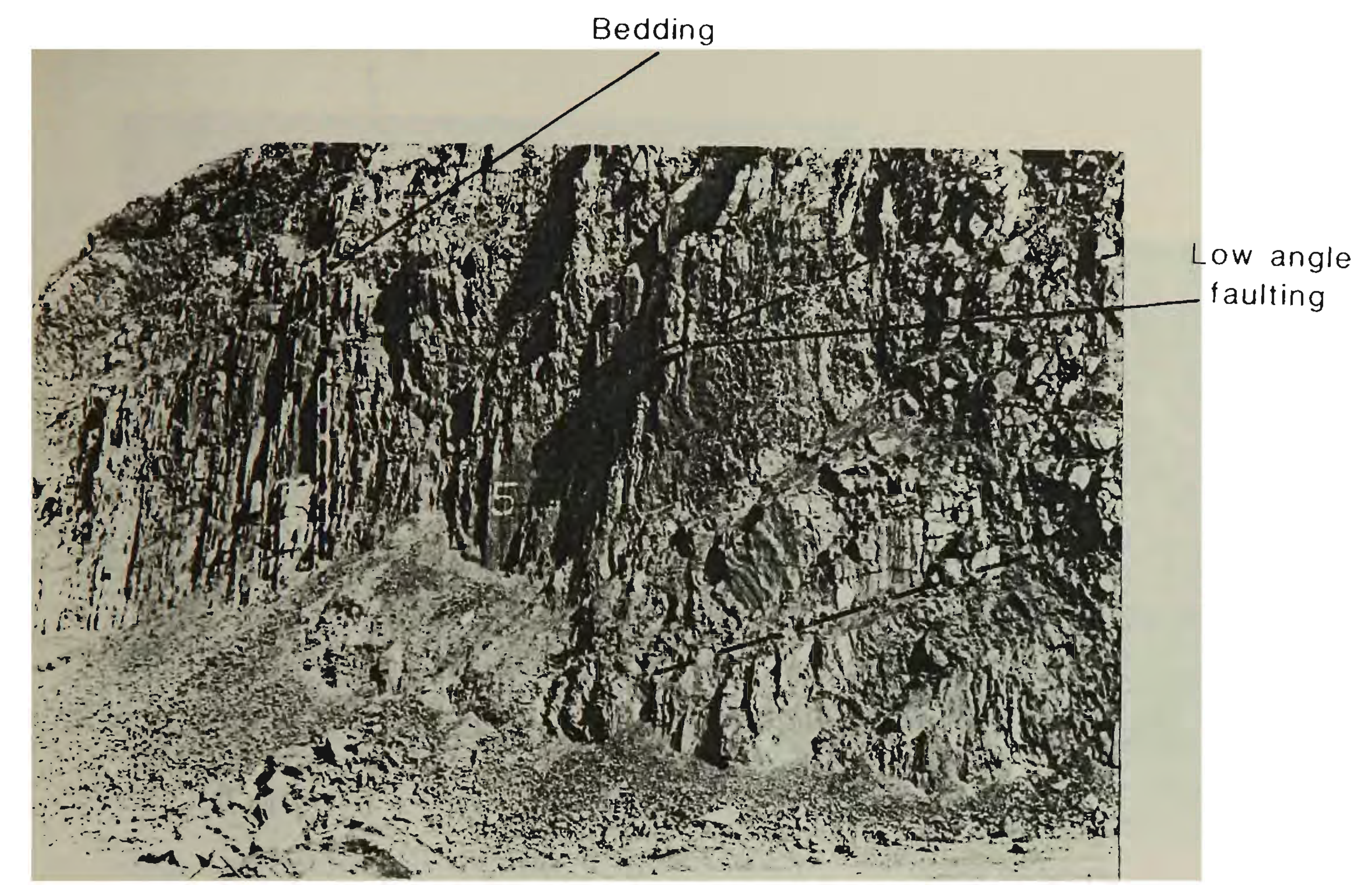


Plate 6.10 Fold style to the south of the Whaleback Fault Zone (Whaleback Shale sequence).
View to the west with the north limb locally overturned.



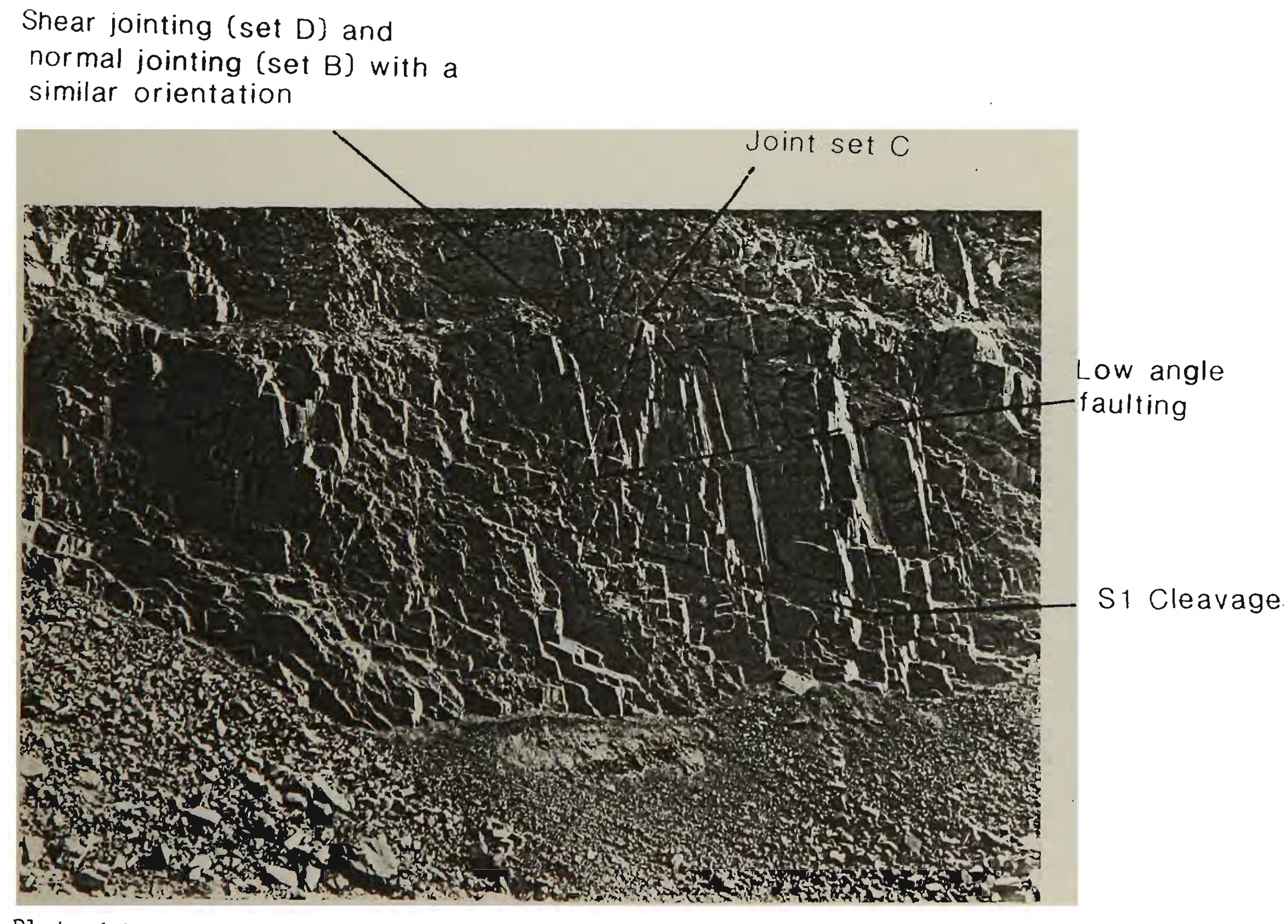


Plate 6.11 Discontinuity system developed in the Joffre Member beyond the influence of the Whaleback Fault Zone (Survey S0005).
Lower view towards the NW.

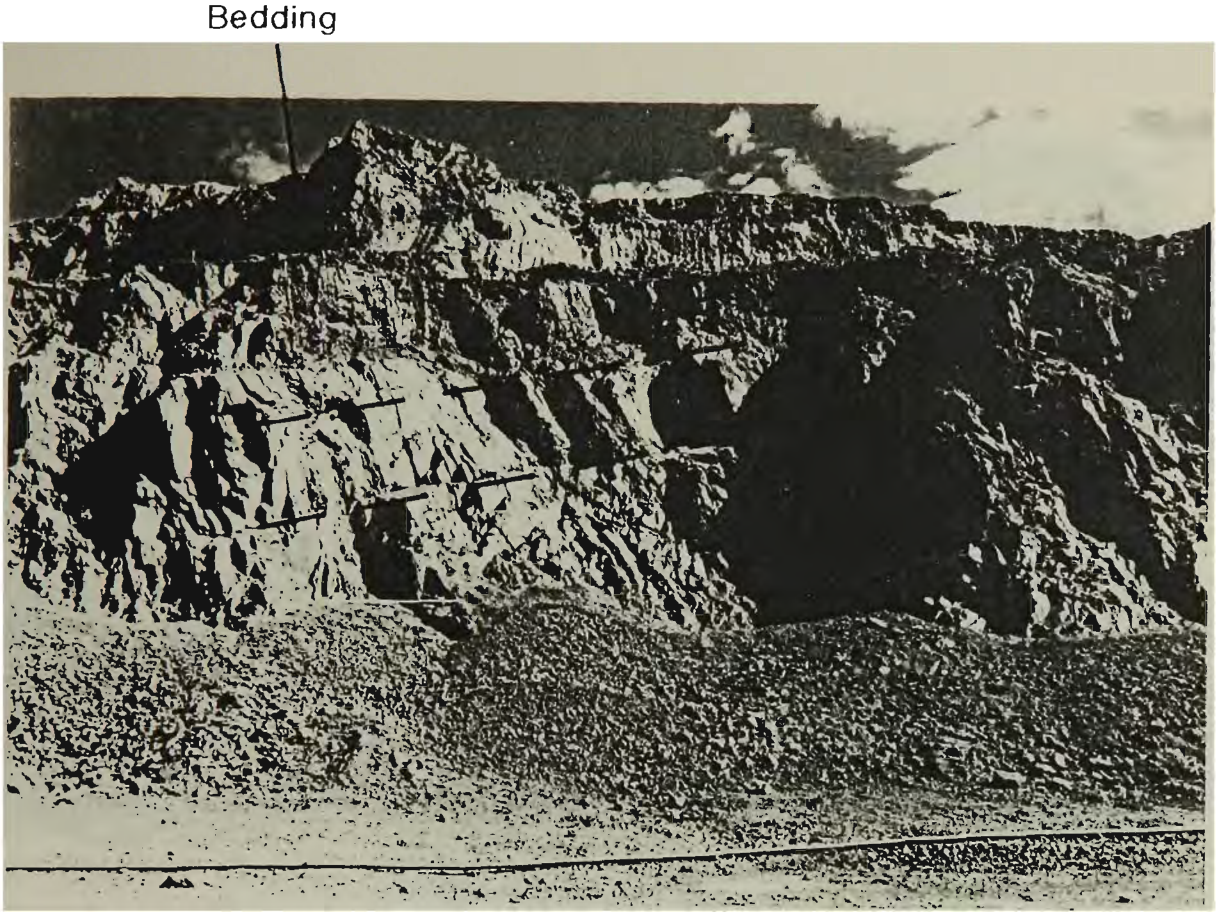


Plate 6.12 Low angle faulting developed on a steeply-inclined, northern fold limb in the Joffre Member (Survey S0006).

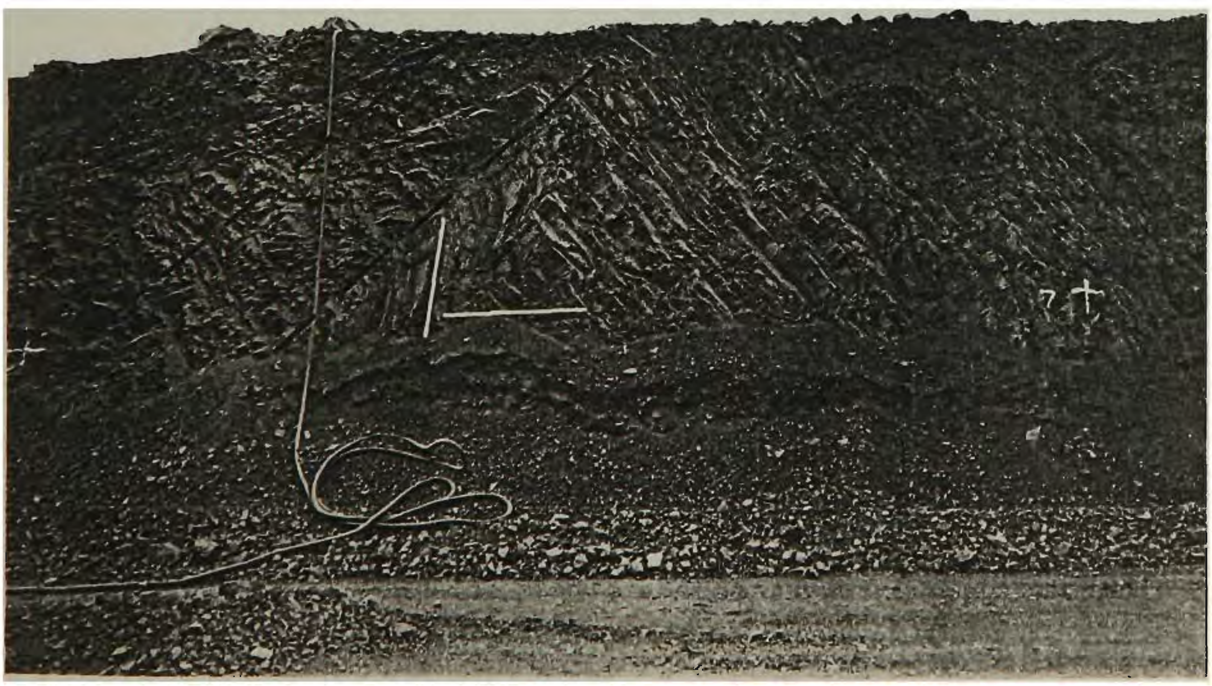


Plate 6.13 Axial plane shearing associated with minor folding in the Joffre Member (Survey S0007).

Plate 6.14 Discontinuity system developed in the "disturbed" zone of the Joffre Member due to the Whaleback Fault (Surveys S0008 and S0009).
Both views to the NW with scanlines located on shallow-dipping, southern fold limbs.

CHAPTER 7

GROUNDWATER AND ITS IMPLICATIONS

7.1 INTRODUCTION

7.2 PREVIOUS WORK

7.3 PIEZOMETER INSTALLATIONS AND PACKER TESTING

7.4 FALLING HEAD TESTS

7.5 PACKER TESTING

7.6 CALCULATED PERMEABILITIES

7.7 CONSIDERATION OF DEPRESSURIZATION OF THE NORTH WALL

7.8 CASE STUDIES

7.9 CONCLUSIONS

7.1 INTRODUCTION

It has been recognised for a long time that groundwater can adversely influence the stability of slopes in a number of ways. This was briefly discussed in Chapter 2, Section 6. Until 1984 all previous mining activity on Mt Whaleback was above the water table which is at Bench 19 (525RL) in the main orebody. Present pit plans indicate that the pit will go to at least Bench 32 in the extreme west of the present study area (5800E) and even deeper further west. Since the present standing water level in the North Wall is between 550 - 575RL, the potential head of water in the North Wall could be as high as 240m in the western part of the study area.

Currently, pit dewatering of the orebody takes place in order to dewater ahead of mining. But the orebody is substantially more permeable than the rocks of the North Wall and this has caused some concern over the difficulty of dewatering the North Wall rocks and the consequent effect on stability (Refs 7.1, 7.2). This has guided the groundwater investigations of the North Wall and the scope of works has been as follows:

- Examination of the nature and extent of existing hydrogeological data;
- Establishment and definition of the aquifers and aquicludes which may effect the pit slope design;
- Examination of percussion and diamond drilling data collected during the geotechnical exploratory drilling programme;
- Determination of hydraulic parameters of different stratigraphic horizons and structural features within the final slope.

These hydraulic parameters were subsequently used in a preliminary design of a depressurization system (Ref 7.2) although this has now been shown to be dependent upon other factors which are described in Chapters 10 and 11. Work conducted as part of this Chapter has enabled a realistic assessment of the effect of groundwater conditions on the stability of the North Wall.

7.2 PREVIOUS WORK

7.2.1 Drilling

Both percussion and diamond drilling has previously taken place within or near the proposed North Wall slope design options. This drilling has principally been used to define the orebody, but has also been used to determine standing water levels. A list of all the boreholes used for water level monitoring on the North Wall is given in Table 7.1 whereas the boreholes which have piezometers installed in them are given in Table 7.2.

As part of these groundwater investigations, additional deep percussion and diamond drilling was completed to provide further geological and geotechnical data on the nature of the materials which lie to the north of the Whaleback Fault (see Chapters 4 and 6). Several of these bores were used for the installation of simple piezometers, and considerable packer testing of the various lithological units was carried out.

The permeability results have been obtained from these holes although the North Wall of Mt Whaleback is the location of the main waste dumps and boreholes can only be sited in 'windows' in the waste. The above factors coupled with the continuing excavation process at Mt Whaleback, have made it difficult to obtain ideal locations for water monitoring bores and hence their widespread distribution as shown in Figure 7.1. Nevertheless reliable groundwater data has been obtained as part of these investigations.

Table 7.1: Current Water Level Monitoring Bores
North Wall Mt Whaleback

BORE	CO-ORDINATES (E/N)	REDUCED LEVEL (m)	PRESENT SWL (m RL)	REMARKS
G947	8205.8/5914.4	595.0	544.1	Piezometer PfjE/S(1)
G684	8195/5875	565.0	534.7	Piezometer PfjE/S(1)
G529	8000/5868	595.6	560.8	Piezometer PfjEa
G709	7559/5702	625.0	558.1	Open Hole PfjSa/Eb
G708	7480/5730	639.2	534.2	Open Hole PfjSa/Eb
G491	7178.2/5759.9	594.7	552.9	Piezometer PfjSa
G496	7179/5691	597.0	571.4	Open Hole PfjEb/Sa
G493	6719/5646	597.0	567.8	Open Hole PfjSb
G553	6400/5522	610.6	566.9	Open Hole PfjEc/Sb
G552	6280/5500	613.2	564.9	Open Hole PfjSb/Eb
G551	6280/5555	596.7	566.2	Piezometer PfjSb/Eb
G694	6199.2/5607.6	591.4	565.8	Open Hole PfjEc/Sb
G695	6196.4/5502.4	610.8	564.4	Open Hole PfjEc/Sb
G1241	8200.6/5858.0	551.8	525.76	Piezometer PfjE/S

NOTES:

PfjEc Jeerinah Dolerite C
PfjSb Jeerinah Shale B
PfjEb Jeerinah Dolerite B
PfjSa Jeerinah Shale A
PfjEa Jeerinah Dolerite A
(1) Whaleback Fault Zone

Table 7.2: Completion Details of Piezometer Installations

BORE HOLE	CO-ORDINATES E/N (m)	RL (m)	TEST INTERVALS (m)	ROCK TYPE
G1422	7680.1/5890.0	588.3	101.6 - 140.2	Pfj - Dolerite A
G1423	7156.2/5778.2	595.7	145 - 164.8	Pfj - Dolerite A /Shale A Contact
			52.3 - 60.0	Pfj - Shale A
G1424	7159.7/5686.8	594.9	190 - 205.5	Pfj - Dolerite A /Shale A Contact
G1427	7156.3/5547.0 60 N	610.0	176.2 - 187.7	Pfj - Shale A
G1502	7320.0/5728.9	639.6	178.8 - 188.8	Pfj - Dolerite A
G1507	7400.5/5779.9	639.7	159.0 - 169.9	Pfj - Dolerite A
			88.0 - 95.0	Pfj - Shale A
G708	7480.0/5730.0	639.2	143.3 - 158.3	Pfj - Dolerite A
G947	8205.8/5914.4	595.0	N/A	
G684	8195/5875	565.0	74 - 78	PfjE/S(1)
G529	8000/5868	595.6	104 - 110	PfjEa
G491	7178.2/5759.9	594.7	156 - 162	PfjSa
G551	6280/5555	596.7	N/A	PfjSb/Eb

NOTES:

PfjSb Jeerinah Shale B
PfjEb Jeerinah Dolerite B
PfjSa Jeerinah Shale A
PfjEa Jeerinah Dolerite A
(1) Whaleback Fault Zone

Field procedures involved drilling either diamond or percussion holes, locating the test horizon from either drill chips or core, confirming the location of the horizon with the geophysical logs, conducting packers tests if appropriate and then installing piezometers.

Piezometers were installed by backfilling the hole to the base of the selected horizon, installing a lower bentonite grout seal, placing the slotted PVC tube over the height of the test horizon, and then installing an upper bentonite seal. This simple method of field installation has proved to be successful for the piezometers installed as part of the groundwater investigations.

7.2.2 **Standing Water Levels**

The bores used to monitor standing groundwater levels are shown in Figure 7.1 and these are presently being monitored on a regular basis (monthly or three monthly) by Mt Newman Mining Co personnel. These standing water levels vary due to mining operations as well as due to variations in groundwater recharge.

It should be pointed out that some previous bores have also been destroyed due to mining operations, but the existing bores do give a good indication of the piezometric surface.

7.2.3 **Piezometric Surface**

Piezometric surface maps are simply contour maps of the piezometric groundwater levels and maps have been constructed for May 1984 and May 1986 in order to show variations with time (Figures 7.2 and 7.3). They indicate that there is a steep hydraulic gradient along the North Wall of the pit which corresponds to the location of the Whaleback Fault Zone (WFZ).

The WFZ is therefore acting as a partial aquiclude to groundwater flow even though the permeability of some of the friable units in the WFZ is high. However the extensive shearing of the WFZ has caused it to have a highly anisotropic permeability. For example, thin clay zones exist throughout the WFZ and are orientated parallel to the fault zone itself and probably create a partial barrier to groundwater flow. In addition, recharge to the dolerites and shales of the Jeerinah Formation from runoff from Mt Whaleback via the talus slope materials may also account for the substantially higher groundwater levels to the north of the fault zone (Figure 7.3).

Comparison of Figure 7.2 and 7.3 clearly shows that there has been a significant fall in the groundwater levels in the orebody itself due to in-pit dewatering. This has created a steep gradient in the piezometric surface and this will continue to develop as mining continues. The maximum level of the groundwater table immediately north of the WFZ is 560RL (see Figure 7.3) and this may be causing some of the damp patches visible on the North Wall on Bench 17 in the East pit.

However localised damp patches are probably also a result of perched water tables within the WFZ as revealed by the standing water level data. These localised perched water tables are shown in Figure 7.4 and are thought to be associated with faulting in the WFZ.

7.3 PIEZOMETER INSTALLATIONS AND PACKER TESTING

A list of the piezometers installed on the North Wall of Mt Whaleback is given in Table 7.2 and it can be seen that most of the piezometers are installed in either the Jeerinah Dolerite or the Jeerinah Shale.

Falling head tests on these piezometers has also been undertaken and these tests show the calculated permeability of the Jeerinah Dolerite and Shale as shown in Table 7.3. These results range from 10^{-8} to 10^{-11} m/s which are very low permeability values.

Table 7.3: Summary of Falling Head Tests

BORE HOLE	TEST INTERVALS (m)	LITHOLOGY	CALCULATED PERMEABILITY (m/sec)	SWL (m RL)
G1422	101.6 - 140.2	Pfj - Dolerite A	3.1×10^{-8}	551.2
G1423	145 - 164.8	Pfj - Dolerite A /Shale A Contact	8.0×10^{-8}	551.6
	52.3 - 60.0	Pfj - Shale A	2.9×10^{-8}	547.5
G1424	190 - 205.5	Pfj - Dolerite A /Shale A Contact	9.0×10^{-9}	566.3 (?)
G1427	176.2 - 187.7	Pfj - Shale A	1.0×10^{-9}	533.7
G1502	178.8 - 188.8	Pfj - Dolerite A	1.2×10^{-10}	542.9
G1507	159.0 - 169.9	Pfj - Dolerite A	1.0×10^{-8}	548.7
	88.0 - 95.0	Pfj - Shale A	1.0×10^{-9}	551.8
G708	143.3 - 158.3	Pfj - Dolerite A	3.4×10^{-11}	554.6
G491	156 - 162	Pfj - Shale A	6.9×10^{-8}	
G529	104 - 110	Pfj - Dolerite A	6.9×10^{-5} (?)	

They can be compared to the permeability results obtained from the packer testing programme. Table 7.4 shows all of the packer testing results for the North Wall. It should be noted that all of the packer tests were undertaken in diamond holes (designated with the prefix D) whereas the piezometers are all in percussion holes (designated with the prefix G). The packer tests carried out during the drilling programmes were either a single packer with a falling head test or a straddle packer with a water pressure test at various pump-in pressures. In some cases both types of test have been carried out in the same hole over the same interval with generally good agreement in the results.

Table 7.4: Summary of Packer Testing Results

Drill Hole	Co-Ordinates	Interval (m)	Stratigraphy	Permeability (m/sec)
D134	7592E/5599N (75°N)	98.7 - 105.1	Phbj J3 (Ore)	2.3×10^{-6}
		121.6 - 126.5	Phbj J3 (Ore)	1.8×10^{-6}
		138.4 - 143.3	Phr/Phs (WFZ)	2.8×10^{-8}
		143.3 - 151.2	Phr/Phs (WFZ)	7.0×10^{-7}
		152.4 - 158.5	Phr/Phs (WFZ)	8.2×10^{-9}
		164.6 - 171.0	Phr/Phs (WFZ)	2.9×10^{-7}
		178.6 - 182.9	Pfj Ea	8.0×10^{-9}
		197.2 - 204.5	Pfj Ea	4.3×10^{-9}
		219.4 - 225.6	Pfj Ea	4.0×10^{-9}
		240.3 - 246.9	Pfj Ea	3.0×10^{-9}
		262.1 - 268.2	Pfj Ea	3.0×10^{-9}
		283.4 - 292.6	Pfj Ea	1.0×10^{-8}
		298.7 - 304.8	Pfj Ea	1.1×10^{-8}
D251	7480E/5400N (73°N)	96.3 - 102.3	Phbj J1 (Ore)	4.2×10^{-8}
		109.5 - 115.5	Phbw	1.4×10^{-8}
		143.7 - 149.7	Phbd/Phr (EFFZ)	-
		150.7 - 156.7	Phr (EFFZ)	-
D252	7328E/5522N	110.0 - 123.0	Phbj J2/Phr (WFZ)	4.0×10^{-8}
		119.0 - 123.0	Phr (WFZ)	-
		134.0 - 141.0	Phr (WFZ)	3.9×10^{-9}
		161.0 - 168.0	Pfj Sa	-
		170.0 - 182.0	Pfj Sa	7.1×10^{-9}
D253	7198E/5419N (78°N)	177.0 - 187.0	Phbj J1 (Ore)	2.7×10^{-7} (min $= 1.8 \times 10^{-7}$) (max $= 3.2 \times 10^{-7}$)
		188.0 - 196.3	Phbj J1/Phbw	2.3×10^{-9}
		209.0 - 223.7	Phr (WFZ)	2.2×10^{-8}
D204	7199E/5470N	113.0 - 119.0	Phbj J2 (Ore)	1.3×10^{-6}
				9.1×10^{-7}(S)
				1.2×10^{-7}(S)
		137.0 - 140.2	Phbj J1 (Ore)	4.8×10^{-7}
				3.2×10^{-7}(S)
				8.0×10^{-7}(S)
		149.0 - 155.3	Phr (WFZ)/Pfj Sa	5.0×10^{-8}
D205	7000E/5487N (81°N)	119.0 - 126.5	Phs (WFZ)	1.4×10^{-8}
		197.0 - 205.0	Pfj Sa	4.2×10^{-10}
D206	6830E/5440N (81°N)	131.0 - 138.6	Phbj J2/Phr (WFZ)	3.3×10^{-8}
D207	7399E/5506N (75°N)	97.0 - 97.2	Phbj J2	7.2×10^{-8}
		113.0 - 120.0	Phbj J1	1.2×10^{-7}
		136.0 - 162.5	Phr (WFZ)	-
D208	6879E/5378N (80°N)	188.0 - 195.0	Phbj J2	1.4×10^{-6}
		218.0 - 223.2	Phr/Phs (WFZ)	3.0×10^{-8}
D209	7000E/5390N (82°N)	197.0 - 203.4	Phbw	6.4×10^{-9}
		241.5 - 247.0	Phr (EFFZ)	7.8×10^{-9}
D210	7194E/5360N (81°N)	182.0 - 187.1	Phbj J1	7.1×10^{-8}
		194.0 - 198.0	Phbw/Phbd	9.6×10^{-9}
		209.0 - 215.5	EFFZ	4.6×10^{-8}
D211	7473E/5566N (80°N)	77.0 - 82.0	Phbj J1	2.6×10^{-9}
		113.0 - 120.0	Phs (WFZ)	1.8×10^{-8}
		137.0 - 141.0	Phs (WFZ)	2.8×10^{-7}
		155.0 - 163.0	Pfj Ea	2.4×10^{-9}
D212	6718E/5435N (79°N)	83.0 - 87.9	Phbj	2.9×10^{-8}
		152.0 - 249.0	Pfj Sb/Eb	2.8×10^{-10}
		242.0 - 249.0	Pfj Eb	-
D213	7399E/5462N (80°N)	83.0 - 91.2	Phbj J2	2.9×10^{-8}
		116.0 - 122.1	Phbj J1	1.0×10^{-7}
		133.0 - 140.5	Phbw	-
		143.0 - 148.2	Phbd	-
		160.0 - 170.3	Phbd	2.0×10^{-7}
		185.0 - 191.0	Phr (EFFZ)	1.1×10^{-7}
D190	6714E/5478N	176.0 - 185.3	Pfj Eb	1.5×10^{-8}(S)
D191	7040E/5610N	81.5 - 103.1	Pfj Eb	4.1×10^{-9}(S)
		174.9 - 187.1	Pfj Sa	3.4×10^{-9}(S)
D192	7338E/5600N (70°N)	216.9 - 229.1	Pfj Ea	1.0×10^{-8}(S)
D230	7639E/5592N)	51.0 - 59.2	Phr (WFZ)	-
		77.5 - 84.4	Phs (WFZ)	1.4×10^{-6}
		114.0 - 120.0	Pfj Eb	-
D231	7561E/5548N)	89.0 - 93.7	Phr (WFZ)	9.7×10^{-8}
		125.0 - 130.6	Pfj Sa/Pfj Ea	3.0×10^{-11}
D232	7800E/5665N	63.0 - 75.8	Phs (WFZ)	2.7×10^{-8}
D234	7920E/5640N	44.0 - 60.4	Phr/Phs (EFFZ/WFZ)	7.1×10^{-8}(?)
		50.0 - 60.4	WFZ/Phs	5.9×10^{-8}
		60.4 - 122.9	Phs/Pfj/Phd (WFZ)	5.0×10^{-9}
		84.0 - 122.9	Pfj/Phd (WFZ)	1.3×10^{-8}
		104.0 - 122.9	Phd (WFZ)	1.2×10^{-7}
D235	6872E/5481N	44.8 - 58.6	Phbj	2.3×10^{-7}
		214.8 - 223.6	Pfj Sa/Pfj Eb	2.6×10^{-8}
		280.0 - 312.9	Pfj Sa/Pfj Ea	6.0×10^{-9}
		290.0 - 312.9	Pfj Ea	9.0×10^{-9}
D236	7400E/5600N	114.0 - 126.7	Pfj Sa	4.0×10^{-9}
D237	7721E/5544N	86.0 - 90.8	Phr (EFFZ)	6.1×10^{-8}
		88.0 - 119.2	Phr/Phs	7.0×10^{-9}
		100.0 - 119.2	Phs	4.0×10^{-9}
		175.0 - 204.5	Phd/WFZ/Pfj Ea	1.0×10^{-9}
D238	7582E/5632N	38.0 - 47.8	Phr (WFZ)	7.8×10^{-8}
D239	6992E/5369N	144.0 - 269.0	Phr/Phs (EFFZ)	3.0×10^{-9}
		190.0 - 269.0	Phr/Phs (EFFZ)	3.0×10^{-9}
		206.0 - 269.0	Phr/Phs	2.0×10^{-9}
D260	8360E/5811N	26.0 - 33.9	Phs (WFZ)	-
		55.8 - 62.2	Phd (WFZ)	-
		75.0 - 93.0	Phd (WFZ)	-
D261	6258E/5177N	203.9 - 219.9	Phbd/Phr	-

Notes:

Phbj	Joffre Member	)
Phbw	Whaleback Shale Member	) Brockman Iron Formation
Phbd	Dales Gorge member	)
Phr	Mt McRae Shale	
Phs	Mt Sylvia Formation	
Pfj Sb	Jeerinah Shale B	)
Pfj Eb	Jeerinah Dolerite B	)
Pfj Sa	Jeerinah Shale A	) Jeerinah Formation
Pfj Ea	Jeerinah Dolerite A	)
Phd	Wittenoom Dolomite	
WFZ	Whaleback Fault Zone	
EFFZ	East Footwall Fault Zone	
(S)	Straddle Packer Test	

The packer test results also show low permeability results for the Jeerinah Formation generally ranging from 10^{-8} to 10^{-9} m/s. It can be seen that units south of the WFZ have much high permeabilities in the range 10^{-6} to 10^{-7} m/s.

7.4 FALLING HEAD TESTS

The field permeability values have been calculated using the method proposed by CANMET (Ref 7.3). The coefficient of permeability is calculated by knowing the time taken for equalization of water pressure conditions in a piezometer as follows:

$$k = 0.133\ S\ r^2 / L \quad \text{m/s}$$

where
- k = permeability, m/s
- r = radius of standpipe, m
- L = length of test interval, m
- S = slope of graph, $\log h_t / h_e$ vs time for one log cycle
- h_t = excess head at time $t = t$
- h_e = excess head at time $t = o$

The test is simply conducted by raising the water level in the standpipe and measuring the time taken to reach the static water level. The permeability values calculated by this method are shown in Table 7.3.

7.5 PACKER TESTING

Packer testing was conducted on selected horizons in diamond holes by using a wireline packer which could be hydraulically inflated once it passed through the end of the drill rods. The drilling mud was flushed out with water prior to the test taking place and so the horizon of interest was defined by the packer at the top and the bottom of the hole. After the test, further drilling proceeded in the normal manner.

7.6 CALCULATED PERMEABILITIES

Both the field work undertaken as part of these groundwater investigations and previous field work has been used to calculate the permeabilities of the different stratigraphic horizons encountered on the North Wall and the summary of results is shown in Table 7.5.

Table 7.5 Stratigraphic Summary of Calculated Permeability

	NO OF TESTS	RANGE OF PERMEABILITY (m/sec)	AVERAGE PERMEABILITY (m/sec)
1. OREBODY UNITS			
Joffre Formation (Phbj)			
J3	-	-	-
J2	8	2.9×10^{-8} - 2.3×10^{-6}	8.9×10^{-7}
J1	6	4.2×10^{-8} - 1×10^{-7}	1.8×10^{-7}
Whaleback Shale Member (Phbw)			
WB	5	2.3×10^{-9} - 1.4×10^{-8}	6.9×10^{-9}
Dales Gorge Member (Phbd)			
BD	1		2×10^{-7}
2. FOOTWALL UNITS			
East Footwall Fault			
EFF	1		4.6×10^{-8}
Mt McRae Shale (Phr)	4	9.7×10^{-8} - 7×10^{-9}	5.5×10^{-8}
Mt Sylvia Formation (Phs)	7	1.4×10^{-6} - 4×10^{-9}	6.2×10^{-8}
Wittenoom Dolomite (Phd)	1		1×10^{-9}
3. WHALEBACK FAULT ZONE			
a) Footwall Units			
Mt McRae Shale	-		-
Mt Sylvia Formation	10	7×10^{-7} - 8.2×10^{-9}	8.8×10^{-8}
Wittenoom Dolomite	2	1.2×10^{-7} - 1.3×10^{-8}	6.6×10^{-7}
b) Jeerinah Formation Units	3	2.7×10^{-8} - 5.0×10^{-9}	2×10^{-8}
4. JEERINAH FORMATION (Pfj)			
Dolerite C	-	-	-
Shale B	-	-	-
Dolerite B	4	3.2×10^{-11} - 1.5×10^{-8}	4.8×10^{-9}
Shale A	9	1×10^{-9} - 2×10^{-7}	4.6×10^{-8}
Dolerite A	21	1×10^{-8} - 3×10^{-11}	1.2×10^{-9}

Taken in isolation, these permeability results can be misleading, and so it is also useful to look at the standing water levels, the structure and the permeability data in order to form a complete picture of the permeability of the North Wall. For example, the permeability of the rock itself is secondary and faults, joints and bedding planes will have the major effect on rock mass permeability. Examination of the diamond drill core also indicates that the primary permeability of the rock is extremely low, and that the secondary permeability will be highly anisotropic and dependent upon the predominant orientation of discontinuities.

7.7 CONSIDERATION OF DEPRESSURIZATION OF THE NORTH WALL

Before addressing the problem of depressurizing pit slopes in the North Wall, it is important to consider if such depressurization is necessary for slope stability considerations.

Present slope stability calculations indicate that all formations south of the WFZ will need to be depressurized whereas the Jeerinah Formation north of the WFZ will not (refer to Chapter 10). However the overall stability of the Jeerinah Formation is not certain and therefore there may still be a requirement to depressurize north of the WFZ.

With the current rate of mining of approximately one bench per year (ie. 15m) depressurization of formations south of the WFZ is occurring by gravity drainage (refer to Figures 7.2 and 7.3). This confirms the permeability data for the Joffre Member of 10^{-6} to 10^{-7} m/s which indicates that gravity drainage will work effectively.

However the permeability data for structures north of the WFZ (ie. 10^{-7} to 10^{-9} m/s) indicate that gravity drainage and simple pumping will not work and that some form of artificial depressurization such as vacuum assist may be necessary.

Detailed analytical and numerical modelling of the likely requirements for pit slope depressurization have been undertaken by Australian Groundwater Consultants (Ref 7.2) and their work indicated that horizontal drain holes 40m long spaced 10m apart on every bench would be the most effective way of depressurizing the Jeerinah Formation. This was based on a bulk permeability of 10^{-9} m/s for the dolerites and 10^{-8} for the shales.

However this proposed depressurization scheme suffers from several major disadvantages as follows:

° The cost of installing such a large number of horizontal drains would be high.

° It would only depressurize a relatively narrow skin of the North Wall immediately adjacent to the pit face even if it were to work successfully. It would do nothing to stabilise potentially deep-seated failures in the Jeerinah Formation.

° There is no guarantee that such a depressurization scheme would work. Conversely strain relaxation due to stress relief may actually increase the permeability such that depressurization occurs naturally anyway.

Given the above, a re-assessment of the feasibility of depressurizing the North Wall was undertaken (Ref 7.2). This work indicated that although a general theoretical approach to the depressurization of large slopes has been given by Brown (Ref 7.4), it is not always universally applicable. Brown relates the coefficient of consolidation, c_v , to the permeability k, and the specific storage S_s as follows:

$$c_v = k/S_s \tag{7.1}$$

where

- k the coefficient of hydraulic conductivity (permeability) is defined as quantity of water which flows through a unit area of material under unit head gradient. It has the units [L/T].

- S_s the specific storage is defined as the volume of water which is produced from a unit volume of saturated material when it is subjected to a unit head reduction. It has the units $[L^{-1}]$.

- c_v is called the coefficient of consolidation in soil mechanics with units $[L^2/T]$.

This relationship indicates that a material with a high permeability and a small storage capacity will be easy to drain. Conversely, a low permeability and a high storage capacity material will be difficult to drain. The coefficient of consolidation, c_v, is essentially a parameter used in the field of soil mechanics and is a measure of the time taken for a change in head to occur in the ground and is therefore useful when considering depressurization of slopes.

If we consider typical values for k and S_s for the North Wall of Mt Whaleback we get: $k = 10^{-8}$ m/s and $S_s = 10^{-5}\ m^{-1}$, and from the above equation we would get $c_v = 10^{-3}\ m^2/s$. Brown (Ref 7.4) quotes rocks with such c_v values as being in the marginal class for depressurization by horizontal drains.

Another factor to consider is the possibility of depressurization by unloading. This effect is common in excavated clay slopes where negative pore water pressures or suction pressures are generated due to the low permeability of the clay and the reduction in overburden stress. However, the c_v values for the North Wall rocks are much too high for this to occur.

In order to determine actual c_v values in the field either vertical holes or horizontal drains could be used. However pump testing in vertical holes is likely to produce very low flows. Flows from horizontal drains are likely to be more effective but the piezometric head should also be monitored in this case. Even though the mass permeability of the North Wall rocks is low, most of the water movement is confined to small joints and discontinuities. Therefore even a very low flow in terms of quantity of water could produce a dramatic effect in terms of depressurization. However the time taken for full depressurization of the slope up to and including the natural phreatic surface may still take several years because of the overall low permeabilities.

7.7.1 Flow Through A Single Fracture

Permeability of jointed rock has been studied by several workers (Refs 7.5, 7.6, 7.7, 7.8). The most commonly accepted equation for flow through a single fracture is based on the theory of flow between two parallel plates. The cubic law, as this equation is frequently called, is

$$q = i e^3 / 12u \tag{7.2}$$

where

- q is the flow rate per unit width of the fracture,
- e is the aperture separating the fracture surfaces,
- u is the absolute viscosity of the fluid, and
- i is the head gradient. This may be related to Darcy's law.

$$q = k\,i\,a = k\,i\,1/n \tag{7.3}$$

where

- a is the width of material which would contain one fracture.
- n is the number of fractures per metre

Thus, from equations 7.2 and 7.3 the relationship between k and n is given by

$$k = n e^3 / 12u \tag{7.4}$$

Taking the viscosity of water as $u = 10^{-7}$ ms, the theoretical relationship between fracture aperture (e), frequency (n) and permeability (k) is shown in Table 7.6.

TABLE 7.6

THEORETICAL APERTURE PERMEABILITY RELATIONSHIP

Frequency	Fracture aperture e (mm)		
n (m^{-1})	$k=10^{-7}$(m/s)	$k=10^{-8}$(m/s)	$k=10^{-9}$(m/s)
0.3	0.074	0.035	0.016
1.0	0.050	0.023	0.010
3.0	0.035	0.016	0.007

Because of the roughness of real joint surfaces, hydraulic flow is not as efficient as that assumed in the parallel plate theory. The actual average joint aperture to produce a given material permeability is likely to be in the range 1.5 to 2.5 times the tabulated values (Ref 7.9). Even after applying this factor, the joint apertures required to produce permeabilities in the range 10^{-9} to 10^{-7} m/s are very small.

The reduction in stress caused by mining reduces the overburden stress which effects both the vertical and horizontal stress components. Discontinuities orientated parallel to the slope will tend to open very slightly in the zone of stress reduction and hence increase the overall permeability. This can be quantified very approximately by using the cubic law, ie. equation 7.2 above.

To demonstrate the effect of overburden removal, it can be assumed that the overburden stress beneath the ground would be 3 MPa for every 100m depth. Further it can be assumed that the normal stiffness on rocks would be approximately 80,000 MN/m^3 in the stress range 1 to 5 MPa, then a stress reduction from 4 to 1 MPa would cause an increase in the average aperture from 0.39mm to 0.43mm (Ref 7.9). According to the cubic law this would be expected to increase the permeability by 34% which is very significant. The actual stiffness and joint apertures for North Wall rocks may be slightly different to those quoted in the example above but nevertheless this still indicates the effect of overburden removal.

7.8 CASE STUDIES

Other open pit mines around the world have also presented significant pit slope depressurization problems in low permeability rocks.

Work by Hoek (Ref 7.10) has shown that Lornex Mine in British Columbia has rock mass permeabilities around 10^{-7} m/s and there has been a great deal of argument on whether the slopes can be drained. Alternative evaluations have been carried out on the merits of drainage adits versus horizontal drains and there has been no clear resolution of this issue on a theoretical basis.

Horizontal drains, up to 300m long have been installed, following a trial drainage program, and the general consensus is that they have resulted in significant depressurization - at least enough to have justified the expenditure and to permit safe mining beneath the slopes. The drainage mechanism appears to be related to the penetration of zones or compartments of higher permeability within a rock mass which, on a global scale, appears to be impermeable. Early borehole pumping tests and permeability measurements on this site were not very encouraging.

Ref 7.11 describes work at Twin Buttes mine in Arizona where depressurization of a clay rich rock slope was carried out by means of an adit with fans of drill holes. The overall permeability of the material was around 10^{-8} m/s. The fans of drill holes are about 120m apart. Depressurization occurred slowly over many months. Beyond the Whaleback Fault Zone the North Wall rocks at Mt Newman are not expected to be clay rich therefore, depressurization is expected to be easier at the Mt Newman mine than at the Twin Buttes mine in Arizona.

The paper by Sharp (Ref 7.12) on the Jeffrey Asbestos mine in Quebec explains how a combination of drainage methods was used to stabilise a major rock slide. It had previously been concluded that drainage of the intact rock would be of little benefit because of its low permeability. However, once the major slide had commenced, a fairly well defined shear zone with high permeability (due to dilation of the rock on shearing) developed. Surface water drainage was introduced to minimise recharge into the slide area. Moreover, horizontal drain holes were drilled and those penetrating the base of the slide produced significant flows. Temporary closure of one of these produced a water pressure of 520 kPa. After three months drainage, closure of the hole produced a pressure of only 35 kPa. It is inferred from these groundwater investigations that drain holes in the correct locations, would be useful in controlling a rock slide if it commenced, even if low permeability made depressurization of the intact rock mass uneconomic.

Hoek (Ref 7.10) also quotes work being undertaken at Fushun West open pit in China, horizontal drain holes are being used to depressurize a slope in shale. The shale foliation dips into the wall at about 20° and the permeabilities values are low across bedding (10^{-8} m/s) but much higher parallel to bedding (3×10^{-6} m/s). Groundwater was close to the crest at each bench, apparently dammed up on shale beds. Horizontal drain holes (actually inclined up to cut through beds) have successfully drained this material.

7.9 CONCLUSIONS

- The permeabilities of the North Wall rocks are low and on an initial assessment would indicate that depressurization would not be practicable. The permeabilities obtained for the Jeerinah Dolerites in particular are extremely low (7×10^{-9} m/s) and it is possible that these results are low because of the falling head test method adopted. Future groundwater work should confirm these results with the rising head method.

- Horizontal drain holes are likely to be useful to depressurize the North Wall rocks. However, if the ratio of permeability to specific storage k/S_s of the rock mass is less than 10^{-4} m^2/s, depressurization will be relatively slow and difficult.

- The exact layout of a horizontal drainage system is dependent upon the size of the potential unstable zone. Considering the potential size of these failure zones as outlined in Chapter 10, it is likely that horizontal drains up to 200m long will be required.

- Horizontal drains are <u>theoretically not feasible</u> because of low permeability of the in-situ rock (the Jeerinah Formation). However, use of such drains may still have beneficial effects because:

  (a) an increase in the aperture of joints aligned approximately parallel to the rock slope may occur due to stress relief,

  (b) the ability to drain will be enhanced on any slide surfaces on which slight shearing accompanied by dilation, has occurred.

- Recharge of groundwater due to surface water infiltration should be prevented by careful surface earthworks to prevent ponding. This is important because any water that gets in, however small, would create further problems of depressurisation of this low permeability rock.

- Groundwater will have a significant effect on the stability of the North Wall and stability analyses should be undertaken assuming no drawdown for the Jeerinah Formation. This would be the worst case at least in the short term. This aspect has in fact been considered in Chapter 10 where the effect of this assumption on overall stability analyses is explained. However it should be re-emphasised that stability is the primary factor determining whether depressurization is necessary or not. High groundwater pressures are not a problem on their own but only in relation to their effect on stability.

- Previous Observation Bores
- ⊙ Piezometer Bores completed during North Wall Study.

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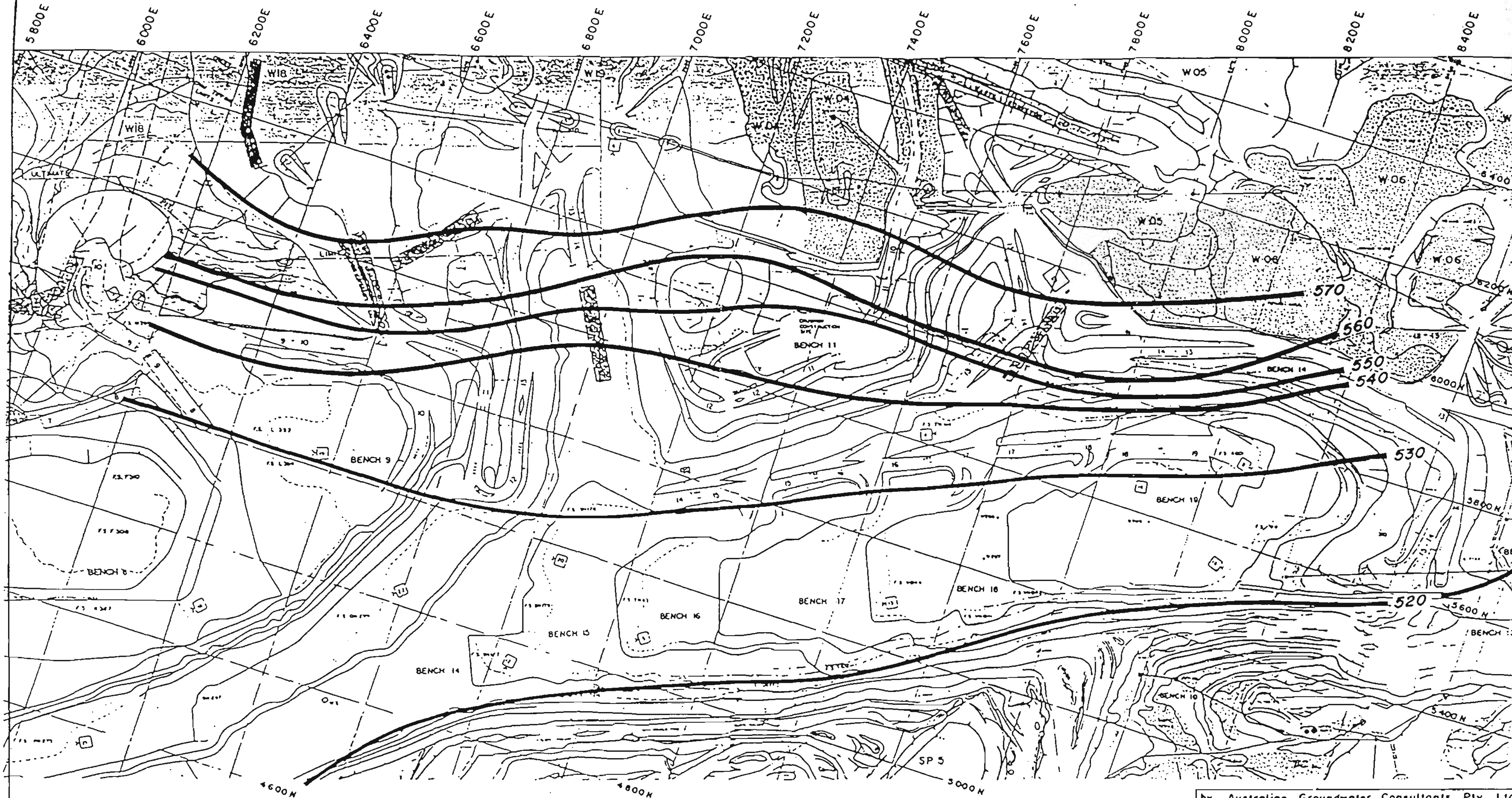
Dwg. No. 1270 - 9
Date October, 1986

Scale 1:7500	BORE LOCATIONS RELEVANT TO NORTH WALL STUDY	FIGURE 7.1
Drn		
Dwg No.		



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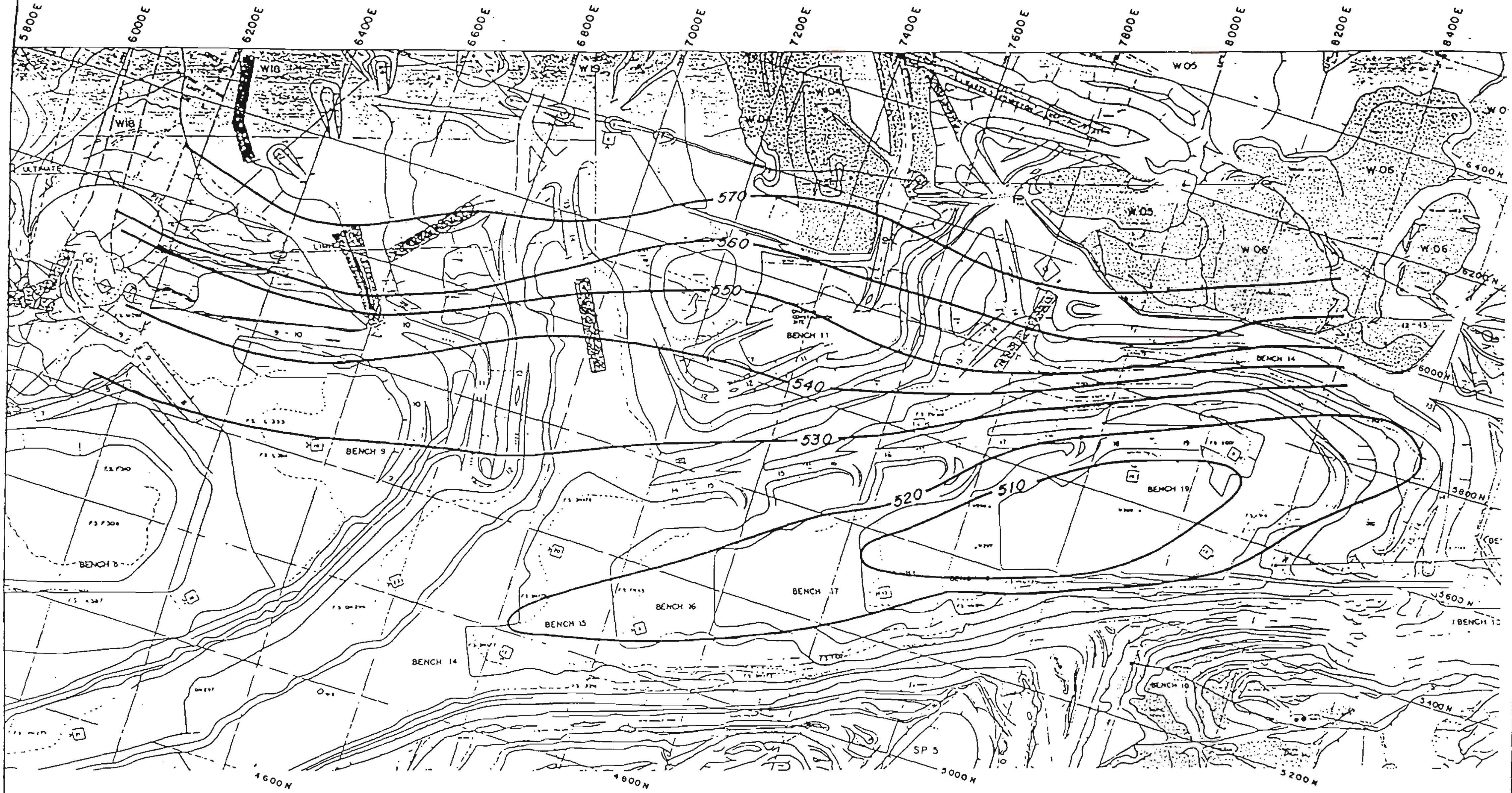
Date October, 1986

Scale 1:7500	MT. WHALEBACK NORTH WALL POTENTIOMETRIC SURFACE MAY 1984	FIGURE 7.2
Drn		
Dwg No.		



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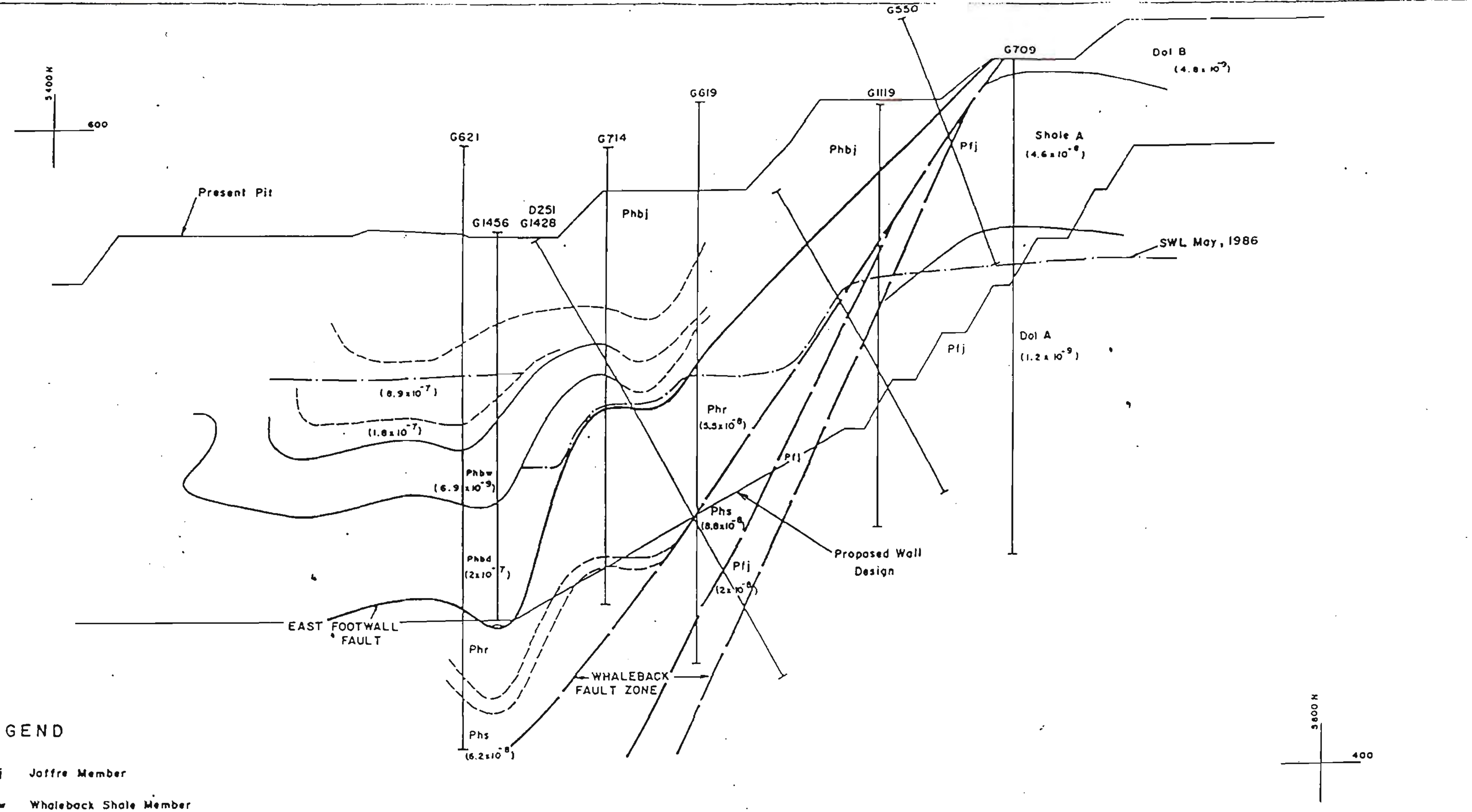
Dwg. No. 1270-12
Date October, 1986

Scale 1:7500	MT. WHALEBACK NORTH WALL POTENTIOMETRIC SURFACE MAY 1986
Drn	
Dwg No.	

FIGURE 7.3



LEGEND

Phbj	Joffre Member
Phbw	Whaleback Shale Member
Phbd	Dales Gorge Member
Phr	Mt. McRae Shale
Phs	Mt. Sylvia Formation
Pfj	Jeerinah Formation
( )	Average Calculated Permeability (m/sec)

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Date October, 1986

Scale 1:1250	GEOLOGICAL INTERPRETATION NORTH WALL SECTION 7560 E	FIGURE 7.4
Drn		
Dwg No.		

CHAPTER 8

PHYSICAL PROPERTIES AND ROCK MASS SHEAR STRENGTH

8.1 INTRODUCTION

Previous test work (Refs 8.2, 8.8, 8.9, 8.13, 8.16) and stability analyses (Ref 8.14) highlighted the problem of potential instability on the North Wall due to the relatively low shear strengths assigned to possible failure surfaces. Specifically, the Jeerinah Shale A and Shale B units dip towards the south behind the Whaleback Fault Zone (WFZ) and the potential for failure along these Jeerinah Shale units became a critical factor in the stability analyses. For large scale overall slope failure, potential failure surfaces have most of their length in Jeerinah Shale and a relatively small proportion of their length in either the WFZ or the Brockman Iron Formation. Hence the shear strength of the Jeerinah Shale became of paramount importance for the determination of the overall stability of the North Wall.

Shear strength values assigned to the Jeerinah Shale in previous work (Refs 8.12, 8.14) were based on :

(a) shear strength test data of both discontinuities and reconstituted material, and

(b) back analyses of previous failures on shale units on Mt Whaleback. This however did not include any back analyses of failures within the Jeerinah Shale unit since no failures have occurred within it.

The above sources of information gave shear strength parameters for the Jeerinah Shale of $c = 40$ kPa, $\emptyset_r = 19°$. Using these strength values it was clearly demonstrated that the stability of the North Wall was marginal at best. Groundwater pressures played a key role in the stability analyses and would have to be decreased dramatically for the North Wall to be even marginally stable.

This conclusion appeared to be very conservative and detailed testing of the weakest zones ('disturbed' zones) within Jeerinah Shale was considered to be most desirable. There are some important points to note in relation to the strength of these 'disturbed' zones as follows:

(a) Laboratory testing of these 'disturbed' zones involved failure of core samples in either uniaxial or triaxial compression. The classical conical failure of a core specimen was rarely developed unless the disturbance was completely ubiquitous. In some cases the samples simply broke apart rather than failing due to a shearing mechanism.

(b) All previous failures on Mt Whaleback have involved relatively low strength materials with failure occurring along pre-existing discontinuities. This has almost always involved failure along bedding at the toe of a slope although failure through a combination of joint sets has occurred higher up along the failure surface. It is almost certain that no failures have occurred through intact rock.

(c) Back analyses of all previous failures have indicated shear strengths considerably lower than those obtained from the present testing of samples of rock material which, in its geological history, has been 'disturbed' (referred to as 'disturbed' samples in all subsequent discussions).

(d) It should also be emphasised that there is a wide variation in strength values obtained for these 'disturbed' samples. This is to be expected since different locations have undergone varying degrees of disturbance. However this does create problems when efforts are made to group data sets together and determine realistic shear strength values. This point is discussed in detail in Section 8.3.

The above points may simply indicate that failure along these 'disturbed' zones in the Jeerinah will not occur, rather than indicate that strength values obtained from laboratory testing of 'disturbed' samples (reported in this Chapter) are an overestimate of the true strength. Nevertheless, these points are important and have led to a cautious approach for pit wall design.

8.2 PREVIOUS TEST WORK

Considerable physical testing of rocks from Mt Newman has taken place over the past 15 years and a comprehensive list of this previous test work with the results is given in Table 8.1.

The initial test work was undertaken by Dames & Moore in 1972 and subsequent work has been undertaken by Golder Associates, Julius Krutschnitt Mineral Research Centre, Dames & Moore and BHP Engineering (Refs 8.2, 8.8, 8.9, 8.13, 8.14, 8.16).

Early work by Dames & Moore in 1972 concentrated on the Jeerinah Formation, Mt McRae shale and the Brockman Formation materials. Further work by Dames & Moore in 1981/82 also concentrated on Mt McRae shales and the Dales Gorge Member shales. Work by the Julius Krutschnitt Mineral Research Centre in 1983 concentrated on the Jeerinah Formation and on the Fault Shale. Golder Associates have tested many different lithologies, including the Jeerinah Formation, Mt McRae shale, Whaleback shale and Fault zone shale.

It can be seen from Table 8.1 that the majority of the early test work was either unconfined compressive strength tests (UCS) or Hoek shear box tests. The UCS test is a useful indicator of rock strength but is relevant only to intact rock samples. Very often it is only the intact competent rock samples that are tested in the UCS test, and not weak rock samples.

On the other hand, Hoek shear box tests are always conducted on existing discontinuities or on sawcut samples. Additionally, the Hoek shear box test can prove to be unreliable in many cases, unless extreme care is taken with the testing procedure and the analysis of the results. (For example, the normal loads used on samples are often too high, the strain rate cannot be controlled easily, and the sample size is often very small in relation to the size of the cast block resulting in sample rotation problems, etc etc.)

The results of this initial test work indicated that it was reasonably straight-forward to determine strengths of discontinuities (although past results have been the subject of some criticism - Ref 8.11); the residual strengths of sheared, remoulded material; and the strength of the intact rock. However it was not easy to determine the rock mass shear strength relevant to pit wall stability and this initial test work was really based on an over-simplified view of rock mass shear strength and its measurement.

8.2.1 Previous Results

A tabulated summary of the previous test results is given in Table 8.1. This Table indicates the lithology, the sample type and the test type as well as the source of the data. It can be seen that there is a wide range in the results particularly for Hoek shear box tests (designated SHC in Table 8.1). The friction angle results obtained are normally in the range of 18 - 30° for Mt Whaleback rocks.

There are of course some exceptions with clays generally having lower friction values (the lowest being 9° for a conventional shear box test on remoulded clay). However for rocks, the range of 18 - 30° appears to be the right order of magnitude.

Nevertheless, some previous shear tests on discontinuity surfaces had fairly low friction angles but high to very high cohesion values (see Table 8.1). In general these values of apparent cohesion quoted for shear tests on joints and bedding are in the order of 100 - 500 kPa.

The validity of these results must be questioned and this data has already been critically examined for the Mt McRae Shale and found to have serious deficiencies (Ref 8.11). In addition, the contributions of sample size, ram friction, high normal stresses and variations in the roughness angle on the sample, have not been considered properly. Thus there are significant uncertainties in regard to these and the results cannot be considered reliable.

Therefore it was considered difficult to select a realistic cohesion value to represent rock mass shear strength based on previous shear strength tests.

The unconfined compressive strength results are also of variable reliability and those tests which looked suspect were excluded from consideration. From the data presented in Table 8.1, there is a considerable number of tests for the Mt McRae Shale (31), the Jeerinah Shale A (10), the Jeerinah Dolerite B (12) and for Fault Shale (31). These results also show a wide range of values, however, reasonable confidence can be placed on only some of them. For example, the Mt McRae Shale and the Jeerinah Shale A have UCS values of approximately 50 MPa, whereas the Dolerites have much higher strengths and range from 80 – 118 MPa. The Fault Shale had a measured UCS value of 10 – 15 MPa, although the actual UCS of the Fault Shale may be considerably lower than this since only reasonably competent samples would have been selected for testing.

It is important to stress here the meaning of the UCS results quoted in Table 8.1. They are only the compressive strength values of the sample selected for testing. The ISRM recommends at least five specimens be tested before one UCS result is quoted in order to overcome the natural variability in rock strength.

However where the sample is 'disturbed' or weathered, there can still be a wide variation in UCS values. For the purposes of determining rock mass strength, it is therefore important to consider the condition of the rock and its compressive strength, as well as its friction angle, or in the case of non-linear failure criteria, its m and s values (see Appendix B).

Table 8.1 also lists m and s values which have been calculated for the Mt McRae Shale and the Joffre Shale. For the Mt McRae Shale the values were described in detail by Hoek (Appendix C) and for the Joffre Shale were described by BHP Engineering (Ref 8.16). These curved failure envelopes were considered to be useful to assess the rock mass shear strength of these particular lithologies and are therefore quoted in Table 8.1.

Finally, it should also be noted that some of the earlier shear box test results did not include the values of apparent cohesion or the non-linear nature of the shear strength envelope for the various defects tested. A single parameter of shear strength (such as a single value of internal friction angle) makes it impossible to determine the actual rock mass shear strength. Although it is appreciated that discontinuities should theoretically have zero cohesion, in practical terms they may have a small value of cohesion. Even if Patton's bi-linear failure criterion is used (Ref 8.1), where two friction angles are quoted, one for use in the low stress region and the other angle used in the higher stress range, a cohesion value should be quoted for the second (high stress) friction angle so that the shape of the failure envelope can be determined. Without this cohesion intercept the point of intersection of the two friction angles cannot be determined. Alternatively, the effective normal stress at which the slope failure envelope changes should be quoted.

Having critically examined the uncertainties and limitations of previous testing methods and test results, a significantly different approach was used to determine the rock mass shear strength of the North Wall. The main strategy was to perform tests on carefully selected specimens of the actual 'disturbed' rock material.

8.3 CURRENT TESTING PROGRAMME

8.3.1 Background

The current testing programme was designed to obtain information about the shear strength characteristics of rocks on the North Wall of Mt Whaleback. Table 8.1 shows that the majority of previous testing on North Wall rocks has been on the most competent materials, ie. the dolerites and the intact shales. The current testing programme was designed to provide information on the weakest materials on the North Wall since these were the most likely to cause stability problems. These materials were either soft, weathered or 'disturbed' rocks, shear zones, weak bedding or cleavage planes and clay horizons. Hence samples of the above materials were taken for testing. Extensive UCS testing of intact rock was not planned since it was considered that failure would not occur through this material.

Previous test work had also indicated that for failure to occur along bedding or along a shear zone in the Jeerinah Formation, residual shear strengths would almost certainly be too conservative, and intact shear strengths would be far too optimistic and not applicable.

Samples of the 'disturbed' rock were therefore obtained from within both the Jeerinah Formation and the Brockman Iron Formation, as it was considered very important to determine the shear strength of the rock mass which was 'disturbed' in its original condition in the field. These samples had various degrees of 'disturbance'. Some for example, had extensive visible fractures, whereas others were highly 'disturbed' and were similar to a loosely compacted breccia.

In addition to this 'disturbance', the Jeerinah Shale exhibits anisotropy with two predominant cleavage directions plus bedding. The condition of the HQ core from these 'disturbed' samples was such that an 'intact' sample of the 'disturbed' material could be obtained, but they could not be cut and prepared in the normal manner. Special core preparation procedures were established following advice from Dr Evert Hoek (Ref 8.17). This involved both careful end-capping techniques and special selection of the testing rate and the confining pressures.

8.2.1 Testing Procedures

The shear strength parameters of a discontinuity are determined by casting a suitable sample in plaster and then shearing in either a Hoek Shear Box or a conventional 300mm shear box. The method of Hoek shear box testing and the subsequent analysis of the results, has been the subject of much debate in the literature (Refs 8.1, 8.3, 8.4 and 8.10). In order to be successful, testing and subsequent analysis of the results has to be undertaken with extreme care. For this reason Hoek shear box tests were not undertaken as part of this testing programme since their relevance to rock mass shear strength is questionable.

The shear testing of discontinuities in a conventional 300mm shear box is not without difficulties, but there is much more control on the strain rate and on the normal stress than in a Hoek box. Additionally the sample can be repeatedly sheared without undue difficulty to obtain residual strength values.

The residual strengths of sheared remoulded material are determined by reconstituting broken material in a conventional 100mm shear box and then repeatedly shearing until residual strength parameters are obtained. Since the material is already broken or crushed, it is simply packed into the shear box prior to testing. Shearing samples in this manner means that there is also a high degree of control on both the strain rate and the normal stress. Moreover, as stated earlier, the sample can be repeatedly sheared to obtain residual strength values. This is the same as for the cast samples in the 300mm box.

This technique is conventionally used in soil mechanics and has been widely used and verified throughout the world (Ref 8.7). However, it can only be used for weak broken materials. It cannot be used for intact rock samples.

The strength of intact rock is determined by testing intact core samples in either uniaxial compression, to determine the unconfined compressive strength, or in triaxial compression to determine the increase in strength with increase in normal stress (triaxial strength). This technique is also in widespread use throughout the world and is the de-facto standard measure of intact rock strength.

The residual shear strength referred to above represent the 'lower bound' strength of a material, whereas the intact shear strength represents the 'upper bound' strength of a material. In other words the strength of any rock could not be lower or higher than these two values. Obviously there is a large difference in strength between this 'lower bound' strength and this 'upper bound' strength. Realistically the actual strength of a rock mass will lie between these two limits.

The difficulty arises in determining the actual rock mass shear strength since it is impossible to simulate all the variables pertaining to field conditions in the laboratory. However an attempt was made to determine the relevant rock mass shear strength for the North Wall by performing special tests on carefully selected samples of 'disturbed' rock.

8.3.2 Scope of the Testing Work

After samples of 'disturbed' rock were carefully prepared in the laboratory, they were tested in both uniaxial and triaxial compression. In addition, large scale shear tests along the discontinuity direction were also carried out. The number and type of tests conducted in this current program are listed in Table 8.2 and it can be seen that most of these tests have been conducted on rocks north of the Whaleback Fault or just south of the Whaleback Fault, at the toe of any proposed slope design.

Tests on Jeerinah Shale comprised approximately 200 specimens prepared for triaxial tests on 'disturbed' samples, six for large 300mm x 300mm shear box tests on sawcut samples, and 36 for remoulded residual shear tests. The number of specimens of each lithology are also summarised in Table 8.2.

It should be noted that the triaxial tests were not conducted in accordance with ISRM Suggested Methods due to the state of the core. There were many core samples which had existing slickensided shear planes through them. This necessitated using specimens which in many cases did not satisfy the requirement for at least a two to one length to diameter ratio.

Furthermore, the shear planes were at quite steep angles to the core, and in order to test the core it was necessary to cap the end surfaces with a high strength, quick setting, non-shrink plaster, thus violating another requirement of the ISRM, viz. that end surfaces should not be capped.

Furthermore, the triaxial tests were generally carried out so that the normal stresses on the specimens would be similar to those in-situ in the North Wall.

Specimen descriptions were checked from bore core logs and lithologies were reviewed from the structural geology sections produced as part of the overall geotechnical programme. The lithologies from previous test programmes were also revised using the new structural geology sections.

8.3.3 Test Results

° General

The samples tested in this current programme were either taken from diamond drill core or from lump samples obtained in the mine from the excavated surface. Complete lists of both diamond and percussion holes relevant to this thesis are given elsewhere (Ref 4.3). The samples used in the testing program were obtained from the diamond drill holes which were part of the North Wall Study (Ref. 8.16).

This current test programme had a wide variation in rock strengths particularly for the 'disturbed' materials. This is to be expected since the degree of disturbance and fracturing is variable within the rock mass. As stated earlier, this variability creates difficulties in trying to assign representative values to the shear strength parameters for use in stability analyses.

A major part of this testing programme involves triaxial testing of rock cores. For the competent materials testing has presented no problems and they have been tested in accordance with ISRM Standards. However for the highly 'disturbed' shales special preparation and testing procedures were necessary as has already been mentioned.

Four examples of typical test specimens are shown in Plates 8.1 to 8.4 for the intact Jeerinah Dolerite, for the intact Jeerinah Shale, for the 'disturbed' Jeerinah Shale and for Fault Shale. It can be seen that the intact shale and dolerite (Plates 8.1 and 8.2) do not require end-capping and fail in the conventional manner. However, the 'disturbed' Jeerinah Shale and the Fault Shale (Plates 8.3 and 8.4) do require end-capping and the failure planes developed are irregular.

The test results from this current programme have been combined with results from previous test work where it could be established that previous test results were reliable. These combined results are produced in Table 8.3 for rock types relevant to the North Wall.

From a casual glance at Table 8.3 it can be seen that there is a wide variation in rock strengths even within the same material type. The variability is dependent upon the type of test conducted. The shear strength in the field depends on the likely mode of failure through that particular rock type and this fact is important for stability analyses (see Chapter 10).

The complete details of results from this current test programme are too numerous to be included here but are given in tabulated and graphical forms in Ref 2.63 (North Wall Geotechnical Study, Volume 6, BHP Engineering) and the reader is referred to it for more detailed information.

The results for specific formations and lithologies are discussed below.

° **Jeerinah Dolerites and Wittenoom Dolomite**

The Jeerinah Dolerites and the Wittenoom Dolomite are shown in Figure D1 through to Figure D5 respectively in Appendix D. It should be noted that the normal stress range on the intact samples is from 0 to 60 MPa whereas the normal stress range on the 'disturbed' samples is from 0 to 30 MPa.

A summary plot of the intact Dolerites and Wittenoom Dolomite is given in Figure 8.1. This figure shows a linear fit to the data and a Hoek-Brown fit on its overlay. The fit to this data is extremely good and the strength of the intact dolerite and the dolomite can be represented by c = 13 MPa, $\emptyset$ = 60°.

The 'disturbed' Dolerite A and Dolerite B results are summarised on Figure 8.2, again in terms of both a linear and a curve fit to the data.

Figures 8.1 and 8.2 show that the intact triaxial strength of Dolerite B and Dolerite A are almost identical, but that the results for the 'disturbed' dolerites are variable as shown in Figure 8.2. They range from c = 6.8 MPa, Ø = 43.5° to c = 15.4 MPa, Ø = 36.5°.

- Jeerinah Shales

The Jeerinah Formation is the most critical rock unit for the stability of the North Wall and as such has been subjected to particular scrutiny in this current test programme. For the Jeerinah Shale A and B there were a total of 42 separate triaxial tests performed on rock samples, six large shear box tests performed on rock lumps cast in plaster, and 15 remoulded direct shear tests. These remoulded direct shear tests were, in most instances, performed on the rock fragments derived from the triaxial tests. In this way a direct comparison was possible between the triaxial strength and the residual strength of the Jeerinah Shales.

The detailed triaxial results for the Jeerinah Shales are given in Figure D6 to Figure D8 in Appendix D.

- **Intact Strength**

  The intact strength of the Jeerinah Shale is shown in detail in Figure D6 in Appendix D and is summarised in Figure 8.2. This data set shown in Figure D6 is quite good and produces a linear fit of c = 15.4 MPa, Ø = 36.5°.

  If we consider this in terms of a curved m and s failure envelope we get m = 3 and s = 1.0 as shown in Figure 8.2. The value for the Uniaxial Compressive Strength (UCS) obtained from this analysis is 64 MPa. The value obtained for m is somewhat lower than normal for an intact argillaceous material.

° **Disturbed Strength**

The intact strength was only determined for the Jeerinah Shale A although a 'disturbed' strength was obtained for both Jeerinah Shale A and Shale B and they are shown in Figure D7 and D8 in Appendix D. The 'disturbed' strengths for the Jeerinah Shale A and B are summarised in Figure 8.3.

Figures D7 and D8 show similar results for a 'disturbed' Jeerinah Shale with Shale A having properties of $c = 2.5$ MPa, $\emptyset = 34.7°$ whereas Shale B has values of $c = 1.6$ MPa, $\emptyset = 39.5°$. Their m and s values are also listed on Figure 8.3 but for all practical purposes these raw data points could be grouped together as will be discussed subsequently.

° **Residual Shear Strength**

Results from the previous test programme conducted by BHP Engineering were included with test results obtained as part of the current test programme. The raw data was then combined to obtain both cohesion and friction angles for the crushed reconstituted shale material. The detailed results for residual shear box tests on Jeerinah Shale are given in Figures E1 and E2 in Appendix E for Shales A and B respectively. These results are: $c_r = 0$, $\emptyset'_r = 31.5°$ for Shale A and $c_r = 12.3$ kPa, $\emptyset'_r = 26.2°$ for Shale B.

It should be noted that the stress range for these residual tests is much lower than for triaxial tests with a maximum normal stress of only 2 MPa. In addition, the data fit for these residual tests on all shales and clays is in general much higher than the fit for triaxial tests on rock samples.

° **Shear Tests on Large Joint Samples**

Shear tests were conducted on cast samples in a 300mm x 300mm shear box. The samples were trimmed, and a sawcut failure plane was produced parallel to cleavage.

The sawcut tests on large blocks are shown in Figures E9 and E10 respectively in Appendix E. The results for large shear box tests on Shale A were $c = 34.8$ kPa, $\emptyset = 29.6°$ and for Shale B $c = 31.6$ kPa, $\emptyset = 37.7°$.

A summary of the residual shear strength of all shales and clays is given in Figure 8.4.

° **Fault Shale**

Material from the fault zone at Mt Whaleback is extremely varied in nature and its original fabric and structure is often completely destroyed. However, in many cases the origin of the material can be recognised. Hence, in past test programmes, fault zone material has often been described as fault shale derived from another lithology such as Mt McRae, Mt Sylvia, Jeerinah, etc. In the current test programme most fault shales were derived from Jeerinah shales and some from Mt McRae shales.

° **Disturbed Strength**

The Fault Shales are shown in Figures D9, D10, D11 and D12 in Appendix D for fault shale derived from Mt McRae, Mt Sylvia, Jeerinah and from unspecified origin respectively. Figures D9 to D12 show the raw data as well as linear and curve failure envelopes. These strength values are summarised in Figure 8.3 and the detailed strength values obtained are:

$c = 0.8$ MPa, $\emptyset = 55.2°$ for the Mt McRae derived fault shale,
$c = 1.3$ MPa, $\emptyset = 39.9°$ for the Jeerinah derived fault shale,
$c = 1.7$ MPa, $\emptyset = 44.3°$ for the Mt Sylvia derived fault shale,
$c = 1.9$ MPa, $\emptyset = 60.7°$ for fault shale of unspecified origin.

Figures D9 and D12 indicate the highly variable nature of the strength of the Whaleback Fault Zone (WFZ). Not only are the cohesion values relatively low compared to rocks on either side of the WFZ, but also their calculated friction angles are extremely high (55° and 60° respectively). Because Figure D9 has an anomalously high friction angle, it has been separated out from the other fault shales and plotted separately in Figure 8.5.

Figure D12 shows that the Mohr's circles at any given normal stress vary enormously, which reflects on the nature of the shale within the fault zone itself. Put simply, there are some comparatively fairly strong rocks within the fault zone as well as extremely weak ones.

However, the data in Figure D12 is so variable that it has been omitted from our assessment of the shear strength of the fault shale (see Appendix A).

° **Fault Shale Residual Strength**

The broken fragments of rock from the triaxial tests on the Fault Shales was also reconstituted and tested in a shear box to determine residual strength parameters. These are shown in detail in Figures E3, E4 and E5 in Appendix E and are summarised on Figure 8.4. The average residual strength of this Fault Shale is $c'_r = 16.7$ kPa, $\emptyset'_r = 26.6°$.

This was almost identical to the residual strength of Jeerinah Shale B and very similar to other residual strength values for shales. This is obviously consistent with the origin of the Fault Shale material.

° **Dales Gorge BIF & Shales**

Triaxial test results of the Dales Gorge BIF are shown in Figure D13 in Appendix D. It can be seen that there is also a high degree of variability with this data. Linear strength parameters are $c = 1.3$ MPa, $\phi = 41°$ with a Hoek-Brown failure envelope being very similar to this at low normal stress levels. This data is grouped together with other 'disturbed' shales in Figure 8.3.

The Dales Gorge BIF and Shales are selected samples of weak material near the Footwall Fault which have been sampled from borecores in this stratigraphic location. Considering the sheared and highly 'disturbed' nature of the rocks in this stratigraphic location the test results are surprisingly good.

° **Residual Strength of Clays**

The detailed residual shear box tests on clays are given in Figures F1 through to Figures F7 in Appendix F. They are also shown in summary in Figure 8.4 as having average properties of $c_r = 6.4$ kPa, $\phi_r = 13.1°$.

Generally the shear strength results for the clays show an excellent fit to the linear strength envelope except for the fault zone Mt McRae derived clays which had a residual friction angle of 14.8°. Clays were also tested from the Jeerinah Shale A ($\phi_r = 9.4°$), from the Mt Sylvia ($\phi_r = 10.2°$), from the Dales Gorge ($\phi_r = 13.3°$), from the Joffre J2 ($\phi_r = 14.3°$), and from the Joffre J3 ($\phi_r = 10.8°$).

8.4 ROCK MASS QUALITY

In addition to the detailed laboratory testing that has been undertaken, the rock mass shear strength has also been assessed on the basis of the 'Rock Mass Quality' or 'Rock Mass Rating' system.

The rock mass rating system was first proposed by Bieniawski (Ref 8.3) and has subsequently been modified by Hoek (Refs 8.5 and 8.6). It is a technique whereby parameters which can be quantified in the field or the laboratory, can be used to quantify the rock mass shear strength which is something that cannot be measured directly.

The rock mass shear strength can conveniently be described by the Hoek–Brown material constants 'm' and 's' which are used in the Hoek–Brown empirical failure criteria as given below:

$$\sigma_1' = \sigma_3' + [\, m\, UCS'\, \sigma_3' + s\, UCS'^2]^{1/2}$$

where:

σ_1'	=	major principal stress
σ_3'	=	minor principal stress
UCS	=	uniaxial compressive strength of the intact rock
m & s	=	empirical constants

They can be rélated to the rock mass rating system by the following expressions:

For **Disturbed** rock masses:

m/m_i = exp [(RMR – 100)/14]

s = exp [RMR – 100/6]

and for **Undisturbed** rock masses:

m/m_i = exp [(RMR – 100)/28]

s = exp [(RMR – 100)/9]

where:

RMR	Rock Mass Rating (Ref 8.3)
m and s	Hoek–Brown material constants
m_i	is the value of m for intact rock.

The above relationships are described more fully in Appendix B. However, the RMR classification scheme includes an assessment of the following engineering geology parameters:

- uniaxial compressive strength of intact rock (UCS)
- rock quality designation (RQD)
- spacing of discontinuities
- condition of discontinuities
- groundwater conditions
- orientation of discontinuities

Values of the above rock mass parameters for both the unweathered Jeerinah Formation, and the weathered Jeerinah adjacent to the WFZ are included in Table 8.4 together with the appropriate ratings indicated by the RMR scheme. This rock mass classification approach produces the following rock mass quality ratings for the Jeerinah Formation:

Lithology	Rating
Unweathered Jeerinah Dolerite	Very good quality
Weathered Jeerinah Dolerite next to WFZ and Unweathered Jeerinah Shale	Fair to good quality
Weathered Jeerinah Shale	Poor quality

The relationships outlined above have been used to determine the Hoek-Brown material constants m and s for both weathered material adjacent to the WFZ and unweathered material. These values are summarised in Table 8.5 together with the values of m for intact rock (mi) determined from laboratory testing.

The relationship between the Rock Mass Rating scheme and Hoek-Brown constants is also shown graphically on Figure 8.6. The calculated m and s values included in Table 8.5 for Dolerite (both weathered and unweathered) and unweathered shale correlate with the curves for undisturbed rock masses shown on Figure 8.6. Significantly, the calculated m and s values for the weathered shale adjacent to the WFZ correlate with the 'disturbed' rock mass curves. This emphasises the overall reduction in rock mass strength of the Jeerinah Shale material adjacent to the WFZ.

It is interesting to compare the m and s values obtained from the RMR system and those obtained from laboratory testing. These are shown in Table 8.6 and it can be seen that the RMR-based values are generally lower than the laboratory results. This is to be expected in the case of intact laboratory samples compared to unweathered field samples since field samples also include the effects of joints. However, for 'disturbed'/weathered samples, the RMR-based values are less different from the laboratory values.

8.5 DISCUSSION

The test results for this current programme relate to materials ranging from extremely strong rocks (the dolerites and dolomites) to extremely weak residual clays. It is, therefore, important not only to know if these test results are realistic but also to determine which material properties will be important in subsequent stability analyses. This latter point will be discussed further in Chapter 10 under Stability Analyses. However, the wide range of strength results obtained from this current testing programme are shown in Figure 8.7 which shows a combined plot of all test data.

8.5.1 Summary of Test Results

Figure 8.7 shows that some different material types have very similar strength properties. For example, the intact Dolerite A, the intact Dolerite B and the intact Wittenoom Dolomite have almost identical shear strength properties. Similar comments also apply to some of the 'disturbed' shale samples. Therefore for all practical purposes, this shear strength data can be grouped together into six different strength values for six different major material types. This is probably within the limits of the accuracy of the testing programme anyway. These six strength types are shown in Figure 8.8 and their grouped average shear strengths are as follows:

- Intact Dolerites and Dolomites — $c = 13$ MPa, $\phi = 60°$
- Mt McRae Fault Shale ('Disturbed') — $c = 0.8$ MPa, $\phi = 55°$
- Intact Jeerinah Shale & 'Disturbed' Dolerite — $c = 13$ MPa, $\phi = 40°$
- 'Disturbed' Jeerinah, Dales Gorge & Mt Sylvia Fault Shales (lower bound) — $c = 1.3$ MPa, $\phi = 39°$
- Residual Strengths of Jeerinah and Fault Shale — $c = 0$ MPa, $\phi = 30°$
- Residual Clays — $c = 0$ MPa, $\phi = 13°$

8.5.2 The Importance of the Disturbed Jeerinah Shale

The findings of this testing programme are very significant. If consideration is given to the relative importance of the results listed above, and how they may contribute to potential instability of the North Wall, it can be concluded that the 'disturbed' Jeerinah Shale and Fault Shales, are likely to be the materials of greatest concern.

The average shear strength of these shales is c = 2 MPa, Ø = 39° as determined from the current test programme. Using these strength values in any stability analysis it can be concluded that large scale failure within the Jeerinah will not occur. In fact, stability of slopes is reasonably assured for all cross sections under most reasonable groundwater conditions. Even if the lower bound strength for these shales is used (ie. c = 1.3 MPa, Ø = 39°) the slopes within the Jeerinah Shale are still likely to remain stable. This conclusion refers to only overall stability. However local stability in the buttress zone is in question as explained in Chapter 10 (Section 10.6.4).

These points will be discussed further in the Stability Analysis undertaken as part of Chapter 10, but for this discussion it is important to consider how much reliability can be placed on the strength parameters of c = 1.3 MPa, Ø = 39°.

Figure 8.3 shows a combined plot of all the 'disturbed' shales. This plot shows both the linear c and Ø plots as well as m and s plots to this data. It can be seen that for the stress range of 0 to 10 MPa the lower bound strength results quoted above are applicable either for c and Ø or m and s values. This is crucial for subsequent stability analysis.

Therefore the lower bound strengths for both the 'disturbed' Jeerinah Shale and the Fault Shale are c = 1.3 MPa, Ø = 39°. These results are significantly higher than the strengths used in the stability analysis for the North Wall in Ref 8.14 (ie. approximately c = 40 kPa, Ø = 19°). Moreover, the raw data points indicate that lower strength values than this could not be assumed even if an alternative strength criterion was used (for example a bi-linear strength criterion after Patton (Ref 8.1).

8.5.3 Comparison with Back Analyses Results

The shear strength parameters determined from back analyses of previous failures on Mt Whaleback are shown in Table 8.7 and are also discussed in the next Chapter. These all indicate fairly low shear strengths at the time of overall failure. The first three Formation Contacts which are shown in Table 8.7 are not normally encountered on the North Wall (unless possibly in the Fault Zone). The contact of the East Footwall Fault is also found adjacent to the Whaleback Fault Zone. It is important to point out that back calculated shear strength parameters of $c = 8$ kPa, $\emptyset = 30°$ were determined for this contact. During this present work the values of residual strength parameters for both the Jeerinah Shale and the Fault Shale were found to be $c = 0$ kPa, $\emptyset = 30°$. It is interesting that these test results are very close to those determined from back analysis of failures and, therefore, could represent the actual field shear strength parameters. However, in other locations the use of residual shear strength parameters may be too conservative as stated previously.

8.5.4 Concluding Remarks

The strengths of the other materials on the North Wall are not so significant as the 'disturbed' Jeerinah and the Fault Shales as described above. The weakest materials, the residual clays and residual Jeerinah Shales, occupy a relatively small percentage of any failure plane, whereas the strongest materials, the intact shales and dolerites, are so strong that failure will not occur through them.

The strength of the North Wall rocks has also been evaluated on the basis of their engineering geology properties and a rock mass quality rating has been established. This rock mass quality rating indicates that the m and s values are lower than those determined from laboratory results of intact samples. It should be emphasised that one would expect this to be the case when laboratory test results are for intact. In contrast, the tests on 'disturbed' laboratory samples give values of m and s which are less different from RMR-based values.

Although the rock mass rating is not a direct measure of rock strength, it does act as a useful guide. This is significant for the subsequent wall design where a cautious approach has been adopted.

8.6 CONCLUSIONS

Following the detailed laboratory test programme as well as an evaluation of existing strength data for North Wall rocks, the conclusions which can be drawn from the results are presented in Figure 8.8. They can be summarised as follows:

1 The shear strength data can conveniently be represented by a Mohr–Coulomb criterion. There appears no advantage in using a Hoek–Brown criterion particularly at the stress ranges encountered on the North Wall.

2 The data sets derived from the test programme are generally very good and can be described accurately by the shear strength parameters proposed. However, there are some variations to this particularly for the 'disturbed' and fault shale materials.

3 The intact strength of the Jeerinah Dolomites as well as the Wittenoom Dolomite is very strong. The test results show a remarkable similarity and they can be grouped together as the same data set. Their strength is c = 13 MPa, Ø = 60°.

4 The intact Jeerinah Shale and the 'disturbed' dolerites can conveniently be grouped together in the same data set. This can be represented by average strength values of c = 13 MPa, Ø = 40°.

5 The 'disturbed' Jeerinah Shales, the Fault Shale material and the Dales Gorge BIF and Shales can also be grouped together, and the lower bound strength of this group is c = 1.3 MPa, Ø = 39°. The strength of the Fault Shale in this group is for failure through the Fault Shale rather than for failure along any existing shear surface or discontinuity. The shear strength of this group is the most significant for the whole North Wall design.

6 Fault Shale derived from Mt McRae Shale surprisingly showed a higher strength than other fault shales and has been grouped separately with shear strength parameters of c = 0.8 MPa, $\emptyset$ = 55°. This apparent high strength is probably due to hard bands caught up in the fault zone.

7 Residual shear strength tests were also undertaken on the Jeerinah Shales and Fault Shales, and these can be conveniently represented by $c_r = 0$, $\emptyset_r = 30°$. This would be for failure along the direction of shearing in the WFZ, the East Footwall Fault or along a fault in the Jeerinah Shale itself.

8 A range of clays from various stratigraphic zones has also been tested and these have residual friction angles which range from 9° to 15°. Their average properties are $c_r = 0$, $\emptyset_r = 13°$.

9 Comparison of these strength values with a recent failure on the East Footwall Fault indicates that the back analyses results obtained from it (c = 8 kPa, $\emptyset$ = 30°) are almost identical to the strength parameters described in Point 7 above.

10 An evaluation of the engineering geology parameters for disturbed Jeerinah Shale and Fault Shale, indicates a lower rock mass strength than that based on relevant laboratory tests (c = 1.3MPa, $\emptyset$ = 39°).

11 The strengths obtained from the triaxial testing of these 'disturbed' shale samples (Point 5) are higher than was intuitively expected on the basis of experience. Accordingly, a cautious approach should be adopted for the North Wall design.

8.7 RECOMMENDATIONS

The following recommendations are made in relation to the shear strength of rocks on the North Wall.

1 The shear strength data presented in this Chapter is generally reliable and further additional detailed laboratory testing is not recommended. There are no new laboratory tests that can quantify rock mass strength in any greater detail than those performed in this testing programme.

2 However there are some new geophysical techniques becoming available which may be able to give a better indication of rock mass shear strength and it is recommended that these be considered for use on the North Wall. Discussion of these techniques for their use is, of course, outside the scope of this thesis.

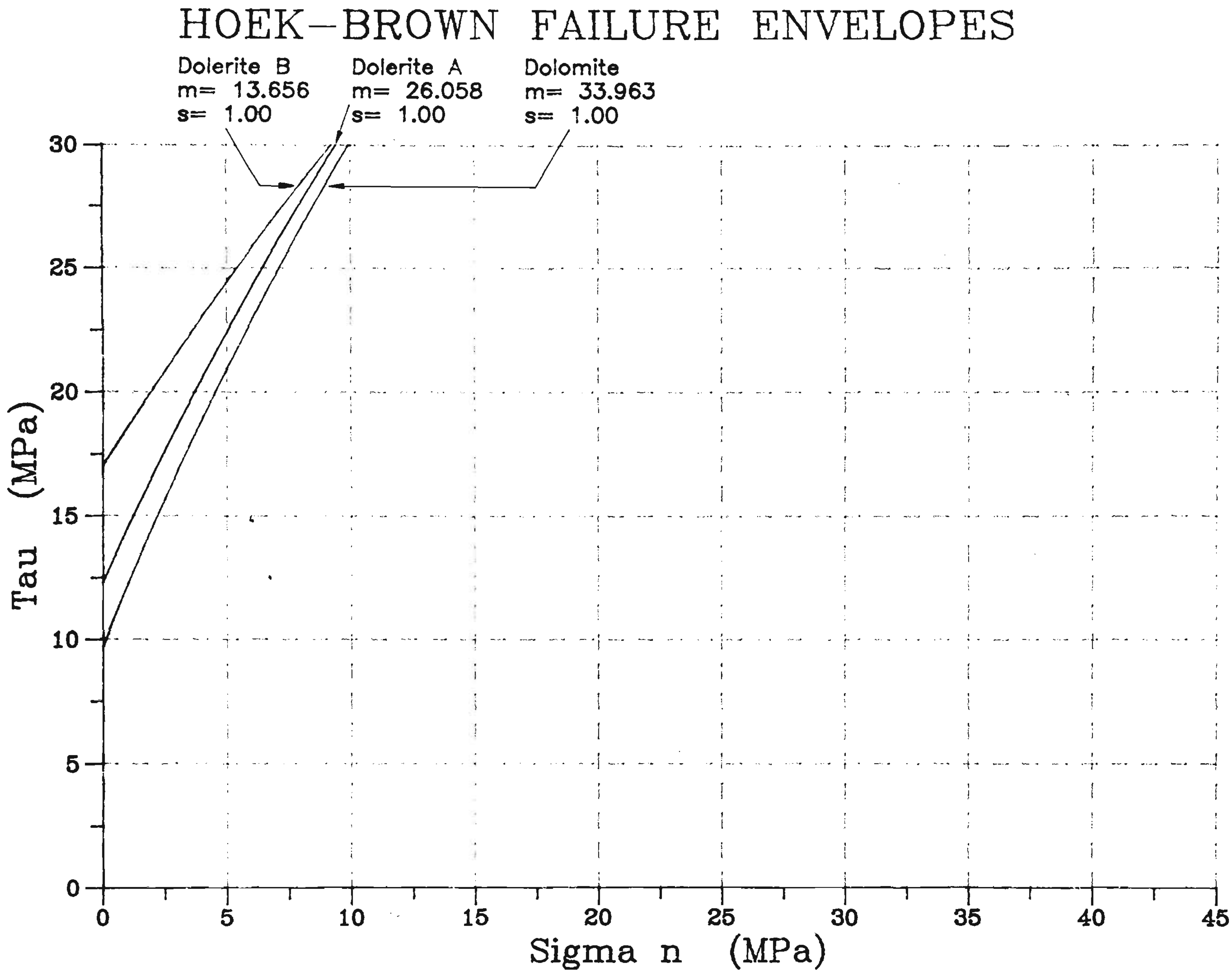
3 It is also strongly recommended that an induced failure in the pit be tried in the Jeerinah Shale. There are several areas in the pit where this trial could take place which are above the main in-dipping Jeerinah Shale sequence. Hence information could be obtained before a large scale wall was exposed in the Jeerinah Shale. Even if failure does not occur it would provide useful information on threshold strength values.

4 The strength results from this Chapter can be used in the stability analyses on the North Wall. For critical zones, lower bound shear strengths have normally been recommended and anomalously high values (eg. Mt McRae Fault Shale) should not be used.

5 The strength of all intact rocks on the North Wall is such that large scale failure will not occur through them. Failure will only occur either through 'disturbed' zones, fault zones or along discontinuities. Stability analyses reported in Chapter 10 have, therefore, been based on failure mechanisms which take account of these realities and are consistent with the structural geology.

6 It is important that the mechanism of any potential failure be considered and that the appropriate shear strength values be applied to it. For example, the same lithology will often have anisotropic strength characteristics and this has been taken into consideration in the stability analyses where appropriate.

7 It is obvious that the most important shear strength results are those for the 'disturbed' Jeerinah Shale and for the Fault Shales. Therefore any new pit failures on or along these materials should be back analysed to determine in-situ strength values which will further enhance the confidence in the shear strength parameters to be used for future stability analyses.


HOEK-BROWN FAILURE ENVELOPES
Dolerite B
m= 13.656
s= 1.00
Dolerite A
m= 26.058
s= 1.00
Dolomite
m= 33.963
s= 1.00
Tau (MPa)
0
5
10
15
20
25
30
35
40
45
Sigma n (MPa)

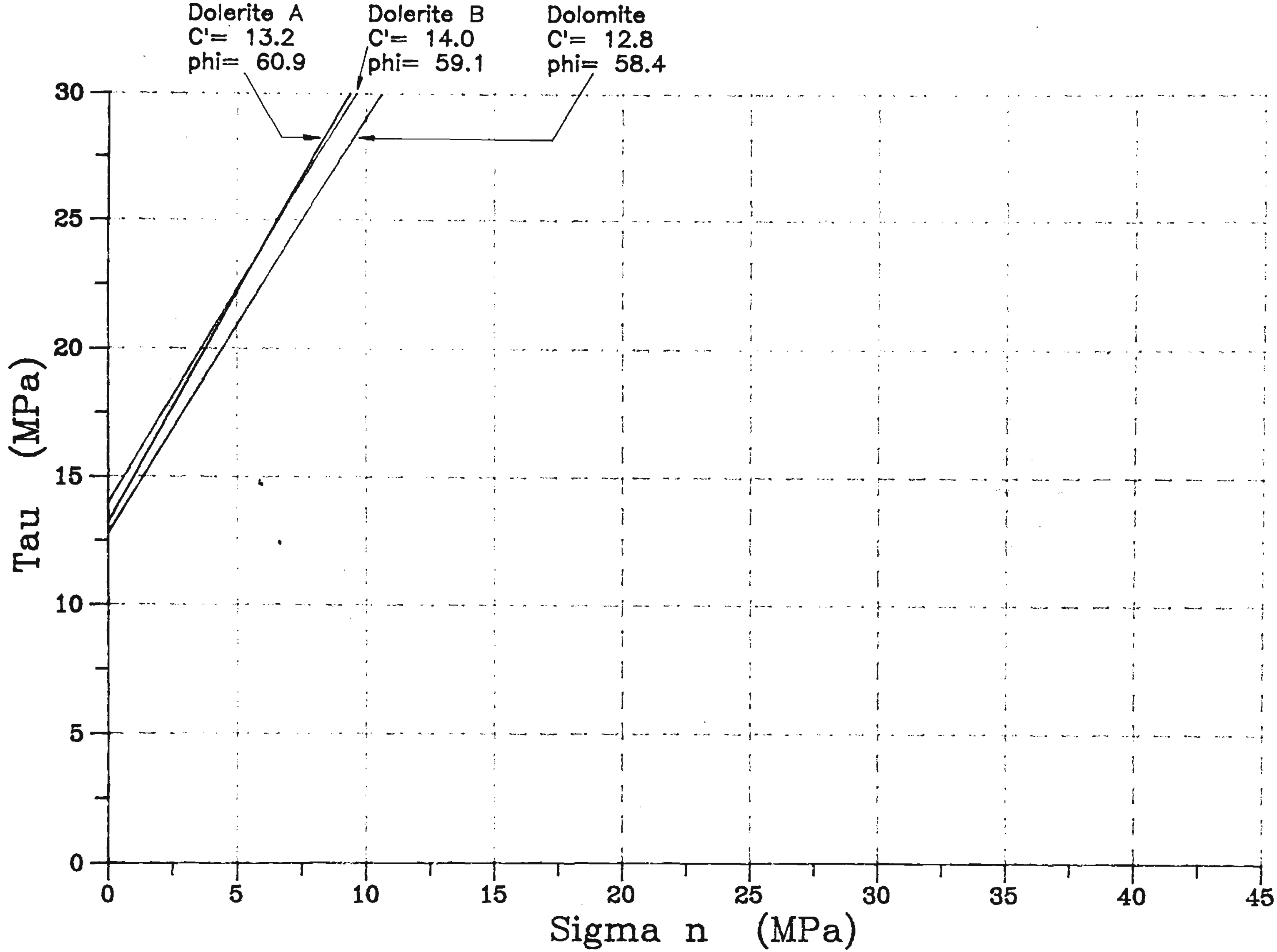
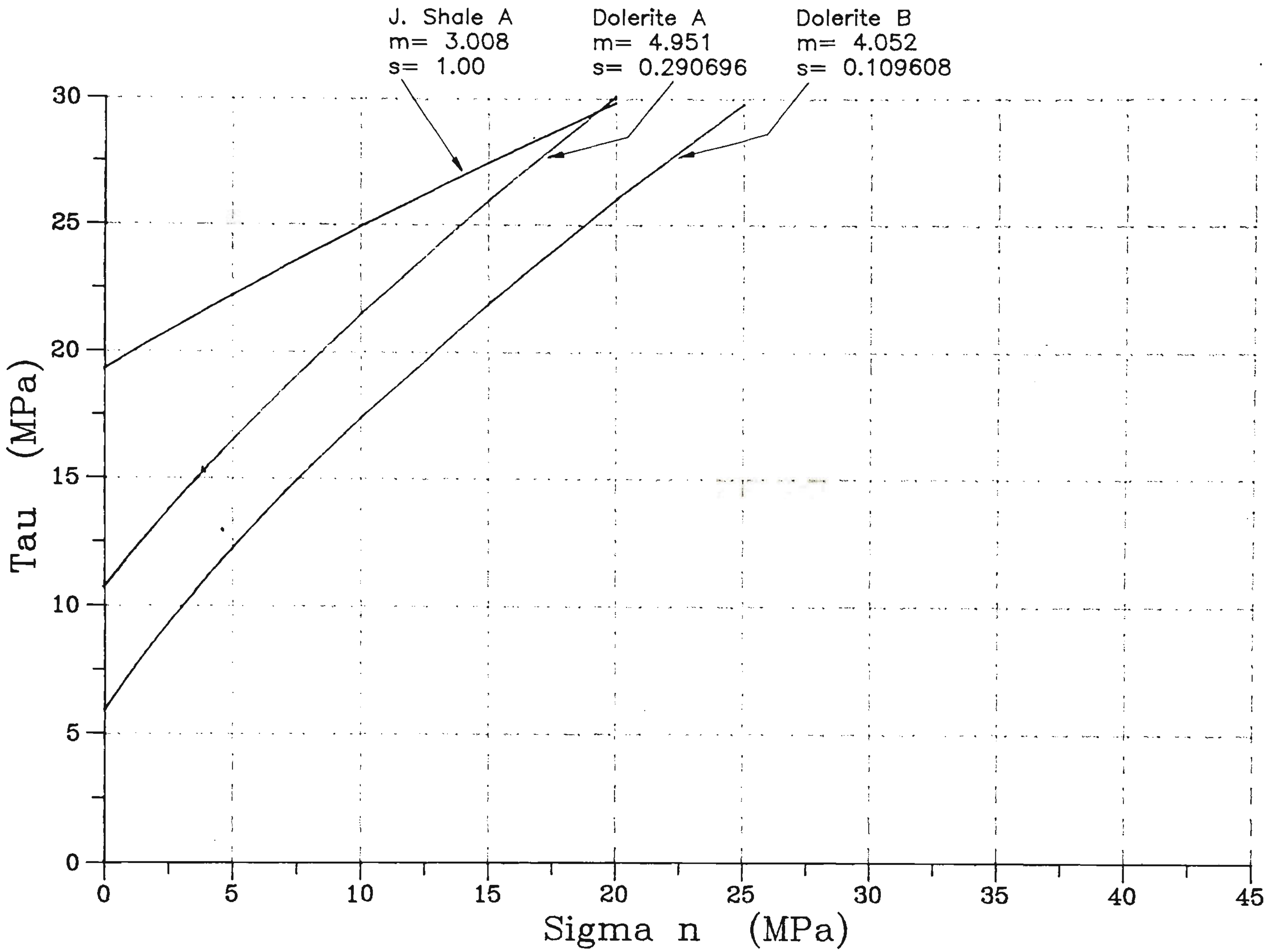

Dolerite A
C'= 13.2
phi= 60.9
Dolerite B
C'= 14.0
phi= 59.1
Dolomite
C'= 12.8
phi= 58.4
Tau (MPa)
30
25
20
15
10
5
0
0
5
10
15
20
25
30
35
40
45
Sigma n (MPa)

Figure 8.1

HOEK–BROWN FAILURE ENVELOPES



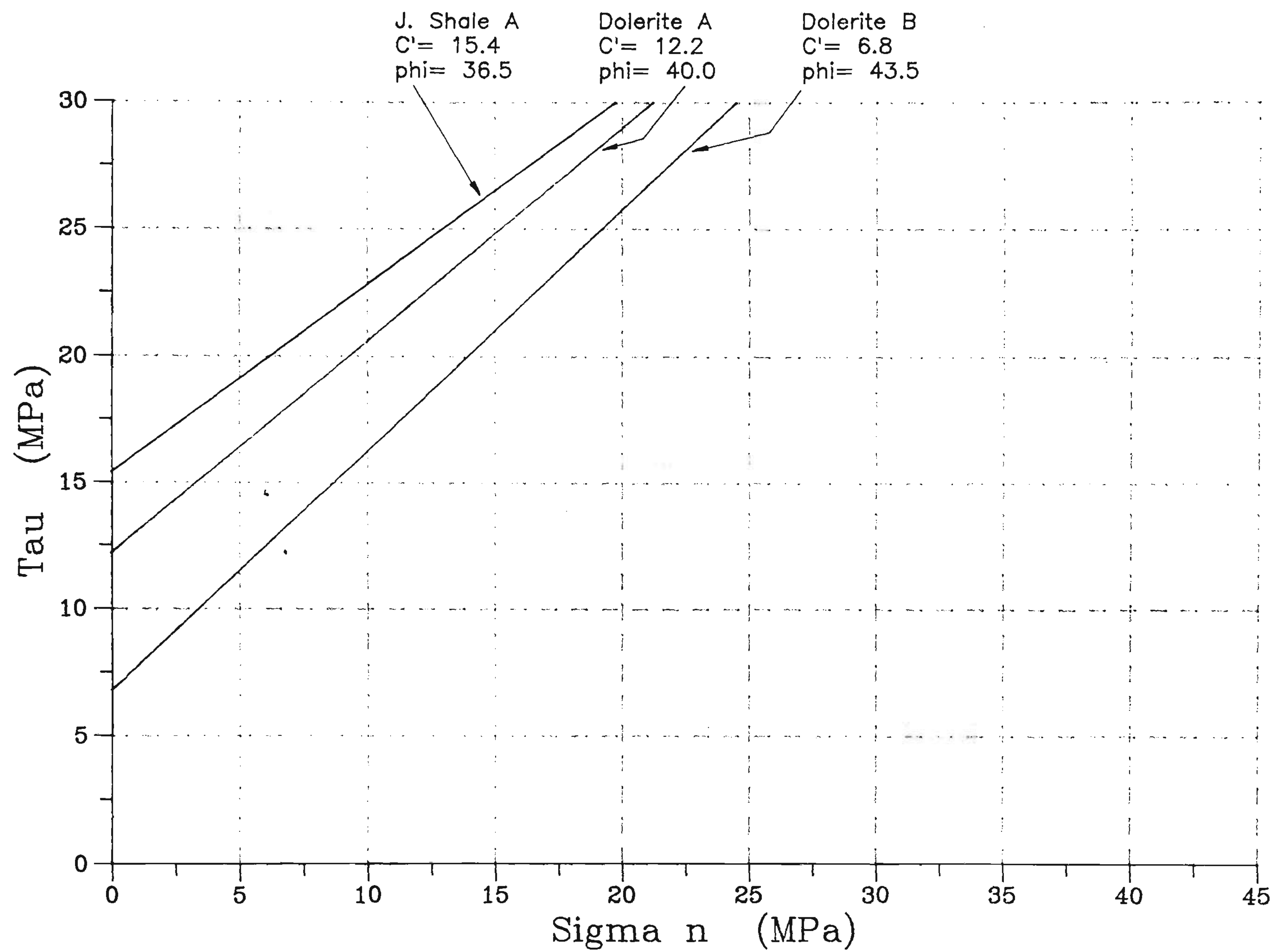

J. Shale A
C'= 15.4
phi= 36.5
Dolerite A
C'= 12.2
phi= 40.0
Dolerite B
C'= 6.8
phi= 43.5
30
25
20
15
10
5
0
Tau (MPa)
0
5
10
15
20
25
30
35
40
45
Sigma n (MPa)

Figure 8.2

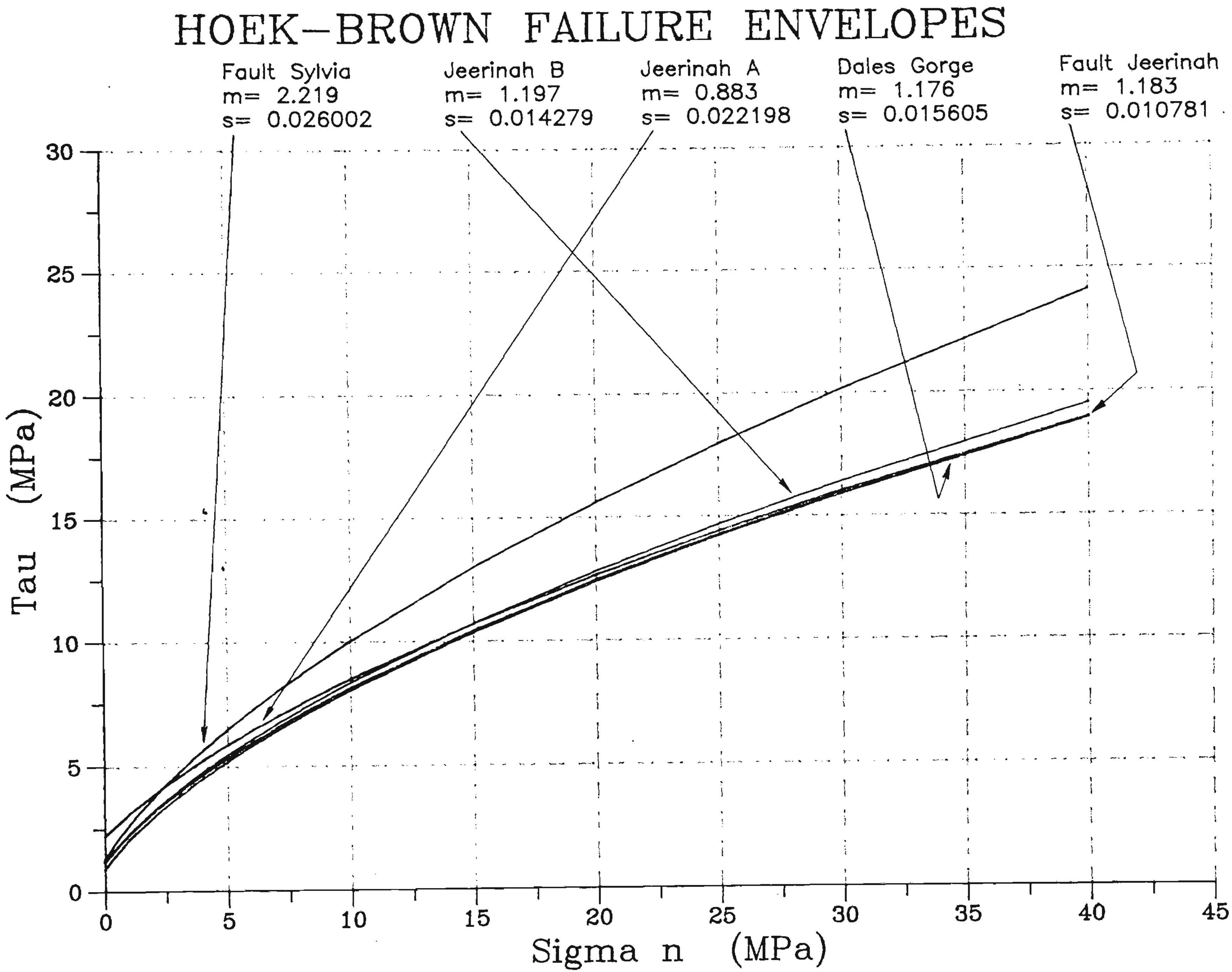


Figure8.3Overlay

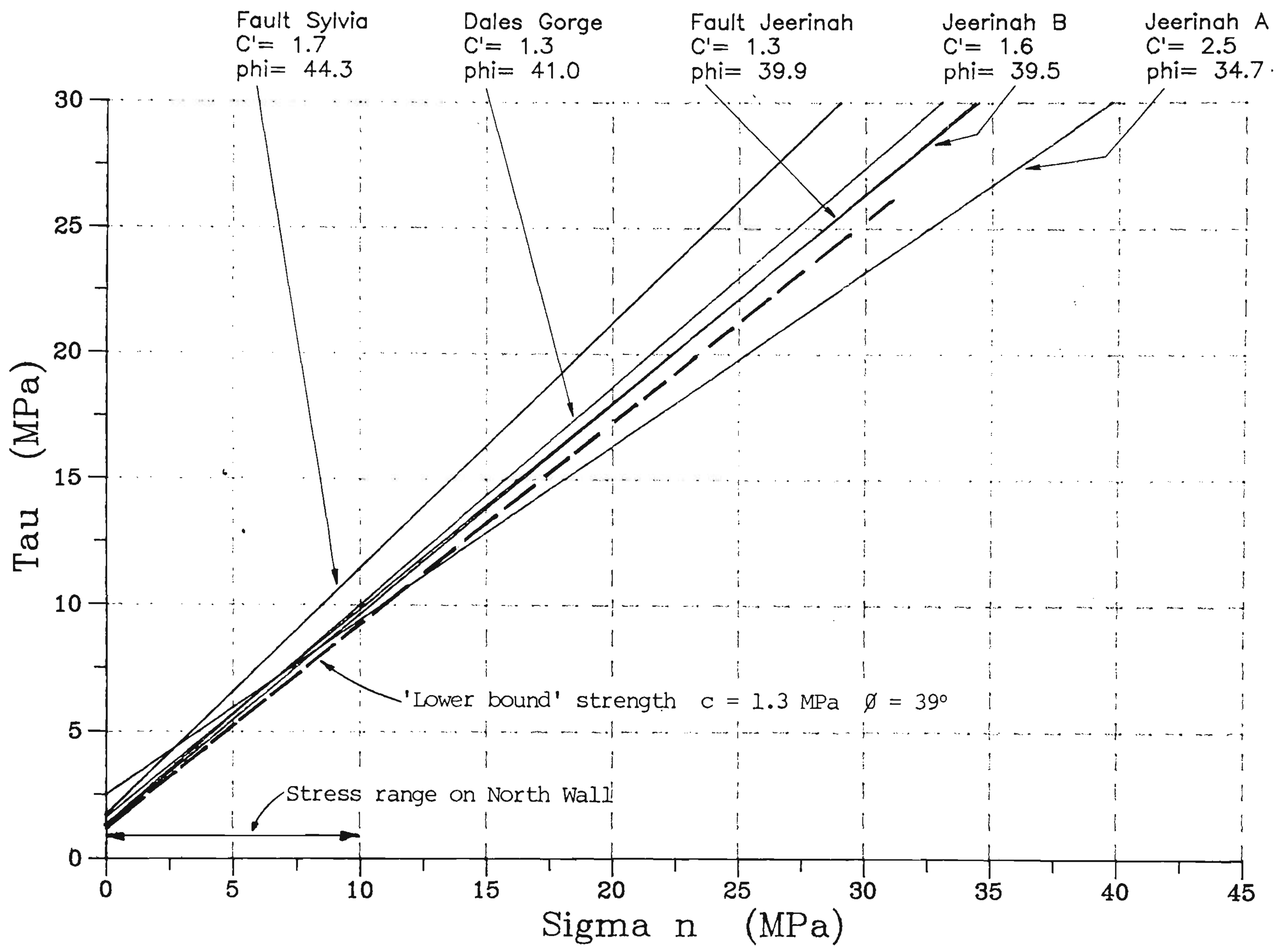

Fault Sylvia
C'= 1.7
phi= 44.3
Dales Gorge
C'= 1.3
phi= 41.0
Fault Jeerinah
C'= 1.3
phi= 39.9
Jeerinah B
C'= 1.6
phi= 39.5
Jeerinah A
C'= 2.5
phi= 34.7
'Lower bound' strength c = 1.3 MPa Ø = 39°
Stress range on North Wall
Tau (MPa)
Sigma n (MPa)
0
5
10
15
20
25
30
35
40
45

Figure 8.3

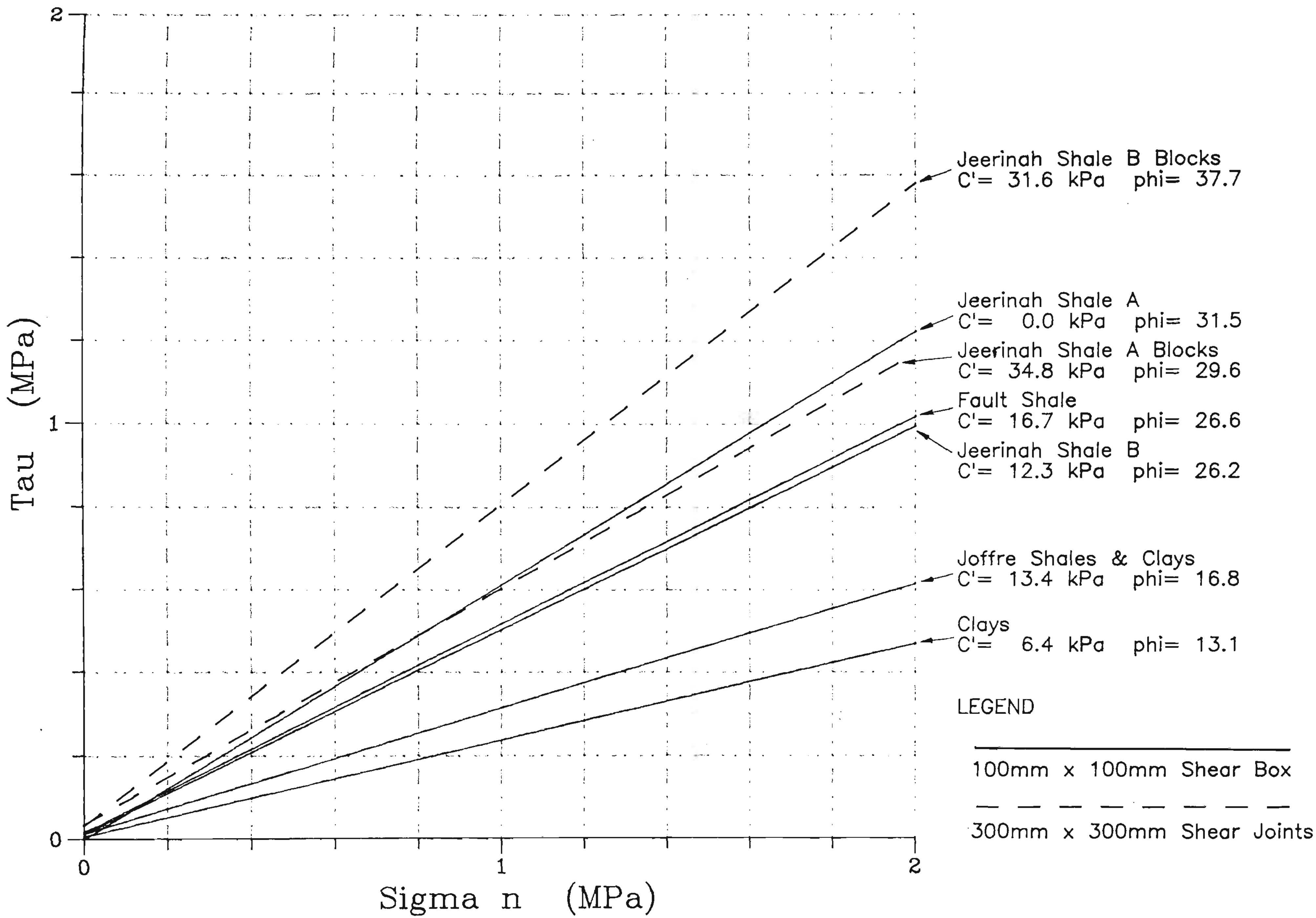

2
1
0
Tau (MPa)
0
1
2
Sigma n (MPa)
Jeerinah Shale B Blocks
C'= 31.6 kPa phi= 37.7
Jeerinah Shale A
C'= 0.0 kPa phi= 31.5
Jeerinah Shale A Blocks
C'= 34.8 kPa phi= 29.6
Fault Shale
C'= 16.7 kPa phi= 26.6
Jeerinah Shale B
C'= 12.3 kPa phi= 26.2
Joffre Shales & Clays
C'= 13.4 kPa phi= 16.8
Clays
C'= 6.4 kPa phi= 13.1
LEGEND
100mm x 100mm Shear Box
300mm x 300mm Shear Joints

Figure 8.4

HOEK-BROWN FAILURE ENVELOPES

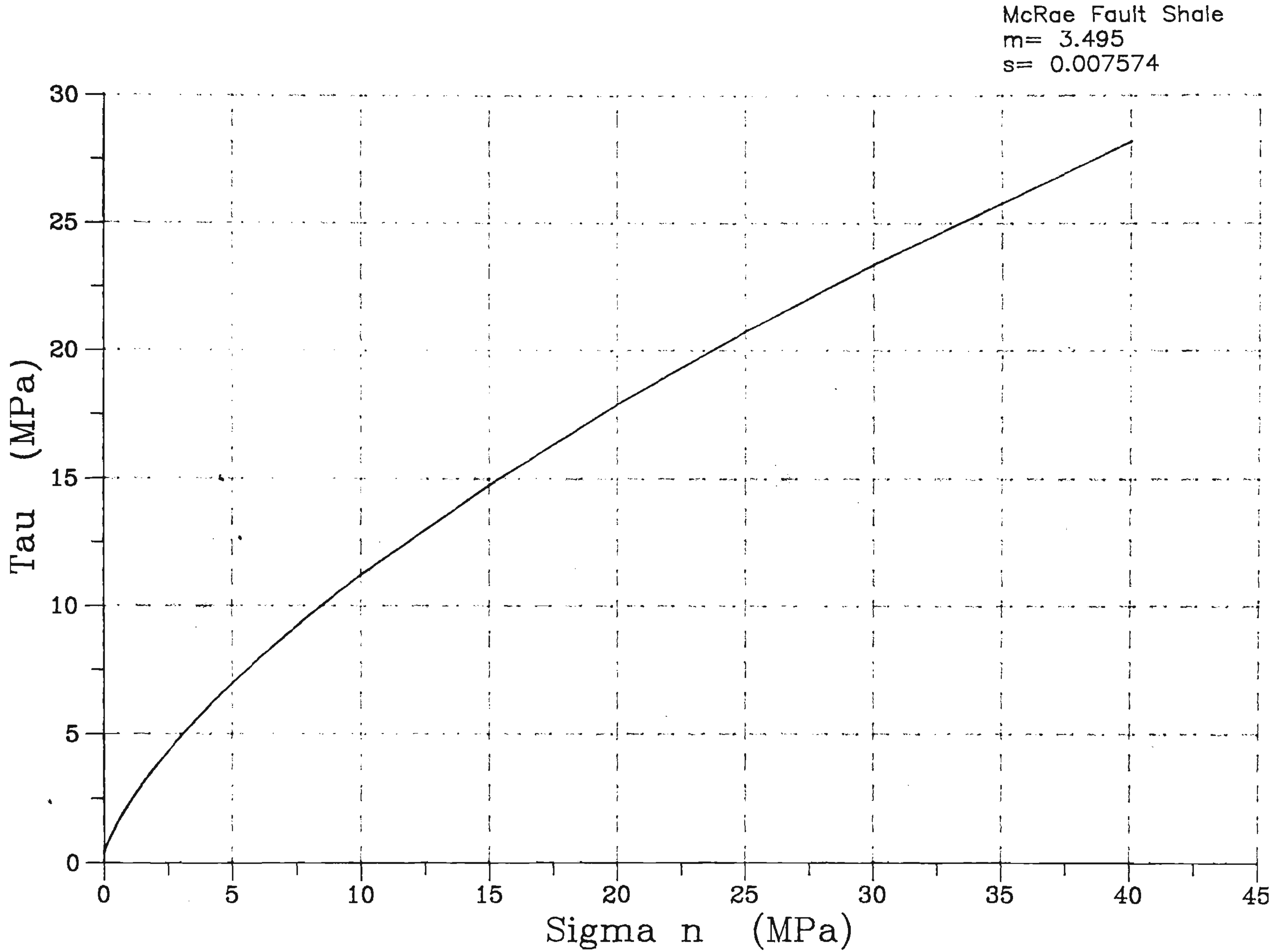


Figure8.5Overlay

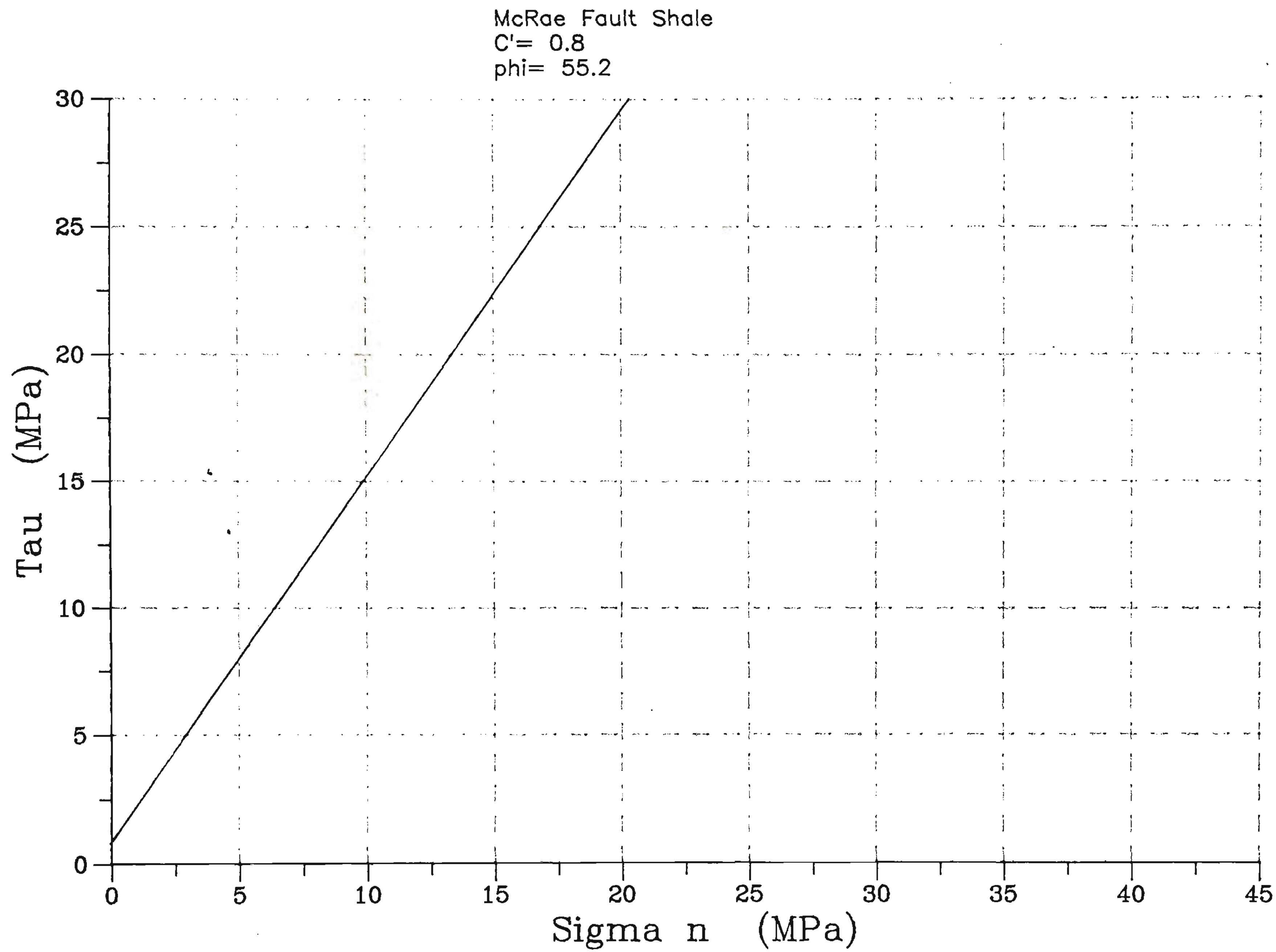
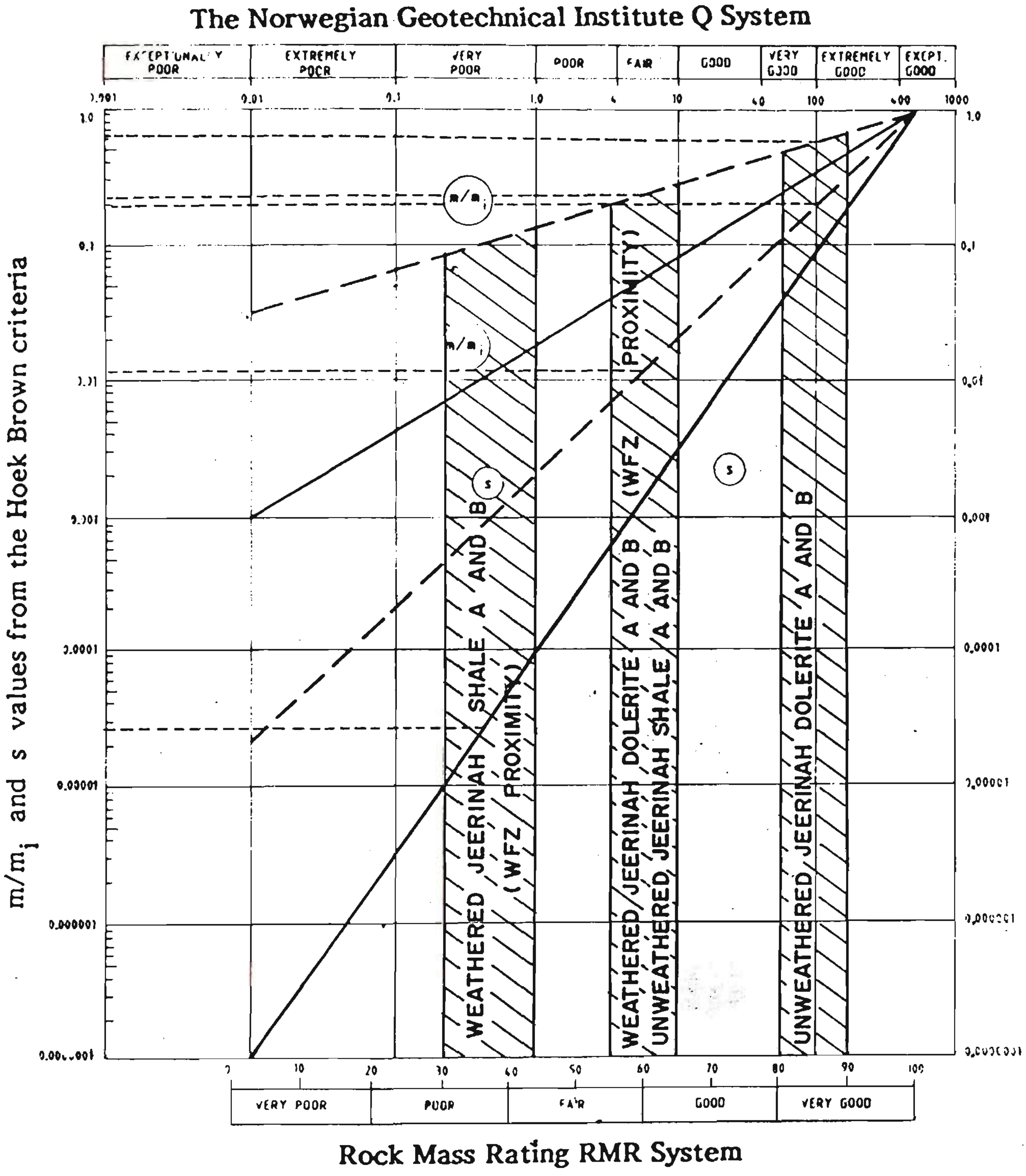

McRae Fault Shale
C'= 0.8
phi= 55.2
Tau (MPa)
Sigma n (MPa)
0
5
10
15
20
25
30
35
40
45

Figure 8.5

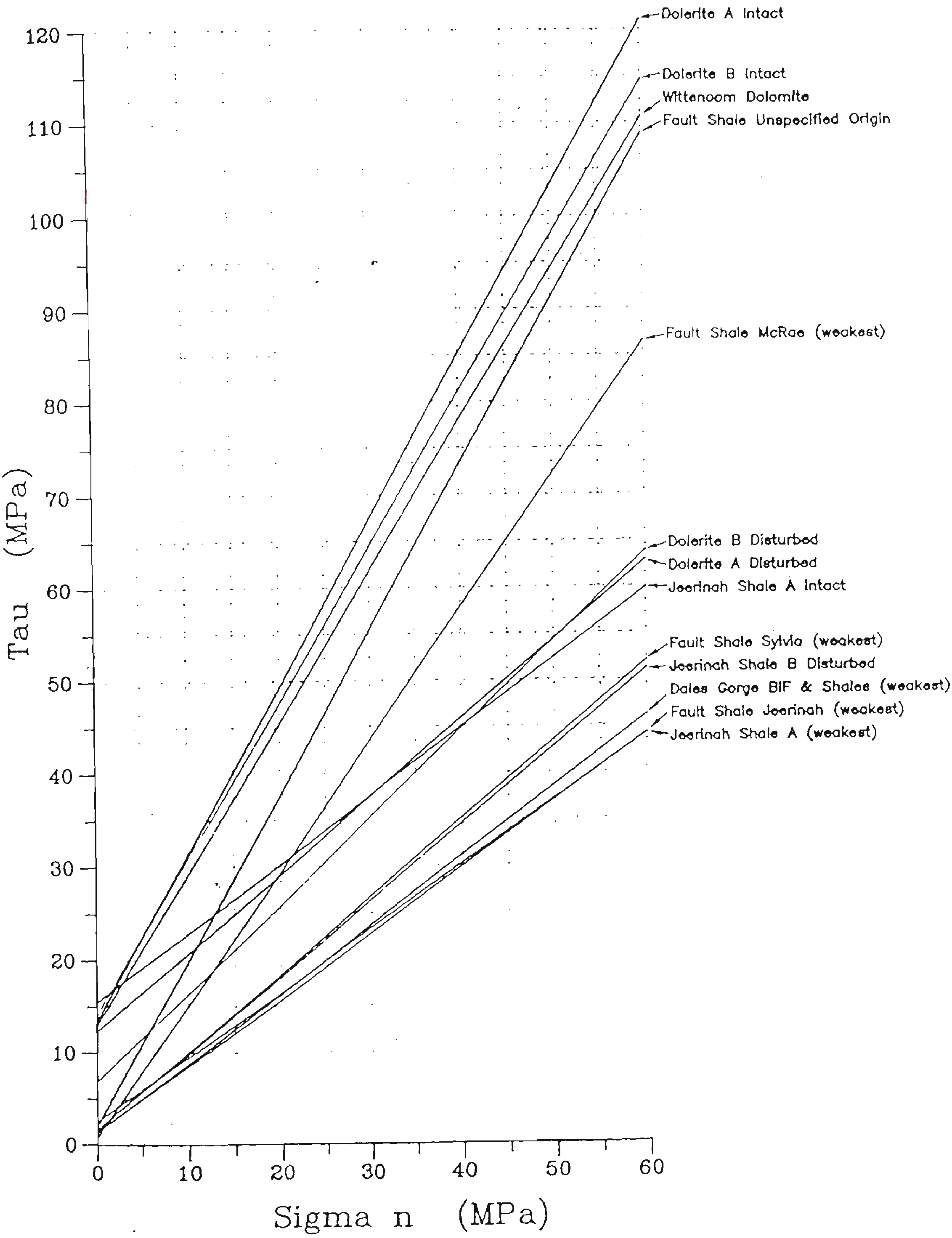


Undisturbed rock masses.

Disturbed rock masses.

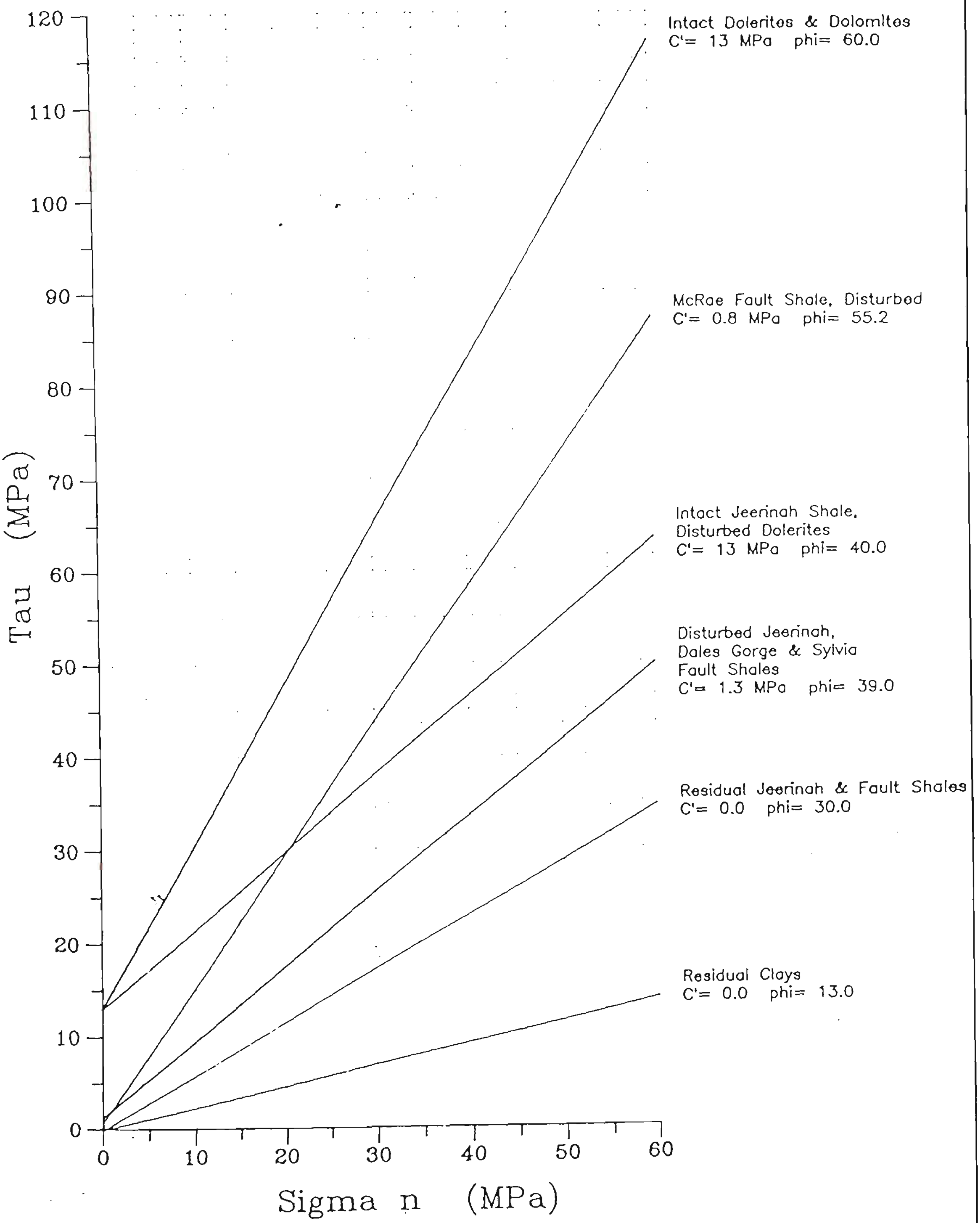
Scale	RELATIONSHIP BETWEEN HOEK-BROWN CONSTANTS m AND s, AND ROCK MASS CLASSIFICATIONS FOR THE JEERINAH FORMATION.	FIGURE 8.6
Drn		
Dwg N		

NEWMAN NORTH WALL STUDY
Individual Plot of Results



Scale		FIGURE 8.7
Drn		
Dwg. No.		

NEWMAN NORTH WALL STUDY
Simplified Plot of Results



Scale		FIGURE 8.8
Drn		
Dwg. No.		

TABLE 8.1

SUMMARY OF PROPERTIES

MT WHALEBACK ROCK TYPES : PAST TEST WORK/ANALYSES

Lithology	Sample Type	Test Type	No of Samples	UCS(MPa)	c(kPa)	Ø (deg)	m	s	Source
Brockman Iron Formation									
Whaleback Shale	Intact	U	1	13.5	-	-			BHPE 1986
	Intact	U	6	26	-	-			D&M 1972
	Bedding	SHC	9	-	*	27			D&M 1972
	Joints	SHC	1	-	*	35			D&M 1972
	Bedding Sawcut	SHC	1	-	41	23			Golders 1985
	Bedding Sawcut	SHC	1	-	24	16			Golders 1985
Dales Gorge Member Shale	Bedding Planes	SHC	7	-	*	20			D&M 1982
Mt McRae Shale (Unspecified)	Intact	U	3	17.5	-	-			BHPE 1986
	Intact	U	17	51	-	-			Golders 1986
	Intact	U	6	86	-	-			D&M 1972
	Distorted	U	5	17	-	-			D&M 1972
	Bedding	SHC	11	-	*	28			D&M 1972
	Bedding Sawcut	SHC	2	-	76	16			Golders 1985
	Joint	SHC	2	-	*	26			D&M 1971/72
	Cross Bedding			50			0.18	0.001	Hoek 1986
	Rough Joints			50			0.04	0.0	Hoek 1986
Upper Zone	Bedding Planes	SHC	13	-	*	22			D&M 1972
	Bedding		-	-	241	21.5			GMS 1985
	Joints	SHC	5	-	*	24			D&M 1982
	Joints		-	-	0	27			GMS 1985
Nodule Zone	Bedding Planes	SHC	10	-	*	28			D&M 1982
	Bedding		-	-	216	30			GMS 1985
	Joints	SHC	3	-	*	20			D&M 1982
	Joints	SHC	-	-	270	28			GMS 1985
				-	185	24			Golders 1985
Chert Zone	Bedding Planes	SHC	8	-	*	25			D&M 1982
	Joints	SHC	1	-	*	24			D&M 1982
	Bedding Oxid		-	-	392	21			GMS 1985
	Bedding Unox		-	-	0	32			GMS 1985

Lithology	Sample Type	Test Type	No of Samples	UCS(MPa)	c(kPa)	Ø (deg)	m	s	Source
Lower Zone	Bedding Planes	SHC	4	-	*	21			D&M 1982
	Bedding Oxid		-	-	0	31			GMS 1985
	Bedding	SHC	-	-	285	18			Golders 1985
	Joints	SHC	3	-	*	25			D&M 1982
	All defects		-	-	98	29			GMS 1985
Mt Sylvia Shale	Intact	U	2	35					Golders 1986
	Intact	U	5	16	-	-			D&M 1972
	Joints		-	-	176	33			GMS 1985
	Bedding	SHC	6	-	*	31			D&M 1972
	Bedding/ shear sawcut	SHC	3	-	0	15			Golders 1985
	Bedding - natural	SHC	1	-	44	13			Golders 1985
	All bedding and joints		-	-	181	31			GMS 1985
Wittenoom Dolomite	Shales		-	-	0	31			GMS 1985
	Shale from fault zone - joint	SHC	1	-	37	18			Golders 1985
Jeerinah Formation									
Shale A	Intact	U	10	52	-	-			JKMRC 1983
	Joints and bedding	SHC	43	-	260	25			D&M 1982
	Bedding		-	-	318	19.5			GMS 1985
	Bedding - natural	SHC	3	-	155	18			Golders 1985
	Shear - natural	SHC	1		547	17			Golders 1985
	Schistosity		-	-	289	21			GMS 1985
	Saw cuts		-	-	160	27			GMS 1985
	Remoulded shale	SSS	3	-	26	24			BHPE 1986
	Remoulded clay	SSR	1	-	19	9			BHPE 1986
Shale B	Intact	U	1	42	-	-			JKMRC 1983
	Bedding		-	-	474	19			GMS 1985
	Schistosity		-	-	361	26			GMS 1985
	Saw cuts		-	-	330	25			GMS 1985
	Remoulded shale	SSS	2	-	17	25			BHPE 1986
Shale (Unspecified)	Joints	SHC	2	-	*	31			D&M 1972
	Bedding	SHC	3	-	*	31			D&M 1972

Lithology	Sample Type	Test Type	No of Samples	UCS(MPa)	c(kPa)	Ø (deg)	m	s	Source
Dolerite A	Intact	U	4	118	-	-			JKMRC 1983
	Joints		-	-	0	31			GMS 1985
	Saw cuts		-	-	89	30			GMS 1985
Dolerite B	Intact	U	9	96	-	-			JKMRC 1983
	Weathered	U	3	80	-	-			JKMRC 1983
	Saw cuts		-	-	25	31			GMS 1985
Dolerite (Unspecified)	Joints	SHC	2	-	*	38			D&M 1972
	Joint in dolerite from fault zone	SHC	1	-	223	25			Golders 1985
Fault Shale	Disturbed	U	5	10.5	-	-			D&M 1972
	Disturbed	U	7	10.4	-	-			JKMRC 1983
	Joints/Bedding	SHC	1	-	*	16			D&M 1972
	Remoulded clay	SSR	1	-	0	14			BHPE 1986
Fault Shale - McRae	Sawcut bedding/ shear	SHC	6	-	37	20			Golders 1985
	Disturbed	U	2	16.0	-	-			D&M 1972
	Disturbed	U	7	10.2					BHPE 1986
	Bedding - sawcut	SHC	12	-	0	27			Golders 1985
	Bedding - natural	SHC	4	-	97	13			Golders 1985
	Joint - natural	SHC	1	-	166	21			Golders 1985
	Remoulded shale	SSS	6	--	19	27			BHPE 1986
	Remoulded clay	SSR	6	-	11	15			BHPE 1986
Fault Shale - Sylvia	Disturbed	U	10	38	-	-			BHPE 1986
	Bedding - natural	SHC	2	-	59	14			Golders 1985
	Bedding/shear sawcut	SHC	2	-	61	16			Golders 1985
	Remoulded shale	SSS	3	-	24	23			BHPE 1986
Fault Shale - - Jeerinah	Remoulded shale	SSS	2	-	27	24.5			BHPE 1986
	Remoulded clay	SSR	1	-	1	10			BHPE 1986
Joffre BIF	Disturbed	U	2	35	-	-			D&M 1972

Lithology	Sample Type	Test Type	No of Samples	UCS(MPa)	c(kPa)	Ø (deg)	m	s	Source
Joffre Jaspilite	Disturbed	U	6	25	-	-			D&M 1972
Joffre Shale	Bedding - natural	SHC	1	-	39	25			Golders 1985
	Sawcut shear/ bedding	SHC	1	-	31	18			Golders 1985
	Remoulded shale J3	SSS	8	-	11.5	16	0.007	0.0	BHPE 1986
	Remoulded shale J2	SSS	1	-	4	18	0.009	0.0	BHPE 1986
	Remoulded shale J1	SSS	3	-	20	17.5	0.009	0.0	BHPE 1986
	Remoulded clay J3	SSR	3	-	1	11	0.002	0.0	BHPE 1986
	Remoulded clay J2	SSR	3	-	0	14	0.005	0.0	BHPE 1986
	Disturbed	U	3	3.5	-	-			BHPE 1986

* Cohesion not quoted

D&M	Dames & Moore 1972 :	Report on Rock Slope Design
	Dames & Moore 1982 :	Report Stability Analysis South Wall Mt Whaleback Open Pit Mine
GMS	Gray, MacFarlane, Slepecki 1985 :	Geotechnical Investigations at Mt Whaleback and Implications for Future Mining
JKMRC	Julius Kruttschnitt Mineral Research Centre 1983 :	A Report on Mt Whaleback Rock Strength Parameters - Slopes 7 and 8 for Fault Shale - Jeerinah Formation (Shales A and B and Dolerites A and B)
Golders	Golder Associates 1985 :	Report to Mt Newman Mining Company on Direct Shear Testing North Wall Mt Whaleback Mine
	Golder Associates 1986 :	Base Case Design Study, Mt Whaleback South Wall. Interim Report No 1 Area 2 Design
BHPE	BHP Engineering 1986 :	Laboratory Testing of Selected Samples for North Wall Investigations
Hoek	Dr Evert Hoek	

Legend: Test Type

TI	Triaxial tests on intact material
TD	Triaxial tests on disturbed material
TWD	Triaxial tests on weakest disturbed material
SHC	Hoek shear test on cast sample
SLC	300mm x 300mm shear box test sample cast, sawcut parallel to cleavage
SSS	100mm x 100mm residual direct shear test on shale crushed prior to testing
SSR	100mm x 100mm residual direct shear test on clay crushed and remoulded prior to testing
U	Uniaxial Compression Test (UCS)

TABLE 8.2

SUMMARY OF SPECIMENS TESTED CURRENT PROGRAMME

Lithology	Test Type	No of Specimens Tested
Dales Gorge Member Shales	Triaxial	18
	Remoulded Direct Shear	6
Mt Sylvia Shale	Triaxial	-
	Remoulded Direct Shear	-
Wittenoom Dolomite - within fault zone	Triaxial	7
	Remoulded Direct Shear	-
Jeerinah Shale A	Triaxial	30
	Large Shear Test on Sawcut	2
	Remoulded Direct Shear	9
Jeerinah Shale B	Triaxial	12
	Large Shear Test on Sawcut	4
	Remoulded Direct Shear	6
Jeerinah Dolerite A	Triaxial	28
	Remoulded Direct Shear	-
Jeerinah Dolerite B	Triaxial	19
	Remoulded Direct Shear	-
Fault Shale		
- derived from Jeerinah	Triaxial	26
	Remoulded Direct Shear	6
- derived from Mt Sylvia	Triaxial	17
	Remoulded Direct Shear	6
- derived from McRae	Triaxial	30
	Remoulded Direct Shear	-
- derivation unspecified	Triaxial	10
	Remoulded Direct Shear	3

TOTALS

Triaxial tests = 197

Remoulded Direct Shear Tests = 36

Large shear tests (300x300mm)
on sawcuts = 6

TABLE 8.3

SUMMARY OF RESULTS OBTAINED FROM COMBINED TEST PROGRAMMES

Lithology	Type of Test	UCS(MPa)	c(kPa)	Ø(degrees)	UCS(c)(MPa)	m	s
Jeerinah Shale A	U	59.5					
	TI		15400	36.5	64.37	3.0	1.0
	TWD		2500	35	64.37	0.883	0.022
	SLC		35	30	64.37	0.075	0.0
	SSS		0	31.5	64.37	0.042	0.0
	SSR		19	9.5	64.37	0.002	0.0
Jeerinah Shale B	U	53.0					
	TWD		1600	39.5	53.0	1.20	0.014
	SLC		32	38	53.0	0.229	0.0
	SSS		12	26	53.0	0.029	0.0
Jeerinah Dolerite A	U	119.8					
	TI		13200	61	103.3	26.1	1.0
	TD		12200	40	103.3	4.95	0.291
Jeerinah Dolerite B	U	99.6					
	TI		14000	59	105.7	13.7	1.0
	TD		6800	43.5	105.7	4.05	0.110
Fault Shale derived from Jeerinah	U	11.0					
	TWD		1300	40	50	1.18	0.011
	SSS		15	28	50	0.040	0.0
Fault Shale derived from Mt McRae shale	U	10.2					
	TWD		770	55	50	3.50	0.008
	SSS		19	27	50	0.026	0.0
	SSR		11	15	50	0.006	0.0
Fault Shale derived from Mt Sylvia shale	U	34.1					
	TWD		1700	44	50	2.22	0.026
	SSS		20	24	50	0.022	0.0
	SSR		1	10	50	0.003	0.0

Lithology	Type of Test	UCS(MPa)	c(kPa)	Ø(degrees)	UCS(c)(MPa)	m	s
Fault Shale	U	15.5					
- origin unspecified	TD		1900	61	50	11.8	0.196
	SSR		0	14	50	0.006	0.0
Wittenoom Dolomite	U	80.6					
	TI		12800	58	92.7	34.0	1.0
Brockman Formation							
Dales Gorge	U	6.1					
Member Shale	TWD		1300	41	50	1.18	0.016
	SSR		3	13.3	50	0.005	0.0
Joffre Shales & Clays	SSS & SSR		13	16.8			

Legend: Test Type

TI	Triaxial tests on intact material
TD	Triaxial tests on disturbed material
TWD	Triaxial tests on weakest disturbed material
SLC	300mm x 300mm shear box test sample cast, sawcut parallel to cleavage
SSS	100mm x 100mm residual direct shear test on shale crushed prior to testing
SSR	100mm x 100mm residual direct shear test on clay crushed and remoulded prior to testing
U	Uniaxial Compression Test (UCS)
UCS	Unconfined compressive strength from actual test
UCS(c)	Unconfined compressive strength calculated from LABDATA program

ROCK MASS QUALITY OF THE JEERINAH FORMATION

Rock Mass Parameter	Jeerinah Dolerite				Jeerinah Shale			
	Unweathered		Weathered		Unweathered		Weathered	
	Value	Rating *	Value	Rating *	Value	Rating *	Value	Rating *
UCS (MPa)	100-120	12	30	4	50	4	20	2
RQD (%)	90-100	20	40-80	13	40-80	13	0-40	5
Discontinuity Spacing	1m	15	60-200mm	8	40-200	8	50-200	6
Discontinuity Condition	Very rough	30	Slightly rough Slightly weathered	25	Slightly rough Slightly weathered	25	Polished Moderately weathered	15
Groundwater Condition	Damp	10	Damp	10	Damp	10	Damp	10
Total Rating		87		60		60		38
Rock Mass Quality	Very good		Fair - good		Fair - good		Poor	

Note : * indicates rating value assigned under Bieniawski's System (Ref 3)

TABLE 8.5

SUMMARY OF ROCK MASS CLASSIFICATION OF THE JEERINAH FORMATION

		RMR	$\frac{m}{mi}$	mi	m	s
Dolerite A	Weathered	60	0.24	26.1	6.26	0.012
	Unweathered	87	0.63	26.1	16.44	0.24
Dolerite B	Weathered	60	0.24	13.7	3.29	0.012
	Unweathered	87	0.63	13.7	8.63	0.24
Shale A	Weathered	38	0.012	1.2	0.014	0.00003
	Unweathered	60	0.24	1.2	0.29	0.012
Shale B	Weathered	38	0.012	3.0	0.036	0.00003
	Unweathered	60	0.24	3.0	0.72	0.012

Notes:

RMR — Rock Mass Rating (Ref 3)

mi — Refers to the 'm' value for intact rock and is typically that value obtained from laboratory specimens

m and s — Empirical constants to define shape of shear strength envelope defined by Hoek (Ref 5, 10)

TABLE 8.6

COMPARISON OF ROCK STRENGTH BETWEEN LABORATORY RESULTS AND ROCK MASS RATING RESULTS

Rock Type		Laboratory Results m	Laboratory Results s	Rock Mass Rating m	Rock Mass Rating s
Dolerite A	Intact/Unweathered	26.058	1.0	16.44	0.24
	Disturbed/Weathered	4.951	0.2906	6.26	0.012
Dolerite B	Intact/Unweathered	13.656	1.0	8.63	0.24
	Disturbed/Weathered	4.052	0.1096	3.29	0.012
Wittenoom Dolomite	Unweathered	33.963	1.0	-	-
Shale A	Intact/Unweathered	3.008	1.0	0.29	0.012
	Disturbed/Weathered	0.883	0.022198	0.014	0.00003
Shale B	Intact/Unweathered	-	-	0.72	0.012
	Disturbed/Weathered	1.197	0.01428	0.36	0.00003

Note:

'Intact' results refer to Laboratory Results where s = 1.0

'Unweathered' results refer to field conditions which includes the effects of jointing, spacing, orientation, etc and therefore is always less than 1.0

'Disturbed' results refer to core samples with a varying degree of disturbance which were tested in the laboratory

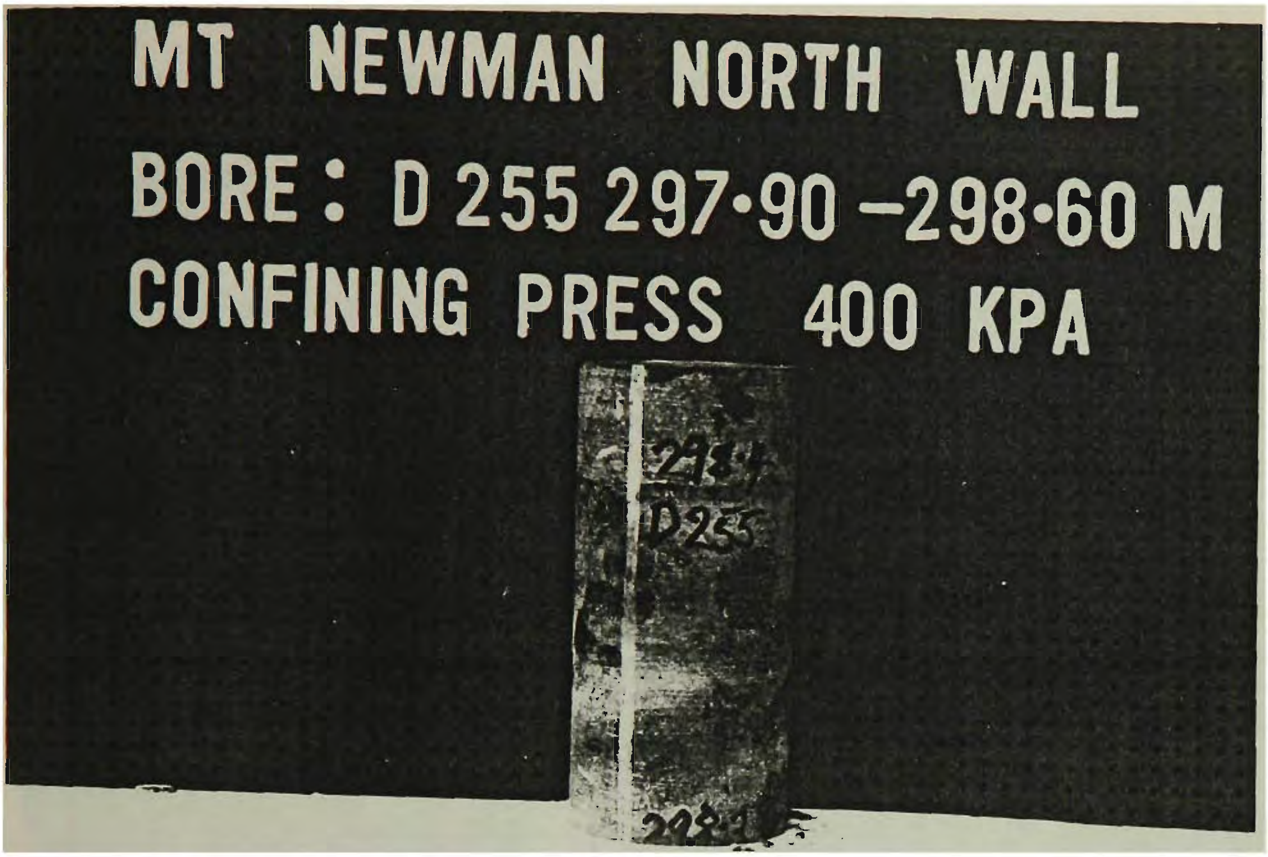
'Weathered' results refer to weathered field conditions

TABLE 8.7

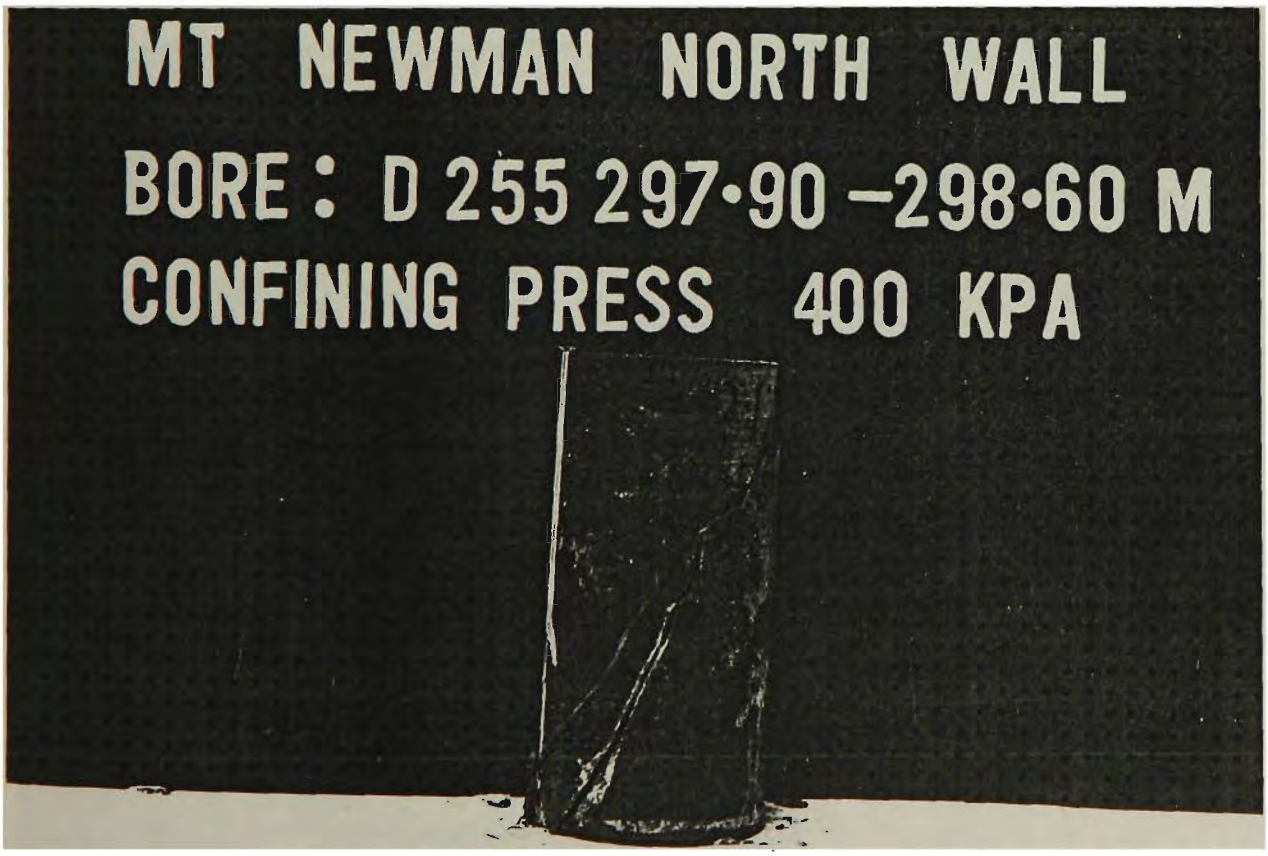
SUMMARY OF SHEAR STRENGTH PARAMETERS FROM BACK ANALYSES

	Formation Contacts	Calculated c(kPa)	Shear Strength Ø(deg)
1	Dales Gorge/Mt McRae Contact	35 - 50	18.0 - 21.5
2	Bruno's Band/Mt Sylvia Formation	30 - 45	25.5 - 29.5
3	Mt McRae/Mt McRae and Mt McRae/ Bruno's Contact	5 - 15	24.0 - 26.5
4	East Footwall Fault Failure East Pit	8	30

Taken from Gray, MacFarlane, Slepecki

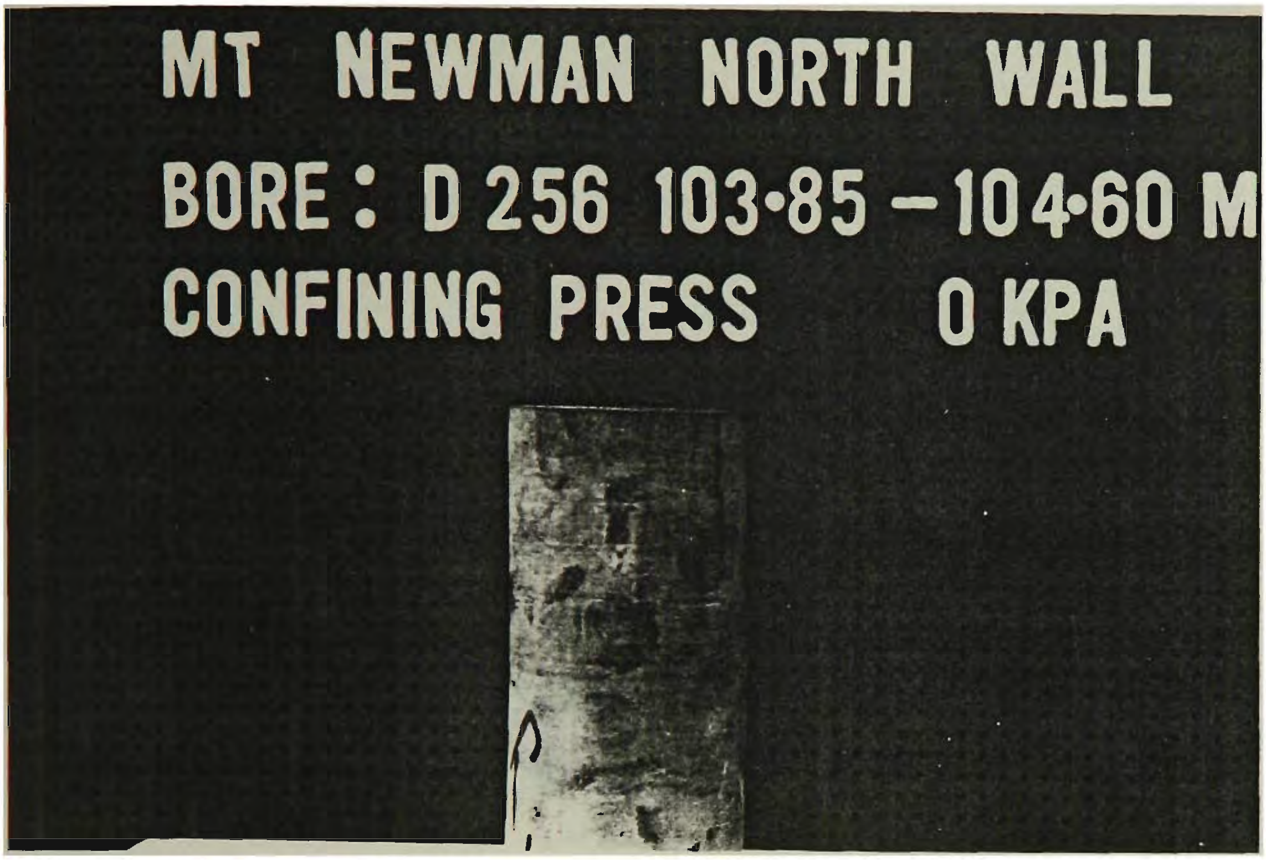


Before

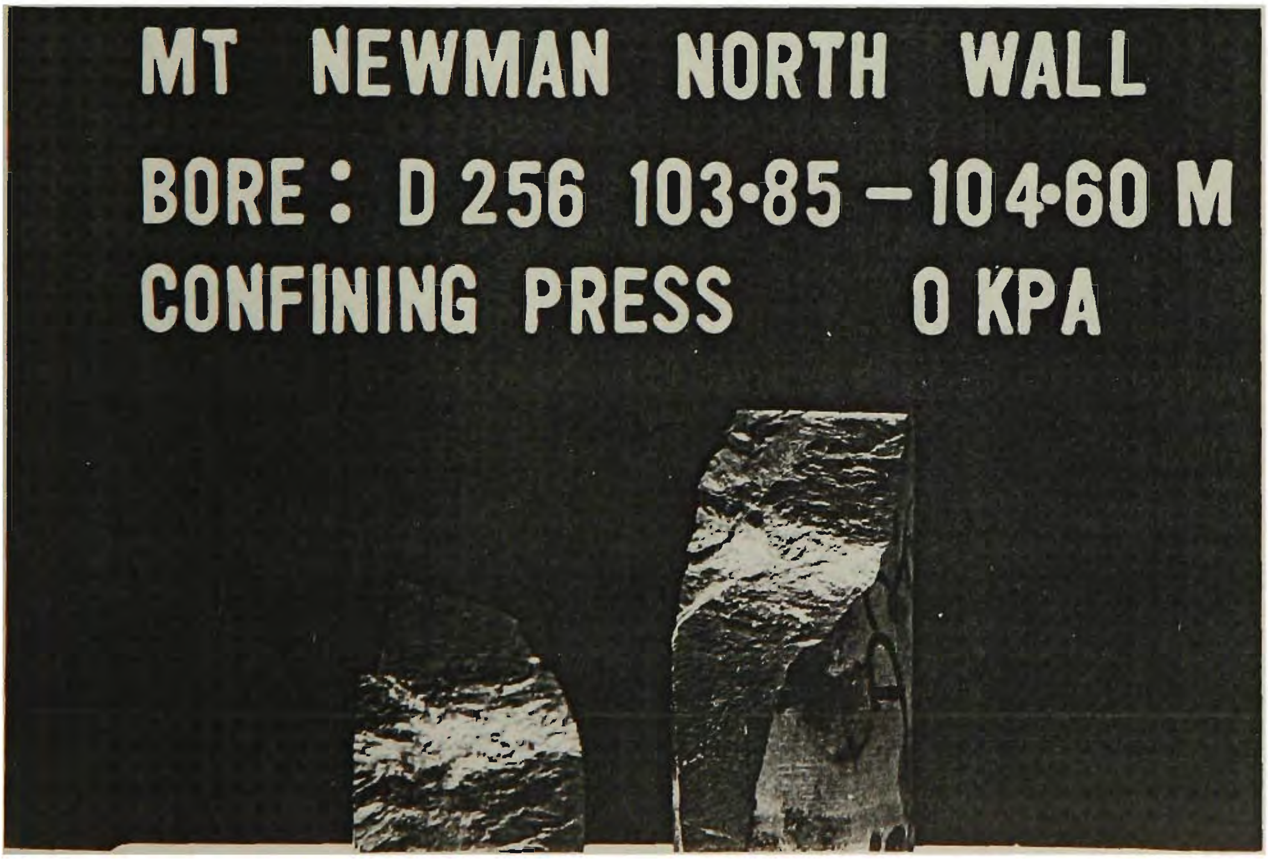


After

Plate 8.1 Jeerinah Dolerite A - Intact

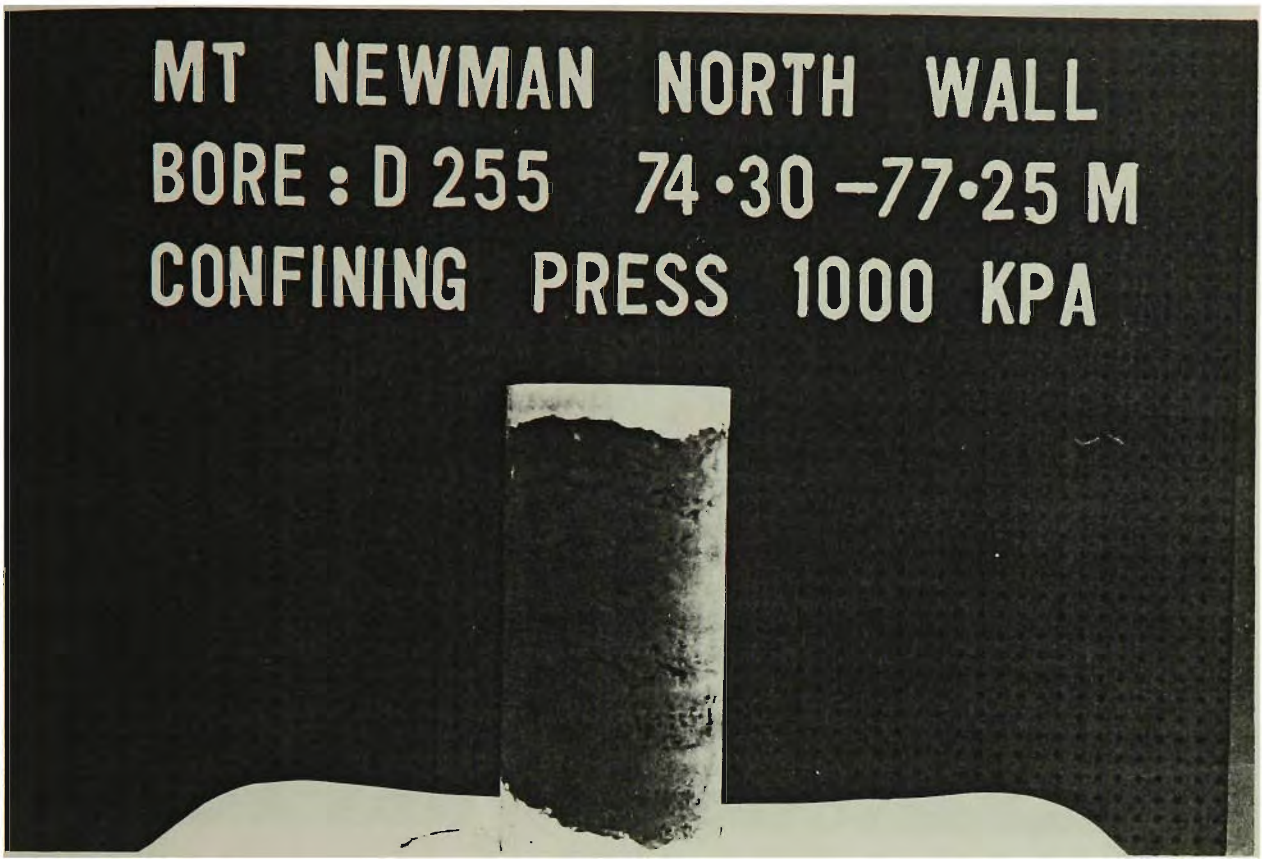


Before



After

Plate 8.2 Jeerinah Shale A - Intact



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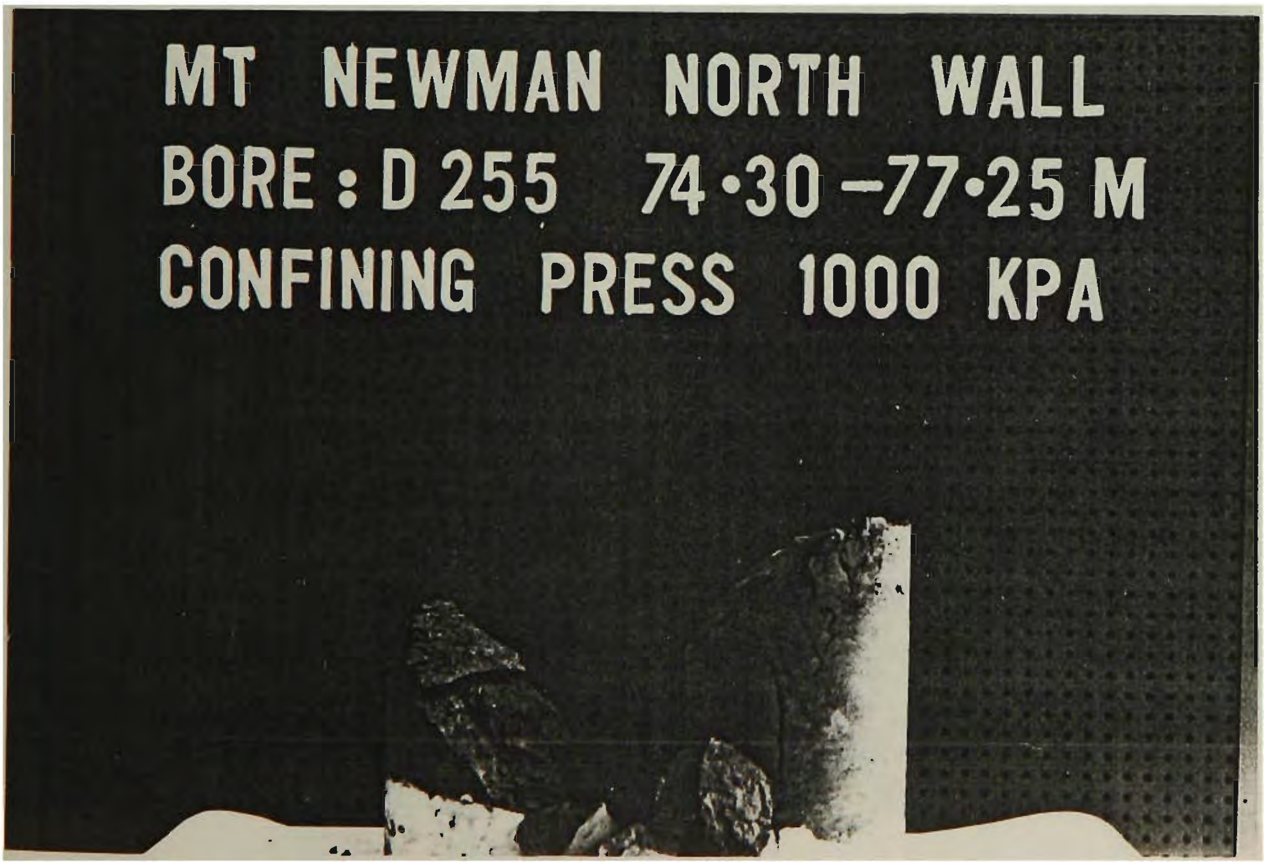
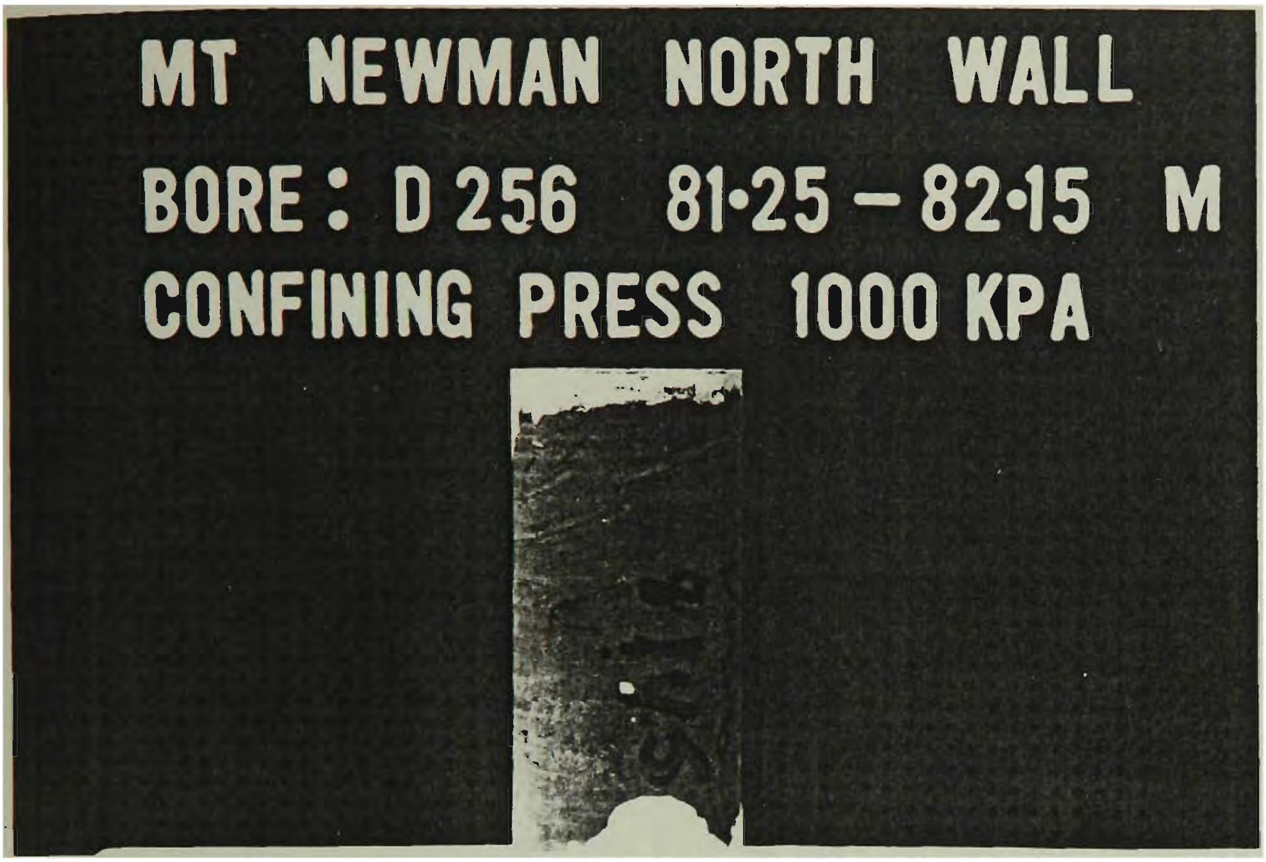
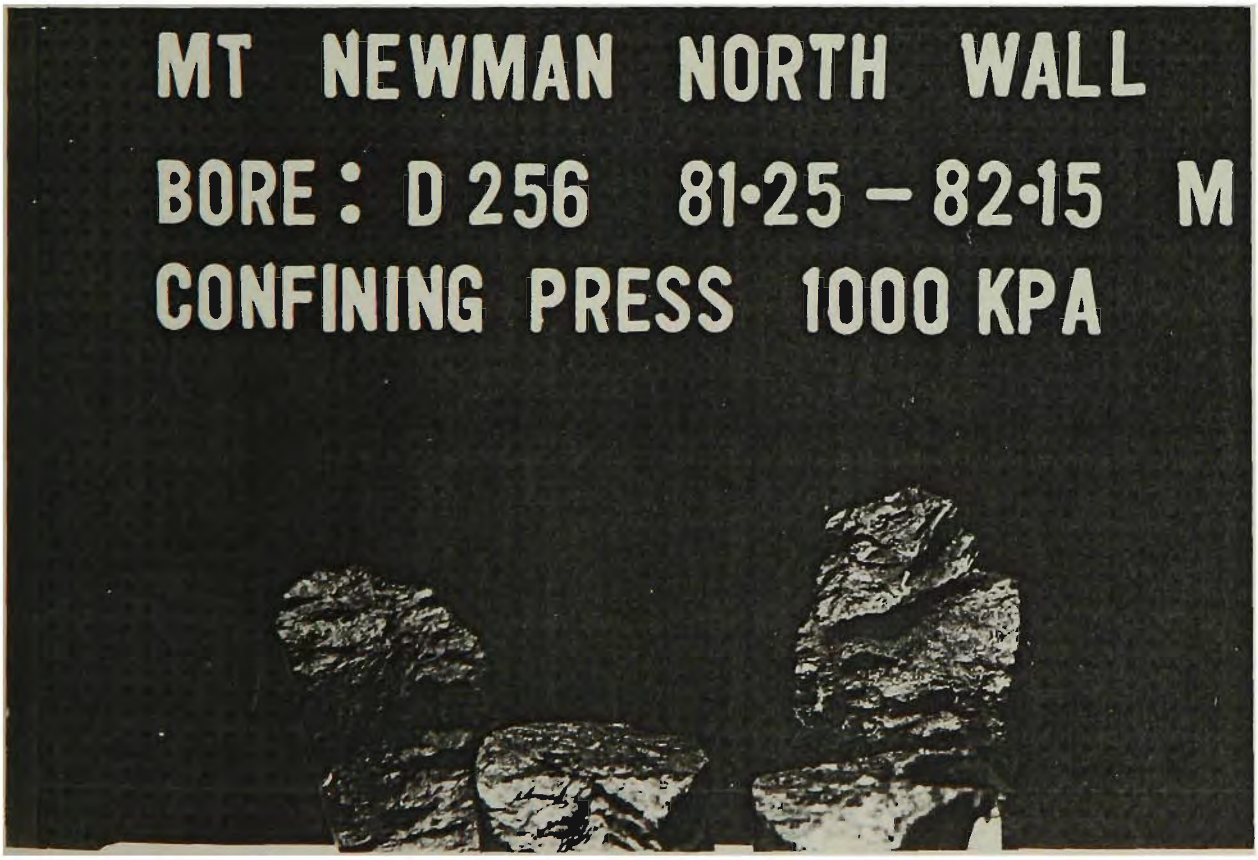


After

Plate 8.3 Jeerinah Shale - Disturbed



Before



After

Plate 8.4 Fault Shale

CHAPTER 9

BACK ANALYSES OF FAILURES

9.1 INTRODUCTION

Mt Whaleback is one of the largest single pit iron ore mines in the world and the current pit slopes on the South Wall in the East Pit are in excess of 200m high. Pit slopes on the North Wall are much smaller and are generally interim rather than final pit slopes. All except one of the slope failures have occurred on the South Wall. The exception is a failure on the East Footwall Fault Zone. These failures range in size from small wedges of rock only several metres across, up to 3 bench (45m high) complex rock slides. The small slides of rock are either generally ignored or easily cleaned up, whereas most of the larger failures (anything greater than about half a bench height, ie. 8m) are normally back analysed to determine their mechanism of failure and their shear strength. The relevance of this Chapter arises from the fact that some lithologies in the North Wall are the same as in the South Wall.

A total of nine of these slope failures have been back analysed and these are listed in Table 9.1. It can be seen that most of these failures occur on some pre-existing weakness surface (bedding, fault, shear zone, etc) which 'daylights' at or near the toe of the failure. Predominant pre-existing discontinuities are not necessary near the crest of the slope in order for failure to develop.

9.2 FAILURE MECHANISM

Stress analysis studies of open pit slopes (eg. Refs 2.16, 2.19, 2.21, 2.45, 2.78 from Chapter 2) clearly shows that shear stresses are generated at the toe of a slope whereas tensile stresses are generated near the crest. Moreover failure surfaces near the crest of a slope are normally sub-vertical, normal stresses on them are low, movement is therefore tensile rather than in shear, and hence the roughness of the failure surface at the crest of the slope is of little significance.

Table 9.1 Failures on Mt Whaleback

Horizon	Failure Mechanism	Dip Angle	Approx Date of Failure	c	Ø
1 DG/MCR*	Bedding slide on synclinal keel and joints across bedding	?	Aug 77	35.0–50.0	18.0–21.5
2 MCR/BB*	Bedding slide on anticlinal nose with tension crack	35°	June 78	5.0–15.0	24.0–26.5
3 DG/MCR*	Flat bedding slide	34°	May 80	35.0–50.0	18.0–21.5
4 MCR/MCR*	Curved bedding slide	30 to 70°	Aug 81	5.0–15	24.0–26.5
5 BB/SYL* 6	Bedding slide on synclinal keel and joints across bedding	Variable	Oct 82	30.0–45.0	25.5–29.5
7 NOD	Toppling		Aug 84	–	–
8 MCR/MCR*	Flat bedding slide	39°	Sept 84	5.0–15.0	24.0–26.5
9 EFFZ*	Slide on in–dipping fault zone	Variable	Mar 87	5.0–10.0	25.0–30.0

* Note:

The 'Horizon' indicates the horizon in which the main sliding occurred. Where failure occurred between two stratigraphic horizons, then the upper horizon is listed first and the lower horizon is listed last. The abbreviations represent the following stratigraphic formations:

DG/MCR	– Dales Gorge / Mt McRae Shale
MCR/BB	– Mt McRae Shale / Bruno's Band
MCR/MCR	– Mt McRae Shale / Mt McRae Shale
BB/SYL	– Bruno's Band / Mt Sylvia Formation
NOD	– Nodule zone in Mt McRae Shale
EFFZ	– East Footwall Fault Zone

For this reason it is relatively simple for a failure surface to propagate near the crest of slope since only the tensile strength of the weakest link need be exceeded. A predominant discontinuity at the crest of a slope is therefore not normally required for failure to occur. Numerous examples of this are available at Mt Newman, where failures have occurred where tensile failure has developed in the upper part of the failure surface along a series of complex joint sets forming an extremely 'rough' failure surface near the crest of the slope.

However the opposite applies at the toe of a slope. Here normal stresses are relatively high, shear stresses are at a maximum and predominant, relatively smooth discontinuities are normally required for failure to occur. Failure through intact rock bridges is possible but unlikely given the relatively high intact strength of the rock (UCS greater than 60 MPa) compared to the relatively low shear stresses imposed on them (a few MPa at the most Ref 9.1).

For the failures listed in Table 9.1, the mechanism of failure normally involves sliding along a relatively smooth bedding surface near the toe and propagating up through the slope through a series of joint sets. Due to the complex structural geology of Mt Whaleback each of the failures is different and those which do not fit into this model exactly are close to it. It indicates that it is the strength of these relatively smooth predominant discontinuities which has the controlling influence on slope stability on the South Wall of Mt Whaleback rather than any other rock strength. It is this strength which has been determined from the back analysis results and compared to laboratory values.

9.3 BACK ANALYSIS RESULTS

The back analysis results listed in Table 9.1 are also presented in Figure 9.1 and the method of back analysis used is as follows.

The information required from a back analysis is to determine the shear strength along the failure surface at the moment of failure. This shear strength is most commonly defined in terms of c and Ø, and therefore there are two variables that can define the shear strength at any given normal stress. If we take the normal stress acting along an entire failure surface, it varies from zero at the two ends to some peak normal stress value approximately in the lower third of the failure surface. However, if c and Ø are varied in order to determine a factor of safety of 1.0, it is apparent that there are many combinations of c and Ø that will produce a factor of safety of 1.0 for the same failure surface. In this case the back analysis does not present a unique solution to the problem.

However if more than one failure has occurred on the same or a similar stratigraphic horizon, ie. the failure surface material properties are similar, then a back analysis can be undertaken for each failure and the results plotted together as shown in Figure 9.1. This figure shows a plot of c against Ø for nine different back analysis results from Newman. Where the lines in Figure 9.1 intersect, a unique combination of c and Ø values are given for that particular failure surface material type.

Figure 9.1 shows three main material types and the circles represent an approximate range in material strengths. It should be noted that the c and Ø curves have been truncated for clarity. Given the large number of variables between different failure surfaces, the results are in good agreement both with each other and with laboratory test results as discussed below.

9.4 COMPARISON WITH LABORATORY TEST RESULTS

Comparison of results from back analyses with laboratory test results is shown in Figure 9.2 for Mt McRae Shale which is one of the major rock types involved in these failures at Newman.

Lilly (Ref 9.5) presented the results of shear tests on Mt McRae Shale at Mt Tom Price and Paraburdoo in WA and stated that 'pit slopes in Mt McRae Shale were almost entirely controlled by the orientation and shear strengths of the bedding planes'. Lilly's shear test results for smooth bedding surfaces are also included in Figure 9.2 as $c = 110$ kPa, $\emptyset = 24°$ (his data points have been omitted for clarity).

MacFarlane (Ref 9.3) has also critically reviewed the large amount of shear test data available at Mt Newman and concluded that much of it is not relevant due to excessively high normal stresses during the shear test or other poor test procedures. However he demonstrated that where laboratory tests are undertaken at normal stresses similar to those existing in the field and that erroneous test results are not included, the laboratory results match the back analysed results very well. MacFarlane's acceptable data points are also shown in Figure 9.2.

It can be seen that Lilly's curve is a reasonable lower bound fit to MacFarlane's data between the normal stress range 0.5 – 2.5 MPa, but possibly overestimates strength in the normal stress range 0 – 0.5 MPa. It should be pointed out that Lilly (Ref 9.5) recognised this and drew a curve down to $c = 0$ in this low stress range.

However if we compare the back analysed shear strength for failures along bedding surfaces in the Mt McRae Shale we get $c = 15$ kPa, $\emptyset = 26°$ and this is an excellent lower bound fit to MacFarlane's data. A detailed investigation has also been undertaken by Golder Associates (Ref 9.4) to determine the shear strength of Mt McRae Shale bedding surfaces. After evaluating detailed surface roughness data they concluded that a curve fit almost identical to the back analysis results shown in Figure 9.2 was appropriate.

Table 9.1 can also be compared with the strength results that were actually used for the stability analyses on the South Wall (Ref 9.4). The actual data points used by Golder Associates are shown in Table 9.2 and it indicates that they used different c and Ø values for increasing normal stress. The range of strengths that they used was c = 10 – 50 kPa, and Ø = 20 – 34°. Therefore it can be seen that the back analyses results, the laboratory test data and the strength values actually used in the stability analyses are all of the same order of magnitude (Refs 9.1, 9.2).

Table 9.2 **Strength Parameters of Upper Mt McRae Shale Bedding Planes Used for Stability Analysis**
(after Golder Associates 1986)

Normal Stress kPa	Cohesion c (kPa)	Friction Angle Ø (degrees)
250	10	29
500	30	28
750	35	27
1000	50	26

It should be noted that the Mt McRae and the Mt Sylvia Formations are also caught up in the WFZ and the EFFZ. Hence the shear strengths obtained for these shales are quite relevant to this thesis and are, of course, crucial for stability south of the WFZ.

Finally, we can also fit a Hoek-Brown curve to this data as also shown by the dotted lines in Figure 9.2. This curve is defined by parameters UCS = 50 MPa, m = 0.075, s = 0) and although it is a good average fit to the data, it may overestimate the strength of the actual failure surface, since failure in this instance appears to be governed by the lower bound laboratory strength rather than the average strength.

Therefore it is quite clear that the linear c and Ø plot of shear strength from back analyses is applicable both to the in-situ strength of bedding surfaces and to the lower bound strength of the laboratory data for South Wall lithologies.

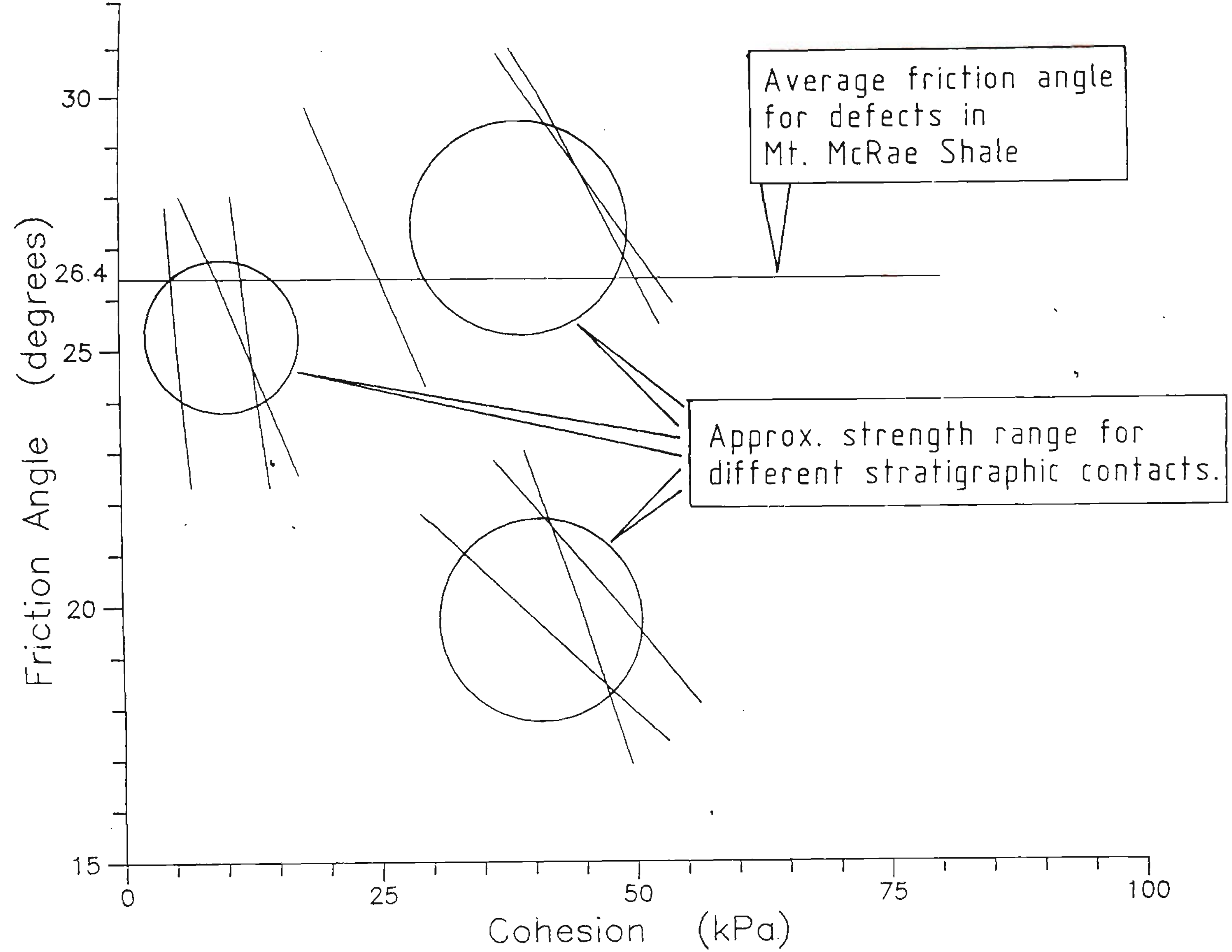
9.5 CONCLUDING REMARKS

The back analyses of the previous failures on the South Wall have correlated well with the laboratory test data and have been used for slope stability determinations on the South Wall (Ref 9.4). Nevertheless, investigations undertaken as part of this thesis indicate that the strength values obtained from back analyses of previous failures are not directly relevant to the rock mass shear strength of the Jeerinah Formation (ie. north of WFZ). They are however relevant to the shear strength of the WFZ and the EFFZ at the toe of the North Wall.

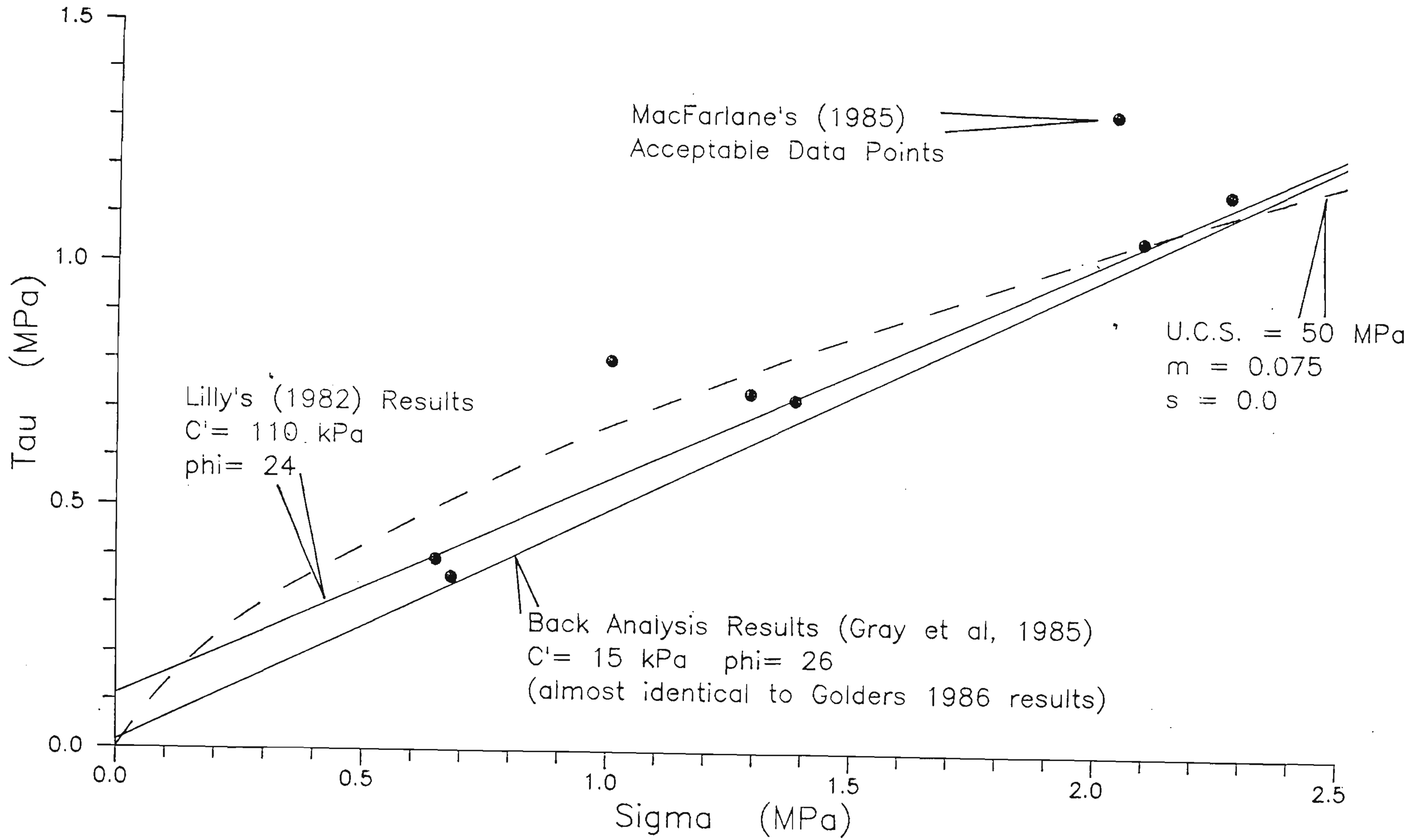
It should also be noted that the shear strength listed in Table 9.1 for the EFFZ is very close to the residual shear strength obtained for fault shales listed in Chapter 8 and indicates that both the WFZ and the EFFZ must be close to the residual strength as determined by laboratory values.

However the same cannot be said for the Jeerinah Shale which has much higher laboratory strengths. The value of the back analysis results has therefore been to only confirm laboratory strength values for the EFFZ and the WFZ. They do not indicate the strength of the Jeerinah Shale which is crucial for the stability analysis as will be shown in Chapter 10.

Finally, a study of the failures has not revealed any significant mechanism of failure which may be described as progressive. This is not surprising since 'progressive failure' (as the term is used for failures in clays and some soft rocks) is not considered to be a significant aspect of potential or actual instability in hard rock slopes.



Scale	Back Analysis Results for Newman Slope Failures	FIGURE 9.1
Drn		
Dwg No		



Scale		**Comparison of results from back analyses with laboratory test results for Mt McRae Shale**	FIGURE
Drn			**9.2**
Dwg No			

CHAPTER 10

SLOPE STABILITY STUDIES FOR FUTURE MINING

10.1 INTRODUCTION

The aim of this Chapter is to present the detailed stability analyses which have been undertaken for the North Wall of Mt Whaleback. It should be pointed out that the assessment of the stability of the North Wall has been undertaken previously (Refs 10.3, 10.4, 10.12, 10.13, 10.14, 10.15, 10.16). Some of this previous work indicated the potential for large scale instability particularly in the WFZ and Jeerinah Formation. This provided a major impetus for further geotechnical research relating to the North Wall of Mt Whaleback.

Previous stability work (Refs 10.16) was based on strength data derived from shear box testing of discontinuities and also from back analyses of previous failures on the South Wall. These investigations indicated relatively low shear strength parameters. In terms of a linear Mohr–Coulomb failure criteria, the friction angles used were variable, ranging between 15 and 30°, and the apparent cohesion values used were all less than about 50 kPa.

These strength values represented failure along discontinuities either from shear box tests or from actual failures on the South Wall. In fact, most failures that have occurred on Mt Whaleback have involved failure along discontinuities and not failure through the intact rock mass. The strength of the intact rock mass would be considerably higher than the values quoted above.

For a realistic design of the North Wall at Mt Newman a decision concerning the real shear strength of the rock mass in the field was crucial and yet not easy. This is because no large scale failures have occurred in the Jeerinah Shale and, therefore, back analyses cannot be used as a tool at this stage.

Therefore the strength parameters that were used in the earlier stability analyses of the North Wall undertaken in 1985 (Ref 10.16) were based on previous laboratory test data and on inference of rock mass strength based on South Wall back analysis results of previous failures.

This approach led to very conservative design based on low strength values. In fact, these values were close to the residual strength values for the Jeerinah Shale. For failure to occur in the Jeerinah Shale, failure would have to take place within existing shear or disturbed zones. These zones have been recognised in borecores but their widespread extent has not been confirmed. Nevertheless if it is assumed that they are extensive throughout the Jeerinah Shale, then the shear strength of these disturbed zones is of key importance to the stability of the North Wall. It cannot be assumed that this rock mass strength of disturbed shale is the same as the strength along discontinuities determined in previous work associated with the South Wall.

These disturbed zones in the Jeerinah Shale were selectively sampled from borecores and tested in the laboratory to determine their triaxial strength. These tests and the strength results obtained from them are described in detail in Chapter 8. However in summary, the shear strength results obtained from this testing programme were much higher than the shear strength results used in previous stability analyses.

Specifically, the lower-bound value of cohesion determined for this disturbed shale material was 1.3 MPa compared to cohesion values used previously (Ref 10.16) of less than 50 kPa. This confirms that previous designs were conservative. Nevertheless a cautious and observational approach has been adopted to the wall design. The structural geology has been reviewed and sections with unfavourable southerly dipping structures have been analysed to determine their stability. These are presented in detail in Section 10.6.

Part of this cautious wall design is the inclusion of a toe buttress in areas where these southerly dipping Jeerinah Shales occur. This toe buttress is designed to be excavated at a later stage of mining if measured high strength of the disturbed Jeerinah Shale can be confirmed further. These high strength values have been used in present designs in order to minimise waste stripping on the North Wall and this is described in detail in Chapter 11.

The stability of the North Wall has been analysed in two major ways as follows:

- Firstly, all of the engineering geological data collected for the North Wall has been considered and the effect on batters and berms excavated through the different lithologies has been determined. This is therefore an assessment of bench scale stability rather than an assessment of overall stability, and is primarily a geomechanics stability consideration of discontinuity data.

- Secondly, larger scale stability of both the Jeerinah and of the Brockman Formation has been considered. Wherever the Jeerinah Shale has an unfavourable orientation in relation to the North Wall, cross sections have been taken and stability analyses performed. Care has been exercised in considering for analysis, (a) north/south sections, (b) down plunge sections as well as (c) sections normal to the wall. The stability of the Brockman Formation principally involves potential failure of the toe buttress area along the WFZ and the East Footwall Fault.

10.2 METHODS OF ANALYSIS

The stability analyses presented in Section 10.6 have been performed using mainly the Spencer-Wright method (Ref 10.2, 10.6). Additional analyses have also been performed using Bishop's method, and Sarma's method (Refs 10.1, 10.5 and 10.10).

The Spencer–Wright method was used because it has several inherent advantages over other methods. Firstly, it can handle non-circular failure surfaces of irregular shape and can therefore follow the Jeerinah contacts reasonably well (there are of course some limits or constraints to the shapes it can handle).

Sarma's method, on the other hand, is cumbersome for curved failure surfaces because it necessitates a large number of slices to follow the failure surface accurately. Secondly, Spencer's method has the facility to perform sensitivity analyses very easily and these are presented in Section 10.6.

However Sarma's method has the advantage that inter-slice shear strengths can be simulated so that one gets a better 'physical' indication of the mechanism of failure in the slope. Nevertheless it is not suited to performing a large number of runs or sensitivity analyses and has therefore only been used to check the results obtained using Spencer's method.

Bishop's method was used to determine stability through the WFZ where a slip surface of circular or near-circular shape is quite likely, since the rock mass strength can be very low. Nevertheless failure along circular slip surfaces is not the only mode of failure assumed for the WFZ since anisotropic strengths have been assigned to the WFZ and, therefore, Spencer's method has also been used.

The engineering geology data has been analysed using the DCONB discontinuity analysis software, using conventional stereoplots as well as following the work of Piteau (Ref 10.8). In addition, wedge and plane failure calculations have been performed using the software developed by CANMET (Ref 10.7) using a probability overlay. This is described in Section 10.5.

10.3 PREVIOUS NORTH WALL DESIGNS

The original design work for the North Wall was completed by Dames & Moore in 1971 and 1972 (Ref 10.3 and 10.4). This design recommended an overall slope angle of 35° and this was supported by stability analyses which gave factors of safety of between 1.2 and 1.7.

However the structural interpretation that Dames & Moore assumed at the time of their study was mainly inferred, since their interpretation was based on very limited drilling north of the WFZ. They indicated small scale folding behind the WFZ and also recognised its major influence on the stability of the North Wall.

The Dames & Moore recommendations remained substantially unchanged until July 1980 when Golder Associates recommended an increase in overall slope angle to 45° (Ref 10.12). The reason for the increase in slope angle by Golders was that there was 'little or no evidence of folding in the Jeerinah Formation north of the boundary fault. Exposures of dolerite . . . suggests generally horizontal bedding attitudes. A fold axis shear in the iron formation can be seen to be well cemented and hence unlikely to act as a potential failure surface'. Moreover Golder Associates also stated that, 'preliminary analyses ... indicate factors of safety in excess of 1.5 for a 45° overall angle for Slope 8' (the East Pit North Wall). The 45° overall slope was only a 'provisional' recommendation but it remained basically unchanged until 1985. Very few stability analyses were undertaken to check the original provisional recommendation for a 45° overall slope angle. Some changes were, however, made between 1980 and 1985. By 1982 the North Wall design was refined by Golders to include different slope angles for different rock types (ie. 42° for the Fault Zone, 48° for the Dolerite and 39° for the Shale, see Ref 10.14).

By November 1983 southerly dipping structures within the Jeerinah Formation were definitely established after the results of a drilling programme became available. Golder Associates (Ref 10.14, 10.15) had previously stated that 'It should be noted that, although the geological sections show the shale–dolerite contacts to be dipping to the south (into the pit) the actual strike of the contacts is at an angle to the wall, and hence sliding failure on the shale foliation contacts is not expected'.

A drilling programme in 1984 confirmed that the Jeerinah Formation generally dips to the south and the west into the pit. Stability analyses undertaken in 1985 (Ref 10.16) indicated that a 45° overall slope for the North Wall would fail based on strength parameters used at that time. It was also pointed out that the North Wall must be designed to follow the structure very closely if stability is to be maintained.

From the work in 1985, two wall designs were proposed for the North Wall. These were the Southern Option which was basically south of the main East Synclinal keel, and the Northern Option which stripped back all the in–dipping Jeerinah Shale north of the WFZ. These two options are shown in Figure 10.1 as well as the current design as of 1985.

It was considered in 1985, before the new testing programme, that both the Southern Option and the Northern Option involve considerable economic disadvantages and also that the design, current at that time, might lead to potential instability. The Southern Option had the disadvantage that it sterilised a great deal of ore and also reduced the operating width in the East Pit. The Northern Option, on the other hand, necessitated stripping back a considerable volume of excess waste. Therefore, it was essential to try and determine an optimum wall design somewhere between these two limits, and the present investigations were initiated in order to achieve such an optimum design.

10.4 PHYSICAL PROPERTIES BASED ON CURRENT TEST PROGRAMME

The physical properties of the rocks on the North Wall have been described in detail in Chapter 8 and therefore will not be repeated here. The two most important strength values which have been used in this Chapter are those which relate to the disturbed Jeerinah Shale and Fault Shales, and the residual strength values for Jeerinah and Fault Shale. In summary, these are c = 1.3 MPa, $\emptyset = 39°$ and $c_r = 0$, $\emptyset = 30°$ respectively and are shown in Figure 8.8 in Chapter 8.

Both a linear and a curved fit to the raw data points has been undertaken in Chapter 8 but for all practical purposes, a linear fit is quite acceptable and hence strength values in terms of m and s parameters have not been used in this Chapter. These strength values have been used in a stability analyses which are described in detail in Section 10.6.

In addition the physical properties of discontinuities have been used to determine the stability of individual benches. In this case the detailed strength properties listed in Chapter 8 have been used. These strength properties are slightly more specific than the more generalised properties and they refer to strengths on discontinuities only. These discontinuity strength properties used in Section 10.5 of this Chapter are shown below in Table 10.1.

Table 10.1 **Discontinuity Strength Properties**

	Unweathered		Weathered	
	c (kPa)	phi°	c (kPa)	phi°
Jeerinah Dolerite	–	–	100 ± 50	48 ± 3°
Jeerinah Shale	35 ± 15	30 ± 3°	15 ± 15	20 ± 3°
Joffre Formation	40 ± 10	25 ± 3°	20 ± 10	17 ± 2°

A waviness component of 8° was applied to the base friction angle of the dolerite (40° + 8° = 48°), whereas no waviness component was applied to the shale. A reasonably high range was also applied to the apparent cohesion values in order to simulate their natural variability in the field. These points are discussed further in Section 10.5.

10.5 BENCH STABILITY BASED ON RECENT WORK

10.5.1 Introduction

Final pit limits in the proposed North Wall designs (Options 1, 2, 3, 4 and 4A) will be established in the Jeerinah Formation to the north of the WFZ, the WFZ itself and various formations of the Hamersley Group (Brockman Iron Formation, Mt McRae Shale, Mt Sylvia Formation) to the south of the WFZ. Interim pit slopes will also be established in these various formations, particularly those of the Hamersley Group. The geological formations listed above include a wide range of rock types with very different rock mass strengths (refer to Chapter 8) and discontinuity characteristics including type, orientation, spacing, persistence, surface roughness, mineral infillings and structure (refer to Chapters 4 and 6).

The discontinuity systems present in the Jeerinah Formation including the Dolerite A and Shale A units, and to a lesser extent the Joffre Member to the south of the WFZ have been shown to be influenced primarily by a range of lithological types and deformational effects related to folding about a variable fold axis direction. The discontinuity systems have also been modified within a 'disturbed' zone, adjacent to the WFZ, where drag folding occurs both within the Jeerinah Formation to the north and the Hamersley Group to the south. Variations in discontinuity orientation, spacing (ie. increased fracturing) and surface roughness have been detected within the 'disturbed' zone and these are also described in detail in Chapter 6.

Penetrative weathering along the WFZ has also been detected at depths up to 300m to the west of 7000E and can be associated with a rock mass strength reduction within the 'disturbed' zone. This rock mass strength reduction can be attributed to both changes in the rock fabric (ie. a general strength reduction) and the formation of weathered mineral infillings (eg. chlorite, clay, etc) on discontinuity surfaces with a subsequent reduction in discontinuity shear strength.

The stability assessment of individual benches outlined in the following sections has considered all of the variations in discontinuity parameters outlined above, where appropriate, together with the effect of moderate groundwater pressures. Sheared stratigraphical contacts are known to exist within the Jeerinah formation and these are assessed separately because of the potential for instability over a larger area.

The variation in rock mass strengths and discontinuity systems of the various stratigraphical units has resulted in several techniques being employed to assess <u>bench</u> stability, as follows:

(a) **Wedge Stability**

In the stratigraphical units where regular discontinuity systems have been developed and where data is available (ie. Jeerinah Dolerite, Jeerinah Shale and Joffre Member), the stability assessment has concentrated on the influence of the dominant discontinuity sets upon wedge instability (refer to Figure 10.2).

(b) **Plane Failure**

Where sheared stratigraphical contacts are known to occur within the Jeerinah Formation, with Dolerite overlying Shale, the instability associated with larger scale plane failure has been assessed (refer to Figure 10.1).

(c) **Rock Mass Failure**

In the WFZ where Hamersley Group sequences have been extensively sheared, altered, oxidised and weathered, a rock mass strength approach has been employed because of the absence of any regular discontinuity system.

The discontinuity sets involved in wedge formation within the various stratigraphic units described above include bedding, cleavage, jointing and faulting. In the Jeerinah Dolerite the 'disturbed' zone adjacent to the WFZ is characterised by jointing which is largely fault related.

Where a Dolerite unit overlies a shale unit this steeply inclined jointing could form a tension crack allowing plane failure to develop along a sheared contact, particularly within the 'disturbed' zone due to increasing dip associated with drag folding. In the Jeerinah Shale the dominant discontinuity type on shallow-dipping, southern fold limbs is an axial plane, slaty cleavage with south-dipping jointing also present. The instability of the Joffre Member appears most likely along the bedding dip on southern fold limbs and locally with axial-plane cleavage and shears in fold hinges and possibly low angle normal faulting.

The potential plane and wedge instability has been assessed by means of limit equilibrium methods using the PLAFAM and WINTAM software developed by CANMET (Ref 10.7). This software allows the variability associated with input parameters (discontinuity orientation, shear strength parameters, unit weight, etc) to be included in the analysis by assuming a normal distribution with the variability about a mean value indicated by the standard deviation. The Monte Carlo simulation procedure is used to perform a minimum of 200 individual factor of safety calculations with the mean value quoted. The probability of failure is also determined by expressing the number of cases which fall below the limiting equilibrium condition (ie. Factor of Safety = 1.0) as a percentage of the total sample size.

The stability assessment for individual benches described in the following sections has enabled the optimum bench design to be determined for the Jeerinah Dolerite, Jeerinah Shale, WFZ material and Joffre Member.

10.5.2 Bench Stability of the Jeerinah Dolerite

The Jeerinah Dolerite is characterised by a regular system of steeply-inclined jointing which is modified to a varying extent by drag folding adjacent to the WFZ. A 'disturbed' zone up to 15–20m wide is indicated by detailed analysis of discontinuity data, in which steeply-inclined, southerly-dipping jointing, of varying orientation, dominates.

To the north of this 'disturbed' zone the incidence of south-dipping jointing is progressively reduced with north-dipping jointing and faulting present.

Two potential failure mechanisms involving jointing within the Jeerinah Dolerite have been identified which are either wedge failure or plane failure as shown in Figure 10.2.

- **Wedge Failure**

The potential for wedge failure involving the Whaleback Fault related jointing is highlighted by existing exposures within the East Pit in the Jeerinah Dolerite. This wedge failure is difficult to analyse in detail because of the variable orientation of jointing as demonstrated by the data for Bench 16 in the East Pit (refer to Figure 10.3).

The area outlined on Figure 10.3 includes the discontinuities that could combine to form individual wedges. (Note that Figure 10.3 shows poles not great circles.) The potential for wedge failure is shown to be higher for a 70° batter face than a 50° face due to the steeply-inclined jointing.

In order to further assess the potential for wedge failure adjacent to the WFZ, the minor discontinuity pole concentrations shown in Figure 10.3 have been assumed to represent distinct sets and their mean orientation determined, together with the variation of both dip and dip direction.

This data is summarised in Table 10.2 and graphically on Figure 10.4 which confirms the increase in the mean dip of jointing within the 'disturbed' zone, adjacent to the WFZ. The mean dip is shown to vary from 58° to 75° for this disturbed zone (see Table 10.2).

Table 10.2 **Variation of Discontinuity Orientation**

		Jeerinah Dolerite (1)				Jeerinah Shale A			
		Dip (deg)		Dip Direction		Dip (deg)		Dip Direction	
Discontinuity		Mean	SD	Mean	SD	Mean	SD	Mean	SD
Jointing	B	-	-	-	-	66	8	272	12
	D	62	5	105	9	80	4	112	8
	E	58	12	131	14	-	-	-	-
	F	63	12	157	14	-	-	-	-
	F1	66	12	184	19	-	-	-	-
	G	-	-	-	-	56	9	168	9
	H	72	12	206	11	-	-	-	-
	I	75	7	229	9	73	7	216	8
Cleavage		-	-	-	-	48	7	210	10

<u>Notes</u>:

(1) Values quoted for the 'Disturbed' Zone

The minor discontinuity pole concentrations are shown as distinct planes on Figure 10.5. This indicates a south to south-easterly plunge of the lines of intersection and hence they plunge directly out of a North Wall design with a trend sub-parallel to the WFZ.

For wedge failure to be kinematically possible, the plunge of the line of intersection must dip at an angle less than that of the slope face in order to 'daylight' and greater than the friction angle of the relevant surfaces.

The outlined area on Figure 10.5 confirms that the potential for wedge formation is greater for a 70° batter face than for 60° or 50° faces. The friction angle shown on Figure 10.5 is the base friction angle determined by triaxial testing of disturbed material and does not include the component due to surface roughness. This was determined by field measurement (refer to Chapter 6) and found to be 8 ± 3° and this has been included the stability analyses described below.

The results of a detailed stability analysis utilising the CANMET software programme WINTAM are summarised in Table 10.3 with details of the piezometric surface used in the 'wet' case shown on Figure 10.6.

Table 10.3 reveals that batter faces with a design angle of 50, 60, 70 or even 80° are all stable in the Jeerinah Dolerite. Values for the probability of failure were found to be very low (less than 2%).

Table 10.3 **Wedge Instability Within the 'Disturbed' Zone of the Jeerinah Dolerite**

Wedge Formation	I/Section Plunge	Sliding	Mean Factor of Safety: 70° Batter Dry	70° Batter Wet	80° Batter Dry	80° Batter Wet
D & I	49/157	Both planes	7.4	6.8	5.6	5.2
E & I	54/161	Both planes	6.9	6.3	4.8	4.4
F & I	63/170	Both or Plane F	9.6	5.1	4.6	
F1 & I	65/175	Plane F1				10
H & I	72/190	Both planes				10
D & H	56/143	Both planes		10	5.1	4.7
F & H	63/155	Both or Plane F	10	6.5	6.0	
F1 & H	63/155	Both planes				10
D & F1	58/138	Both planes		10	5.1	4.7

Notes:
(1) Discontinuity shear strength is C = 100 ± 50 kPa Ø = 48 ± 3°
(2) I/Section Plunge is the Intersection Plunge Angle
(3) The factor of safety of both 50° and 60° batters is greater than 10 for all cases considered.

° **Plane Failure**

Extensive shearing at the Shale A/Dolerite B contact has been detected by diamond drilling between 7320E (D192) and 7400E (D256). A reduction in the intact rock strength of both the Shale and Dolerite adjacent to the contact is also associated with this shearing (refer to Chapter 6). As this contact will be 'daylighted' by the proposed North Wall designs between approximately 7160E and 7360E, the potential for plane failure involving the overlying Dolerite has been assessed. General details of this failure mode are shown on Figure 10.2 with the cross-sectional geometry and position of the piezometric surface shown on Figure 10.6.

A complete set of structural geology cross-sections are included in Appendix A and these indicate a contact dip of about 20–25° for 7160E–7320E, increasing to 40° at 7400E for the Jeerinah Dolerite B overlying the Jeerinah Shale A. The sensitivity of instability to this variation in contact dip has been assessed together with three groundwater regimes (dry, 3m and 6m piezometric heads in a tension crack) and two bench designs being 70° batter/8m berm and 50° batter/12m berm. The results are shown below in Table 10.4 for an acceptable factor of safety of 1.2:

Table 10.4 **Contact Dip Angle Required to Produce a FOS = 1.2**

	70° batter / 8m berm			50° batter / 12m berm		
	Dry	3m	6m Piezo Head	Dry	3m	6m Piezo Head
Unweathered Contact Shale	31°	29°	27°	45°	45°	45°
Clay Coated Contact	19°	18°	17°	20°	19°	18°

It can be clearly seen that instability of batter faces is mainly dependent upon the contact dip angle and the shear strength of the contact shale. Bench design and moderate groundwater pressures have a reduced influence.

The size of any potential failure is also dependent upon the continuity or persistence of jointing, both laterally and vertically in the Dolerite. Field observations indicate that the fault-related jointing adjacent to the WFZ is persistent laterally with irregular jointing traceable for 10–20m.

Persistence in a vertical direction is more limited with the discontinuity data analysis presented in Chapter 6 indicating a mean value of up to 4m with standard deviation values also of 4m. The size of any potential failure is therefore also dependent upon the bench design. A flatter overall slope near the shale/dolerite contact results in the wall being closer to this contact over a greater number of benches than a steeper wall design (see Figure 10.6). Therefore this flatter wall design in this case has a higher probability of failure because of the reduced height of the tension cracks needed to initiate failure (refer to Figure 10.6). For this reason a steep bench design utilising a 70° batter/8m beam is recommended with failure blocks likely to limited to only one bench in height.

10.5.3 Bench Stability of the Jeerinah Shale

The Jeerinah Shale is a medium strong slate/phyllite with a well-developed discontinuity system which is strongly influenced by the style of folding and the direction of the major fold axes. Revised interpretations of the structural geology presented in Appendix A reveal three factors of significance for bench stability:

- The WFZ generally intercepts shallow-dipping, southern fold limbs in which the dominant foliation is a well-developed, axial-plane slaty cleavage. This cleavage is occasionally co-planar with bedding and south-dipping jointing is also present on the southern fold limbs.

- The direction of the major fold axes rotates from northwest - southeast in the extreme east of the East Pit to approximately east-west to the west of 7750E. A similar rotation in the discontinuity system of the Jeerinah Shale is expected to occur but this cannot be confirmed until suitable exposures become available.

- Drag folding occurs adjacent to the WFZ, to the west of approximately 7550E, resulting in localised increased southerly-dip of the Jeerinah A and B units.

Examination of the surface outcrop plans (see Appendix C) reveals that the WFZ, and hence the North Wall pit designs rotate by approximately 35° to 40° to the west of 6480E. To assist with the stability assessment of the Jeerinah Shale, three structural domains have been defined on the basis of the trend of the major fold axes and that of the WFZ, as follows:

Eastern Domain – East of 7750E

Consists of the Shale A unit with a northwest – southeast fold axis system and hence a south-westerly dipping axial-plane cleavage. The WFZ has a trend of approximately 070 – 075°.

Central Domain – Between 6480E – 7750E

Consists of both Shale A and Shale B units with an east-west fold axis system and possibly a southerly-dipping axial-plane cleavage. The WFZ has a trend of approximately 070 – 075°.

Western Domain – West of 6480E

Consists of the Shale B unit with an east–west fold axis system/southerly dipping axial–plane cleavage. The WFZ has rotated with a trend of 105–115°.

° **Eastern Domain**

The discontinuity system determined for the Jeerinah Shale A by detailed mapping of the area to the north–east of the East Pit is shown on Figure 10.7. This figure indicates that wedge failure involving a number of joints sets and cleavage is kinematically possible for batter faces dipping at more than 60°. The mean orientation values of the discontinuity sets involved are summarised in Table 10.2 and the direction of the lines of intersection for the various wedge combinations are included in Table 10.5. Table 10.5 confirms the importance of the axial–plane cleavage and joint set G for wedge instability.

Table 10.5 **Summary of Potential Wedge Formation Types Within the Jeerinah Shale**

	Structural Domain					
	West		Central		East	
Wedge Formation	I/S Plunge	Sliding On	I/S Plunge	Sliding On	I/S Plunge	Sliding On
Jointing B & G	50/184	Both	50/184	Both	50/213	G
Joint B & Cleavage	50/184	Both	50/184	Cl	50/213	Cl
Jointing D & G	54/180	Both	54/180	Both	54/194	G
Joint D & Cleavage	49/180	Cl	49/180	Both	49/194	Both
Jointing I & G	–	–	54/120	Both	54/150	Both
Joint I & Cleavage	–	–	–	–	50/213	G

Notes:
(1) Eastern Domain – East of 7750E
(2) Central Domain – Between 6480E and 7750E
(3) Western Domain – West of 6480E
(4) 50/184 represents dip and dip direction of the plunge of the intersection of wedges
(5) I/S Plunge is the Intersection Plunge Angle
(6) Cl represents Cleavage

The results of a detailed stability assessment are presented on Figure 10.8 for the wedge combinations listed in Table 10.5. A range of batter angles, from 50 to 80° and the effect of moderate groundwater pressures are included in this analysis with details of the piezometric surface adopted, shown on Figure 10.6. The base plot of Figure 10.8 represents unweathered Jeerinah Shale, located away from the influence of the WFZ, with shear strength parameters of $c = 35 \pm 15$ kPa and $\phi = 30 \pm 3°$ being adopted. The most unstable wedge combination is found to involve the axial plane cleavage and the steeply inclined jointing of set D with a mean factor of safety of 1.34 for a 70° batter face and 1.01 for a 80° batter face subjected to moderate groundwater pressures. In summary, the batter scale stability of wedges in the Jeerinah Shale is high.

The instability associated with clay-coated discontinuity surfaces within the 'disturbed' zone adjacent to the WFZ is shown on the overlay to Figure 10.8. This also indicates that a high probability of failure occurs in batter face angles inclined at angles greater than 60°. A 70° batter face is marginally stable wherever the axial plane cleavage and joint set D are well developed with a mean factor of safety of 1.11. This must be considered to be only relevant close to the WFZ.

° **Central and Western Domains**

The level of wedge instability in both the Central Domain and Western Domain is similar due to the controlling factor of a south-dipping axial-plane cleavage.

The discontinuity system associated with a major fold axis trend of east-west is shown on Figure 10.9 for the Central Domain and Figure 10.10 for the Western Domain. The overlays to Figures 10.10 and 10.11 indicate that wedge failure is kinematically possible for batter faces dipping at more than 50° and is therefore potentially more of a problem than for the Eastern Domain.

The results of the stability assessment for the Western Domain are presented on Figure 10.11. The stability analyses for the Central Domain are generally similar to those for the Western Domain and are not presented separately. The base plot of Figure 10.11 indicates that all wedge configurations in unweathered Jeerinah Shale are stable with a minimum mean factor of safety value of 1.56. For weathered Jeerinah Shale material, however, with clay-coated discontinuities instability will occur for batter angles dipping at more than 60°. This is associated with three of the four wedge types analysed (refer to the overlay to Figure 10.11). Instability is also indicated for a 60° batter face involving the axial-plane cleavage and joint set B.

° **Summary of Bench Instability in the Jeerinah Shale**

An axial-plane, slaty cleavage has been identified as the dominant foliation within the Jeerinah Shale and in combination with steeply-inclined jointing the potential for wedge failure has been assessed for three structural domains.

In the Eastern Domain (to the East of 7750E) batter scale stability is generally high, particularly for unweathered Jeerinah Shale. However for highly weathered clay coated joint surfaces the stability of batters decreases rapidly whenever the batter angle is greater than 60° and groundwater pressures are present.

In the Central and Western Domains the batter scale stability is also high in unweathered shale for all batter angles. For clay coated discontinuities next to the WFZ, instability occurs for batter faces greater than 50° even for dry conditions.

10.5.4 Bench Stability Within the WFZ

A discontinuity analysis has not been performed for the WFZ since bench stability will be controlled by material strengths themselves, rather than by discontinuity strengths. This is not to say that discontinuities are totally unimportant since there is very high degree of preferred orientation for discontinuities which is parallel or sub-parallel to the WFZ. Nevertheless, much of the WFZ is highly friable and kaolinitic zones are often present.

Therefore it is reasonable to assume that bench scale failures will occur through weak materials themselves, and the material properties appropriate for the WFZ are $c = 0$, $\emptyset = 30°$ for residual fault shale and $c = 0$, $\emptyset = 13°$ for clays. However, the residual fault shale strength only applies for shearing parallel to the WFZ. For shearing across the WFZ the appropriate shear strength parameters are $c = 1.3$ MPa, $\emptyset = 39°$, ie. a much higher strength.

In addition, the WFZ consists of a series of shears, friable zones, clay layers as well as hard bands that it is impossible to generalise on the stability of individual benches within it. Where shears and clay layers are dipping out of the face at unfavourable orientations, failure will occur as is evident in the East Pit at present. However the overall orientation of the WFZ is sub-parallel to batter faces and therefore the occurrence of in-dipping shears and weakness planes is reduced.

The major problem with bench stability in the WFZ will be the problem of slaking. This is already evident in the East Pit where small scree slopes are forming on benches. Slaking has also been accelerated on the WFZ in the East Pit where surface water has been channeled down the face. These problems can be solved by grading the benches to remove excess material and also to direct the water away from the crest.

These slaking and stability problems have been considered in the wall design and the proposed design has a flat overall slope through the WFZ (31°). Stability analyses through the WFZ are really heavily dependent upon the shear strength parameters used and these are discussed further in Section 10.6.

10.5.5 Bench Stability of the Joffre Member

The discontinuity system present in Joffre Member has been interpreted from a limited amount of mapping data conducted mostly at the western end of the East Pit. The existing data indicates that the discontinuity system is poorly defined with the fold relationships generally indistinct.

Discontinuity data collected close to the WFZ suggests that a 'disturbed' zone is also developed to the south of the WFZ and is of greater width than that developed in Jeerinah Formation rocks.

The width of the 'disturbed' zone indicated by an analysis of discontinuity orientation is at least 30m. The structural elements identified within this 'disturbed' zone are shown on Figure 10.12 with the exception of bedding. The structural elements likely to influence bench stability in the Joffre Member include the following:

- shallow–dipping normal faults which may be locally steepened adjacent to the WFZ due to drag folding (refer to Appendix A for cross–sections to the west of 7120E).

- axial plane shears occurring in the hinge zones of minor folds.

- persistent, axial–plane cleavage which develops in the hinge zones of large folds.

- bedding which may be locally steepened (due to drag folding) and associated with clay infilling material adjacent to the WFZ.

East of 6480E

The overlay to Figure 10.12 indicates that wedge formation to the east of 6480E is kinematically possible involving combinations of the axial-plane cleavage/shearing with north-south trending jointing. Locally steepened low angle normal faulting or south-dipping bedding (at angles greater than 40°) could also combine with north-south jointing to produce wedge instability.

West of 6480E

Plane failure involving axial-plane cleavage/shearing, low angle normal faulting or bedding is also likely to produce local instability wherever east-west orientated benches occur, particularly to the west of 6480E (refer to Figure 10.12). The steeply-inclined, fault-related jointing (Set H) with an ENE-WSW trend would allow the formation of tension cracks for plane failure to the west of 6480E and combine with the north-south jointing in wedge formation to the east of 6480E.

Also the wide variety of potential failure modes described above and the variation in dip produced by drag folding adjacent to the WFZ limits the effectiveness of a detailed stability assessment. The kinematically possible wedges shown in Figures 10.12 and 10.13 are not particularly sensitive to batter angle. However if we consider the typical strength of Joffre discontinuities as being:

- unweathered material are $c = 40 \pm 10$ kPa, $Ø = 25 \pm 3°$,
- weathered, clay-coated surfaces $c = 20 \pm 10$ kPa, $Ø = 17 \pm 2°$ (refer to Chapter 8).

then both wedge and plane failure will be principally controlled by the friction angles quoted above. Whenever the contact dip angle exceeds them, failures are likely.

10.6 OVERALL STABILITY BASED ON CURRENT SHEAR STRENGTH RESULTS

This section describes stability analyses that have been undertaken for potentially large scale failures on the North Wall. The previous section outlined the stability of individual benches in detail, and although these can be a problem, they can be controlled by either local changes in batter angle or by artificial support. This is not the case for large scale failures which may encompass up to 15 benches. Therefore the stability assessment of large scale failures must consider deep seated failure and could involve the Jeerinah Formation, the WFZ and the Brockman Formation.

In practice, the assessment of the overall stability of the North Wall has not been undertaken in isolation. It has been performed by considering the structural geology, the mining constraints of trying to extract most of the ore from the Dales Gorge as well as trying to minimise stripping. Therefore it has been an interactive process between stability analyses, wall design and economic assessment of the different wall options.

The structural geology of the North Wall has been investigated in detail (see Chapter 4) and sections have been taken through the North Wall on potentially unstable structures. This has involved taking sections normal to the wall, in a down-plunge direction as well as in a north-south direction. The results presented in this Chapter are considered to be for the worst case (minimum factor of safety) of any section taken through the North Wall and, therefore, a comprehensive list of all possible cross sections is not given.

All of the north-south cross sections showing the structural geology and the proposed wall designs are shown in Appendix A from Figures A2 to A40. The most important cross sections used in the stability analyses are given in Appendix B in Figures B1 through to B24 and these will be described below.

10.6.1 Methodology and Pit Options Considered

The Jeerinah Formation consists of a series of dolerites (actually amphibolites) and shales (actually slates or phyllites) which are in alternating layers and are folded on a large scale. Immediately north of the WFZ, the Jeerinah Formation dips to the south and the west, ie. into the pit.

It is these in–dipping Jeerinah layers which have been the source of concern for the overall stability of the North Wall. Specifically between approximately 7440E and 6960E the Shale A unit is dipping south into the pit and overlies the Dolerite A. Also between 6360E and 6720E the Shale B unit is also dipping south into the pit on top of the Dolerite B.

Various wall designs have been proposed in order to try and overcome the problems of these in–dipping Jeerinah units and these will be discussed subsequently in Chapter 11. However for the stability analyses conducted in this Section a toe buttress has been incorporated in one of the options (Option 4) and the stability of this buttress has also been investigated.

The various options for overall pit slope design are presented and discussed in Chapter 11 (Section 11.8). However, for the following discussion it is relevant to briefly outline the Options 4 and 4A. Option 4 is a pit design which incorporates a large toe buttress in order to stabilise in–dipping Jeerinah Formation Shale horizons. This toe buttress is located in the vicinity of the WFZ and the EFFZ and 'sterilises' considerable amounts of the Dales Gorge orebody. Option 4A on the other hand, is identical to Option 4 in the upper part of the wall design but completely removes this toe buttress. Thus, the sterilised ore in Option 4 is removed in Option 4A.

Appendix B shows the most unfavourable sections which have been used in the stability analyses. For example, Figures B1 to B5 show the stability analyses which have been undertaken for Section 7200E. Figures B1 and B2 list the stability of the buttress zone for Option 4. This involves sliding along the WFZ and the East Footwall Fault. In this case the material properties used in both B1 and B2 are residual strength values (ie. $c_r = 0$, $\phi_r = 30°$).

It should also be pointed out that the figures in Appendix B are taken from the computer stability modelling and are therefore for illustrative purposes only.

10.6.2 Groundwater

The groundwater conditions shown in Appendix B are either for 'dry', 'wet' or '$r_u = 0.3$' conditions. For 'wet' conditions the water level is at RL 575 as a horizontal dashed line (see Figure B3). Results from Chapter 7 indicate that the worse case conditions are approximately RL 575 behind the North Wall and hence these have been used as a no-drawdown 'wet' case in the stability analyses.

For 'dry' conditions the groundwater table is set at a level below the failure surface. This is also represented by a horizontal dashed line as shown in Figure B1. This simulates the best possible stability that could be achieved by dewatering. It should be noted that these are not in fact dry conditions but conditions of zero uplift pressures.

For $r_u = 0.3$ condition, equivalent pore water pressures are calculated along the failure surface automatically by the computer program. The corresponding piezometric surface (or watertable) is not shown for this case in the figures.

10.6.3 Earthquake Conditions

Earthquake conditions have not been considered for several reasons. Firstly, the earthquake record indicates that earthquakes in the Newman area are infrequent and of low intensity (equivalent to about 0.02g, Refs 10.4, 10.17). Secondly, if the whole North Wall design is dependent upon earthquake events of this low magnitude then the adequacy of the developed design would be seriously in question. In conclusion, it was decided that events of this low magnitude would not significantly influence the type of cautious design that was being developed.

10.6.4 Results

The stability analyses results for Option 4 and 4A are illustrated in Appendix B. These sections are either for the buttress zones of Option 4, the overall stability of Option 4 or the overall stability of Option 4A. Although the detailed stability analysis results for Options 2 and 3 have been undertaken they are not reproduced in Appendix B since these Options are not the crucial options (ie. they are economically unattractive and similar stability criteria apply as Options 4 and 4A).

For potential failure in the buttress zones (Figures B1, B2, B7, B9, B11, B14 and B17 buttress zones), it can be seen that failure is most likely along the WFZ and the EFFZ. Where the EFFZ it is highly irregular, shear strength properties for disturbed fault shale have been used ($c = 1.3$ MPa, $\emptyset = 39°$), eg. Figure B17. This value of shear strength would lead to higher values of the factor of safety.

A summary of the stability results for Options 4 and 4A are given in Table 10.6. It can be seen that the stability analyses of the buttress zone indicate that it is only stable if 'dry' conditions exist. Table 10.6 shows that the buttress zone design is relevant for Option 4 only and fails in most instances even with a piezometric surface equivalent to $r_u = 0.3$ ('wet' conditions would be equivalent to $r_u = 0.5$).

Table 10.6 **Summary of Overall Stability Analyses of the North Wall**

Section No	Direction	Factor of Safety: Buttress Zone Dry	Factor of Safety: Buttress Zone RU = 0.3	Factor of Safety: Overall (Wet)** c = 2.0 MPa	Factor of Safety: Overall (Wet)** c = 1.3 MPa
Option 4					
7200	Normal	1.26	0.88	3.39	2.48
7120	Normal	1.35	0.99	3.07	1.92
7080	Normal	1.27	0.89	2.80	1.99
7040	Normal	1.20	0.83	3.37	2.37
7000	Normal	1.40	0.95	3.43	2.43
6960	Normal	1.69	1.17	3.63	2.80
6480	Normal	*	*	2.93	2.06
6360	Normal	*	*	4.95	3.64
7200	Down	1.04	0.69	3.35	2.41
6720	Down	*	*	6.29	4.13
6600	Down	*	*	4.43	3.34
6480	Down	*	*	5.64	4.25
6360	Down	*	*	8.10	6.04
Option 4A					
7200	North–south	*	*	4.18	2.68
7160	North–south	*	*	3.87	2.79
7040	North–south	*	*	3.22	2.30
7000	North–south	*	*	3.13	2.36

Note:

(1) Buttress Stability is not relevant for this section

(2) (Wet) refers to no drawdown of the watertable and its level is at 575 RL behind the WFZ

(3) Strength properties used:

WFZ and EFFZ	– c = 0, Ø = 30°
Disturbed Jeerinah/Fault Shale	– c = 1.3 or 2 MPa, Ø = 39°
Waste Dump Material	– c = 0, Ø = 35°

Table 10.6 also shows a summary of the analysis results for overall stability. These are all shown for 'wet' conditions (ie. worst case). It can be seen that there are two values of cohesion used for the disturbed Jeerinah Shale, these are c = 2 MPa and c = 1.3 MPa.

These represent the average and lower bound shear strengths for this disturbed Jeerinah Shale material. In all cases it can be seen that the stability is very high for overall slope failure on the North Wall even using these lower bound strengths and assuming wet conditions. The choice of cohesion value (c = 2 MPa and 1.3 MPa) does make a substantial difference to the results, but it is not significant since the factor of safety values are all very high even for c = 1.3 MPa. In addition, partial or full drainage of the slope would also increase the stability significantly.

Each one of the sections listed in Table 10.6 are different and it can be seen from Appendix B that the failure surface may often have to pass through four different material types in order for failure to occur. For example, Figure B3 shows the failure surface passing through the EFFZ, the WFZ, the disturbed Jeerinah Shale and waste dump material.

In reality the lower strength buttress material would tend to slide away from the more competent Jeerinah material as is indicated by the buttress stability results. This has been confirmed by checks we have undertaken using Sarma's method of stability analysis which shows tensile stresses developing along the WFZ (ie. between the buttress zone and the Jeerinah Formation. However, the results indicate that the overall Jeerinah Formation slope is still stable if the shear strength values used are correct.

The overall stability of the North Wall has also been checked by varying the location of the failure surface. In some instances the failure surface follows identified shear or disturbed zones within the Jeerinah Shale, whereas in others cases it follows closely the contact between the dolerite and the shale. In practical terms this makes very little difference to the overall stability results. The stability results listed in Table 10.6 are summarised graphically in Figure 10.14 for both overall stability and for buttress stability for Options 4 and 4A.

Figure 10.14 only shows the critical results for the stability of the buttress and the overall slope and is not intended as a comprehensive list of stability results. It should be noted that stability results for option 4 are presented for both 'normal' sections (ie. sections taken at 90° to the wall) and also for 'down plunge' sections (ie. sections taken in the maximum dip direction of the Jeerinah Formation). Stability results for Option 4A however are shown in Figure 10.14 for north–south cross sections only. These different cross sections are shown in Figure 10.14 because they represent the minimum stability results obtained for each Option and each individual cross section with the potential failure surface is shown in detail in Appendix B. It should be noted that cross sections through the different pit wall options and geology have been taken by using the Intergraph computer graphics system in order to determine critical sections. Also the north–south cross sections are very similar to the down plunge sections since they are approximately in the same direction. Hence stability results are also similar.

10.6.5 Discussion of Results

It is rapidly apparent from the stability results quoted above that the over–riding influence on the stability of the North Wall is the high value of cohesion assigned to the Jeerinah Shale. These physical property results have already been discussed in Chapter 8, but it is instructive to know what shear strengths would be required on the North Wall to produce either an acceptable factor of safety or to produce failure. This is in effect a sensitivity analysis of the shear strength required for the North Wall, rather than an analysis using measured or given shear strength values.

° Buttress Stability

The sensitivity analyses for the buttress zone is shown in Figure 10.15 with a factor of safety of 1.0, and in Figure 10.16 with a factory of safety of 1.25.

These figures show the combination of c and Ø parameters that will produce a factor of safety of 1.0 or 1.25. Therefore the higher the strength indicated by Figures 10.15 or 10.16, the less stable the overall structure actually is.

If the strength values for the WFZ and EFFZ are now considered in relation to Figures 10.15 and 10.16 (ie. c = 0, Ø = 30°), it can be seen that all of the sections are stable in Figure 10.15 but there are some sections which exactly fit the parameters c = 0, Ø = 30° (ie. FOS = 1.25) in Figure 10.16. Figures 10.15 and 10.16 all assume dry conditions for the buttress zone. These results confirm that the buttress zone must be dewatered in order to be stable.

° **Final Comments**

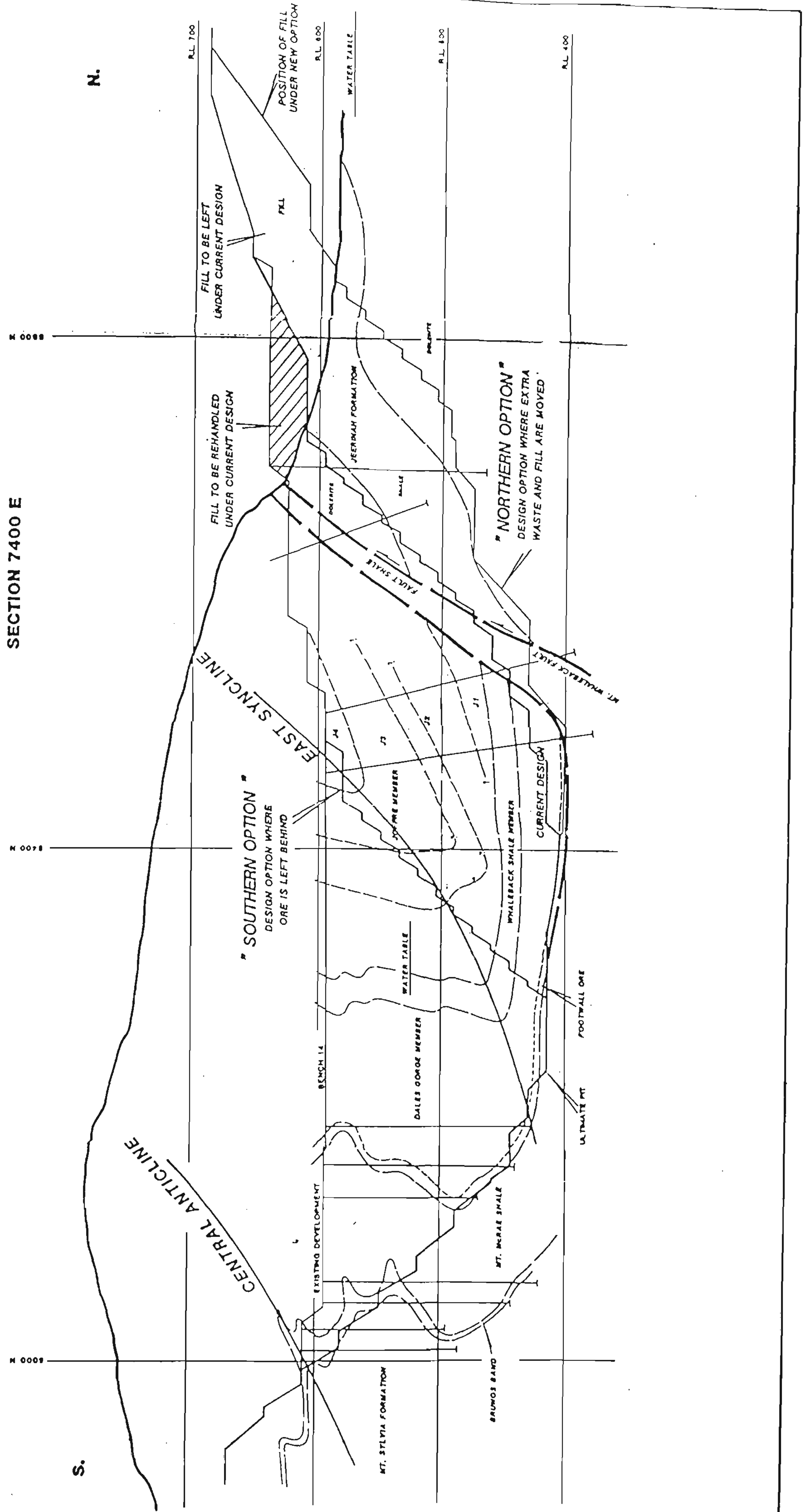
If the overall slope stability is now considered in terms of sensitivity analysis, the results can be seen in Figures 10.17 to 10.20. These are plots of c and Ø required for stability assuming FOS = 1.0. In this case the sensitivity of the disturbed Jeerinah Shale is being considered, which was originally assigned shear strength parameters of c = 1.3 MPa, Ø = 39° in the stability analyses. The strength of the Jeerinah Shale is varied in order to produce a factor of safety of 1.0 whereas the strength of all other materials remains constant (ie. waste material, WFZ, buttress zone where applicable). This sensitivity analysis for overall slope stability also assumes 'wet' conditions (ie. no drawdown) which is the same as the stability results presented previously.

Figures 10.17 and 10.18 show the results for an FOS = 1.0 and it can be seen that the least stable section is 7080E shown on Figure 10.18. If for the moment we assume that 39° is the correct friction angle, then it can be seen that the cohesion value required to just cause failure to occur on section 7080E is approximately 350 kPa Figure 10.18) which is much less than the available c = 1.3MPa.

In fact, it is approximately four times less than the value we have obtained from laboratory testing. Other sections shown in Figure 10.17 are even more stable than those shown in Figure 10.18. For example, for most sections shown in Figure 10.17, strength parameters of only, cohesion equals 350 kPa and a friction angle of 15°, would be required for stability.

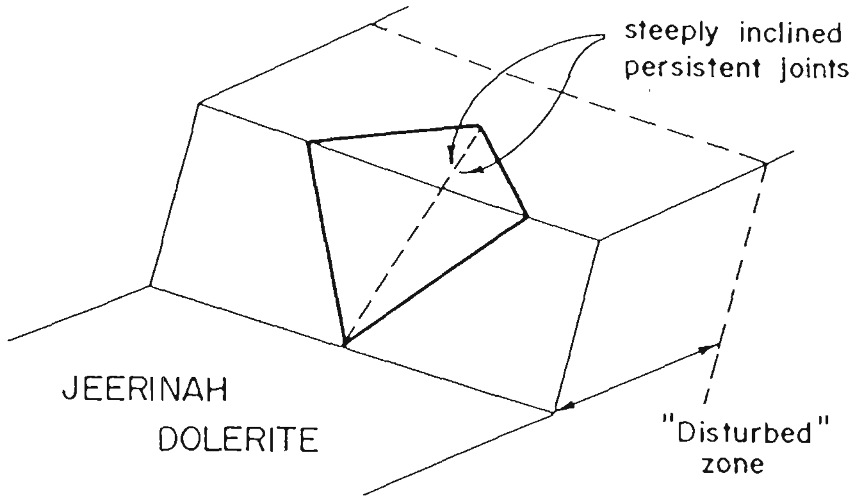
Figures 10.19 and 10.20 show a similar relationship as Figures 10.17 and 10.18 except in this case a FOS = 1.25 has been assumed (ie. an acceptable value of stability). In this case section 7080E shown in Figure 10.19 requires a cohesion value of approximately 580 kPa at a friction angle of 39° to achieve factor of safety of 1.25. This is still less than half the laboratory measured strength for disturbed Jeerinah Shale.

In summary, there is at least one order of magnitude difference between the laboratory strength test results and the shear strengths necessary to cause stability problems for overall slope failure. However, one must accept that on its own the buttress appears to be susceptible to failure under adverse pore water pressure conditions (Table 10.6) although, if such a failure does occur, it would be of limited significance in terms of its extent. The factor of safety of the buttress can be improved by drainage if necessary since the Brockman Formation has a much higher permeability than the Jeerinah Formation. Moreover, by the time the stage involving the full height of the buttress is reached, more information would have been gathered about field conditions.

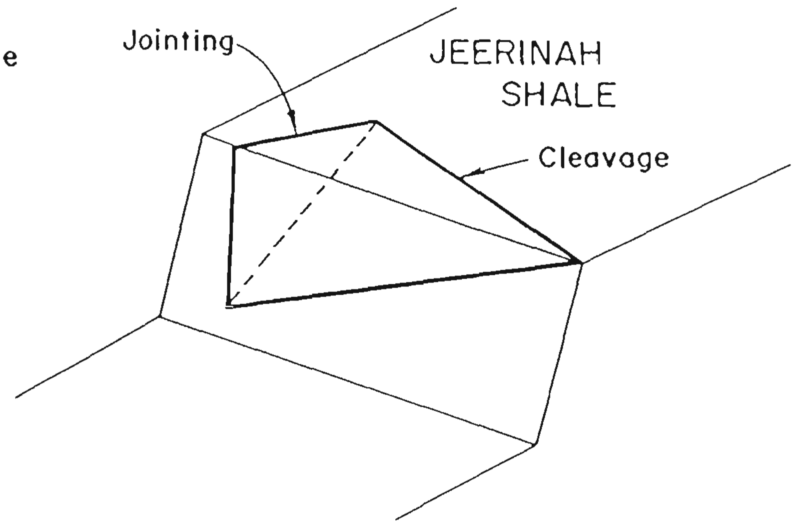


Scale	OVERALL PIT CROSS-SECTION	FIGURE 10.1
Drn		
Dwg. No.		

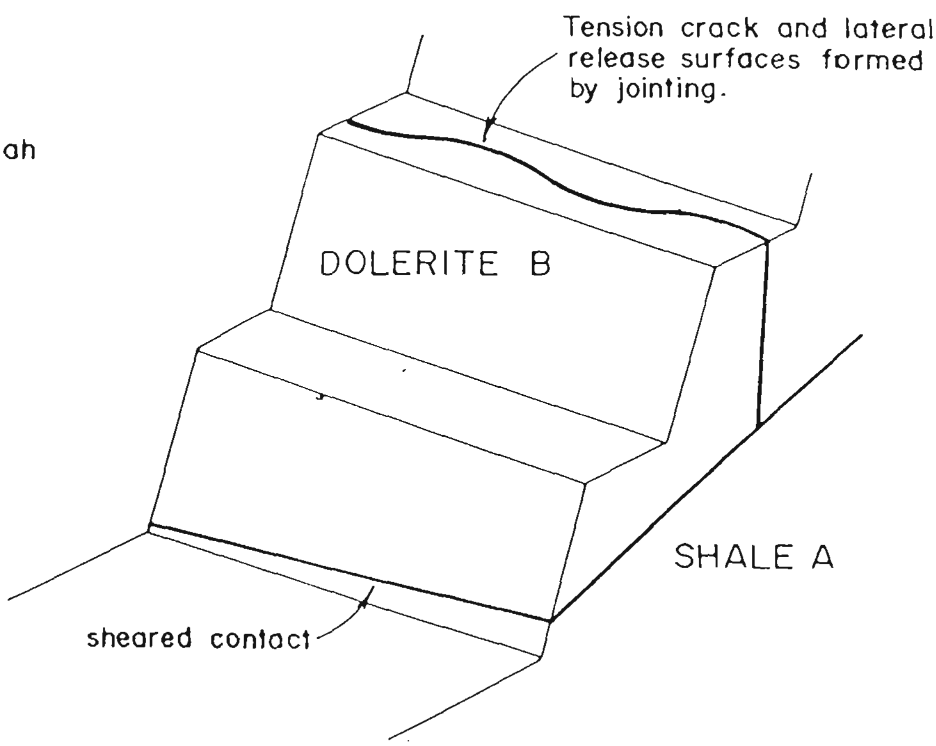
a) Wedge formation involving jointing in the Jeerinah Dolerite.



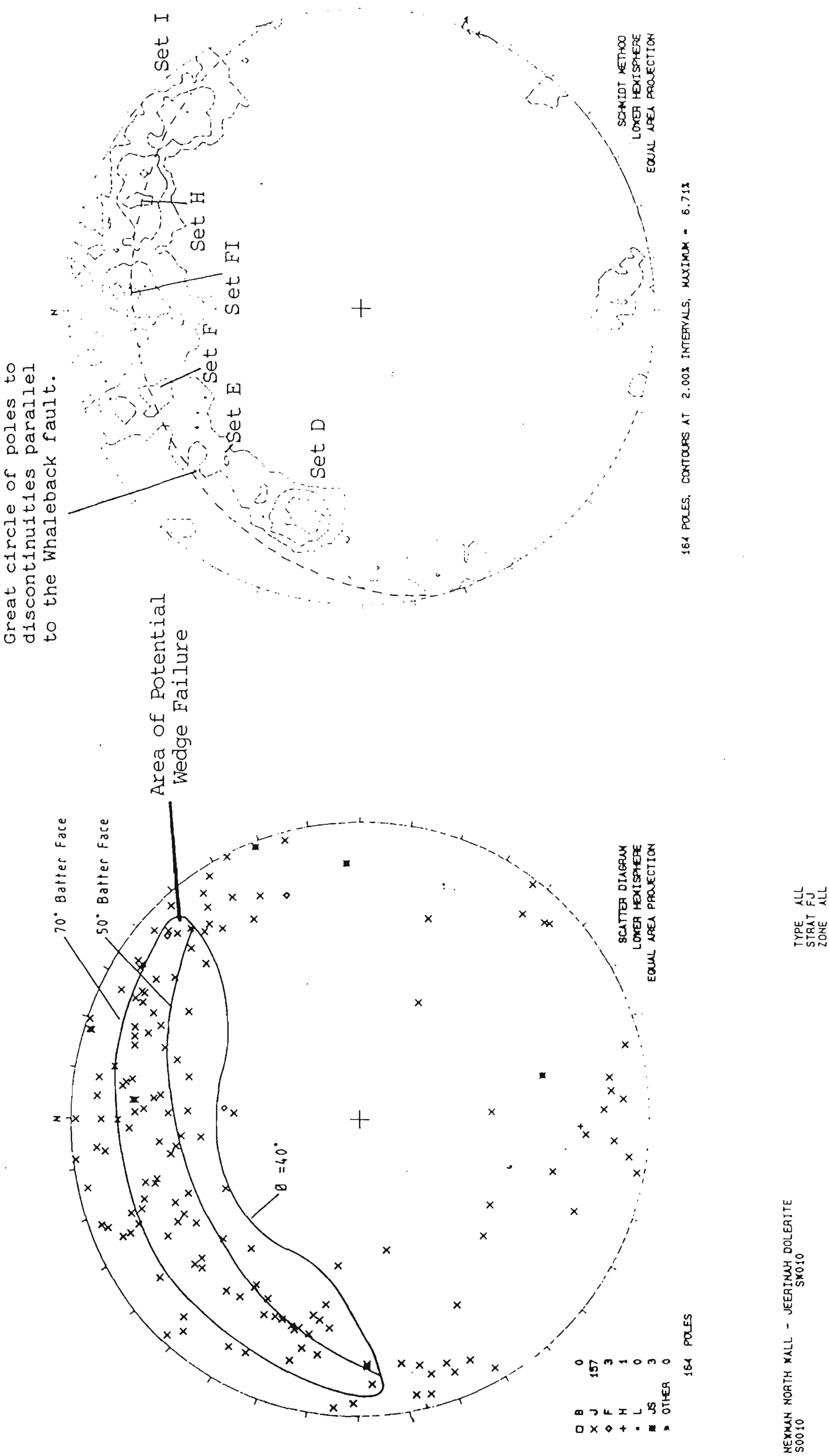
b) Wedge formation involving cleavage and jointing in the Jeerinah Shale.



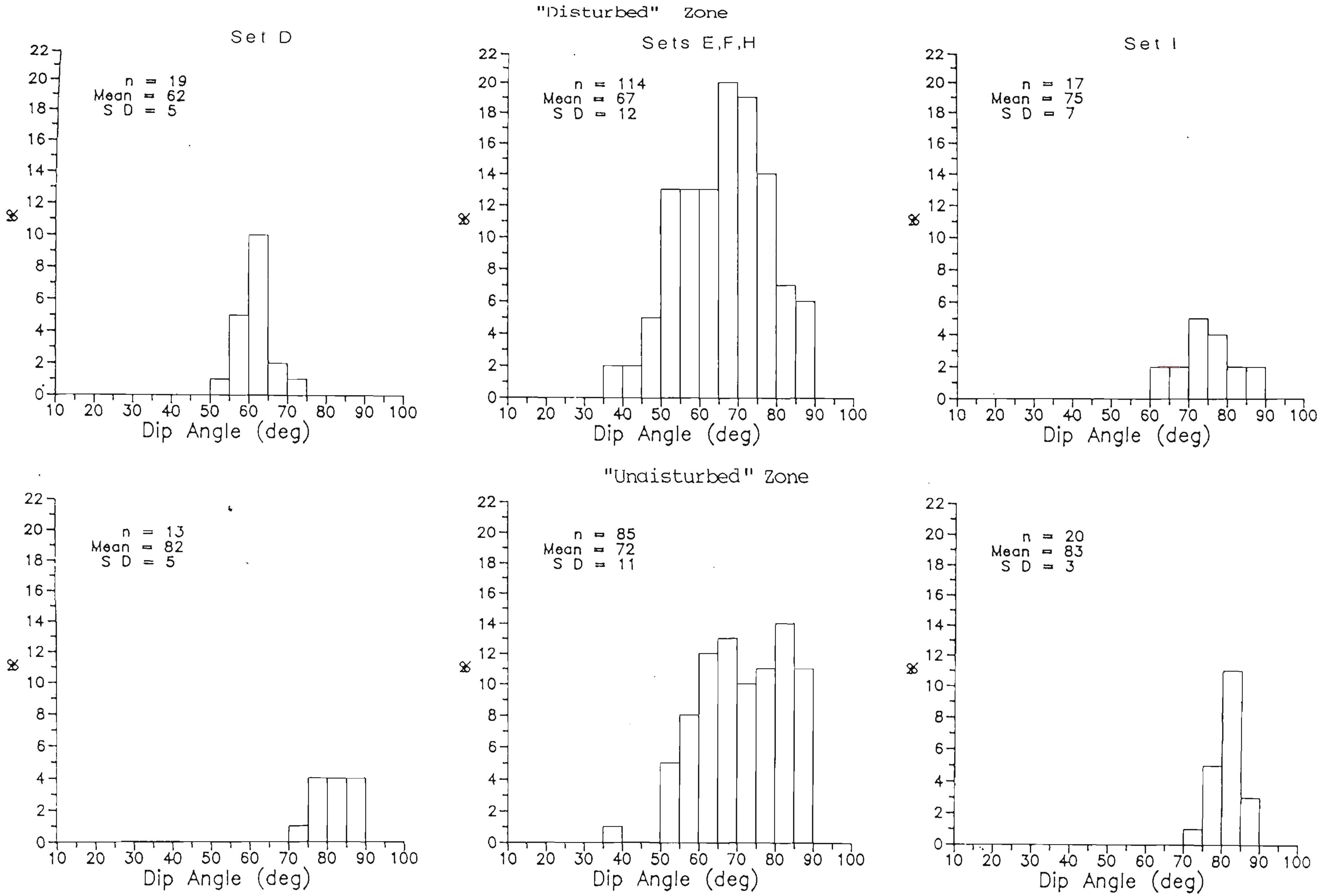
c) Plane failure involving Jeerinah Dolerite overlying a sheared contact with Jeerinah Shale.



Scale	POTENTIAL MECHANISMS OF BENCH INSTABILITY WITHIN THE JEERINAH FORMATION	FIGURE 10.2
Drn		
Dwg. No.		



Scale	POTENTIAL WEDGE FORMATION IN THE JEERINAH DOLERITE A: BENCH 16	FIGURE 10.3
Drn		
Dwg. No.		



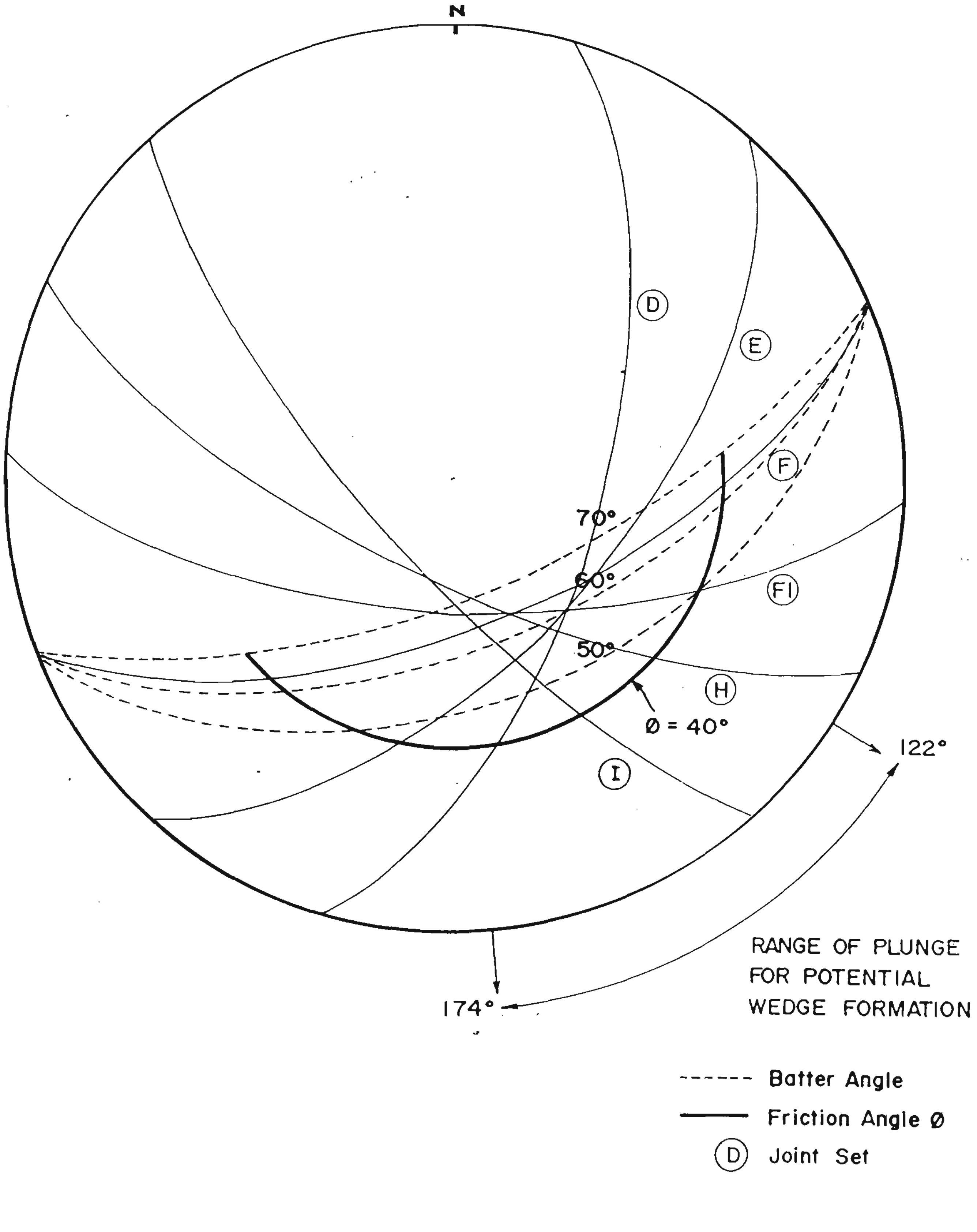
Scale

Drn

Dwg. No.

VARIATION OF MEAN DIP FOR JOINT SETS IN THE JEERINAH DOLERITE

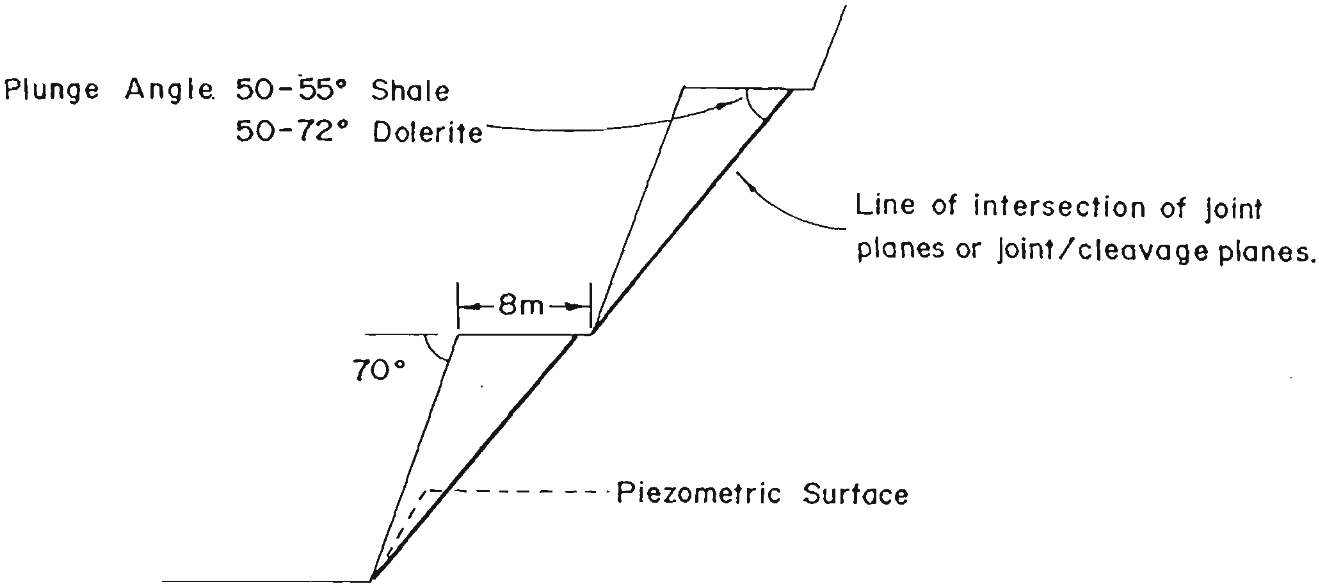
FIGURE 10,4



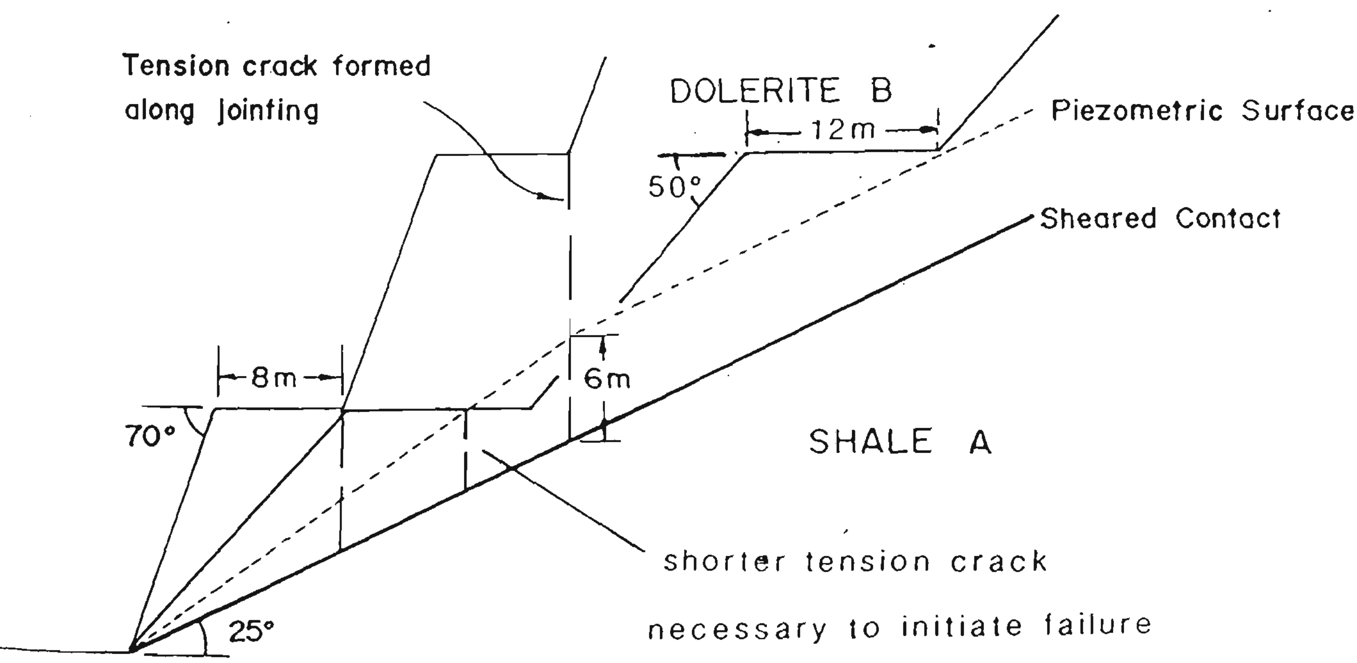
OVERLAY

NORTH WALL BENCH DESIGN; 6480E-8400E

Scale	POTENTIAL WEDGE INSTABILITY WITHIN THE "DISTURBED" ZONE OF THE JEERINAH DOLERITE	FIGURE 10.5
Drn		
Dwg. No		

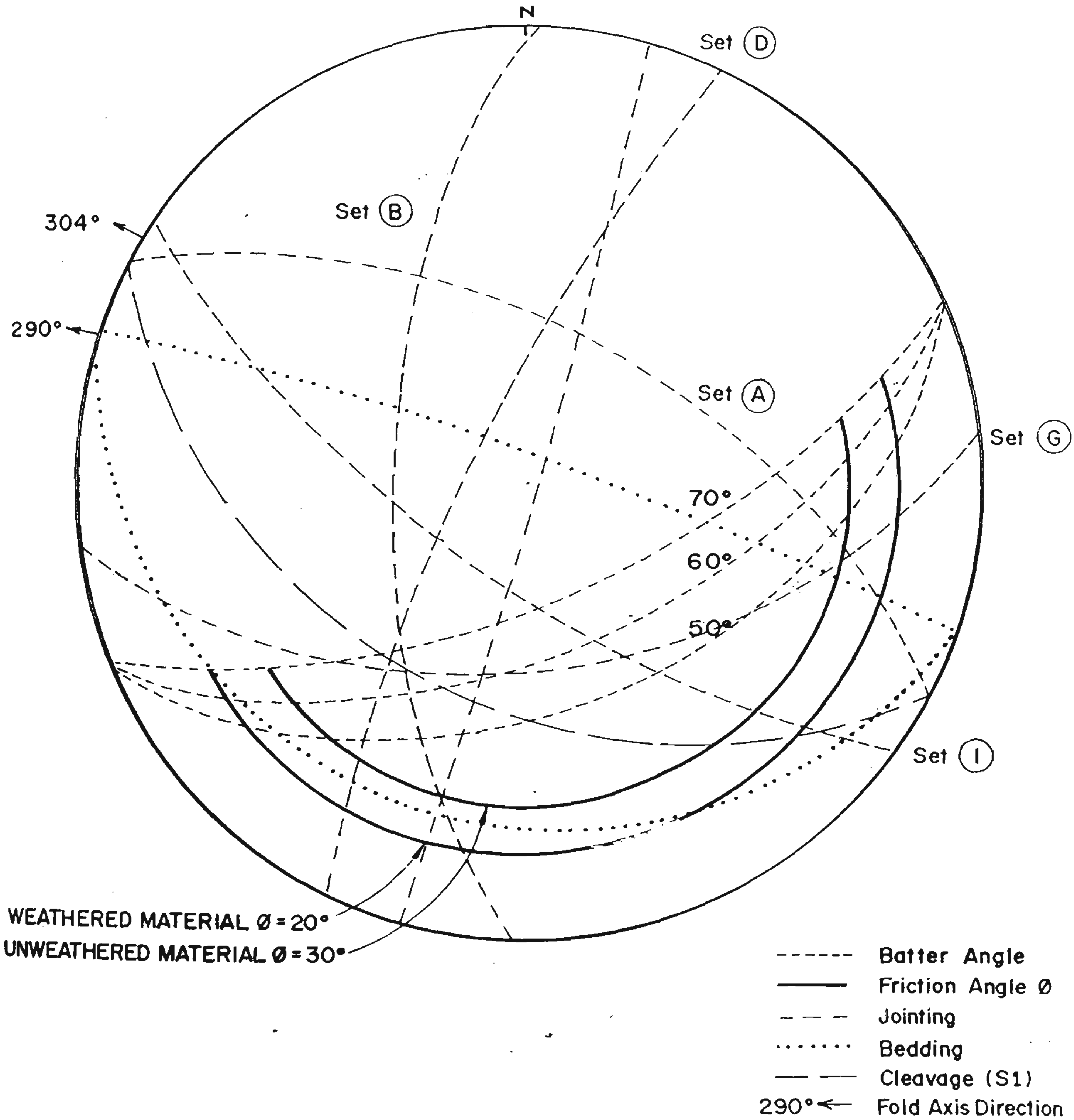


(a) WEDGE FAILURE INVOLVING JOINTING IN THE JEERINAH DOLERITE AND CLEAVAGE + JOINTING IN THE JEERINAH SHALE.



(b) PLANE FAILURE ON THE SHALE A/DOLERITE B CONTACT.

Scale 1:500	DETAILS OF WEDGE AND PLANE FAILURE ANALYSIS	FIGURE 10.6
Drn		
Dwg. No.		

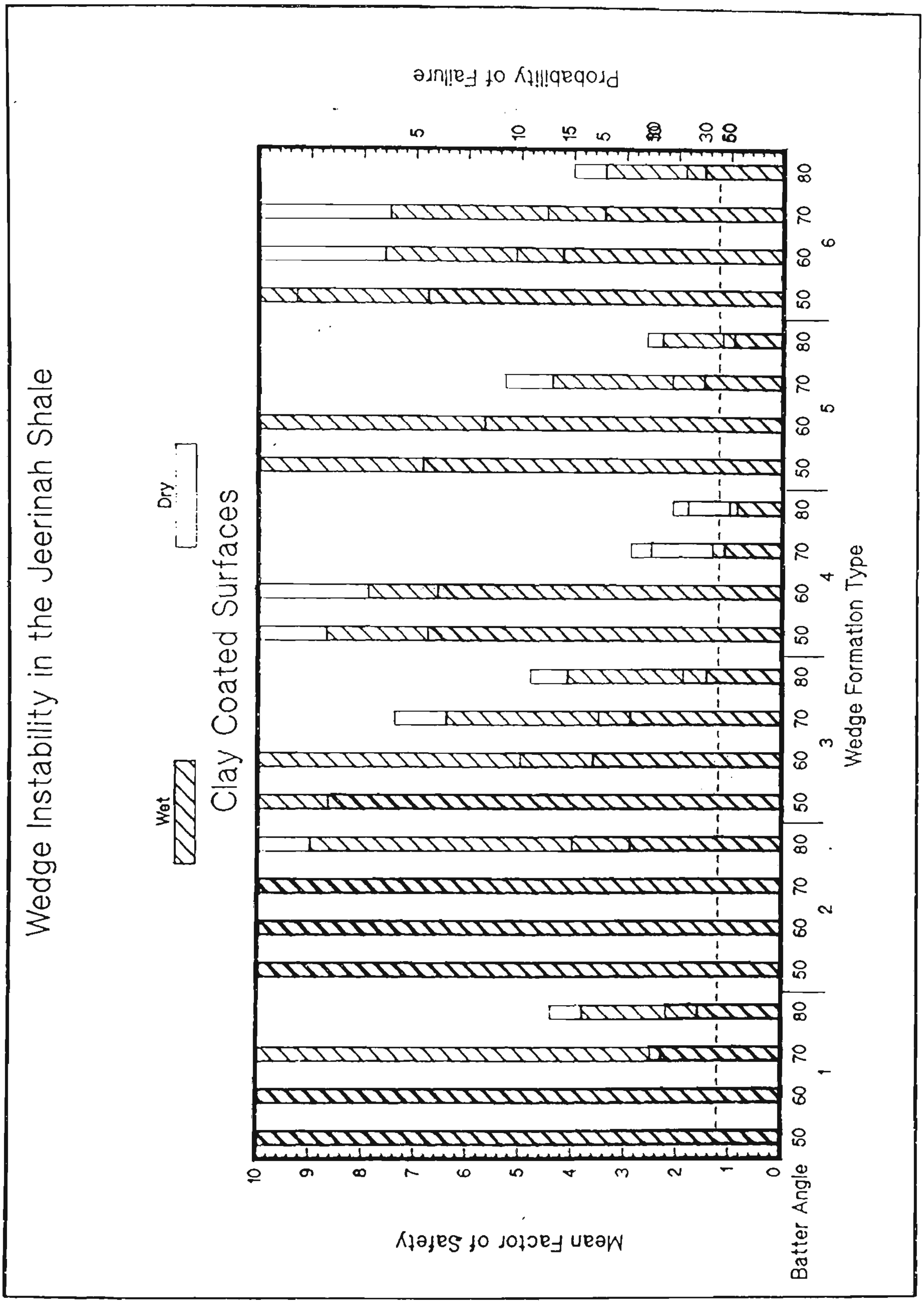


N.B. STABILITY ASSESSMENT ASSUMES COHESION = 0

OVERLAY

NORTH WALL BENCH DESIGN; 6480E–8400E

	Scale	POTENTIAL WEDGE FORMATION WITHIN THE JEERINAH SHALE A: EASTERN DOMAIN	FIGURE 10.7
	Drn		
	Dwg. No.		



NOTES

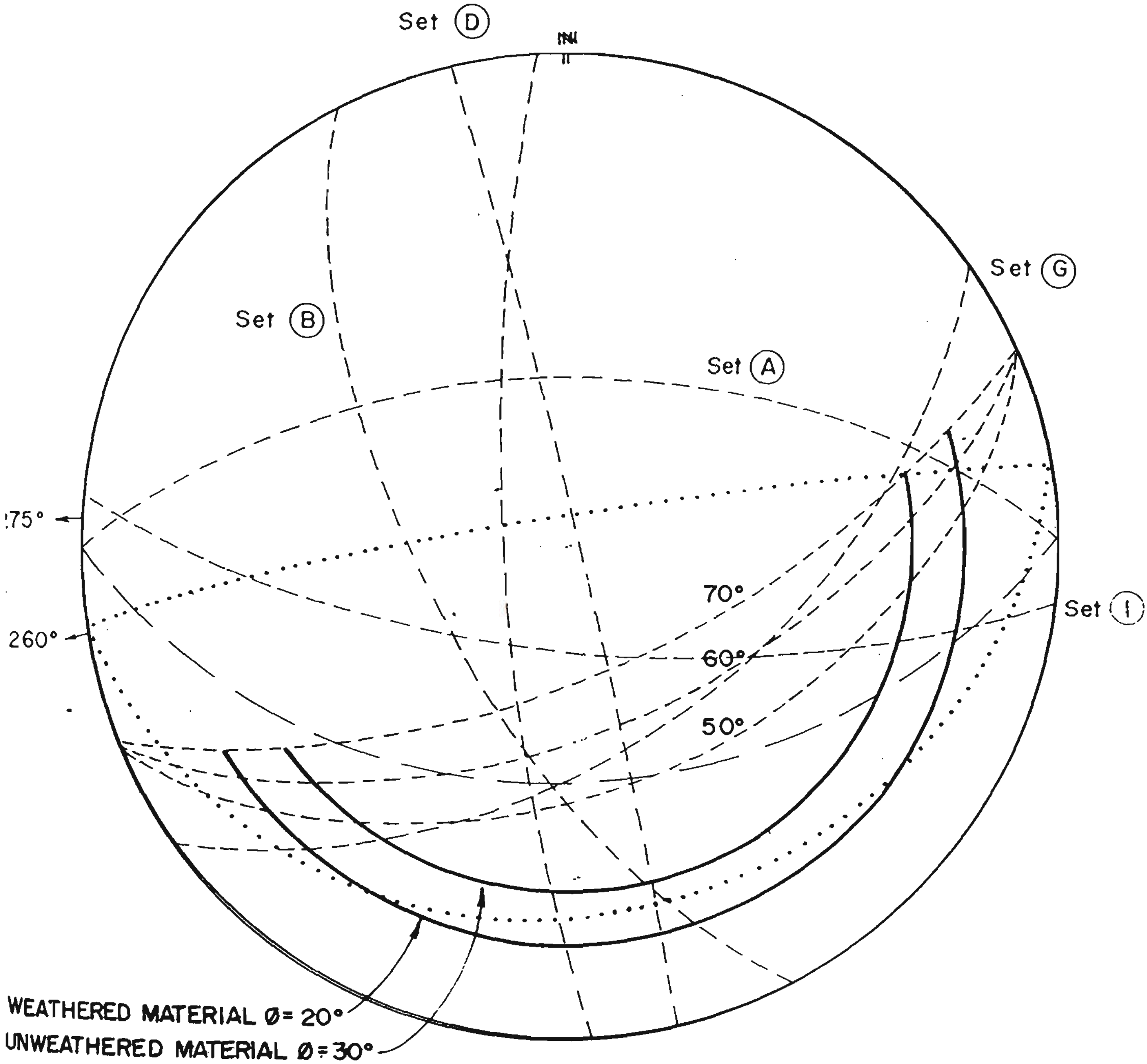
1 Joint Sets B + G
2 Joint Set B + Cleavage
3 Joint Sets D + G
4 Joint Set D + Cleavage
5 Joint Sets I + G
6 Joint Set I + Cleavage

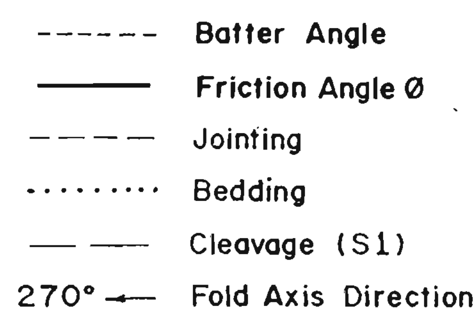
Discontinuity shear strength :

Unweathered - $c = 35 \pm 15$kPa, $\phi = 30 \pm 3°$

Clay coated - $c = 15 \pm 15$kPa, $\phi = 20 \pm 3°$

Scale Drn	POTENTIAL WEDGE INSTABILITY WITHIN THE EASTERN DOMAIN OF THE JEERINAH SHALE	FIGURE 10.8



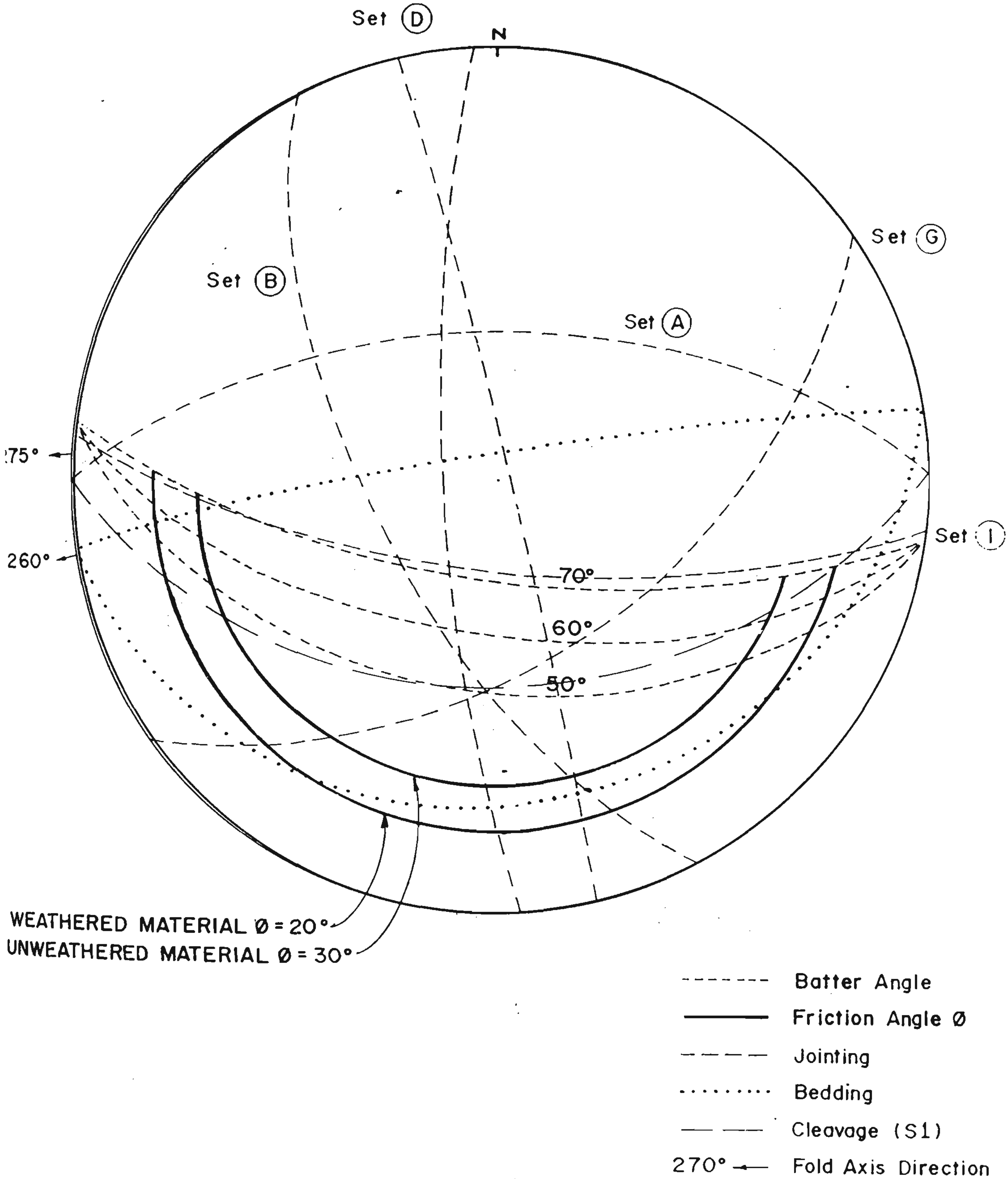


N.B. STABILITY ASSESSMENT ASSUMES COHESION = 0

OVERLAY

NORTH WALL BENCH DESIGN ; 6480 E - 8400E

Scale	POTENTIAL WEDGE FORMATION WITHIN THE JEERINAH SHALE A AND B : CENTRAL DOMAIN	FIGURE 10.9
Drn		
Dwg. No		

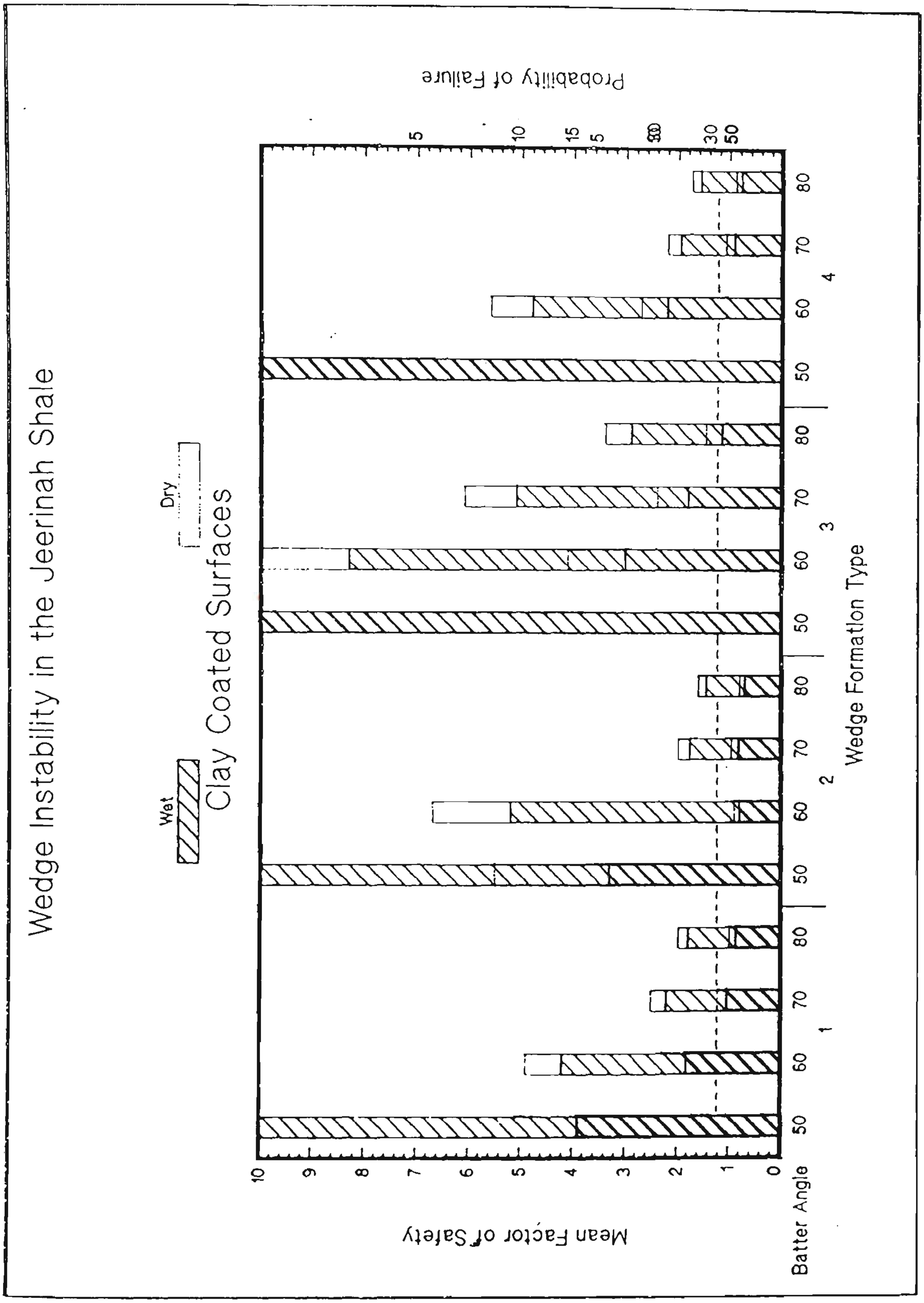


N.B. STABILITY ASSESSMENT ASSUMES COHESION = 0

OVERLAY

NORTH WALL BENCH DESIGN ; 5920E - 6480E

Scale	POTENTIAL WEDGE FORMATION	FIGURE
Drn	WITHIN THE JEERINAH SHALE A	10.10
Dwg No	WESTERN DOMAIN	



NOTES

1 Joint Sets B + G
2 Joint Set B + Cleavage
3 Joint Sets D + G
4 Joint Set D + Cleavage
5 Joint Sets I + G
6 Joint Set I + Cleavage

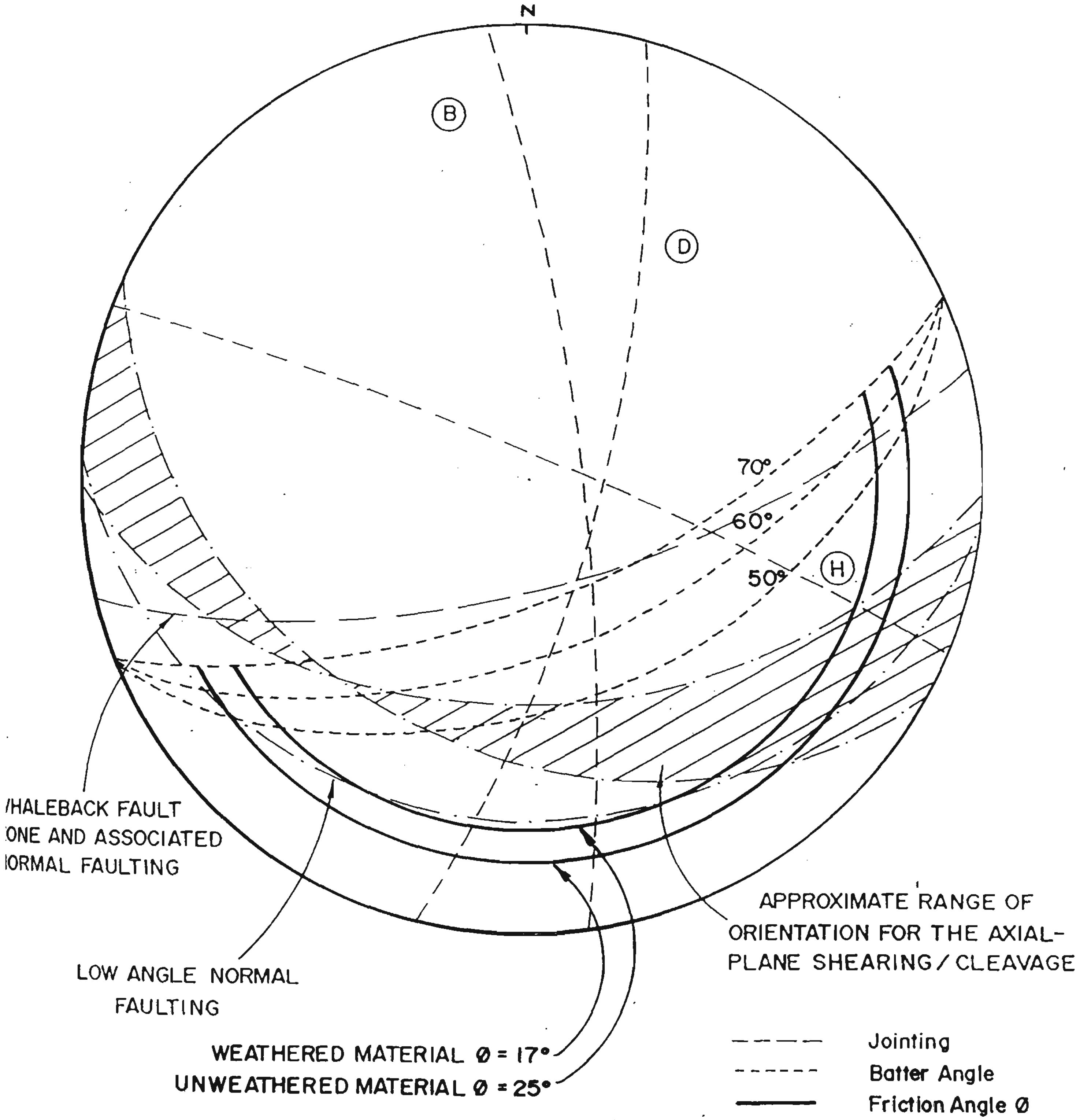
Discontinuity shear strength :

Unweathered - $c = 35 \pm 15$kPa, $\phi = 30 \pm 3^{\circ}$

Clay coated - $c = 15 \pm 15$kPa, $\phi = 20 \pm 3^{\circ}$

OVERLAY

Scale	POTENTIAL WEDGE INSTABILITY WITHIN THE CENTRAL AND WESTERN DOMAINS OF THE JEERINAH SHALE	FIGURE 10.11
Drn		
Dwg. No.		

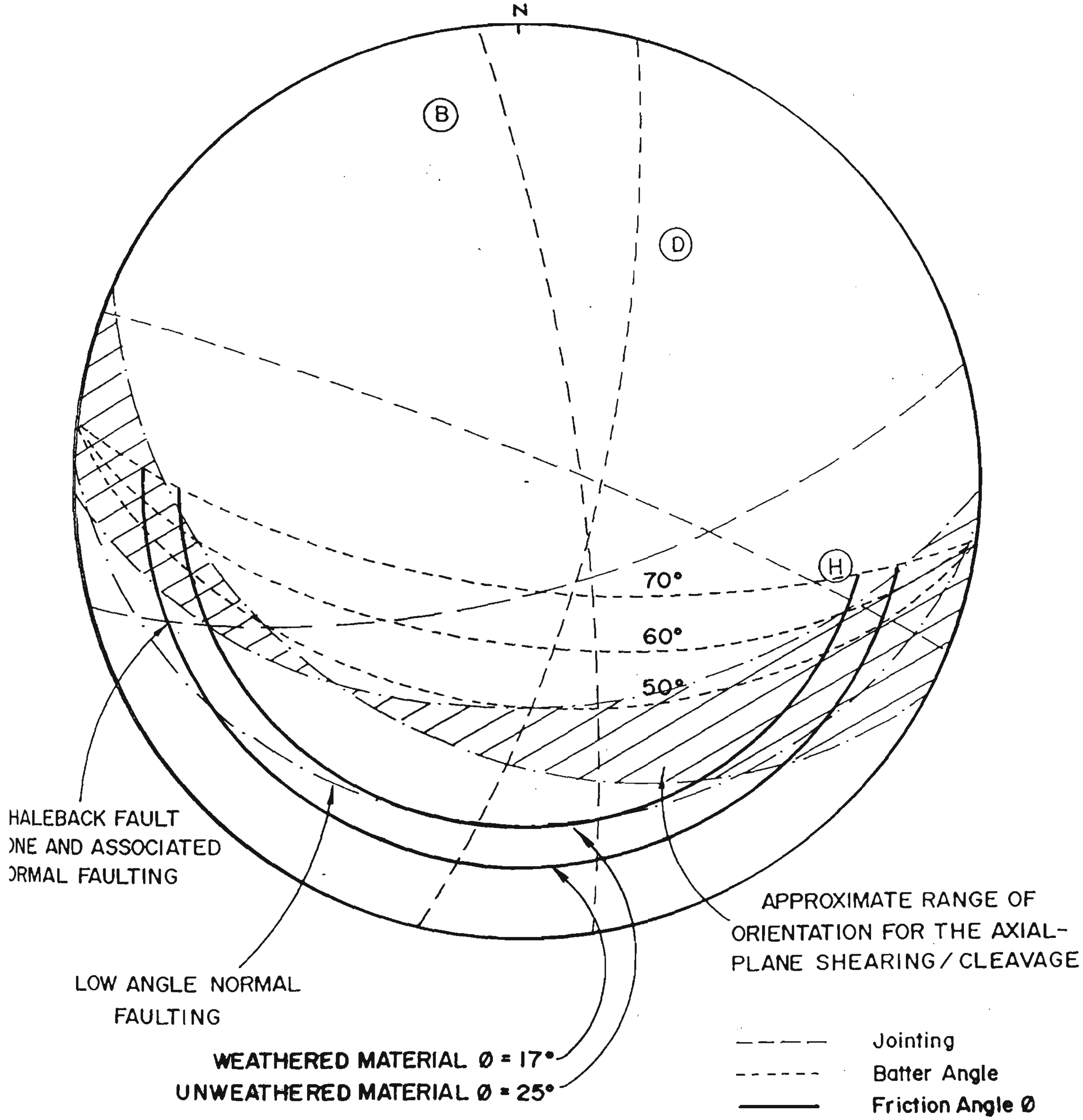


N.B. STABILITY ASSESSMENT ASSUMES COHESION = 0

NORTH WALL BENCH DESIGN; 6480E - 8400E

N.B. BEDDING NOT SHOWN DUE TO VARIABLE DIP.

Scale	POTENTIAL WEDGE FORMATION JOFFRE MEMBER DISTURBED ZONE	FIGURE 10.12
Drn		
Dwg No		



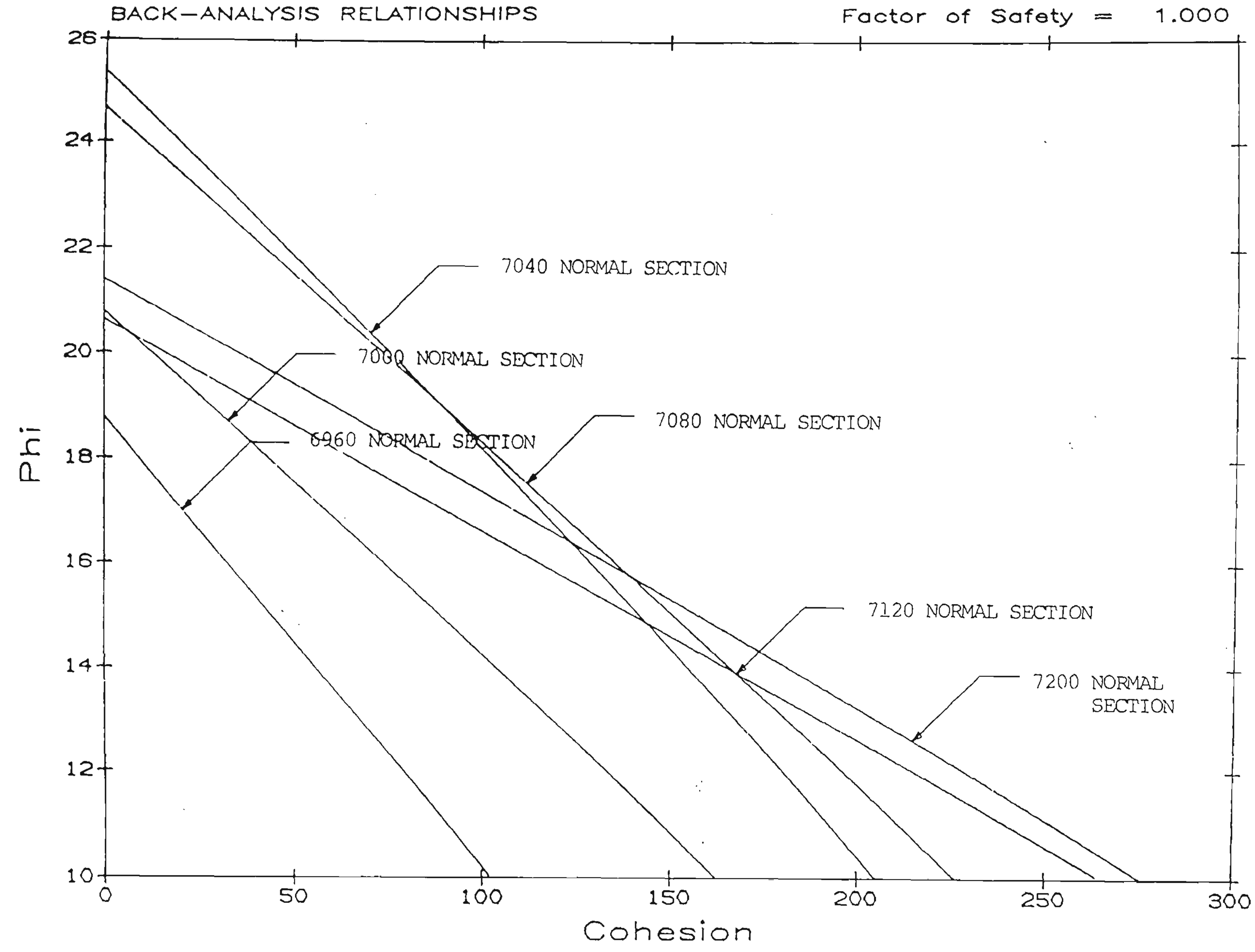
N.B. STABILITY ASSESSMENT ASSUMES COHESION = 0

OVERLAY

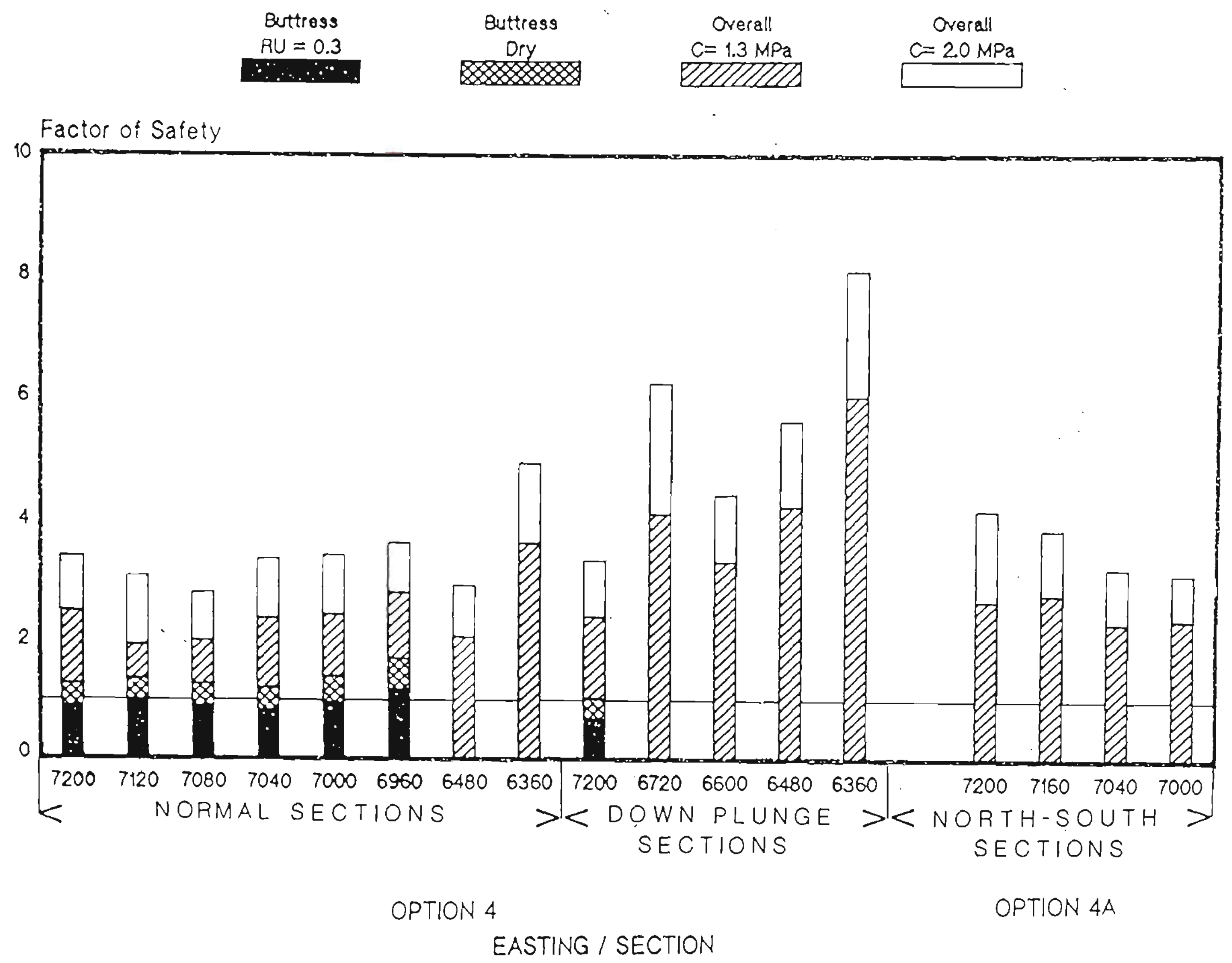
NORTH WALL BENCH DESIGN; 5920E - 6480E

N.B. BEDDING NOT SHOWN DUE TO VARIABLE DIP.

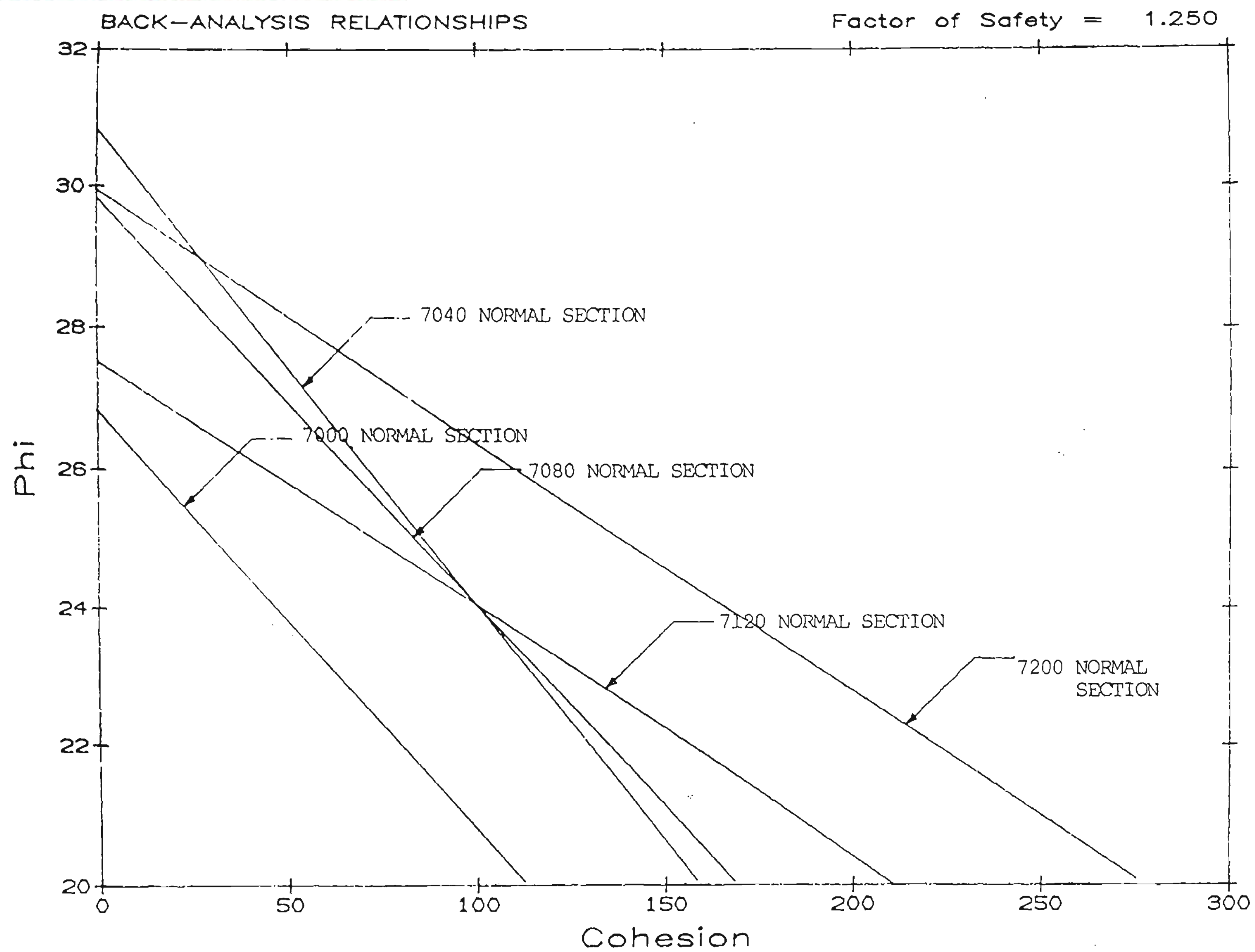
Scale	POTENTIAL WEDGE FORMATION JOFFRE MEMBER DISTURBED ZONE	FIGURE 10.13
Drn		
Dwg No		



Scale	NEWMAN NORTH WALL BUTTRESS ZONE	FIGURE 10.15
Drn	OPTION 4	
Dwg No		



Scale	STABILITY RESULTS FOR NORTH WALL	FIGURE 10.14	
Drn			
Dwg No			

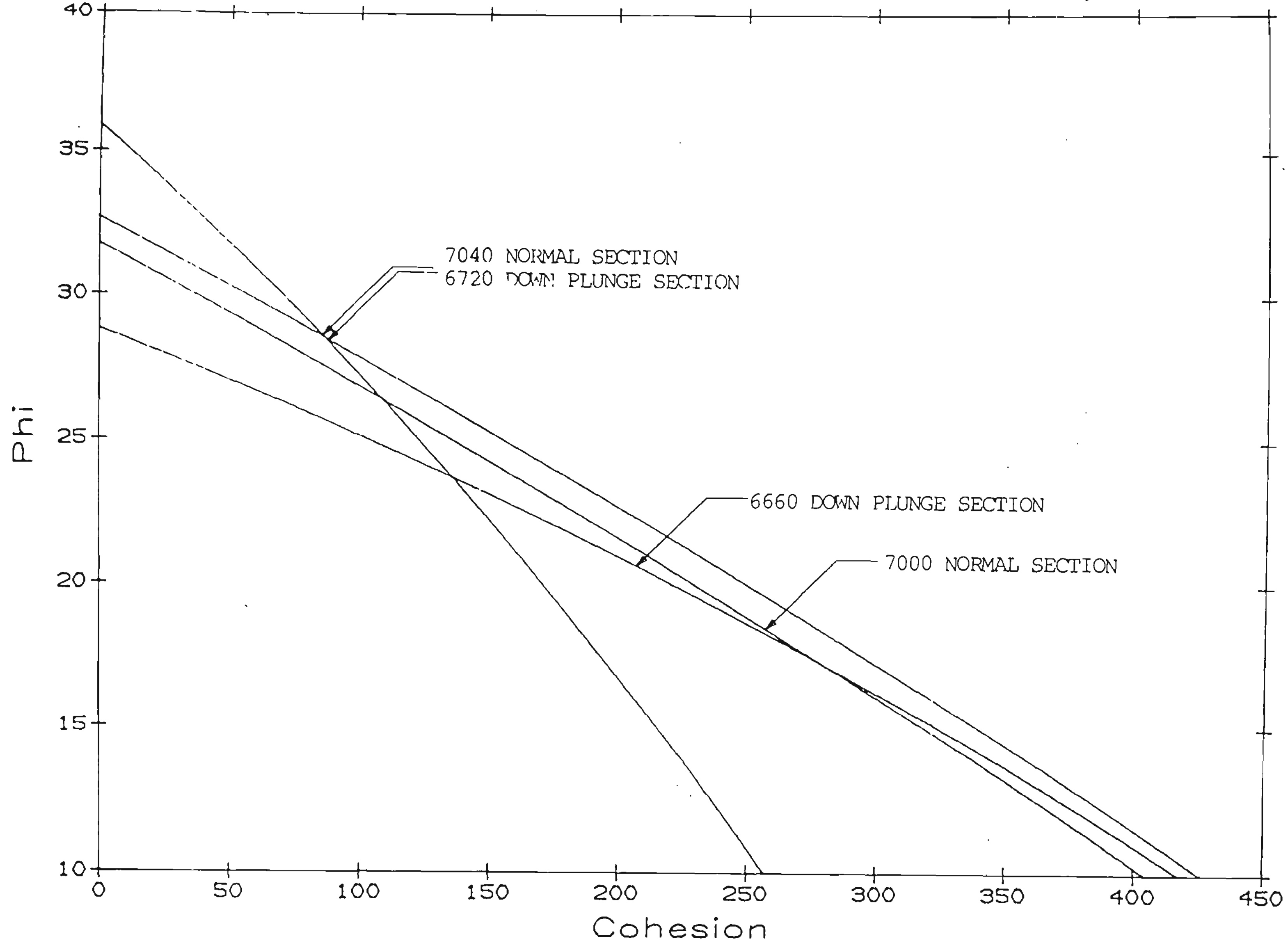


Scale	NEWMAN NORTH WALL BUTTRESS ZONE
Drn	OPTION 4
Dwg No	

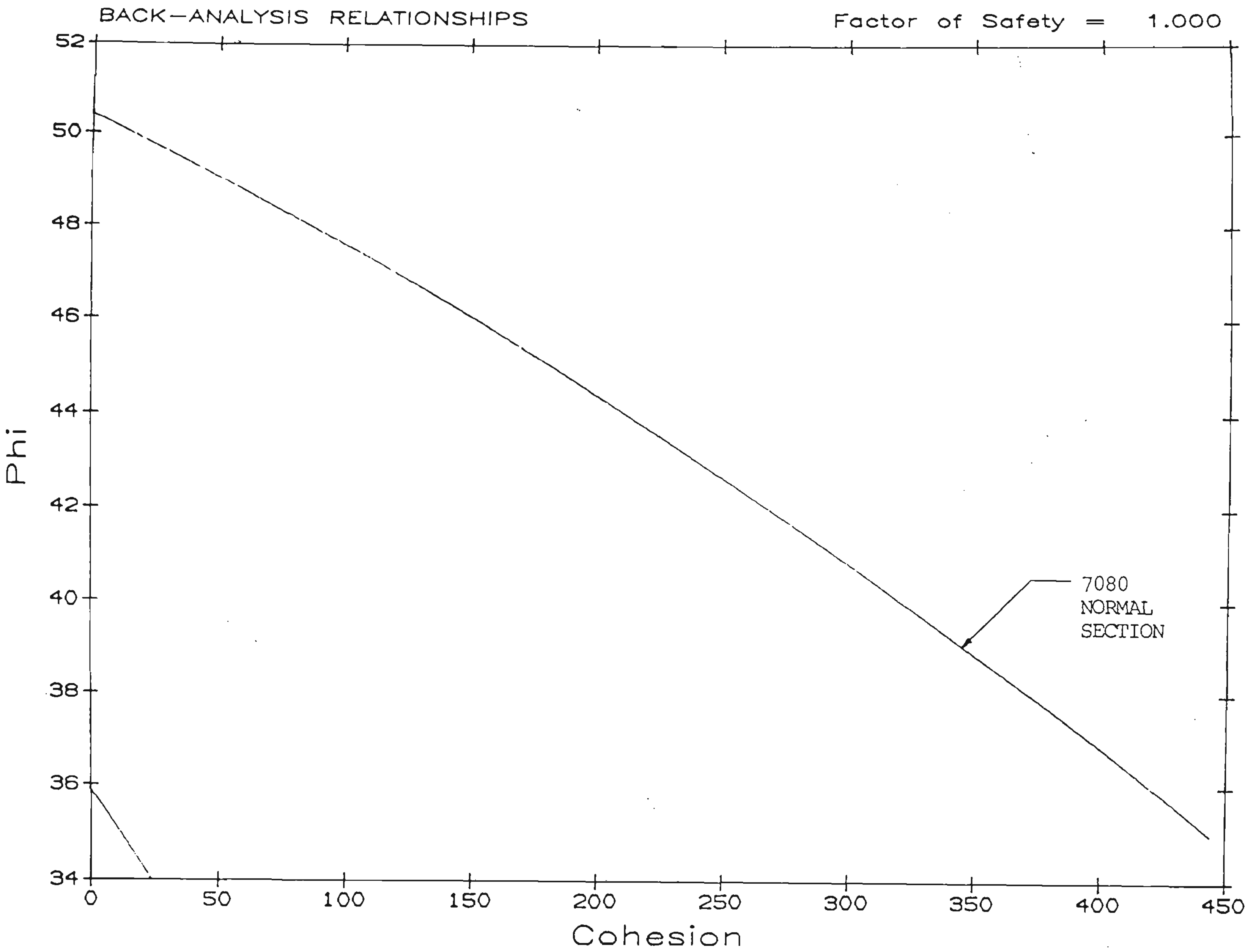
FIGURE 10.16

BACK–ANALYSIS RELATIONSHIPS

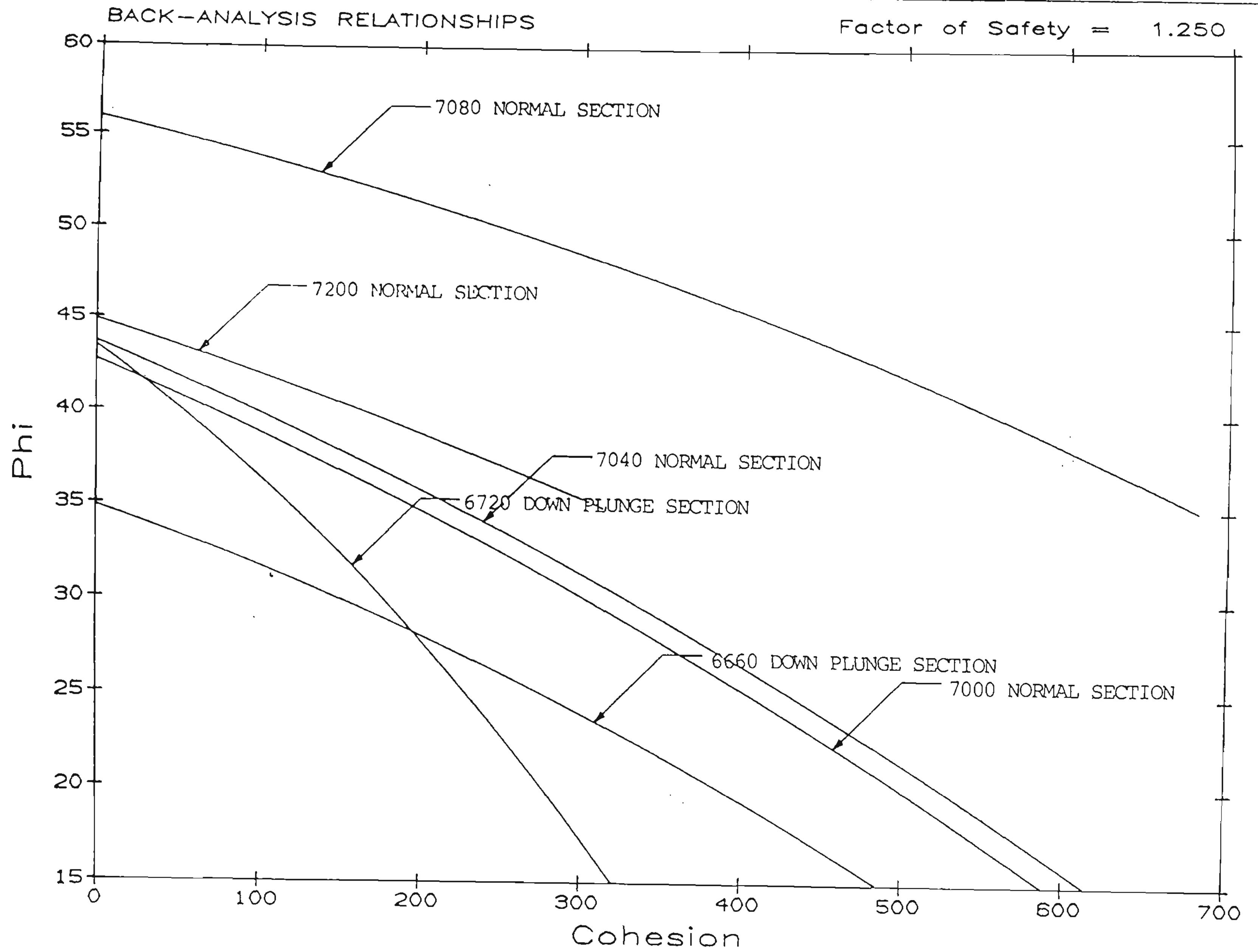
Factor of Safety = 1.000



Scale	NEWMAN NORTH WALL SECTIONS	FIGURE
Drn	OPTION 4 OVERALL	10.17
Dwg No		



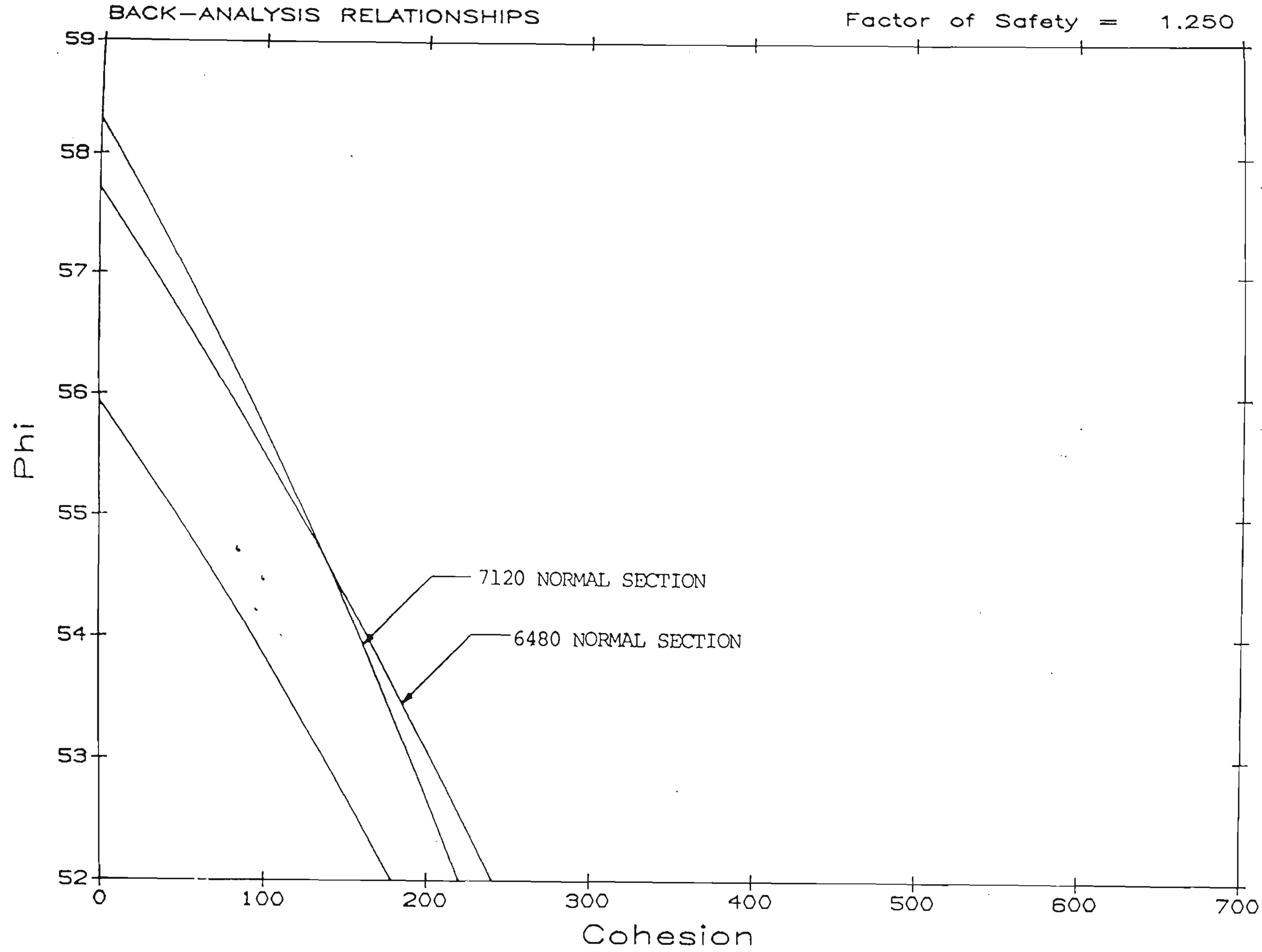
	Scale	NEWMAN NORTH WALL SECTIONS	FIGURE
	Drn		10.18
	Dwg No	OPTION 4 OVERALL	



Scale
Drn
Dwg No

NEWMAN NORTH WALL SECTIONS
OPTION 4 OVERALL

FIGURE
10.19



Scale		NEWMAN NORTH WALL SECTIONS	FIGURE
Drn		OPTION 4 OVERALL	10.20
Dwg No			

CHAPTER 11

PIT DESIGN AND ECONOMIC ASSESSMENT

11.1 INTRODUCTION

Originally, the preferred economic objective was a wall design which would enable extraction of all the ore in the Dales Gorge member and would be as close as practicable to the previous pit wall design. It was to be a wall that was predominantly north of the WFZ although the 'Northern Option' proposed in 1985 was to be avoided if at all possible (Internal Memorandum from Mt Newman Mining Company, 1986). The 'Southern Option' which was also proposed at that time was not to be considered since it sterilised large volumes of ore and also reduced the operational width in the base of the East Pit. Hence the major thrust of this work has been to investigate structures, geology and wall designs north of the WFZ.

11.2 FACTORS INFLUENCING WALL DESIGN ON A BENCH SCALE

The wall design on a bench scale has been designed to minimise the extent of local failures and at the same time take advantage of rock strength and structure. Structural control involving various discontinuity types (bedding, jointing, cleavage, faulting, shearing, etc) dominates the Joffre Member, Jeerinah Dolerite and Jeerinah Shale units implying potential for wedge failures or failures along planar discontinuities. However extensive shearing, alteration and weathering is associated with material within the WFZ resulting in considerable reduction in the intact rock strength. In this very weak rock material there is minimal structural influence from discontinuities and a potential failure along curved surfaces is more likely than along planar surfaces.

For potentially unstable batter faces there are basically three different ways in which rock slope control can be achieved. These are:

(a) Design the slope so that no failures occur by using conservative batter angles and wide benches to achieve a low overall slope angle. This method is consequently uneconomic and involves a high stripping ratio.

(b) Excavate the pit such that the slope geometry (ie. batter angle and berm width) is designed to retain (on benches or berms) any small scale failures that occur. If access is available the debris from localised failures can be periodically removed. This method has been adopted to produce the batter/berm design for the North Wall.

(c) Excavate the slope under controlled conditions and utilise artificial support methods (for example use of cable bolts) to stabilise a steepened wall design. The cost of artificial support is usually minimal when compared with the savings achieved by a reduction in waste stripping. This approach has also been adopted for areas of the North Wall where adverse structural geology conditions are encountered.

The approach adopted for batter scale design in this thesis has been to adopt method (b) above and is similar to the concept proposed by Piteau (Ref 10.8). Piteau defined the optimum slope design as being 'that which enables the slope to be excavated under controlled conditions so that failures are caught on berms and adequate access is provided for maintenance and removal of material'. In this approach the minimum required berm width is related to the maximum cross-sectional area of a potential failure. The approach was developed by considering the plunge of the line of intersection of two discontinuities forming a wedge (refer to Figure 10.2 in Chapter 10). The minimum berm width required is dependent upon the following:

- plunge of the line of intersection
- bench height
- batter face angle
- bulking factor of failed material
- angle of repose of failed material

In the following analyses the angle of repose has been assumed to be 38° and a bulking factor is not included to avoid a conservative bench design. Design curves for a variable plunge angle (40 to 80°), bench height (15 or 30m) and batter face angle (50 to 80°) are shown on Figure 11.1. The Jeerinah Dolerite, Jeerinah Shale, WFZ material and Joffre Member have been assessed separately in order to establish basic guidelines for the base wall design.

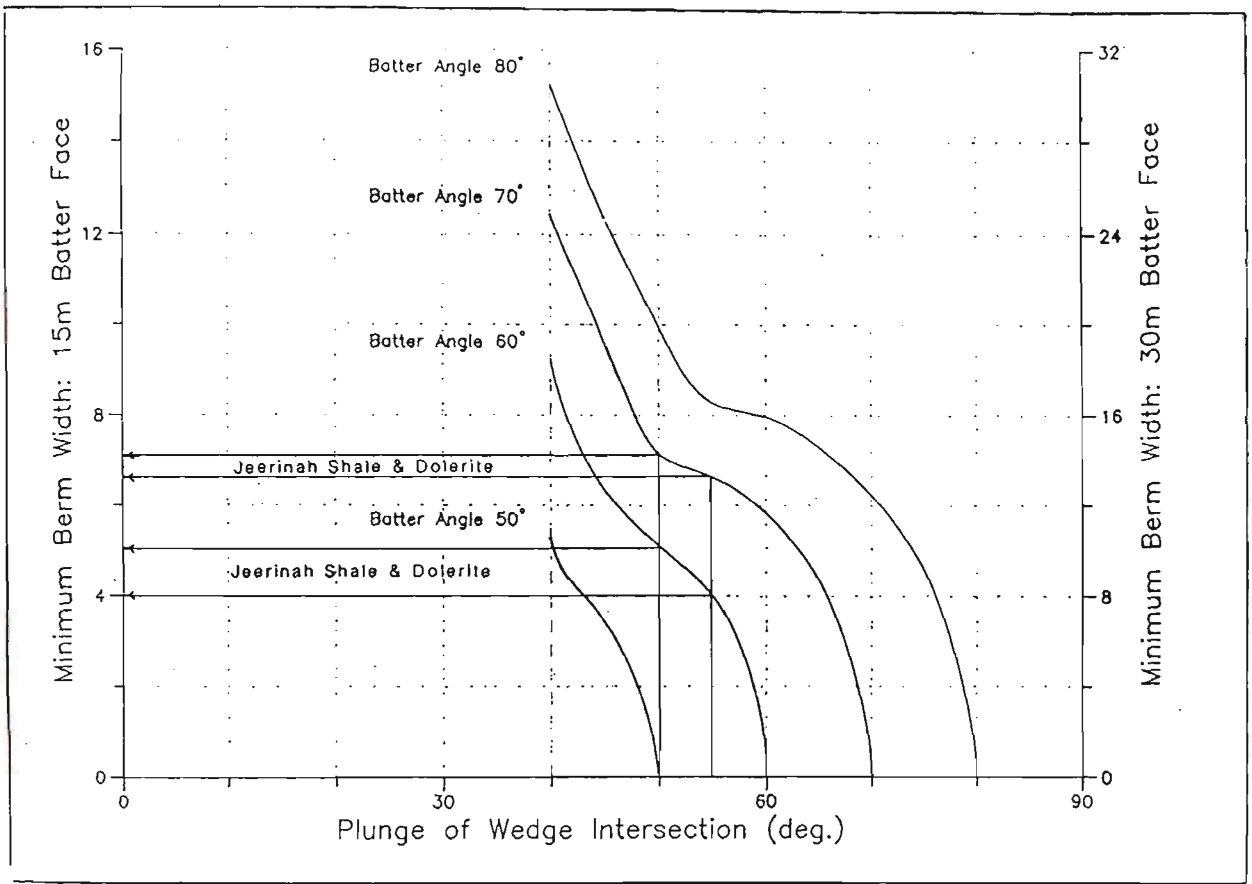


Figure 11.1 Minimum Berm Width Required for Wedge Instability on a 15m or 30m High Bench

11.3 JEERINAH SHALE

The batter scale slope design adopted for the Jeerinah Shale is 70° batters and 8m berms which produces an overall slope of 48°. This is based on the fact that the mean plunge of all kinematically possible wedge combinations varies between 49° and 55°. These plunge angles are summarised in Table 10.5 in Chapter 10, and shown graphically in Figure 11.1.

Figure 11.1 shows the required berm width for a 60° batter angle is 5m and for a 70° batter angle, 7m. This minimum berm width is usually increased to allow for back-break due to blast damage, shovel excavation, general ravelling, etc. Ideally the back-break should be determined by field measurement but in the short term a value of 1m has been assumed. The minimum required berm width is therefore 6m for a 60° batter face and 8m for 70° batter face.

The previous stability assessment presented in Chapter 10 indicated the potential for wedge instability in batter faces excavated steeper than 60° in the Eastern Domain and 50° in the Central and Western Domains for weathered material adjacent to the WFZ. All batter faces excavated in unweathered material have been shown to be stable for the groundwater situations assessed.

A slope design incorporating a 70° batter/8m berm geometry has therefore generally been adopted for the Jeerinah Shale and located beyond the 'disturbed' zone adjacent to the WFZ. This results in an overall slope angle of 48° (refer to Figure 11.2).

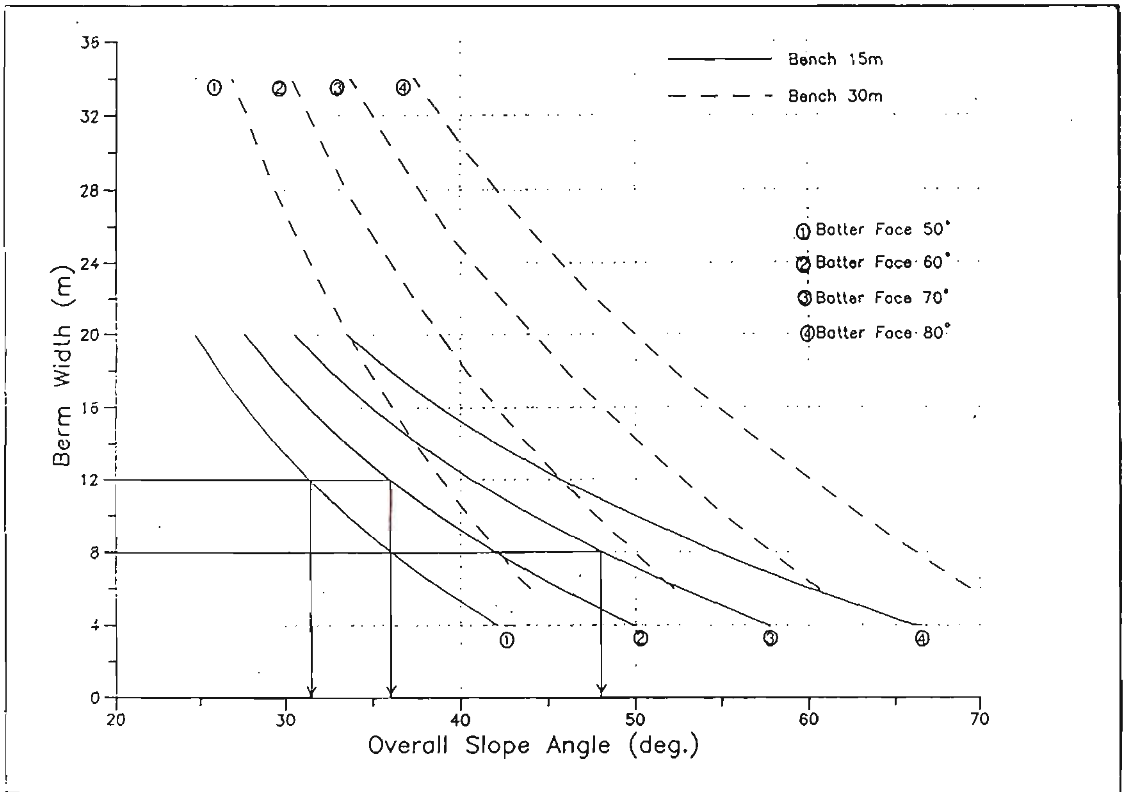


Figure 11.2 Relationship Between Overall Slope Angle, Berm Width and Batter Face Angle

11.4 JEERINAH DOLERITE

The analysis of wedge stability presented in Chapter 10 indicated that all bench designs were stable with respect to wedge failure and hence the steeper wall design utilising a 70° batter/8m berm geometry has been adopted for the bench scale wall design. However, there is still the potential for the removal of wedge material during final limit blasting and excavation, and hence the bench width should be designed accordingly.

The mean plunge angles for the various wedge combinations identified are shown in Table 10.3 in Chapter 10, and generally exceed 50°. The required minimum berm width is therefore the same as that for the Jeerinah Shale and is adequately covered by the 70° batter/8m berm design (ie. an overall slope of 48°) as shown in Figure 11.1.

The potential for plane failure of Dolerite B overlying a sheared contact with Shale A between 7160E and 7400E has been described previously in Chapter 10. The approach developed by Piteau (Ref 10.8) has been extended to assess the minimum berm width required to contain a single bench plane failure limited by a tension crack as shown in Figure 10.6 in Chapter 10. The variation of the required berm width with respect to the contact dip angle has been assessed for bench designs utilising a 70° batter/8m berm and 60° batter/12m berm geometries as shown in Figure 10.6. These berm widths range from 19m for a 70° batter face up to 26m for a 60° batter face and are obviously impractical.

However the potential for bench scale failures of dolerite overlying both weathered and unweathered shale is specifically governed by the dip of the dolerite/shale contact and since this is highly variable, it is not practical for the batter/berm wall design to try and accommodate this. Where localised instability is a problem, artificial support measures are recommended.

11.5 WHALEBACK FAULT ZONE MATERIAL

The wall design in the WFZ consists of 50° batters and 12m berms giving an overall slope of 31°. This design is flatter than that in either the Jeerinah Shale or Jeerinah Dolerite and is considered to be a reasonable balance between stability and an economic design.

Stability calculations for the WFZ still indicate that failures will occur, but these would still occur at even flatter slope angles than the design proposed here. In addition, slaking of the wall in the WFZ will occur and continue to be a nuisance rather than a serious problem. Slaking has already occurred on the pit wall at the East end of the pit in the WFZ and this will continue to occur along the rest of the wall. However the overall slope of 31° is less than the natural rill angle of this material (36°) and can therefore accommodate slaking if it occurs.

11.6 JOFFRE MEMBER

The potential for bench instability in the Joffre Member has been shown to increase where drag folding adjacent to the WFZ results in an increased southerly–dip of persistent discontinuities. The critical dip for instability to develop (factor of safety = 1.2) is found to be 15° for weathered, clay–coated discontinuity surfaces and 25 to 28° (depending on groundwater conditions) for unweathered material. This indicates that artificial support measures may need to be employed to maintain bench stability in the toe buttress region of some of the design options.

A slope design utilising a 60° batter/12m berm has been adopted for final pit limits within the Joffre Member resulting in an overall slope angle of 36°.

11.7 SUMMARY OF BENCH SCALE WALL DESIGN

An assessment of the bench stability for both wedge and plane failure modes has allowed a wall design to be produced based on the berm width required to retain any failed material. The batter/berm configurations for each of major materials varies as follows:

Rock Type	Batter/Berm Configurations	Overall Slope
Jeerinah Dolerite	70° batter/8m berm/15m bench	48°
Jeerinah Shale	70° batter/8m berm/15m bench	48°
WFZ material	50° batter/12m berm/15m bench	31°
Joffre Member	60° batter/12m berm/15m bench	36°

These bench scale wall design configurations have deliberately been kept to a minimum in order to reduce the complexity of the overall wall design, although adverse geological structure, particularly at the western end of the North Wall has resulted in the following variations from this base wall design being considered:

- a reduction in overall slope angle north of the WFZ between 6280E and 6600E in order to excavate Dolerite C overlying a steeply-dipping contact with Shale B (50° batter/12m berm utilised).

- a similar reduction in overall slope angle in order to excavate Dolerite B overlying a steeply-dipping contact with Shale A between 7320E and 7440E (50° batter/12m berm utilised).

- a steepened wall design in the Joffre Member between 5920E and 6280E to reduce the excessive waste stripping to the north of the WFZ (70° batter/8m berm utilised).

- a variable wall design at the toe of the North Wall where the proposed wall design follows the East Footwall/South Whaleback Fault in order to minimise stripping and maximise ore recovery. This incorporates an unbenched slope up to five benches high.

11.8 OVERALL PIT SLOPE WALL DESIGNS

A total of five different overall pit slope wall designs have been considered in this thesis. These different wall designs are based on the above batter/berm configurations and are as follows:

- **OPTION 1:** This is basically the steepest wall design which we believe to be practicable in order to extract most of the ore in the Dales Gorge member.

- **OPTION 2:** Option 2 is similar to Option 1 except that it incorporates a wide berm to act as a buttress to the Jeerinah Formation. It also involves additional stripping over Option 1.

- **OPTION 3:** Option 3 is the flattest wall design proposed and cuts out the majority of the unstable structures. However, it is not as conservative as the previously proposed 'Northern Option'.

- **OPTION 4:** Option 4 is similar to Option 2 except that the upper part of the wall design is generally flatter. It also incorporates a toe buttress in order to allow for the perceived potential instability of the Jeerinah Formation.

- **OPTION 4A:** Option 4A follows the same wall design as Option 4 in the upper part of the pit wall but removes the toe buttress and hence maximises ore recovery.

NOTE:

(1) An extreme southern option for the North wall also deserves some consideration as briefly discussed in Chapter 12 (see also Fig. 12.1). This southern option would have the aim of achieving a high net present value by high-grading the orebody. However it is outside the scope of this thesis.

(2) These wall design options given above, are marked on the cross sections shown in Figures A2 to A40 in Appendix A. They are also shown in plan and isometric views in Appendix C in Figures C1 through to Figures C20 to assist the reader.

The wall designs in the above five options have been produced by drawing wall options on north-south cross sections to fit the structural model and then tying these cross sections together in an east-west direction. The wall design was then smoothed out to avoid unnecessary curvature in the wall and then these modified wall designs were finally plotted back on to the north-south cross sections.

11.9 DETAILED SECTIONS

All available gamma log information as well as assay information for boreholes has been used to produce the north-south cross sections shown in Appendix A although they are not shown on these figures for clarity. They have been reproduced on the large scale working drawings presented to Mt Newman Mining Pty Limited but are not included as part of this thesis.

The cross sections are described briefly below with particular emphasis on Options 4 and 4A because they are the options which have been recommended as a result of this detailed research work. The rationale behind the other pit options is generally self evident. It is therefore useful to read the comments below in association with the figures in Appendix A.

Cross Section Easting	Comments
° 5920E	This is the western most section in the area of study and the structural geology is not known with confidence. The Option 4 and 4A designs are essentially the same with a relatively steep angle north of the WFZ but becoming much flatter through the WFZ itself. Option 1 has potentially major problems with a fault dipping into the pit immediately behind the wall design.
° 6040E	Options 4 and 4A follow a similar position to 5920E. Option 3 is much flatter whereas Option 2 cuts out a major fault. Option 1 still has a potentially major instability in the upper part of the wall. Structural information is very limited at depth.
° 6280E	Option 1 still has potential instability in the upper part of the wall with a major fault. Option 3 is very flat to accommodate wall stripping further east. The Shale B begins to dip into the pit and hence forms a possible boundary for sliding of Shale B on Dolerite B. Option 4A is slightly steeper than Option 4 in this area.

°	6480E	Shale B is now dipping very steeply behind WFZ and Dolerite C appears at top of wall. All wall designs cut out Dolerite C. Option 3 cuts out most of the in–dipping Shale B. Option 4A is steepened at the toe to scavange maximum ore and has two double benches. Structure here at the toe should be checked when pit is deeper.
°	6600E	Exact location of contact between Shale B and Dolerite B should be determined particularly for Option 4A. Option 4A may have problems in contact and require support. This option does recover all ore. All other options very similar and are south of WFZ at toe.
°	6720E	All wall options very similar except Option 4A which recovers all ore. Shale B/Dolerite B contact should be checked in more detail when pit is closer to contact. No other problems envisaged. Buttress area starting to get larger.
°	6880E	All sections virtually identical. Main wall is in Dolerite B and cuts out Shale B. Flat bench across WFZ. Option 4A follows contact at toe.
°	7000E	Again, all options the same except Option 4A which follows the EFFZ contact at the toe. This footwall may require local support. Further drilling is required to define this contact better when closer to this level.
°	7040E	Similar comments apply as previous section.

- 7120E — Wall designs 1 to 3 similar. Dolerite B/Shale A contact had unfavourable dip hence wide buttress as safe guard against potential instability. Wall design above wide bench is in dolerite and should be no problem. Option 4A removes buttress below wide bench and has flat angle through WFZ and then follows EFFZ contact over almost five benches. Catch fences required here. Buttress stability is questionable and needs further verification when additional detailed structural information is available.

- 7160E — Dolerite B/Shale A contact higher up the wall than in 7120E and close behind wall in Options 2 and 4A. Also disturbed dolerite close to WFZ and contact may require localised support. Shale A is also dipping south and may cause problems, hence all wall designs except Option 4A include toe buttress. Unbenched footwall of Option 4A is four benches high. Would almost certainly require some support.

- 7200E — Option 4 has buttress to prevent Dolerite B sliding on Shale A. All other options expose this contact. Shale A also has shear zone which is 'daylighted' in Option 4A. Option 4A solves problem by crossing contact at a flat *ip and having a flat overall wall design through Shale A and WFZ. Unbenched footwall at toe of Option 4A is three benches high and fairly flat. Buttress stability of Options 1 to 4 is poor. Toe of buttress is south of synclinal keel overall structure unfavourable and difficult to accommodate.

- 7320E — Dolerite B/Shale A contact still dipping south. Buttress extends high up the wall to go above these contacts. Option 4A 'daylights' contact and support at toe area here may be necessary if shear strengths on contact are low. Option 4A has at least four benches of unbenched footwall and will require support.

- 7400E — Dolerite B/Shale A contact is removed by all options except Option 1. Option 4A is fairly flat through Shale A and WFZ and then has a three bench high unbenched footwall at the toe. Option 4A wall is sub–parallel with Shale A/Dolerite A contact.

- 7440E–7600E — All wall designs the same except for minor sections near crest. 7440E–7520E is an ideal location to trial induced failure test on Shale A/Dolerite A contact. The toe of the wall has unbenched footwall normally three benches high. This will require localised support particularly near the crest of the unbenched footwall. Section 7600E may require heavier support.

- 7640E–7680E — All options essentially the same. All designs steep through Dolerite A and flat through WFZ.

- 7720E–7880E — This is the area for the crusher and haulroad. Wall is flat through WFZ with minor unbenched footwall following contact at toe. This is two benches high at 7840E. These sections show the present and projected positions of the haul road and the crusher and the overall structure of the Joffre in this area is stable. Dips in Joffre are flat or to the north next to the WFZ unlike further west. Sub–vertical faults are present but these will not cause problems.

- 7920E–8240E The wall design is generally flat through the WFZ and has an unbenched footwall at the toe. This unbenched footwall increases in height from 7920E to a maximum height at approximately 8040E (five benches). Stability of this footwall is not considered to be a problem providing structure is not 'daylighted'. However, catch fences will almost certainly be necessary, particularly at 8000E and 8040E.

- 8240E–8400E Design basically follows existing pit and then follows EFFZ contact. It should be noted that the existing wall design through the WFZ is steeper than that proposed for sections further west.

It is impossible to list in detail all of the design features of each section and the reader is also referred to the plan and isometric views of the wall designs given in Appendix C. These isometric views show the size and extent of both the toe buttress and the unbenched footwall slope.

11.10 COMMENTARY ON WALL DESIGNS

The wall designs presented in this Chapter are concerned with wall options that attempt to extract all ore from the Dales Gorge and will not require major revisions in the future. Therefore they are based on the structure and the stability on a large scale. There is simply insufficient information in the western areas of the North Wall to propose batter and berm configurations that would not be subject to some change.

The batter and berm configurations are considered to be the correct ones in terms of the design concepts outlined above. However local changes could be made to batter and berm designs where appropriate. In particular, this could apply to steepening the wall design in the Jeerinah Dolerite to double bench faces, although such measures should only be applied where there is detailed information. Also the overall angle in the WFZ is also only 31° and if significant hard shale layers were encountered, these too could be steepened. However considering the problems of slaking and the current condition of the WFZ in the East Pit, 31° is an optimal design.

11.11 ECONOMIC ASSESSMENT

A detailed economic assessment of the wall design for the North Wall has also been undertaken. It should be emphasised here that these comparisons may not be absolute, but in relative terms they accurately represent the differences between the different pit wall options. However the financial parameters upon which this economic study is based is of a confidential nature to Mt Newman Mining Co Pty Limited and so these parameters are not presented in this thesis. Nevertheless the method used to produce the economic assessment is listed below.

The different options were compared section by section and the different volumes calculated. Each section was then summed to produce an overall volume for the entire wall design. These volumes were calculated by comparing each option against an arbitrary line drawn in the position of the southern boundary of the WFZ and hence waste volumes were compared as against this benchmark. Since all the volumes are compared against this same benchmark, the comparisons for each wall option are correct in relative terms. The volume of ore was measured as the volume of ore 'sterilised' south of the WFZ, ie. ore which is not recovered. These concepts are illustrated in Figure 11.3.

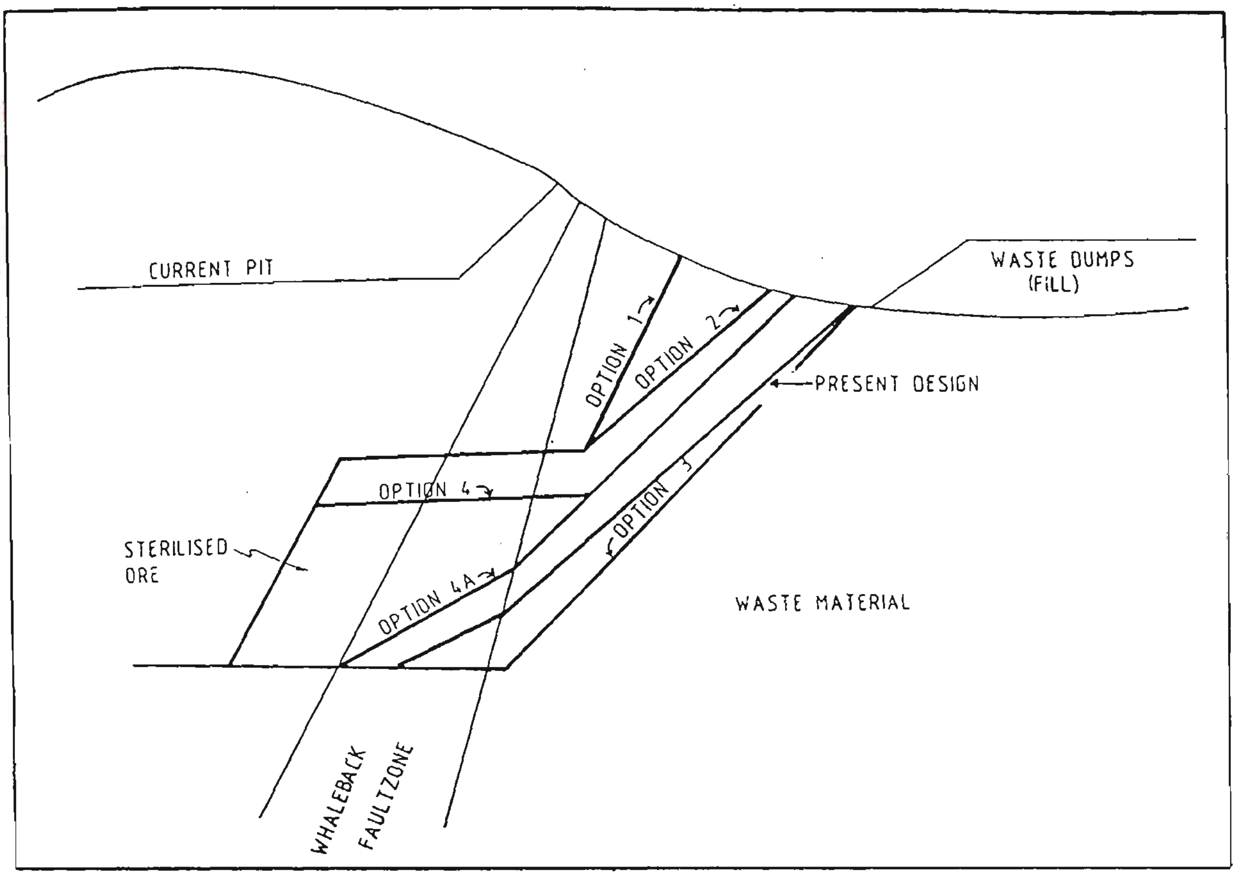


Figure 11.3 Method of Measuring Material Volumes

A summary of the tonnes of waste, high grade ore and low grade ore are given in Table 11.1. For the original economic assessment each cross section was split into a number of mining stages ranging from one to nine. These mining stages were then used to calculate the total amount of ore and waste to be obtained over the next nine years of mining and the net present value was then calculated. This was undertaken using a very detailed economic model although this is not presented in this thesis.

This detailed economic model compares the net present value of Options 1 to 4A. This has been performed by assuming a total pit production of 30 million tonnes of ore and 60 million tonnes of waste and then adding this to the incremental production of each of the Options.

Table 11.1 Comparison of Different Options

Option	STERILISED ORE (Tonnes) High Grade (3.9)	Low Grade (3.5)	Total
1	12,973,350	2,688,350	15,661,700
2	10,764,000	1,274,350	12,038,350
3	9,824,880	1,125,950	10,950,830
4	10,657,140	1,730,750	12,387,890
4A	872,820	0	872,820

Option	REMOVED WASTE In-situ (2.5)	Rehandled (2.0)	Total
1	29,440,100	1,587,280	31,027,380
2	45,940,650	2,273,680	48,214,330
3	87,882,650	7,378,080	95,160,730
4	61,098,200	3,576,680	64,674,880
4A	62,344,100	3,132,480	65,476,580

Option	TONNES RELATIVE TO OPTION 1 Waste	Ore	Stripping Ratio	NPV m$
1	–	–	–	0
2	+17,186,950	+3,623,350	4.7	–35.89
3	+64,133,350	+4,710,870	13.6	–35.66
4	+33,647,500	+3,273,810	10.3	–15.41
4A	+34,449,200	+14,788,880	2.3	+31.09

Option 1 is the steepest option and has zero incremental production. It is therefore the base case for the tonnes of ore and waste mined, ie. 30 and 60 million tonnes respectively (ie. a typical total pit production from Mt Whaleback). Options 2 to 4A have additional ore and waste added to these base case figures which are shown in Table 11.1. The net present value is also shown at the bottom of Table 11.1 for each option which has been calculated from the detailed economic model.

Since it is only the relativities which are important, it does not matter that the total production of the pit is not exactly 30 million tonnes per annum. Similarly, variations in economic parameters used in the calculations are not that critical. Table 11.1 presents the net present value results and assuming Option 1 has a relative NPV of zero, the following NPV values are obtained:

Option 2	–$35.89m
Option 3	–$35.66m
Option 4	–$15.41m
Option 4A	+$31.09m

It can be clearly seen that Option 4A is economically the most attractive since it minimises stripping and maximises ore recovery. It has a low stripping ratio of 2.3 and is at least $31m better than Option 1 and up to $66m better than Options 2 and 3. Option 4A would also be more economically attractive than the present pit design since it recovers more ore and has less waste to remove (see Appendix A).

Nevertheless, it should be pointed out that there may be other wall options further south which may be economically more attractive than Option 4A. This would however result in a 'high grading' of the pit and would sterilise a considerable amount of ore. It is also beyond the scope of this present research investigation.

11.12 CONCLUDING REMARKS

There are several possible options for the geotechnical design of the North Wall. Considering the stability analyses, the geological structure and economic factors, it is considered that Options 4 and 4A are the best two of all the wall designs considered. However there are some problems associated even with these options.

Option 4 follows a toe buttress concept on the basis that the buttress is an insurance policy against large scale overall failure of the Jeerinah Formation. This concept came to light late in the investigations for the thesis (that is the Brockman Formation was not investigated in detail). However the attraction of this buttress concept is that the buttress need not be a permanent feature and can be removed at a later stage in order to remove the ore at the toe of the buttress.

The buttress concept also enables time for the field trials to confirm the shear strengths of the disturbed Jeerinah Shale. In this light, Option 4 was considered to be a cautious approach to the wall design.

However it is now apparent that it has some deficiencies. Firstly, in order to create a 'buttress' to the southerly-dipping Jeerinah Formation, the buttress must follow these in-dipping structures up the wall going in an easterly direction. Therefore what was initially intended as a 'toe' buttress becomes a major wall within itself.

Secondly, the shear strengths of the WFZ and the EFFZ have been confirmed by actual failures and it is therefore known that the shear strengths on these two horizons are in the order of $c = 0$, $\emptyset = 30°$. They are not anywhere near the strength of $c = 1.3$ MPa, $\emptyset = 39°$ as determined from laboratory tests for the disturbed Jeerinah Shale. Some parts of the buttress would, of course, be located in these weak zones. As has been shown in the previous Chapter, the buttress is likely to be unstable in some cases especially under adverse pore water pressure conditions.

If a wall design is adopted without the buttress, ie. Option 4A, the two major problems are those associated with 'daylighting' in-dipping Jeerinah Shale units and problems with the unbenched footwall along the EFFZ contact. 'Daylighting' in-dipping Jeerinah Shale is only a problem if the in-situ shear strength of the Jeerinah Shale is of much smaller magnetude than that indicated by the laboratory test results. A field trial to determine this strength has therefore been strongly recommended to Mt Newman Mining Co.

The problems of the unbenched footwall at the toe of the North Wall will not be trivial, since the footwall itself will consist of highly disturbed EFFZ or WFZ material and be up to five benches high. Nevertheless, Mt Newman Mining Co will have considerable experience in supporting unbenched footwalls by this stage (eg. with the artificial support measures being installed on the South Wall) and the problems of supporting very friable material can be solved (eg. the support work being undertaken at BHP's Koolan Island Mine to stabilise friable sandstones and quartzites).

From an economics point of view, Option 4A is significantly better than all other options considered. It should be noted that the toe buttress has a NPV (net present value) of about $46m. Considering this fact, there will be strong pressure on economics grounds to remove the buttress, ie transition from Option 4 to Option 4A.

It must be concluded that from a stability point of view the buttress concept (Option 4) is the favoured option. It is based on a cautious and conservative approach but one which recognises the immense value of observational data gathered during the progress of a mining project. Such an approach is the cornerstone of modern geotechnical engineering. Ultimately it may be that transition from Option 4 to Option 4A will be possible and that the buttress can be removed without serious threat to stability. Alternatively, it may become possible to accept some small failures as part of such a transition.

CHAPTER 12

SUMMARY, DISCUSSION AND RECOMMENDATIONS FOR FUTURE WORK

12.1 SUMMARY

The research presented in this thesis has been concerned with the important considerations which influence the design of large scale excavated rock slopes. These considerations include engineering geological characteristics, shear strength of the rock mass, traditional stability analyses and practical excavation aspects. Moreover, economic considerations have been considered in this thesis as well.

A total of five new wall designs have been evaluated and the best two of these options have been considered in detail.

Considering the economic constraints to maximise ore recovery, the proposed wall design is the optimal design for the North Wall. However a wall design with a higher net present value may be achieved by adopting an extreme southern option for the North Wall in order to high-grade the orebody. This would necessitate moving the wall much further south and a conceptual economic evaluation of this is worth some consideration (see Figure 12.1). However, such a consideration was outside the scope of this thesis.

Detailed laboratory tests indicate that the shear strength of the Jeerinah Shale is much higher than previously assumed, and this has facilitated a wall design which extracts all ore. However a large scale field trial is recommended in order to confirm these strengths. A decision tree is given in Figure 12.2 showing the North Wall extraction sequence depending on the outcome of the field trial. The results of the field trial will not change the overall wall design, but it will change the extraction sequence. An interim wall design incorporating a toe buttress is proposed for this interim extraction if shear strengths are found to be lower than those predicted by laboratory tests.

Detailed consideration has been given in this thesis to the structural geology, engineering geology, groundwater conditions, physical properties, stability analyses and economic assessments in order to produce an optimal wall design. In addition, innovative techniques have been used to process and present the data including sophisticated computer modelling, cross hole seismic tomographic imaging and computer borelog presentations. The reader is referred to the respective Chapters for further details on the above topics.

12.2 INTRODUCTION

The research presented in this thesis is the result of a comprehensive geotechnical investigation into the North Wall of Mt Whaleback which was funded by Mt Newman Mining Company. A great deal of information has been excluded from this thesis and this includes detailed plans, sections, borelogs and colour slides of all borecores. This additional information is available in the form of reports prepared for the Mt. Newman Mining Company. The author has also had a long involvement with the Newman mining project and some of the ideas in relation to rock slope stability were developed as a result of field experience gained over this period.

In the first part of this thesis, the scope of the research work was outlined and important considerations for rock slope excavations were discussed along with modern concepts of rock mass behaviour. A brief review of the Newman mining project was then presented. This was followed by a discussion of the geology, principally a detailed description of the rock mass. The engineering geology and shear strength characteristics of this rock mass were then described and the influence of groundwater and permeability was explained. The results of back analyses of all the previous failures on Mt Whaleback were presented so that both the strength at failure and the mechanism of excavated rock slope failures could be used for current and future consideration of analysis and design of rock slopes. A range of possible pit slope designs were considered both in terms of stability and economics. Finally, a decision-tree concerning the project has been proposed in the first section of this Chapter (Figures 12.1 and 12.2).

The result of this work has been that two of the best alternatives for wall design have been identified and examined thoroughly. In making a final selection between these two alternatives, there has to be a balance between stability and economics. A toe–buttress pit wall design appears to be the most logical on the basis of stability considerations and be consistent with the observational approach. There can be an eventual transition to the second desirable alternative if and when a decision to remove the toe buttress is taken based on engineering judgement and observation.

12.3 STRUCTURAL GEOLOGY

A comprehensive structural geological investigation has been completed as part of the overall geotechnical study for the North Wall in the East Pit of Mt Whaleback. This investigation has involved both percussion and diamond drilling, pit face mapping, mapping of costeans as well as collation and scrutiny of all existing geological information.

The investigation has been complicated by the fact that there are very few exposures of the Jeerinah Formation in the present pit and much of the crest of the North Wall is covered in waste dumps. Innovative techniques have been used to try and overcome these handicaps including the use of cross hole seismic tomography (see Chapter 4) and the use of sophisticated three dimensional computer graphics.

The main findings of the structural geology investigations are that the Jeerinah Formation is not a simple 'layer cake' model, but is structurally complex being folded on a large scale. These folds generally plunge to the west although this plunge is variable. In addition to this variable fold style, the Jeerinah Formation consists of alternating 'shale' and 'dolerite' units which have been subjected to regional metamorphism resulting in slate/phyllite and amphibolites being present. It is this structural complexity which has the major influence on subsequent pit wall design. A generalised model for the structure of the North Wall is shown in Figure 4.3.

SUMMARY, DISCUSSION AND RECOMMENDATIONS FOR FUTURE WORK

The detailed structural geological findings of this investigation are as follows:

- **The Whaleback Fault Zone.** The structural geology of the site for the North Wall of the Mt Whaleback East Pit is dominated by a steep, south dipping, intense shear zone called the Whaleback Fault Zone (WFZ). This zone constitutes a normal fault with a throw of at least 600m. The WFZ cuts Upper Proterozoic rocks which are deformed by folds which generally plunge gently towards the west and have axial planes which range in dip from 45° southwards down to horizontal. The WFZ also cuts ore bearing horizons and acts as the northern boundary of the orebody.

- **South of the Whaleback Fault Zone.** South of the WFZ the rocks are intensely folded with wavelengths measured in metres. The orebody is truncated at depth by another intense shear zone, the East Footwall Fault Zone (EFFZ), which therefore acts as the lower limit to the orebody. The rocks which constitute the EFFZ are sheared and folded versions of the rocks stratigraphically underlying those horizons which are mineralized.

- **North of the Whaleback Fault Zone.** North of the WFZ are thick meta-dolerites and slates of the Jeerinah Formation. These rocks are the lowest stratigraphically in the area of interest. The wavelength of folds in these rocks is controlled by the thick and relatively massive meta-dolerites and hence is measured in hundreds of metres rather than in metres as on the south side of the WFZ. Otherwise the style of folding is identical north and south of the WFZ except that the axial planes of folds north of the WFZ trend more to the north-west rather than the west as is the case south of the WFZ. The Jeerinah slates are dominated by a strong south-dipping axial plane slaty cleavage.

The WFZ consists of sheared versions of rocks within the EFFZ and can be seen displacing the EFFZ at the east end of the East Pit. The constraint on wall design is that all ore be removed and therefore the toe of the wall must be in the vicinity of the intersection of the WFZ and the EFFZ.

12.3.1 Commentary on Wall Design in Relation to Structural Geology

The orientation of the wall is essentially East–West and with the toe more or less fixed by the above geological and geometrical constraints, the essential problem in designing a wall arises from the fact that different Jeerinah lithologies intersect the wall in passing from east to west along the wall.

This situation occurs because the Jeerinah folds plunge approximately west and the axial planes strike more towards the north–west whilst the wall trends east–west. Hence, the lowest Jeerinah stratigraphic unit, Dolerite A first intersects the wall in the east to be replaced by successively higher Jeerinah stratigraphic units - Shale A, Dolerite B, Shale B, Dolerite C, Shale C - in progressing westwards along the wall.

In the eastern area of the North Wall, it would be excavated in the WFZ and then in Dolerite A but in progressing westwards the geometric problem arises of making the transition from a wall that is essentially in (strong) Dolerite A to a wall that is essentially in (weak) south dipping Shale A and then back to a wall that is essentially in (strong) Dolerite B. The same problem arises in making the Dolerite B/Shale B/Dolerite C transition and, in the future, the same problem will presumably arise in making the Dolerite C/Shale C transition to the west of the area of immediate interest.

The problems of designing a wall in such variable geology have been addressed in detail in Chapter 11.

12.3.2 Conclusions

It should be emphasised that there are very few good exposures of the structure of the proposed North Wall. At the extreme western area of the investigation the structure is only known in broad terms. In particular, the location and number of large faults at depth in this area need to be defined.

It is concluded that much of the information presented in Chapter 4 was both difficult and expensive to obtain. Some of this information was obtained from windows and exposures between waste dumps on the North Wall. Since waste dumping is progressively covering these exposures, it has been strongly recommended to Mt Newman that these surface exposures be mapped in detail prior to future waste dumping.

It has also been recommended that the use of the latest geophysical techniques could greatly assist in future geological investigations and that these techniques would be cost-effective compared to deep diamond drilling.

12.4 CROSS HOLE SEISMIC INVESTIGATIONS

The cross hole seismic investigations were used as an adjunct to the structural geological work in order to confirm postulated structural interpretations. The fact that the North Wall is covered in waste dumps and that there are very few exposures of the Jeerinah Formation has created great difficulties in determining these structures. It is therefore difficult even to find suitable locations to drill in the Jeerinah Formation. It is almost impossible to confirm structure with actual exposures.

However from the drilling that was undertaken and from the original mapping on Mt Whaleback, it was postulated that large scale folding existed in the Jeerinah Formation. The borehole evidence only gave point information and it was essential that the style of folding be confirmed. Surface seismic techniques were of no use because of the large volume of waste covering the crest of the North Wall.

Therefore cross hole seismic tomography was used to determine the structure between strategic boreholes and the model of structural geology was completed successfully.

The specific objective of the survey was to determine the shale and dolerite contacts at various depths within the Jeerinah Formation. During the two week survey, eight pairs of holes were investigated. A total of 400 explosive shots were fired, with seismic signals recorded by a 12 channel data acquisition system.

This was the first time that cross hole seismic tomography had been used at Mt Newman. It is envisaged that confidence in interpretation will improve with experience, and better instrumentation and data processing techniques. This seismic tomographic imaging proved to be both innovative and practical and significantly reduced the amount of additional drilling that would otherwise be required. Thus the cost of necessary investigations was greatly reduced while, at the same time, a new and innovative technique was used successfully.

12.4.1 Results

The field data, in general, are of moderate quality amid large background noise from existing mining activities. However, tomographic imaging was possible with most of the data.

The results indicate that cross hole seismic tomography can detect dolerite/shale contacts. A high velocity region was detected within the shale region in Hole G1481. This was originally thought to be a highly stressed area, but in fact turned out to be a zone of sericitic shale within the generally chloritic Jeerinah Shale.

The processed results indicate that shale/dolerite contacts can be identified by velocity contrasts in the structures. However variations in velocity can also be caused by changes in mineralogy. Therefore these results have been considered with other data available from the boreholes. For example, the gamma log data and the assay data were also useful in determining the overall structure in the Jeerinah Formation.

The present tomographic survey has limited to areas below the water table so that there was an effective geophysical coupling to the rock. This limitation can be overcome by using a system of clampable detectors, together with an appropriate source coupling fluid, such as drilling mud.

In conclusion, good correlation between survey results and known geology from drilling has demonstrated the suitability of cross hole seismic tomography as a tool for mineral exploration and mine planning.

12.5 ENGINEERING GEOLOGY

Both the structure and the engineering geology of the North Wall are complex and have been examined in detail as part of this investigation. The engineering geology investigations have encompassed field mapping of outcrops, discontinuity surveys and costeans, as well as a detailed examination and classification of borehole information.

These investigations have not been solely restricted to the immediate North Wall area, but have included the Jeerinah Formation to the extreme north-east, and a flow unit in the Marra Mamba area to the south-west of Mt Whaleback. Suitable exposures in the Jeerinah Formation in the North Wall area itself have been extremely limited. By following this broad line of investigation, we have been able, for example, to determine the rock mass characteristics of the Jeerinah Formation beyond the influence of the Whaleback Fault Zone (WFZ).

It has been demonstrated during this research that the WFZ is one of the features of engineering geology of the rocks of the North Wall which is most significant for slope stability. The other important features are the style of folding and the slaty cleavage developed in the Jeerinah Shale units. The detailed findings of the engineering geology are as follows:

- The three main stratigraphic units that influence the North Wall design of the East Pit (Jeerinah Dolerite, Jeerinah Shale and Joffre Member) are characterised by different lithologies, discontinuity systems and rock mass strengths.

  The discontinuity systems are generally well defined and reflect not only a lithological influence, but also a strong stratigraphical and structural influence. The discontinuity systems for each of the three stratigraphical units indicate different fold axis directions which may be explained by different fold phases or a rotation of major fold limbs in relation to the principal stress direction.

- Drag folding and a 'disturbed' zone occur in both the Jeerinah Formation to the north and the Brockman Iron Formation to the south of the WFZ. The width of these 'disturbed' zones has been defined by assessing the variation in discontinuity parameters (type, orientation, spacing, roughness) and intact strength reduction due to weathering associated with the WFZ.

- The width of the 'disturbed' zones is variable but generally appears to be greater to west of 7000E and also wider in the Jeerinah Shale and Joffre Member than the Jeerinah Dolerite units. The 'disturbed' zone is typically 15m wide in the Jeerinah Dolerite A and B and up to 30m wide in the Jeerinah Shale B and Joffre Member. Further data is required to confirm the width of the 'disturbed' zone to the west, in the Jeerinah Shale B and Dolerite C units, where weathering effects appear to be more pronounced.

- The drag folding due to the WFZ has resulted in areas of the Jeerinah Formation having an increased southerly dip. This affects overall large-scale slope stability due to the increased dip of the Shale/Dolerite contacts and bench-scale stability due to the increased southerly dip of any well-developed discontinuities.

12.5.1 Conclusions

The main conclusions of the engineering geology investigations are that the rocks of the Jeerinah Formation are generally in 'better' (ie stronger) condition than was previously postulated. However, there are 'disturbed' zones close to the WFZ where the measured engineering geology parameters signify a reduced rock mass strength (refer to Chapter 8).

The WFZ is the most 'disturbed' unit in the North Wall and consequently has the lowest strength. The alteration and weathering within the WFZ has resulted in thin kaolinitic clay layers being present parallel to the direction of faulting. These are the weakest materials within the WFZ.

The Joffre unit of the Brockman Iron Formation does not exhibit the slaty cleavage of the Jeerinah Formation, although it is still 'disturbed' close to the WFZ. The overall engineering geology parameters in the Joffre Member below the present crusher site are favourable for stability.

The discontinuity systems outlined for the Jeerinah Formation are based largely upon the mapping of existing exposures to the east of 7600E where the Dolerite A and Shale A units outcrop. Further data collection in areas to the west has been strongly recommended to Mt Newman as exposures become available and the following specific recommendations are made:

- The effects of the fold axis rotation upon the discontinuity system in the Jeerinah Shale units needs to be determined by additional field mapping.

- The variation in the dip of south-dipping discontinuities, especially the axial-plane cleavage produced by drag folding on the WFZ is not known with confidence and additional data is required by field mapping as suitable exposures become available.

- Sheared shale/dolerite contacts exist and their extent needs to be confirmed by diamond drilling. In particular the Shale A/Dolerite B boundary between 7160E and 7400E requires additional drilling.

- Locate the extent of the 'disturbed' zone and discontinuity characteristics adjacent to the WFZ in the Dolerite B, Shale B and dolerite units further west of the area that has previously been covered by diamond drilling and field mapping.

- Quantify the northern extent of penetrative weathering adjacent to the WFZ and the associated reduction in rock mass strength and occurrence of clay-coated discontinuity surfaces by diamond drilling. This applies particularly to the Jeerinah Shale B unit.

- Additional measurements are required on the south dipping discontinuities in the Joffre Member rocks south of the WFZ particularly for the toe buttress wall design.

12.6 GROUNDWATER AND PIT SLOPE DEPRESSURISATION

Detailed groundwater investigations have been completed to determine what groundwater pressures were likely to be encountered on the North Wall as mining progresses, and also to determine what effect they would have on rock slope stability.

If these investigations indicated a significant influence adverse to stability, then a subsequent step was to determine if these groundwater pressures could be relieved by either natural or artificial means.

The groundwater investigations indicated that the permeabilities of the rocks on the North Wall are extremely low and range from 10^{-7}m/s to 10^{-9}m/s. The WFZ appears to act as a hydraulic barrier between the orebody footwall units to the south and the Jeerinah Formation to the North. However this is not supported by the permeability measurements, since the WFZ has a permeability which is the same or greater than the Jeerinah Formation.

Computer modelling work using this permeability data indicated that natural slope depressurization will be extremely slow. Therefore an artificial depressurization scheme was suggested as a means of reducing pressures behind the proposed pit slope. Horizontal and sub-horizontal drains are considered to be the best means of achieving the required depressurization.

Present seepage from the face above the water table, is considered to be due to local surface seepage. Also the phreatic surface itself is not uniform, and localised perched water tables are present between faults. This is shown in a typical section in Figure 7.4.

A review of the groundwater work indicated that the permeability measurements of the Jeerinah Dolerite have much lower than expected (7×10^{-9}m/s) and additional field permeability measurements are recommended using the rising head method.

Based on permeability measurements alone, drainage appears difficult. However, considering experiences with similar rocks in other parts of the world, it appears that horizontal drains would have a good chance of depressurizing the North Wall in relation to the rate of mining. The lengths and spacing of such horizontal drain holes should be governed by the size of potentially unstable blocks on the North Wall.

Some specific details are also given on the layout of horizontal drain holes and other additional benefits of them. However, their effectiveness cannot be determined by further theoretical or review work, and it is recommended that some trial horizontal drain holes be installed in order to achieve this.

12.6.1 Commentary

The need for either natural or artificial dewatering is determined by the permeability of the rocks on the North Wall. However the requirement to reduce water pressures is determined by the stability analyses of the North Wall. Present data indicates that considering overall large-scale slope stability within the Jeerinah Formation, there is little likelihood of failure even assuming no drawdown of groundwater conditions. There is therefore no need to reduce groundwater conditions within the Jeerinah Formation itself.

However the stability of structures within the Brockman Formation are marginal even in dry conditions. It is therefore essential that the toe buttress or interim slopes south of the WFZ as well as the WFZ itself, be dewatered. As discussed in Chapter 7, this is best achieved by horizontal drainage holes since the experience at other mines indicates that they are usually successful even though permeability measurements may indicate otherwise.

12.6.2 Conclusions

- The permeability of Jeerinah Dolerite (7×10^{-9} m/s) measured by falling head tests and water injection tests with packers in boreholes, is lower than expected. Further measurements have been recommended using the rising head method in borehole test sections drilled without mud.

- The possibility of horizontal drain holes providing useful depressurization at the North Wall, compatible with the rate of mine deepening, appears to be marginal or better. If the ratio of permeability to specific storage k/S_s of the rock mass is less than 10^{-4} m^2/s, depressurization will be relatively slow and difficult.

- The stability of the Brockman Formation is marginal even in dry conditions and therefore the slope will have to be dewatered. However, the Jeerinah Shale is stable even assuming no drawdown and therefore dewatering in the Jeerinah is not required at this stage.

- Horizontal drains may not be required in the Jeerinah Formation but they will be beneficial in the Brockman and WFZ because of:
  (a) an increase in the aperture of joints aligned approximately parallel to the rock slope because of stress relief, and
  (b) the ability to drain any slide surfaces on which slight shearing and dilation has occurred.

- The possibility of diverting surface water away from slope recharge areas should be examined. For example, grading and sealing the surface of waste dumps would be an effective and inexpensive control measure.

- In laying out horizontal drain holes consideration should be given to:
  (a) drilling two or three holes from one drilling position to reduce establishment costs, and
  (b) drilling the hole at 5° below horizontal as recommended in Chapter 7 to reduce drain hole clogging caused by aeration of groundwater containing iron.

12.7 PHYSICAL PROPERTIES AND ROCK MASS SHEAR STRENGTH

A detailed laboratory testing programme has been completed in order to determine rock mass strength of rocks critical for stability. In addition, a detailed review has been undertaken of all previous laboratory and field test work on North Wall rocks and these results have been combined with the results from the current test programme.

The results indicate that there is a wide variation in material strengths on the North Wall from the strongest to the weakest materials. The intact dolerites and intact dolomites are the strongest materials, and the weakest are the clays encountered in various stratigraphic horizons. These results are summarised in Figure 8.8. There is also a wide variation in material strength within the same lithology, and this is particularly true for the disturbed materials in or adjacent to the WFZ. The test results have also shown that within the normal stress range for the North Wall, the strength results can be realistically described by a linear Mohr–Coulomb shear strength envelope although Hoek–Brown non–linear plots have been made as well.

Of crucial importance is the shear strength of the disturbed Jeerinah Shale and for Fault Shale, since these will be the major materials involved in any large scale failure on the North Wall. This current test programme has indicated that the shear strength of these materials can be represented by parameters $c = 1.3$ MPa, $\emptyset = 39°$. This is a lower bound value taken from the test data which was carefully selected for reliability.

All the results from the current testing programme can be conveniently grouped together into six major rock strength types. Of the six major strength types, the two most important are those for the disturbed Jeerinah Shale and the residual strength of Fault Shale. This latter strength is $c = 0$, $\emptyset = 30°$.

A considerable volume of test work has been completed for this and for other test programmes, and it is considered that there is sufficient laboratory test data now available in order to produce reliable strength values. There are also no new laboratory tests that can be used to quantify rock mass strength in any greater detail than those that have been performed as part of this testing programme. Additional detailed laboratory testing is therefore not recommended.

However, there are some indications that the rock mass shear strength of the Jeerinah Shale, in particular, may not be as high as indicated by this test programme. This is, in no way, meant to imply that the laboratory test values are not reliable enough, but simply that it is extremely difficult to represent field conditions by any laboratory test. These indications are firstly, those derived from the engineering geological assessment of the rock mass strength (which in itself, is somewhat qualitative) and secondly, those derived from the strength values obtained from back analyses (which admittedly are not from the Jeerinah Shale).

Therefore it has been recommended to Mt. Newman Mining that the shear strength of the disturbed Jeerinah Shale be confirmed by an in-pit induced slope failure. There are areas on the North Wall where this could be undertaken prior to any major slope exposure of the Jeerinah Shale. This aspect is taken into account by the cautious approach to the design of the North Wall.

Nevertheless, the shear strength results obtained from this test programme can be reliably used in current and future stability analysis provided they are used with care. A summary of the strength results for the North Wall rocks is given in Figure 12.5. In particular it should be noted that similar lithologies have anisotropic strengths and therefore different strength values have been used depending on the direction of the potential failure surface.

12.7.1 Recommendations

The following recommendations are made in relation to the shear strength of rocks on the North Wall.

- The shear strength data used in this thesis is generally reliable and further additional detailed laboratory testing is not recommended. There are no new laboratory tests that can quantify rock mass strength in any greater detail than those performed in this testing programme.

- However there are some new geophysical techniques becoming available which may be able to give a better indication of rock mass shear strength and it is recommended that these be considered for use on the North Wall for future work. Discussion of these new techniques is outside the scope of this thesis.

- It is also strongly recommended that an induced failure in the pit be undertaken in the Jeerinah Shale in order to confirm the laboratory test results. There are several areas in the pit where this trial could take place. (These areas are above the main in–dipping Jeerinah Shale sequence.) Hence information could be obtained before a large scale wall was exposed in the Jeerinah Shale. Even if failure does not occur it would provide useful information on threshold shear strength values.

- The strength results from this thesis can be used in the stability analyses on the North Wall. For critical zones, lower bound shear strengths have normally been recommended and anomalously high values (eg. Mt McRae Fault Shale) should not be used.

- The strength of all intact rocks on the North Wall is such that large scale failure will not occur through them. Failure will only occur either through disturbed zones, fault zones or along discontinuities. Stability analysis work presented in this thesis is based on an appreciation of these facts.

- It is important that the mechanism of any potential failure be considered and that the appropriate shear strength values be applied to it. For example, the same lithology will often have anisotropic strength characteristics.

- It is obvious that the most important shear strength results for the North Wall are those for the disturbed Jeerinah Shale and for the Fault Shales. Therefore any new pit failures on or along these materials should be back analysed to determine their in-situ strength.

12.8 DISCUSSION OF PIT SLOPE DESIGN

Slope stability has been assessed on both a local bench scale and on an overall wall scale. On a local scale, a minimum number of batter and berm configurations have been adopted in order to keep wall designs simple. Batters will generally be stable except in some localised and disturbed zones close to the WFZ. Some localised artificial support may be necessary depending on detailed structure. Also batters in the WFZ will continue to slake and it is impractical to design to avoid these, however the wall design is fairly flat across the WFZ.

The WFZ further to the west is much wider than in the eastern end of the pit and some local steepening may be possible at this location depending on detailed structure. Some local steepening may also be possible in the dolerite although only minor gains can be made by double benching in dolerite since the overall structure must be followed closely.

The interim slope below the crusher and conveyor in the East Pit is stable, although further steepening of this interim wall is not recommended.

A total of five wall designs have been investigated on the basis that all the ore is to be extracted from the Dales Gorge member. They are:

- Option 1 — is the steepest possible option
- Option 2 — is an intermediate option which includes a large buttress
- Option 3 — is a very flat option which removes most of the in-dipping Jeerinah Shale
- Option 4 — is an option which follows the structure closely in the upper part of the wall and then has a variable height toe buttress to stabilise in-dipping Jeerinah Shale
- Option 4A — is an option which is the same as Option 4 except that it removes the toe buttress completely and has an unbenched footwall slope at the toe.

In addition, a further possible option is an extreme southern wall design for the North Wall. This is a wall design that would be a 'high-grade' option and sterilise large amounts of ore in the North Wall. This southern option may be economically attractive and should be considered but it is beyond the scope of these present investigations (see Chapter 11).

Stability analyses conducted on the above options, indicate that from an overall geomechanics stability perspective Option 4 is the best option at this stage of knowledge. However it is important to note that the buttress itself is marginally stable and dewatering it would be essential. The structure is unfavourable and the shear strengths along the WFZ and the EFFZ are known to be at residual values. Also the buttress has to be very large on some cross sections in order to actually buttress the in-dipping Jeerinah Shales. It is therefore considerably more than just a 'toe' buttress.

The shear strength data for the Jeerinah Shale used in the overall stability analysis is much higher than that used in previous studies. Therefore stability analyses of the Jeerinah Formation all indicate that the Jeerinah Shale is stable assuming the measured 'disturbed' strength is truly representative of field conditions. All the evidence currently available suggests this to be the case. The stability of the Jeerinah Formation can also be improved by dewatering, although based on the above, this is not necessary.

However the disturbed strength of Jeerinah Shale has not been confirmed by any back analyses and it is strongly recommended that a large scale field trial be undertaken to confirm the laboratory test results. Areas where this could be undertaken have been identified.

The shear strength of the WFZ and the East Footwall Fault Zone (EFFZ) are known with considerable accuracy and therefore dewatering of a toe buttress or interim slopes south of the WFZ is recommended.

Provided that the shear strengths obtained from the field trial are similar to the laboratory results, Option 4A is recommended as the final wall design. If a field trial indicates significantly lower values, then an interim wall design following Option 4 should be adopted with a subsequent extraction to Option 4A based on further observation and engineering judgement. This subsequent extraction should be closely monitored and threshold movement values should be established. This is shown schematically in the decision tree for the North Wall extraction sequence in Figure 12.2.

Option 4A includes an unbenched footwall in weak or disturbed ground of up to five benches high which will pose its own localised stability problems. However these will not be insurmountable since there is considerable experience already available in relation to the supporting of weak ground. However it is strongly recommended that the basal Dales Gorge/EFFZ and WFZ contact is accurately established prior to commencement of this unbenched footwall.

The structure is less well known at depth particularly to the west of 6600E and therefore more detailed structural investigations should be undertaken in this deep western area to confirm the detailed wall design. Serious consideration should be given to geophysical techniques to do this.

Finally, conceptual wall designs much further south should be considered for the North Wall and simple economic evaluations undertaken on them. However there are serious doubts if such a wall would be as practical and as economically attractive as Options 4 and 4A. Put simply, Option 4A has a good stripping ratio (2.3), maximises ore recovery, maintains a wide pit width and overcomes all of the major stability problems associated with the North Wall given our current level of information. Any wall options further south will inherit considerable problems with all of the above factors. Option 4A is shown in detail in Figures 12.3 to 12.6.

Again, Option 4 is an excellent option since, at this stage, the high shear strength based on the recent laboratory testing programme has not yet been finally confirmed in the field. In fact, from a geotechnical stability perspective it is a better option than Option 4A in as much as it permits transfer to Option 4A as more information and experience is gathered in due course.

12.9 ECONOMIC ASSESSMENT

An economic assessment of the wall design for the North Wall has also been undertaken. It should be emphasised here that these comparisons may not be absolute, but in relative terms they accurately represent the differences between the various options. In addition, the detailed economic assessment was undertaken with the assistance of Mr Lou Tejchman formerly of Mt Newman Mining Co, and economic factors included in the assessment may vary depending on market conditions.

SUMMARY, DISCUSSION AND RECOMMENDATIONS FOR FUTURE WORK

For the economic assessment of the North Wall designs each cross section was split into a number of mining stages ranging from one to nine which represented the remaining life of each cross section in years. These mining stages were then used to calculate the total amount of ore and waste to be obtained over the next nine years of mining and the net present value was then calculated. This was undertaken using a very detailed economic model.

This detailed economic model compares the net present value of Options 1 to 4A. This has been performed by assuming a total pit production of 30 million tonnes of ore and 60 million tonnes of waste and then adding this to the incremental production of each of the Options.

Options 2 through to 4A were then compared to Option 1 (the steepest possible option that could be used) in order to obtain volumes of both waste and ore. For example, the amount of ore for Option 2 was obtained by subtracting the amount of sterilised for Option 2 away from the amount of sterilised ore for Option 1. Other Options were calculated in a similar manner.

A summary of the results is given below:

TONNES RELATIVE TO OPTION 1

Option	Waste	Ore	Stripping Ratio	NPV m$
1	–	–	–	0
2	+17,186,950	+3,623,350	4.7	–35.89
3	+64,133,350	+4,710,870	13.6	–35.66
4	+33,647,500	+3,273,810	10.3	–15.41
4A	+34,449,200	+14,788,880	2.3	+31.09

From this work it can be clearly seen that Option 4A is economically the most attractive and this has therefore been recommended along with Option 4, which is better from a geotechnical stability perspective at the present time. Other costs of each design, such as costs to clean-up failures and artificial support costs have not been quantified at this stage for Option 4A since the detailed extent of failures in the western section of the area cannot be quantified yet. However the above economic factors indicate that Option 4A could accommodate considerable support and clean-up costs and still be more attractive than other options.

12.10 MAJOR CONCLUSIONS

The conclusions of all of these areas of investigation are as follows:

- The bench stability for all of the batter/berm designs proposed is adequate. Some minor problems are likely to be experienced with disturbed material close to the WFZ but these will be limited in extent. Localised artificial support may be necessary depending on detailed structure.

- For highly disturbed clay coated discontinuities, there are likely to be batter scale stability problems. However, these are also considered to be of limited extent close to the WFZ.

- For the WFZ slaking will be a continuing problem as well as minor failures occurring on a bench scale. It is impractical to design to avoid these. Further to the west, the WFZ is much wider and the amount of competent material within it increases.

- Batter and berm configurations are designed to produce the following overall wall angles:

  48° in the Jeerinah Dolerite and Jeerinah Shale
  31° in the Whaleback Fault Zone (WFZ)
  36° in the Joffre Member

  These are considered to be correct on an overall scale but local steepening may be possible particularly in the dolerite and in the WFZ.

- The interim pit wall below the present crusher and conveyor is stable. However it should be noted that the fill for the present haul road has already encroached on this interim wall design and therefore there appears to be little scope operationally for steepening this interim wall. The dip of the Joffre in this location is either flat or dips to the north with both vertical and low angle faults or shears. Since the friction angle along these low angle shears is greater than their dip, the wall is stable. The strength of the Joffre is too high for failure to occur across bedding.

- The shear strength data used in the stability analyses of both benches and overall slopes have been taken from Chapter 8. These indicate that the overall stability of the Jeerinah Formation is high even in 'wet' conditions. The stability of the WFZ and the Joffre Member is only marginal even in 'dry' conditions.

- A total of five wall designs have been proposed which attempt to extract all ore from the Dales Gorge member. They are:

Option 1	is the steepest possible option
Option 2	is an intermediate option which includes a large buttress
Option 3	is a very flat option which removes most of the in–dipping Jeerinah Shale
Option 4	is an option which follows the structure closely in the upper part of the wall and then has a toe buttress of variable height to stabilise in–dipping Jeerinah Shale
Option 4A	is an option which is the same as Option 4 except that it removes the toe buttress completely and has an unbenched footwall slope at the toe.

- The buttress included in Options 2 and 4 has stability problems. They have unfavourable structure and the shear strengths along the WFZ and the EFFZ are known to be at residual values. The buttress also has to be very large on some cross sections to stabilise the in–dipping Jeerinah Shales. This buttress therefore becomes a major wall in itself. Drainage of the buttress is essential.

- Stability analyses of the Jeerinah Formation all indicate that slopes in the Jeerinah will be stable assuming the measured 'disturbed' strength is applicable. All the evidence currently available suggests this to be the case. The stability of the Jeerinah Formation can also be improved by dewatering, although based on the above this is not necessary.

- The disturbed strength of the Jeerinah Shale has not been confirmed by any back analyses.

- Option 4A is contingent upon the in–situ disturbed shear strengths for the Jeerinah Shale being adequate if not as high as the strengths based on the laboratory testing programme.

- Dewatering in the WFZ and the Brockman at the toe of the slope will improve their stability significantly.

- The stability analyses of the different wall options indicates that Option 4A overcomes most of the major stability problems and from an economic point of view is the preferred option. There are some stability problems with Option 4A in particular near disturbed contacts in the Jeerinah next to the WFZ, and with the unbenched footwall proposed at the toe. From an overall geotechnical stability perspective, Option 4 is the preferred option at the present time when the high strength of Jeerinah Shale based on laboratory testing has not, as yet, been confirmed in the field.

- Artificial support measures will almost certainly be required for Option 4A in the two problem areas listed above.

- Option 4A will also have a significant unbenched footwall up to five benches high. This will be in highly disturbed and friable material unlike the South Wall. However this will only be encountered at the base of the wall design. Experience on the South Wall at Mt Whaleback and experiences elsewhere indicate that such a footwall is achievable.

- Option 4A is economically the most attractive since it minimises stripping and maximises ore recovery. It has a low stripping ratio of 2.3 and is at least $31m better than Option 1 and up to $66m better than Options 2 and 3. Option 4A would also be more economically attractive than the present pit design since it recovers more ore and has less waste to remove.

- However, it should be pointed out that there may be other wall options further south which may be economically more attractive than Option 4A.

An economic assessment of this should therefore be undertaken in the very near future so that this wall design can either be dismissed or investigated in greater detail (this is shown schematically in Figure 12.1). A detailed geotechnical investigation of this possible southern wall design is beyond the scope of this thesis.

12.11 KEY RECOMMENDATIONS

The following key recommendations are made as a result of the detailed research investigations:

- The batter/berm configurations and hence overall slope angles proposed as a result of these investigations should be adopted. These configurations should only be modified if detailed, local structural and strength information is available. The possibilities are to steepen the dolerite and the western WFZ or reduce batter angles on highly disturbed or clay coated zones close to contacts. The number of permutations of batter/berm configurations has deliberately been kept to a minimum in order to simplify the wall design.

- The economic analysis of wall options should be compared in more detail in **absolute** terms. The economic assessments undertaken are correct in **relative** terms only. This requires use of Newman's orebody data base and is a detailed task. In addition, consideration of a North Wall design further south is beyond the scope of this thesis. However such a wall option should still be considered on a detailed economic basis including impact on reserves, stripping, operational width, etc. in order to determine if there are other viable wall designs further south. Nevertheless it should be stated that such a wall design would have to be a long way further south, and would sterilise significant reserves.

- The wall design Option 4A can be adopted for the North Wall design if a field trial to determine the shear strength is successful. If the field trial proves that the strength of the Jeerinah Shale is very low, then an initial pit development following Option 4 with a subsequent final scavanging operation to excavate back to Option 4A is recommended. In this case, final pit slopes on Option 4A would have to be monitored carefully for movements and safety threshold values be established. In addition, it is essential that the initial pit development following Option 4 stop at least two benches above the EFFZ at the toe of buttress.

- A field trial to determine the shear strength of the Jeerinah Shale is almost mandatory and is strongly recommended. Areas have been identified where such a trial could be undertaken. The field trial should not necessarily be expected to produce a significant or even small failure. Stability of the slope during the field trial will still indicate minimum threshold shear strength values if a failure does not occur.

- There is sufficient laboratory test data currently available and further detailed laboratory work is not recommended.

- Given the importance of the crusher and, in particular, its proximity to the WFZ and the structure in the Joffre, it is recommended that the interim wall below the crusher not be steepened.

- Subject to confirmation by a field trial, the shear strengths used in the stability analyses indicate that the Jeerinah Formation will be stable. However disturbed Jeerinah (both dolerite and shale) close to the WFZ could present some problems.

Therefore it is recommended that the stratigraphic contacts close the WFZ be determined accurately at least TWO benches ahead of mining. This is particularly relevant for the Shale A/Dolerite A contact at approximately 7500E, the Dolerite B/Shale A contact at approximately 7400E, and the Shale B/Dolerite B contact at approximately 6600E.

- Any buttress of Brockman Formation material left next to the WFZ, either for Option 4 or during intermediate extraction for Option 4A, will be only marginally stable even in dry conditions. Therefore dewatering of the Brockman and the WFZ is essential. Groundwater investigations have revealed perched water tables within the Brockman and these should be dewatered prior to the development of interim or major slopes in it. If natural dewatering is not sufficient, additional artificial dewatering should be used.

  On current information, there is no need to embark on a major dewatering program for the Jeerinah Formation. This conclusion is, of course, subject to the field trial to confirm the strength of the Jeerinah Shale. Groundwater levels should continue to be monitored to check dewatering of North Wall.

- Artificial supports will be necessary for slopes associated with Option 4A particularly for the unbenched footwall at the toe of the North Wall and possibly for disturbed contacts. Artificial supports are not considered practical to stabilise other structures since they are on a scale of hundreds of metres (eg. potentially unstable Jeerinah Shale blocks or large buttress blocks). Design of these artificial supports requires detailed drilling information which is best undertaken only a few benches in advance of mining.

- The unbenched footwall for Option 4A follows the EFFZ contact as well as having the WFZ immediately behind the wall. This is up to five benches high and will pose its own localised stability problems. However these will not be insurmountable since there is considerable experience already available on supporting weak ground. However it is strongly recommended that the basal Dales Gorge/EFFZ and WFZ contacts be accurately established prior to commencement of the unbenched footwall. This applies particularly to areas west of 7400E.

- The structure is not known in sufficient detail in the western area (west of about 6600E) to design detailed batters and berms. However the overall wall design is correct. It is therefore recommended that more detailed structural investigations be undertaken in the western area to confirm the structural model and hence the detailed wall design. Serious consideration should be given to geophysical techniques to locate major faults and stratigraphic boundaries accurately.

- Permanent survey monitoring stations are already in place in parts of the North Wall. These should be extended and continue to be checked for movements. This should apply particularly to structures south of the WFZ which may form part of an interim wall or part of the toe buttress.

DECISION TREE FOR NORTH WALL DESIGN

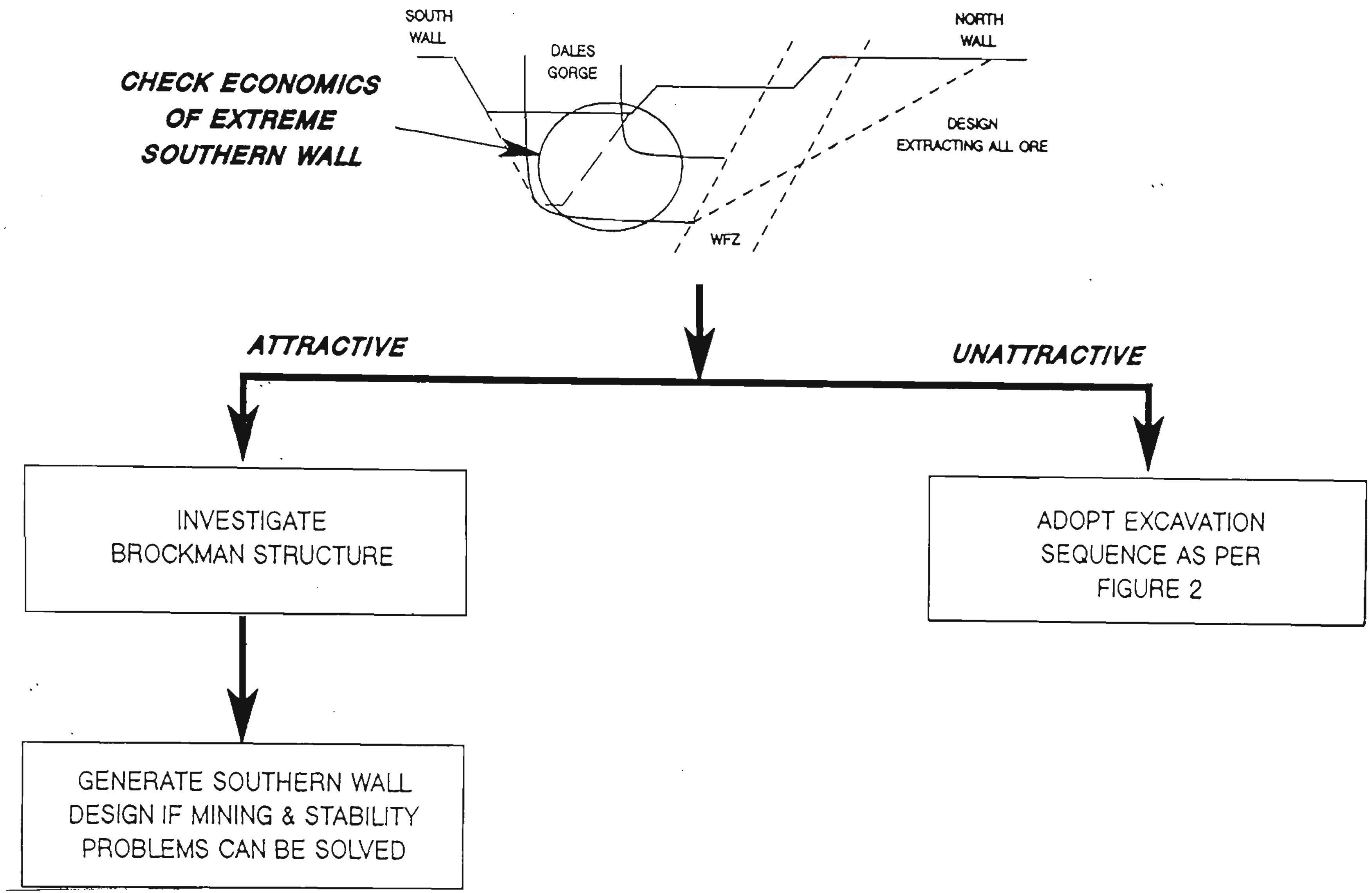


FIGURE 12.1

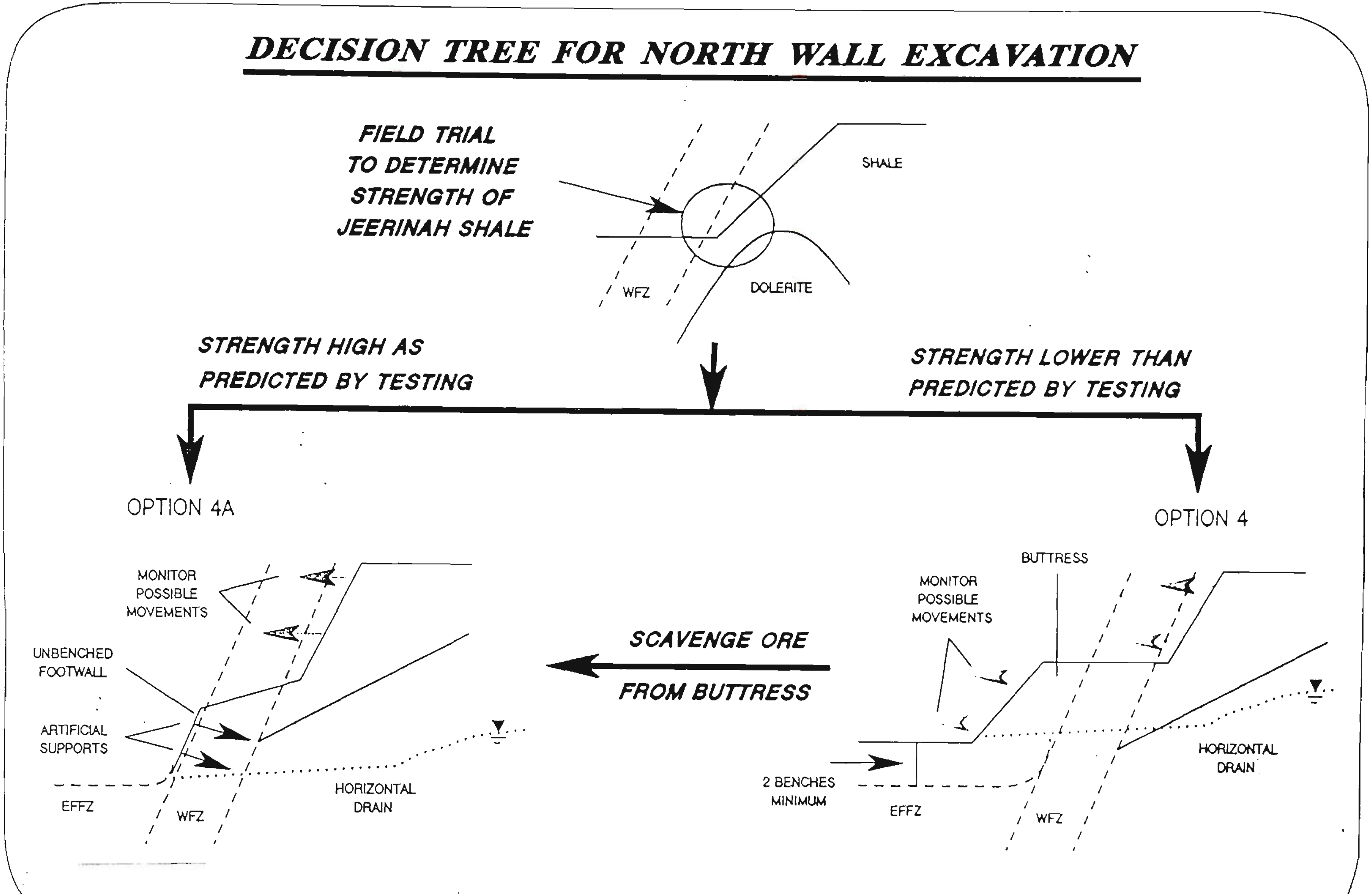
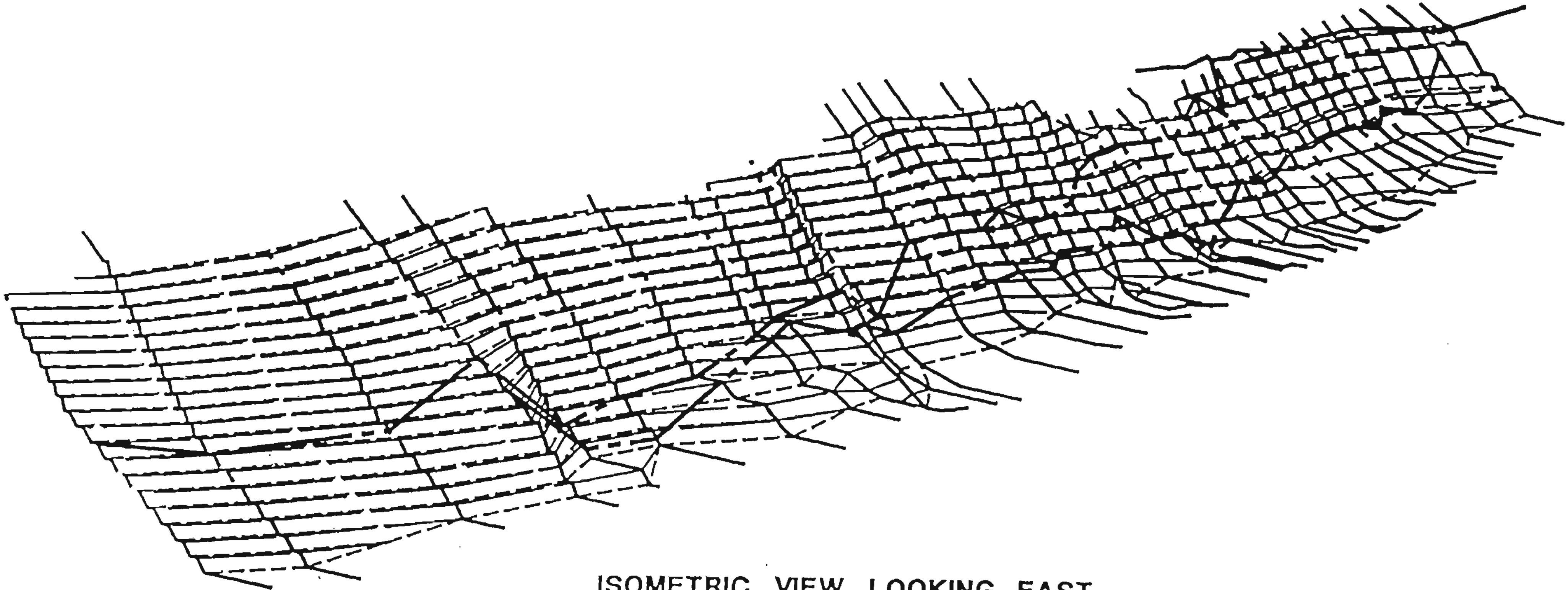

DECISION TREE FOR NORTH WALL EXCAVATION
FIELD TRIAL
TO DETERMINE
STRENGTH OF
JEERINAH SHALE
SHALE
WFZ
DOLERITE
STRENGTH HIGH AS
PREDICTED BY TESTING
STRENGTH LOWER THAN
PREDICTED BY TESTING
OPTION 4A
MONITOR
POSSIBLE
MOVEMENTS
UNBENCHED
FOOTWALL
ARTIFICIAL
SUPPORTS
EFFZ
WFZ
HORIZONTAL
DRAIN
SCAVENGE ORE
FROM BUTTRESS
OPTION 4
BUTTRESS
MONITOR
POSSIBLE
MOVEMENTS
2 BENCHES
MINIMUM
EFFZ
WFZ
HORIZONTAL
DRAIN

FIGURE 12.2

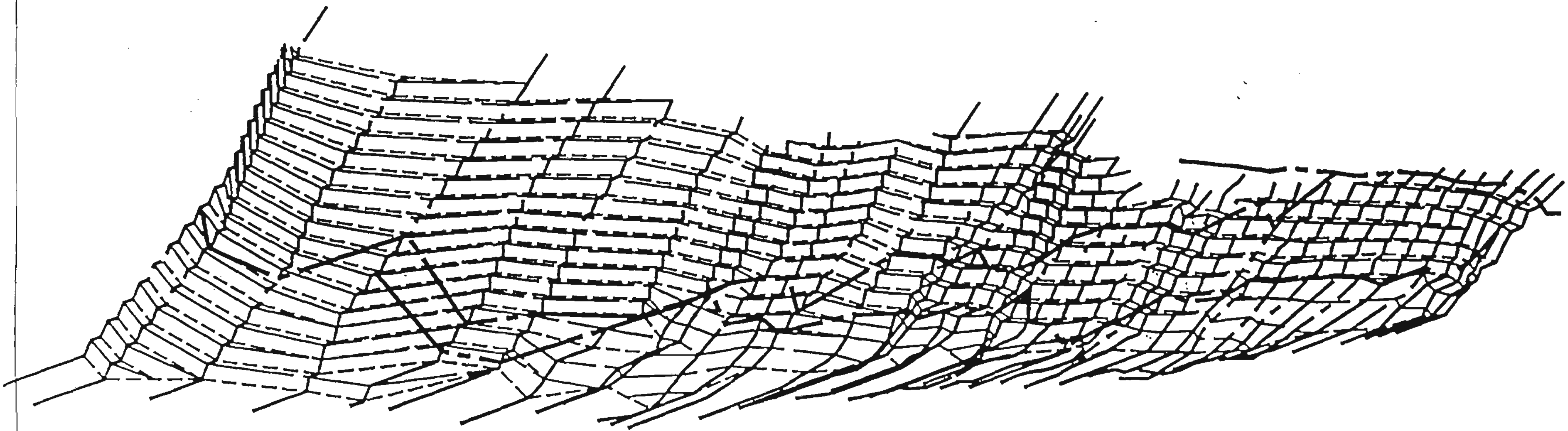
PROPOSED PIT WALL DESIGN OPTION 4A



ISOMETRIC VIEW LOOKING EAST

FIGURE 12.3

PROPOSED PIT WALL DESIGN OPTION 4A



ISOMETRIC VIEW LOOKING WEST

FIGURE 12.4

PROPOSED PIT WALL DESIGN OPTION 4A

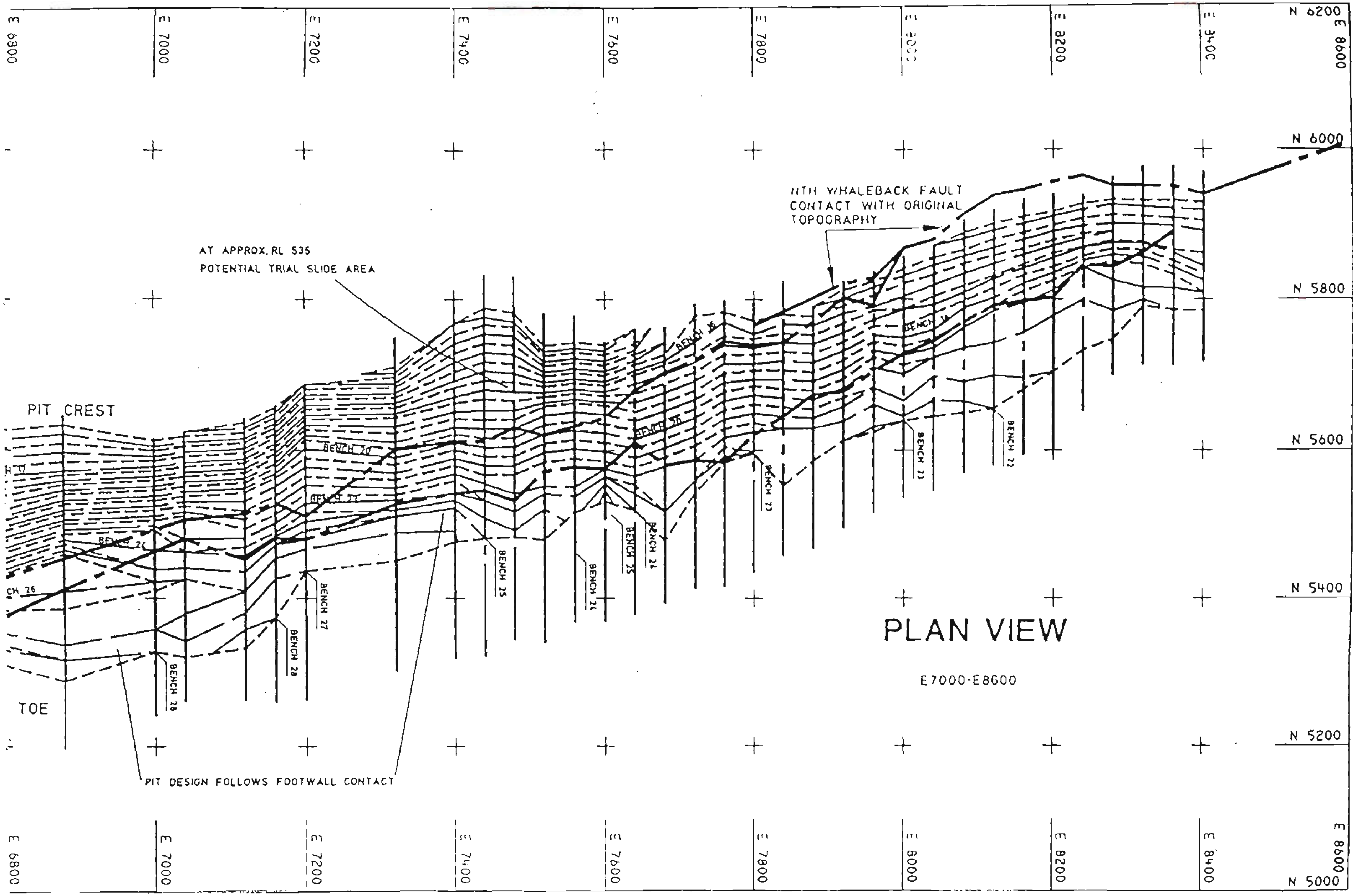


FIGURE 12.5

PROPOSED PIT WALL DESIGN OPTION 4A

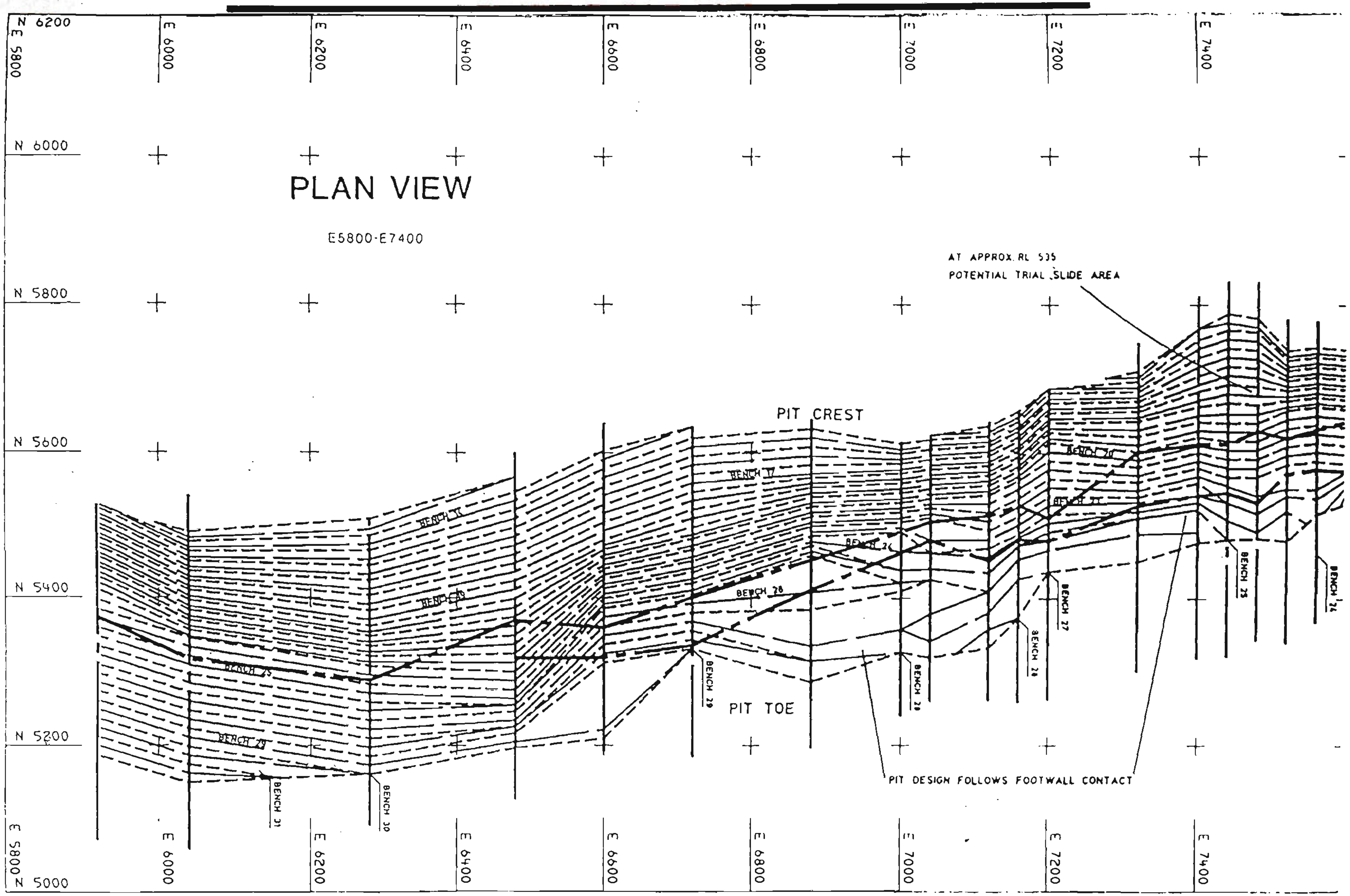


FIGURE 12.6

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GLOSSARY

1.0 STRATIGRAPHIC SYMBOLS

Pfj	Jeerinah Formation
Phmm	Marra Mamba Formation
Phd	Wittenoom Dolerite
Phs	Mt Sylvia Formation
Phr	Mt McRae Shale
Phbd	Dales Gorge Member
Phbw	Whaleback Shale Member
Phbj	Joffre Member
WFZ	Whaleback Fault Zone
EFFZ	East Footwall Fault Zone

2.0 MISCELLANEOUS SYMBOLS

Eigen Vector — For engineering geology applications it is a method to measure the variation about a mean pole of joint data

Fisher's Constant K — Provides a measure of the clustering of discontinuity sets and is estimated by:

$$K = \frac{N - 1}{N - R}$$

(N = number of observations, R = resultant vector)
If there is no preferred orientation, K is close to unity and if all discontinuities are parallel, K approaches infinity. High K values thus indicate a small scatter.

RQD — Rock Quality Designation
A measure of the number of fractures in a length of core. Therefore RQD = 100% represents no fractures.

ISRM — International Society of Rock Mechanics

HQ	Size of drill core, nominally 63mm
UCS	Unconfined Compressive Strength normally quoted in MPa
m and s	empirical constants used to describe a curved shear strength envelope (after Hoek)
costeans	trenches normally cut by a dozer to expose geology
i	roughness angle of joint surfaces. Measured as the difference between a flat plane and a rough surface. The value 'i' is normally added to phi to account for the increase in shear strength of a rough surface over a flat plane
phi	angle of shearing resistance or friction angle. A measure of the rate of increase in shear strength with increasing normal load
c	cohesion. Literally the shear resistance of a material or surface at zero normal load. It is a convenient term which is normally determined simply as the intercept of the failure envelope on the shear stress axis.
Sigma n	the normal stress applying on the shear surface in question. An increase in normal stress will increase the shear resistance unless phi equals zero.

3.0 DEFINITION OF COMPLEX GEOLOGICAL TERMINOLOGY

Term	Description
amphibole	a silicate mineral similar to pyroxene
amphibolite	metamorphic rock consisting of mainly amphibole
anastomosing	cross connecting
anisotropy	properties vary according to direction
axial-plane	the surface joining the hinge lines in adjacent folded surfaces
axial-plane cleavage	slaty cleavage occurring parallel to the actual plane of a fold
anticline	a fold in the form of an arch
asymetric fold	non-symetric fold
antithetic	a term applied to fault planes which dip in the opposite direction to the bedding
boudinage	a minor structure arising from tensional forces which has the appearance of a string of sausages
chlorite	layer lattice mineral such as talc or brucite, normally green
clasts	an inclusion of a pre-existing rock in a sedimentary rock
cleavage	a plane of weakness
chevron fold	fold style typically in a 'Z' shape
coeval	occurring at the same time
conjugate	a term used to describe two sets of structural features formed at the same time but aligned in differing directions

crenulation cleavage	a discontinuous or non-penetrative fabric formed by deformation of an earlier foliation
downthrow	lower side of fault
dextral	describes the relative movement of the side of the fault in this case to the right
diagenesis	changes which take place in the composition and strength of rocks and minerals due to burial
dolerite	a medium grained basic hypabyssal igneous rock
en echelon	a series of parallel or sub-parallel features
epidotes	rock forming silicate minerals normally formed as a result of low to medium grade metamorphism
felspar	the most important single group of rock forming silicate minerals
felspathic	containing felspars
fabric	texture and structure of any rock
foliation	parallel orientation of platy minerals or mineral banding in rocks
fold	a flexure in rock
goethitized	hydrated iron oxide normally a weathering product from other iron minerals (eg hematite)
hinge	change in direction of the fold
hematite	important iron oxide mineral – main ore in Dales Gorge Member
kaolinite	clay mineral
kink fold	folds with planar limbs and very angular hinges

listric fault	faults which are 'spoon shaped' – typically concave upwards
lineation	one dimensional feature in a rock or shown on the rock surface
lamination	thin discreet layers of rock
meta-dolerites	dolerites which have been altered by regional metamorphism
normal fault	a fault with a major dip/slip component in which the hangingwall is on the downthrow side
ophitic	characteristic texture of some basic igneous rocks especially dolerites
oblate	flattened
parasitic folds	smaller scale folds on the limbs of larger scale folds
pitch	angle between horizontal and axis measured on the axial-plane of the fold
phyllite	a cleaved metamorphic rock with affinity to slate and schists
plagioclase	a felspar mineral
plunge	the angle between the axes of a fold and the horizontal line lying in a common vertical plane
pressure solution cleavage	a cleavage formed by the transfer of material in solution in a stressed environment
prolate	extended in width
Proterozoic	an rock age between 570 and 2390 million years before the present
pyroxene	silicate mineral commonly found in amphiboles
recumbent	a fold in which the axial-plane of the fold is sub-horizontal

reverse fault	a fault with a major dip/slip component in which the hangingwall is on the upthrow side
sericite	a fine grained which mica often with a high SiO_2 and MgO content
shear zone	a zone with sub-parallel walls in which high deformations are localised
sinistral	a term applied to tear faults to describe the apparent direction of apparent movement, in this case to the left
slate	low-grade regionally metamorphosed argillaceous rocks which have developed a well marked cleavage but have suffered little recrystallisation so that the rocks are still very fine grained
sub-ophitic	a textural term applied to basic igneous rocks
syncline	a basin shaped fold, opposite of an anticline
tectonic/tectonism	a adjective used to relate a particular phenomenon to a structural orogenic concept
vergence	relationship between bedding and cleavage on fold structures
vug	the cavity in a rock
xenolith	an inclusion of pre-existing rock in an igneous rock

4.0 DEFINITION OF GROUNDWATER TERMINOLOGY

Term	Description
Aquifer	An aquifer is a formation, group of formations or part of a formation that contains sufficient saturated permeable material to yield significant quantities of water to bores and springs.
Confined (or Pressure or Artesian) Aquifer	A confined aquifer is a completely saturated permeable formation of which the upper and lower boundaries are impermeable layers. In a confined aquifer the water is under sufficient pressure to cause it to rise above the aquifer if given the opportunity. The level to which the water rises is called the potentiometric level, the static water level or static head.
Semi-Confined (or Leaky) Aquifer	The confining layers of many pressure aquifers are not completely impermeable thus allowing vertical leakage of water to occur. In this report the aquifer is defined as semi-confined or confined depending on whether the effects of leakage were or were not measurable under the pumping test conditions.
Unconfined Aquifer	An unconfined aquifer is a permeable formation only partly filled with water overlying a relatively impermeable layer. It contains water which is not subjected to any pressure other than its own weight.
Confining Bed	The body of impermeable material stratigraphically adjacent to one or more aquifers. In nature its hydraulic conductivity may vary from zero to some value distinctly lower than that of the aquifer.
Potentiometric Surface	The potentiometric surface is a surface which represents the static head. As related to an aquifer it is defined by the levels to which water will rise in tightly cased wells. The water table is the potentiometric surface of an unconfined aquifer.

Hydraulic Gradient

The hydraulic gradient is the change in static head or hydraulic potential, per unit of distance in a given direction. If not specified the direction generally is understood to be that of the maximum ate of decrease in head.

Dewatering

The removal of groundwater from water-bearing ground, generally to produce unsaturated, drained conditions.

Depressurization (or Pressure Relief)

The lowering of the potentiometric surface.

Depressurization of a confined aquifer can occur without producing any unsaturated ground, however once unsaturated conditions are produced in the aquifer then dewatering (as well as depressurization) occurs at that point.

Transmissivity

The transmissivity (T) of an aquifer is the rate at which water at the prevailing viscosity can be transmitted through a unit strip of aquifer under a unit gradient.

$T = Kb$ where :
K = hydraulic conductivity (also referred to as coefficient of permeability)
b = aquifer thickness

Dimensions of T are: Volume/unit time/unit width
Units used are : $m^3/d/m = m^2/d$

Specific Yield

The specific yield of a rock or soil is the ratio of (1) the volume of water which the rock or soil, after being saturated, will yield by gravity to (2) the volume of the rock or soil. The definition implies that gravity drainage is complete.

In the natural environment, specific yield is generally observed as the change that occurs in the amount of water in storage produced by the draining or filling of pore spaces and is, therefore, dependent upon particle size, rate of change of the water table, time, and other variables. Hence, specific yield is only an approximate measure of the relation between storage and head in unconfined aquifers.

Storage Coefficient (or Storativity)

The storage coefficient is the volume of water an aquifer releases from or takes into storage per unit surface area of the aquifer per unit change in head.

In a confined water body the water derived from storage with decline in head comes from expansion of the water and compression of the aquifer; similarly, water added to storage with a rise in head is accommodated partly by compression of the water and partly by expansion of the aquifer. In an unconfined water body, the amount of water derived from or added to the aquifer by these processes generally is negligible compared to that involved in gravity drainage or filling of pores; hence, in an unconfined water body the storage coefficient is virtually equal numerically to the specific yield.

APPENDIX A

BOREHOLE TYPE

R ⊕	DENOTES REVERSE CIRCULATION DRILLHOLES
G ⊕	DENOTES GRADE DRILLHOLES
YP ⊕	DENOTES GEOLOGY T4 DRILLHOLES
D ⊕	DENOTES DIAMOND DRILLHOLES

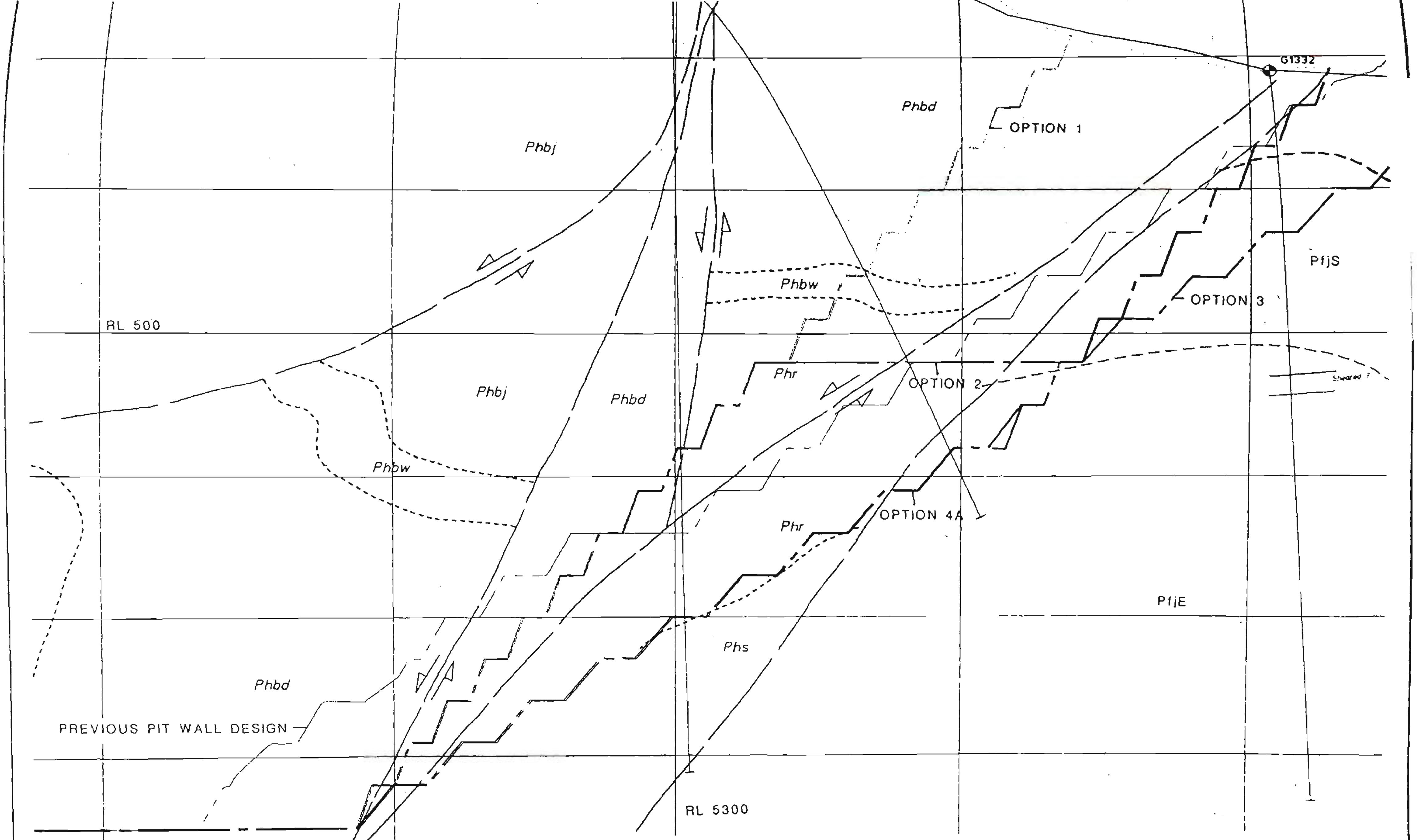
STRATIGRAPHY

LG	LOW GRADE ORE
HG	HIGH GRADE ORE
Phbj	JOFFRE MEMBER
Phbw	WHALEBACK SHALE MEMBER
Phbd	DALES GORGE MEMBER
Phr	MT. McRAE SHALE
Phs	MT SYLVIA FORMATION
PfjS	JEERINAH SHALE
PfjE	JEERINAH DOLERITE
Phd	WITTENOOM DOLOMITE
WFZ	WHALEBACK FAULT ZONE
FWF	EAST FOOTWALL FAULT

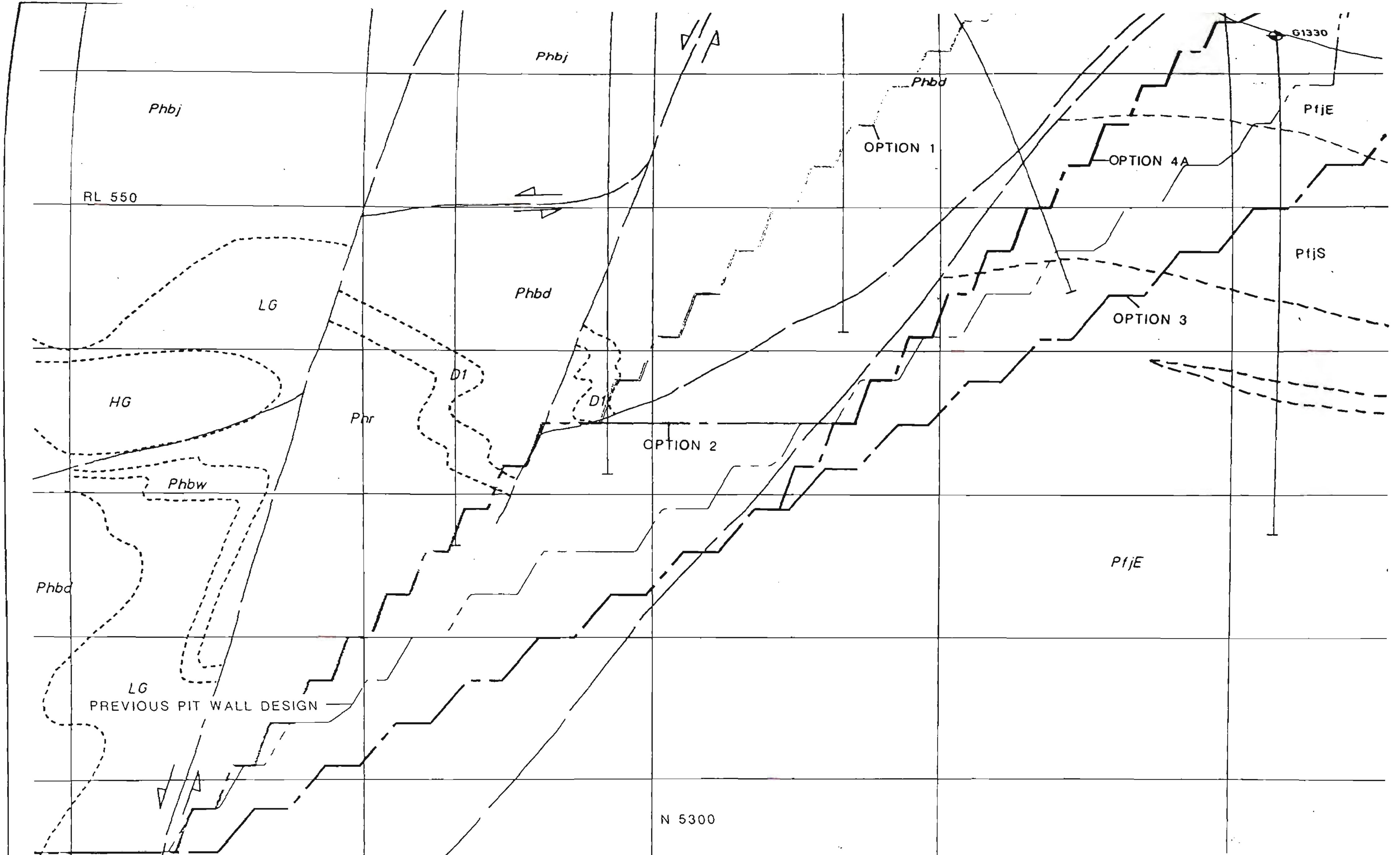
PIT LIMITS

———————	ORIGINAL TOPOGRAPHY
————— — -	OLD ULTIMATE PIT
————— — -	PROPOSED WALL DESIGN
——— ———	FAULTING

Scale	LEGEND TO FIGURES A2 - A40	FIGURE
Drn		A1
Dwg. No.		



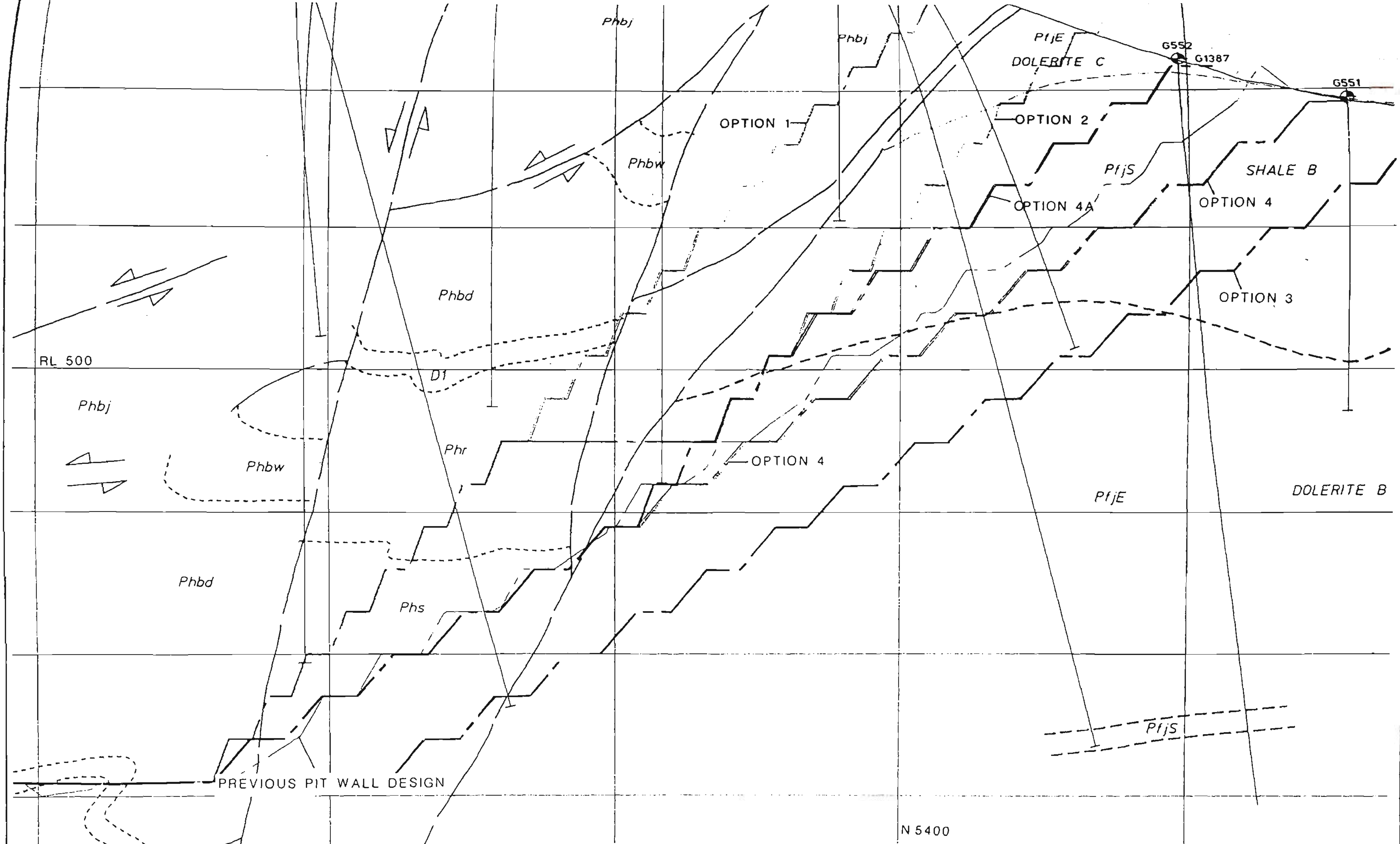
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 5920 E	FIGURE A2
Drn		
Dwg No.		



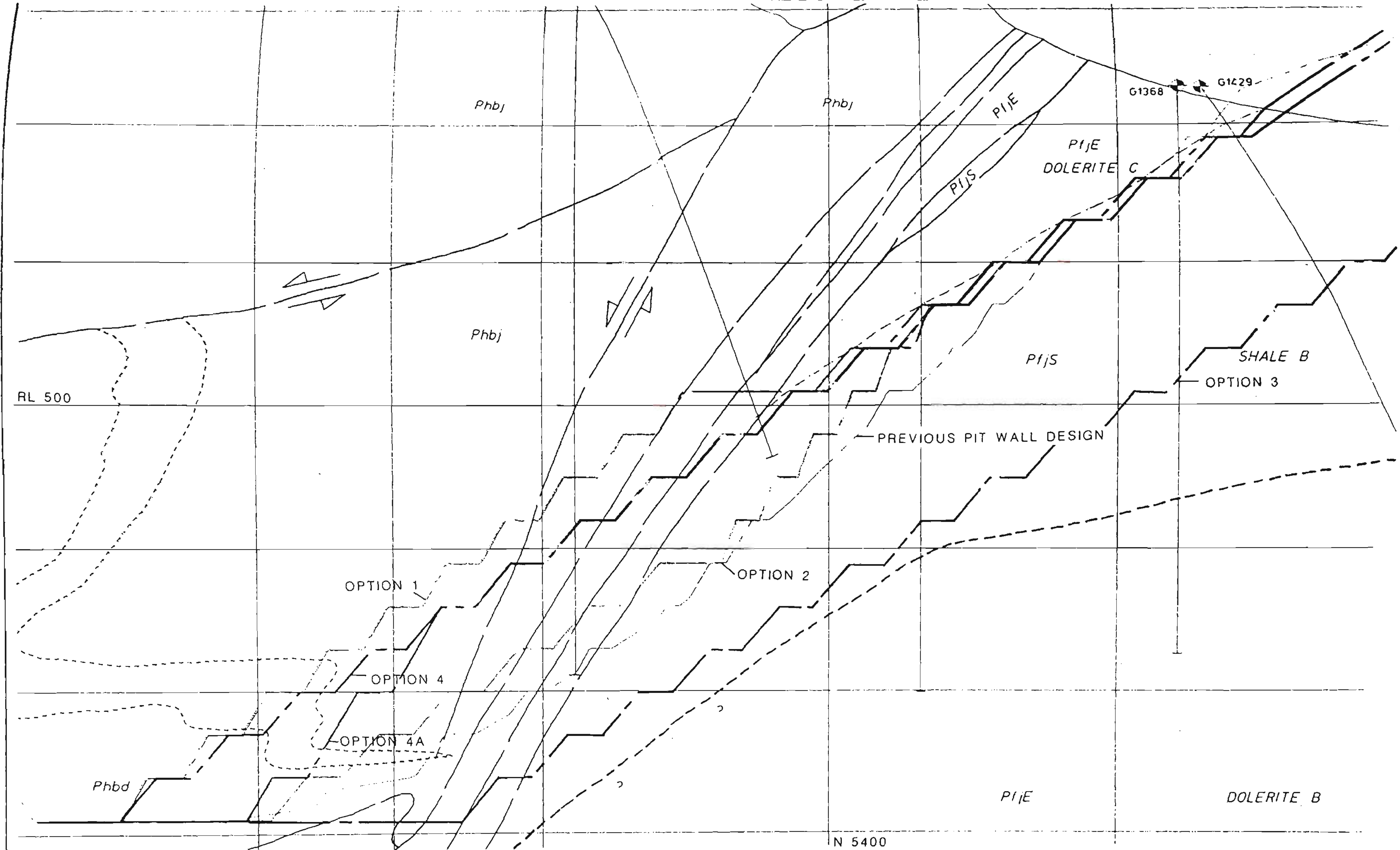
Scale	
Drn	
Dwg No.	

MT. WHALEBACK EAST PIT - NORTH WALL
CROSS SECTION 6040 E

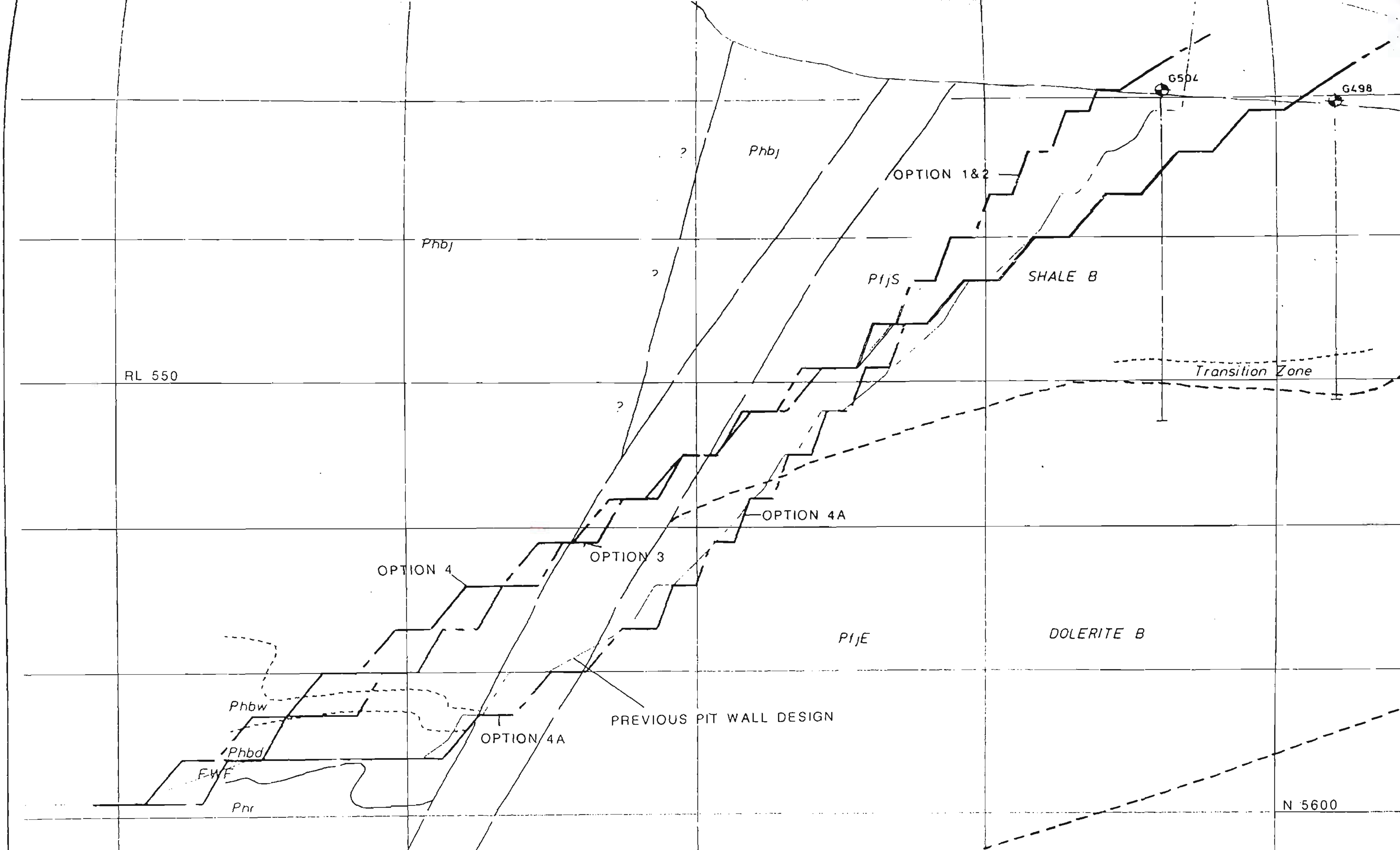
FIGURE
A3



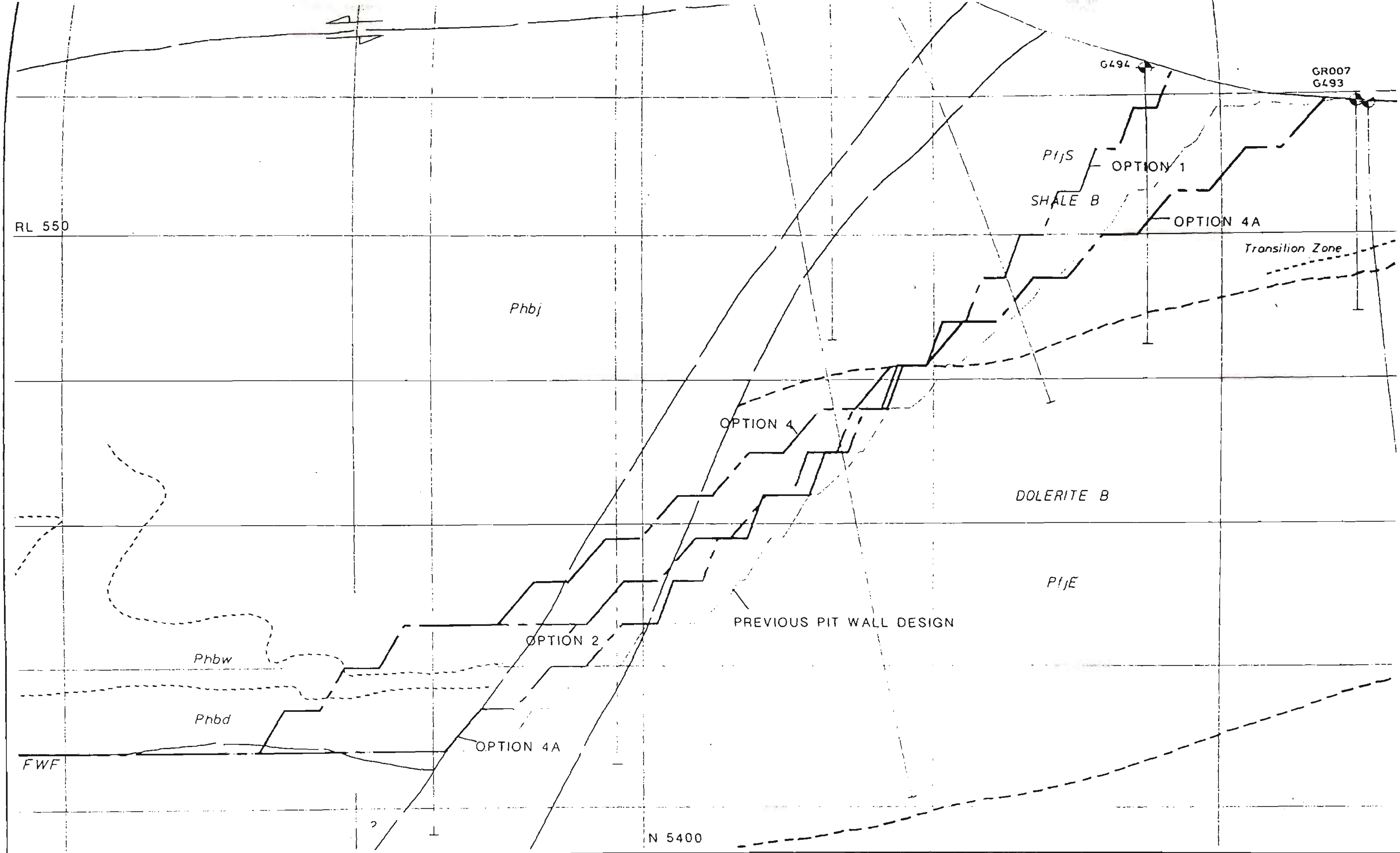
Scale		MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 6280 E	FIGURE A4
Drn			
Dwg No			



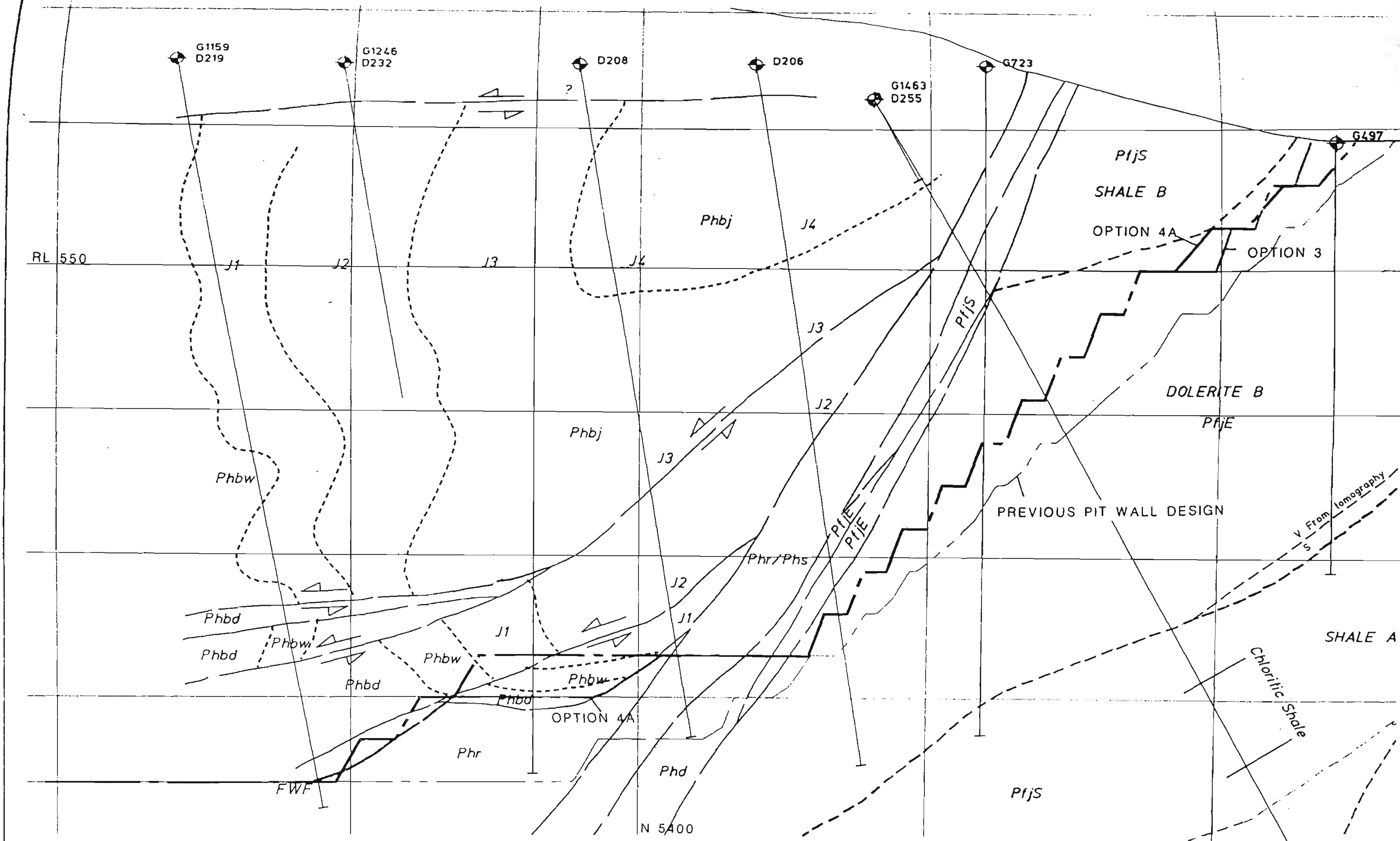
Scale	MT. WHALEBACK EAST PIT - NORTH WALL	FIGURE
Drn	CROSS SECTION 6480 E	A5
Dwg No		



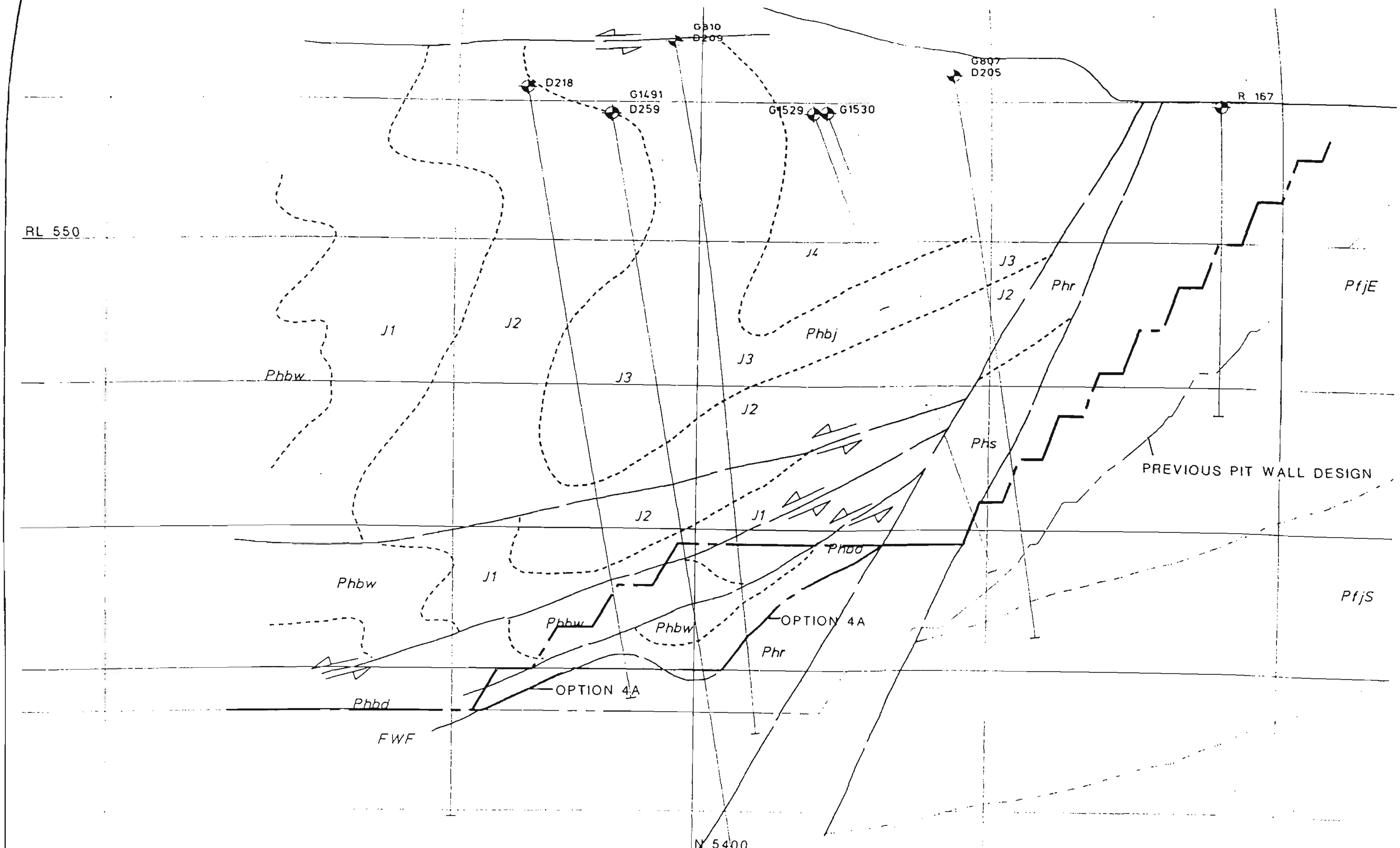
Scale	MT. WHALEBACK EAST PIT - NORTH WALL	FIGURE
Drn	CROSS SECTION 6600 E	A6
Dwg No		



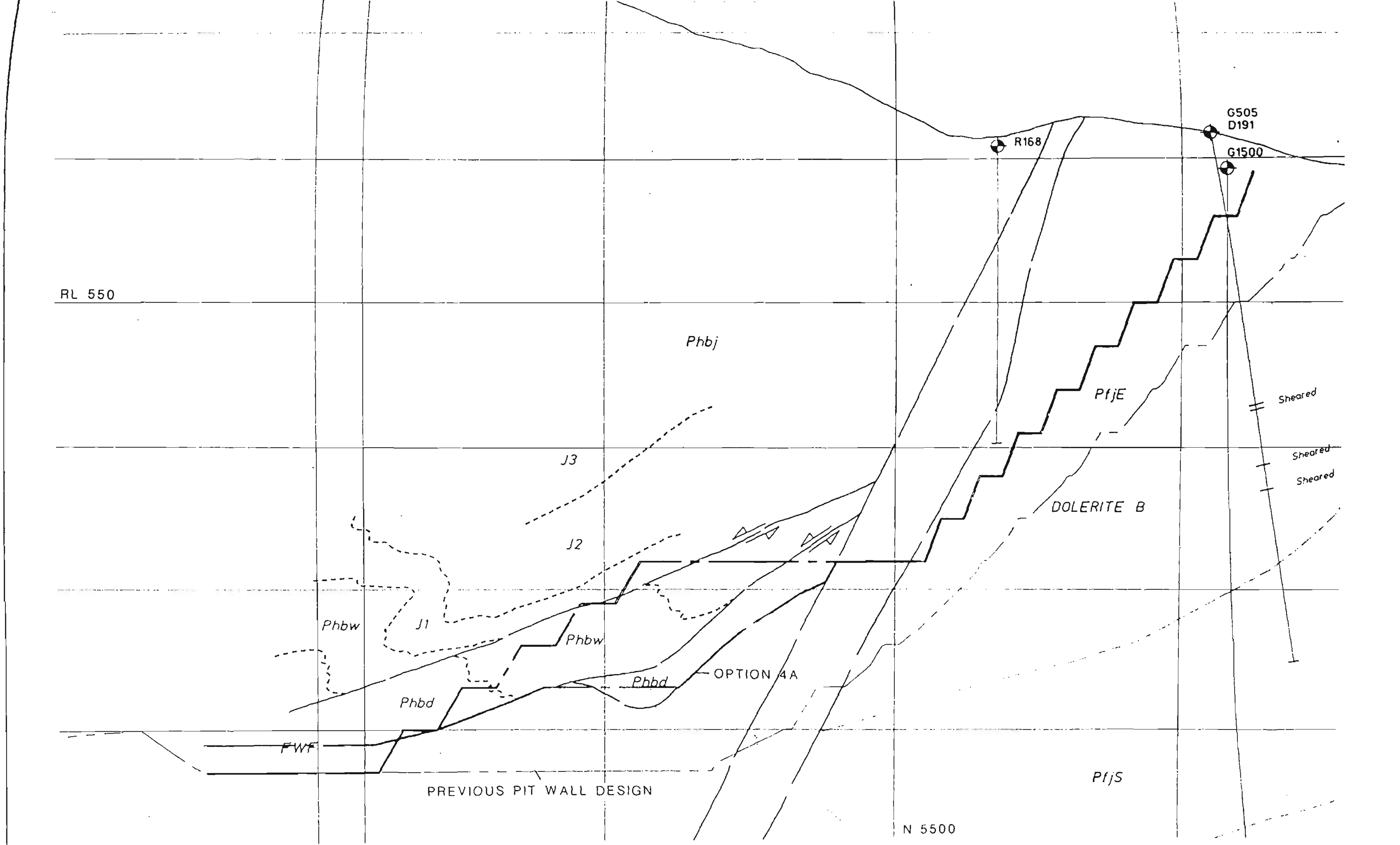
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 6720 E	FIGURE A7
Drn		
Dwg No.		



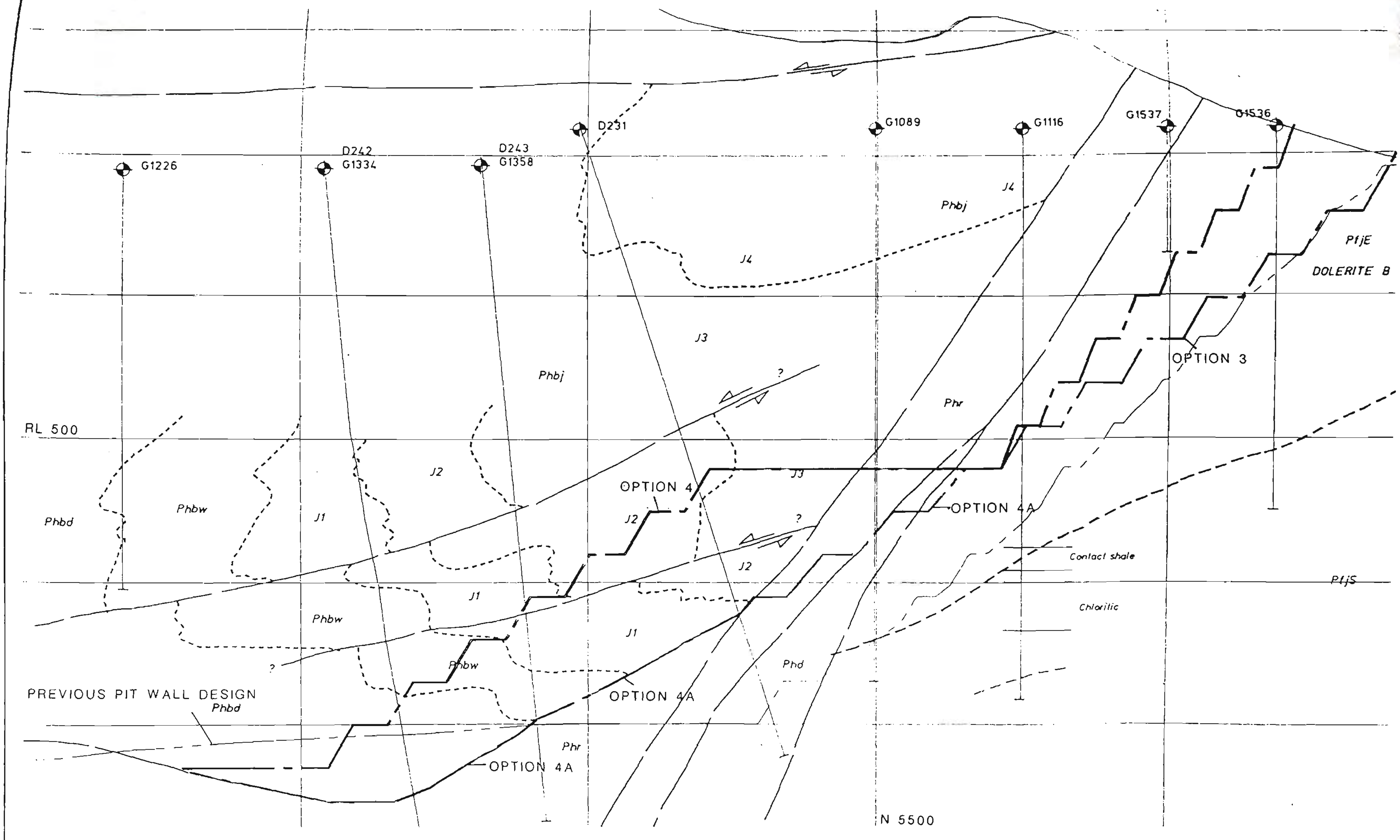
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 6880 E	FIGURE A8
Drn		
Dwg No.		



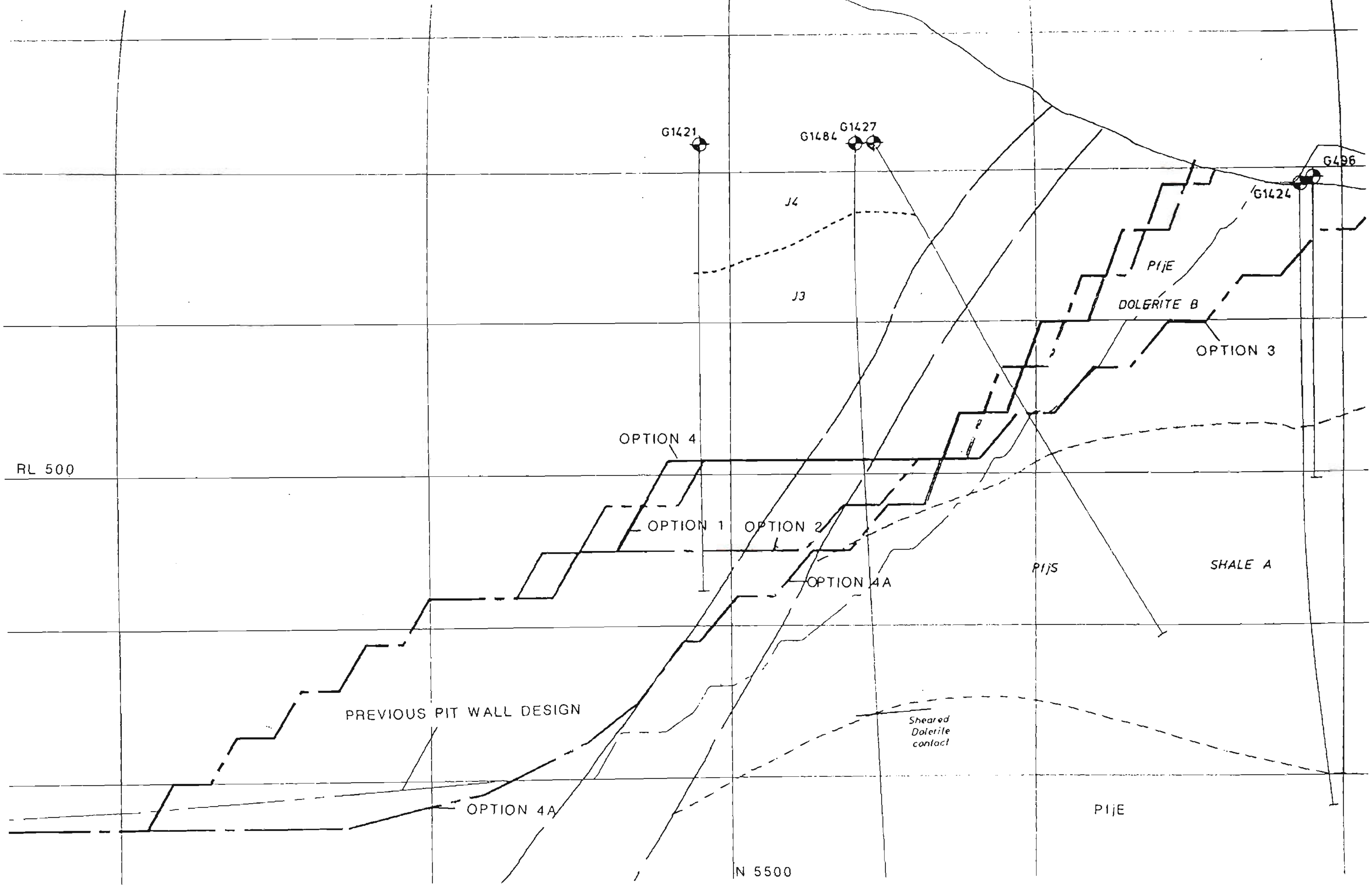
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7000 E	FIGURE A9
Drn		
Dwg No.		



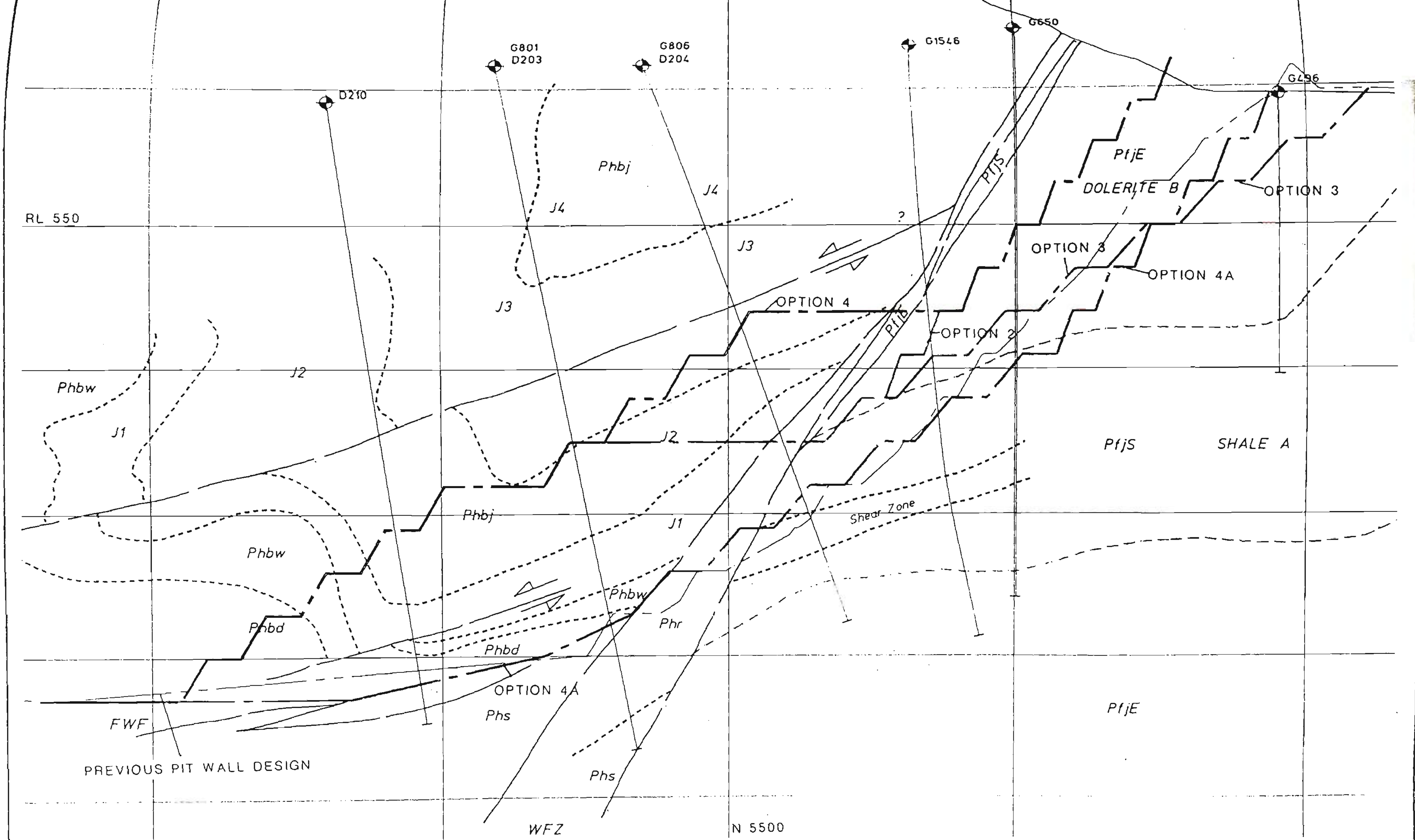
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7040 E	FIGURE A10
Drn		
Dwg No.		



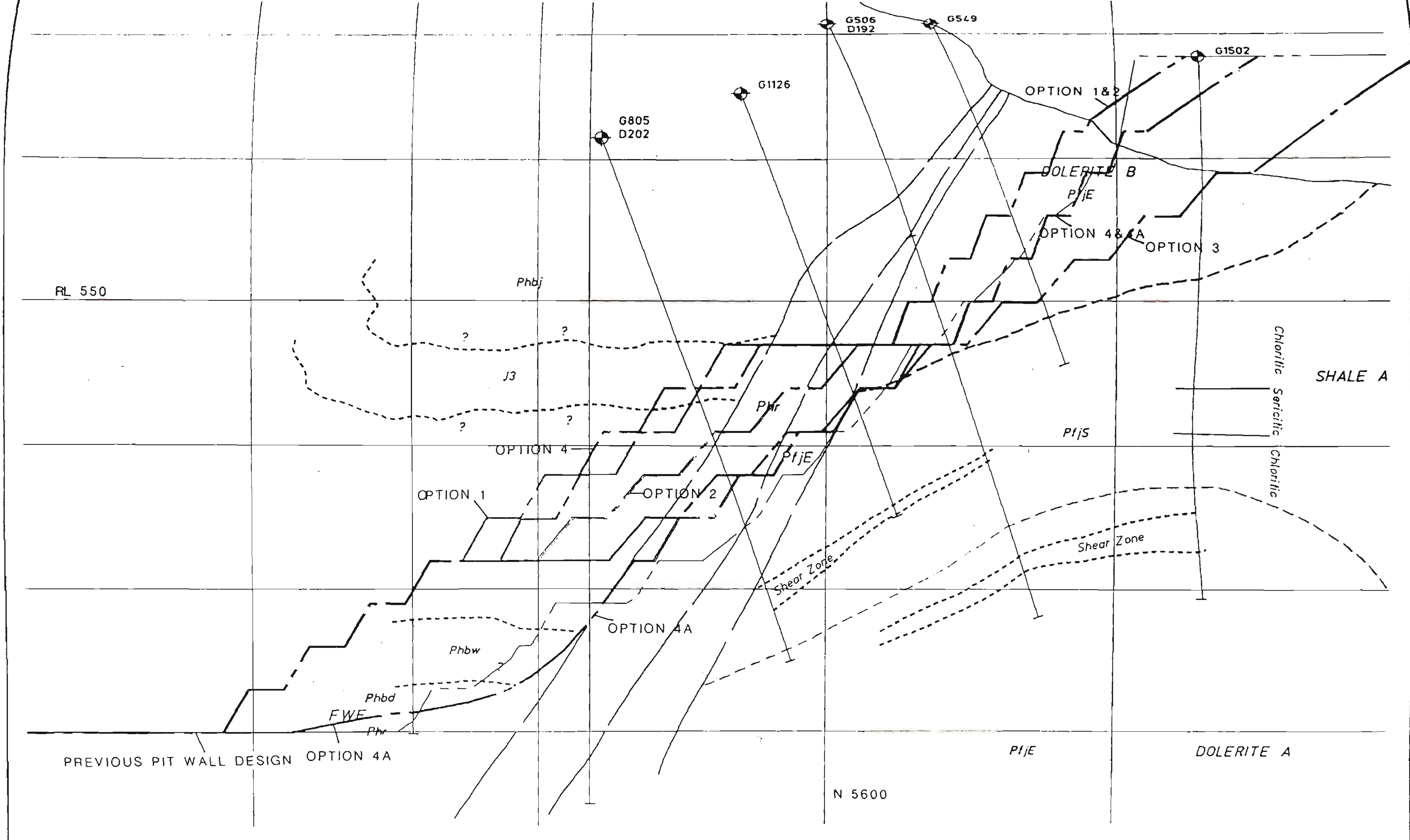
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7120 E	FIGURE A11
Drn		
Dwg No		



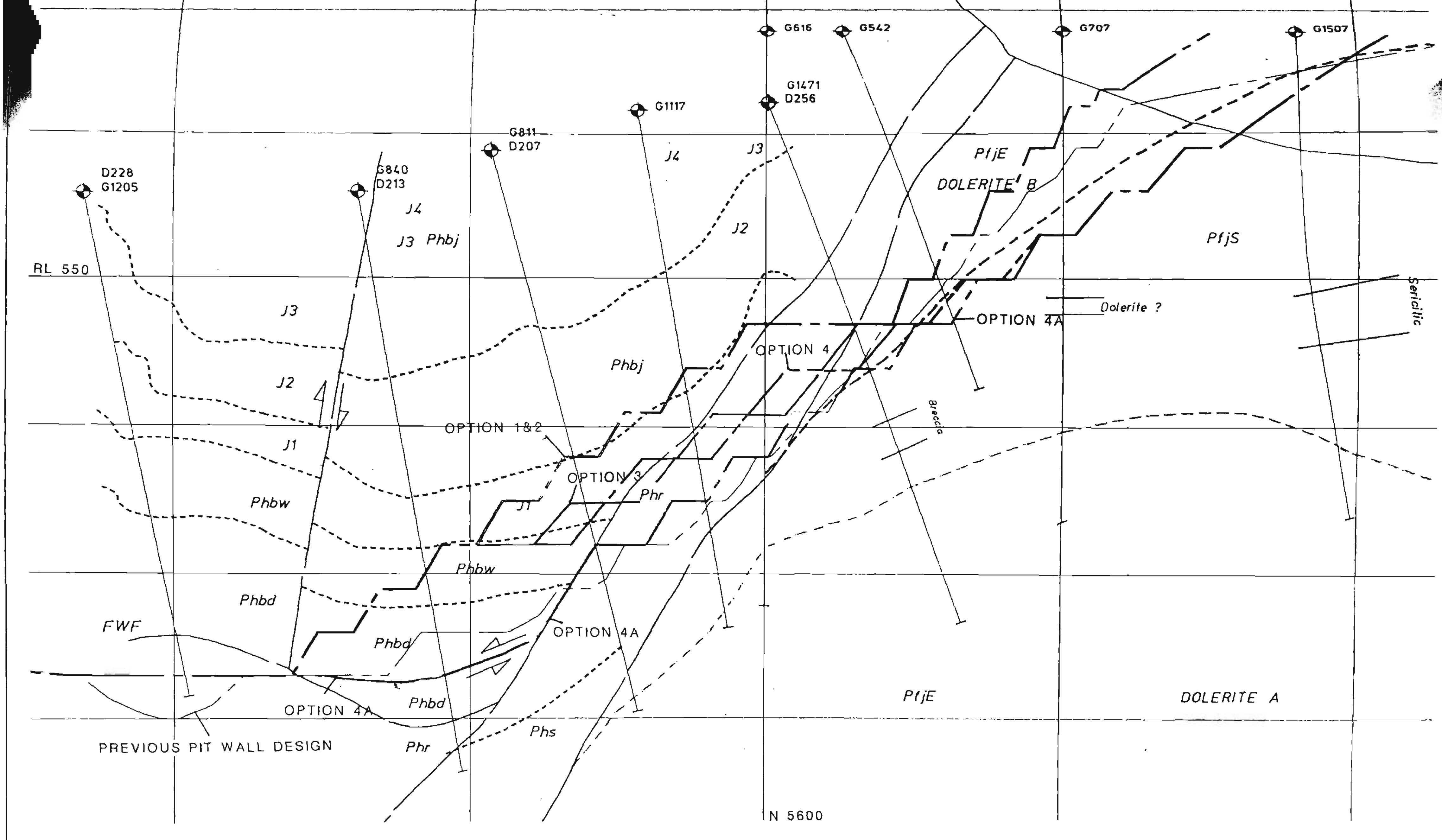
Scale	MT. WHALEBACK EAST PIT - NORTH WALL	FIGURE	
Drn	CROSS SECTION 7160 E	A12	
Dwg No.			



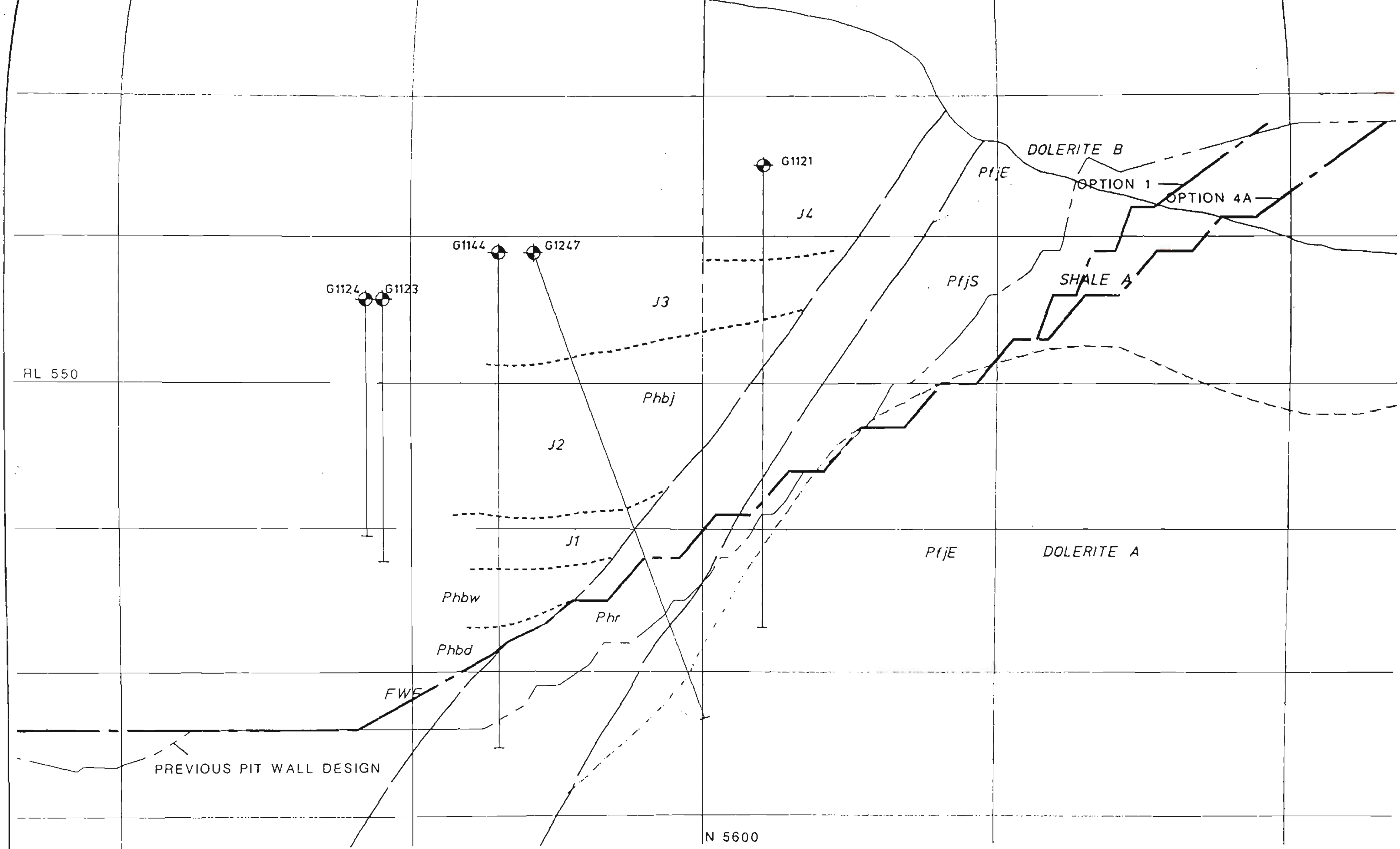
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7200 E	FIGURE A13
Drn		
Dwg No.		



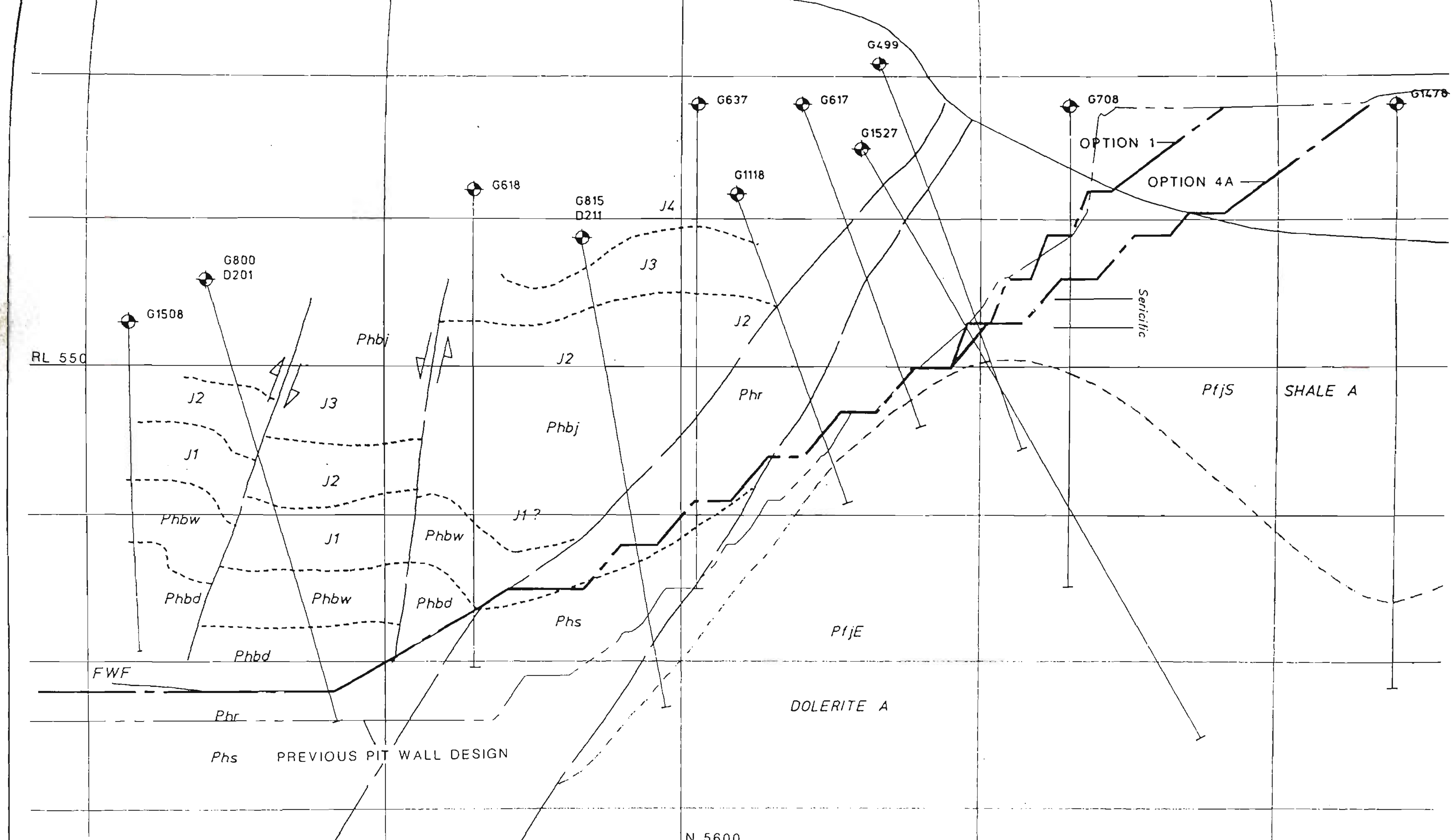
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7320 E	FIGURE A14
Drn		
Dwg No		



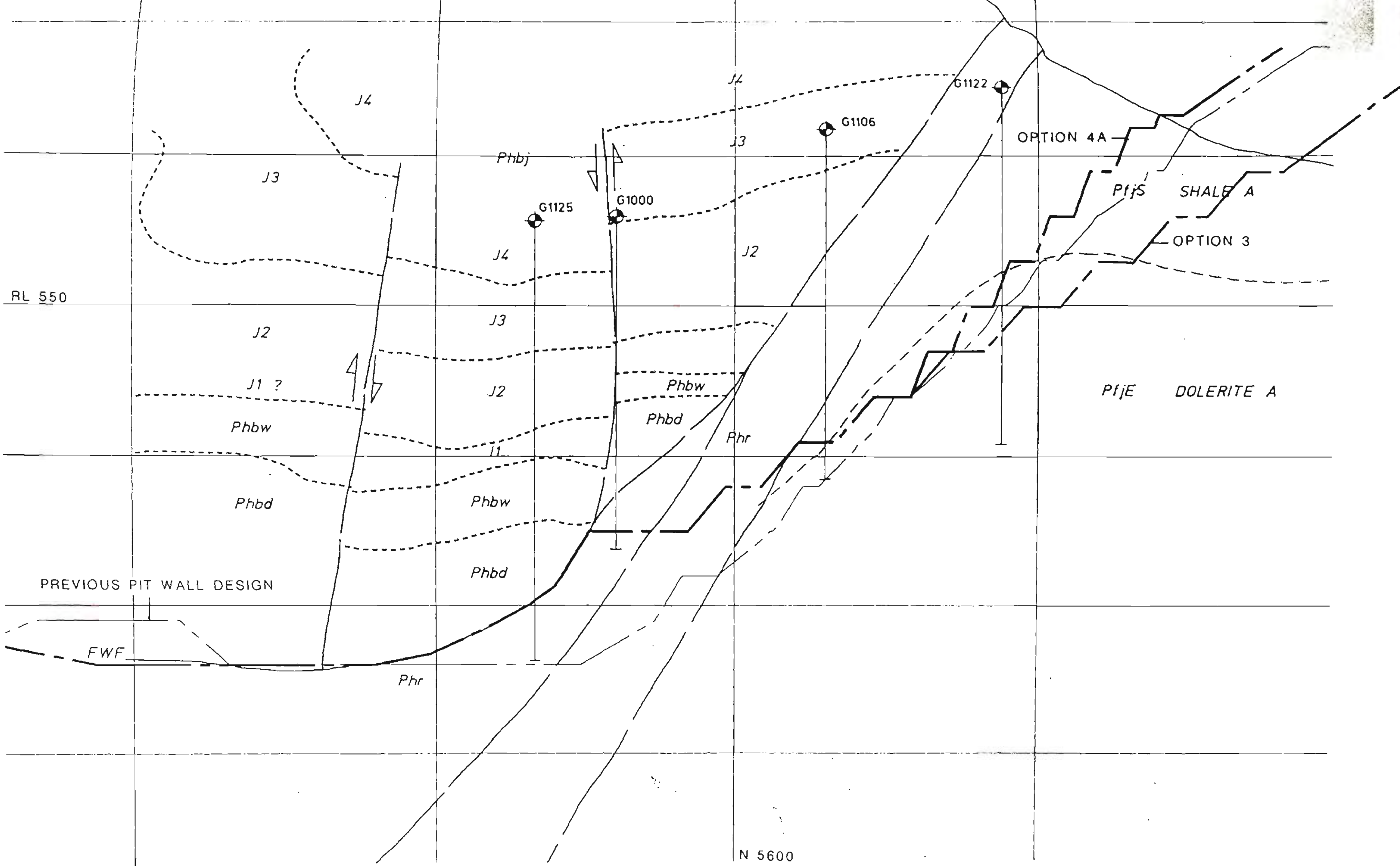
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7400 E	FIGURE A15
Drn		
Dwg No		



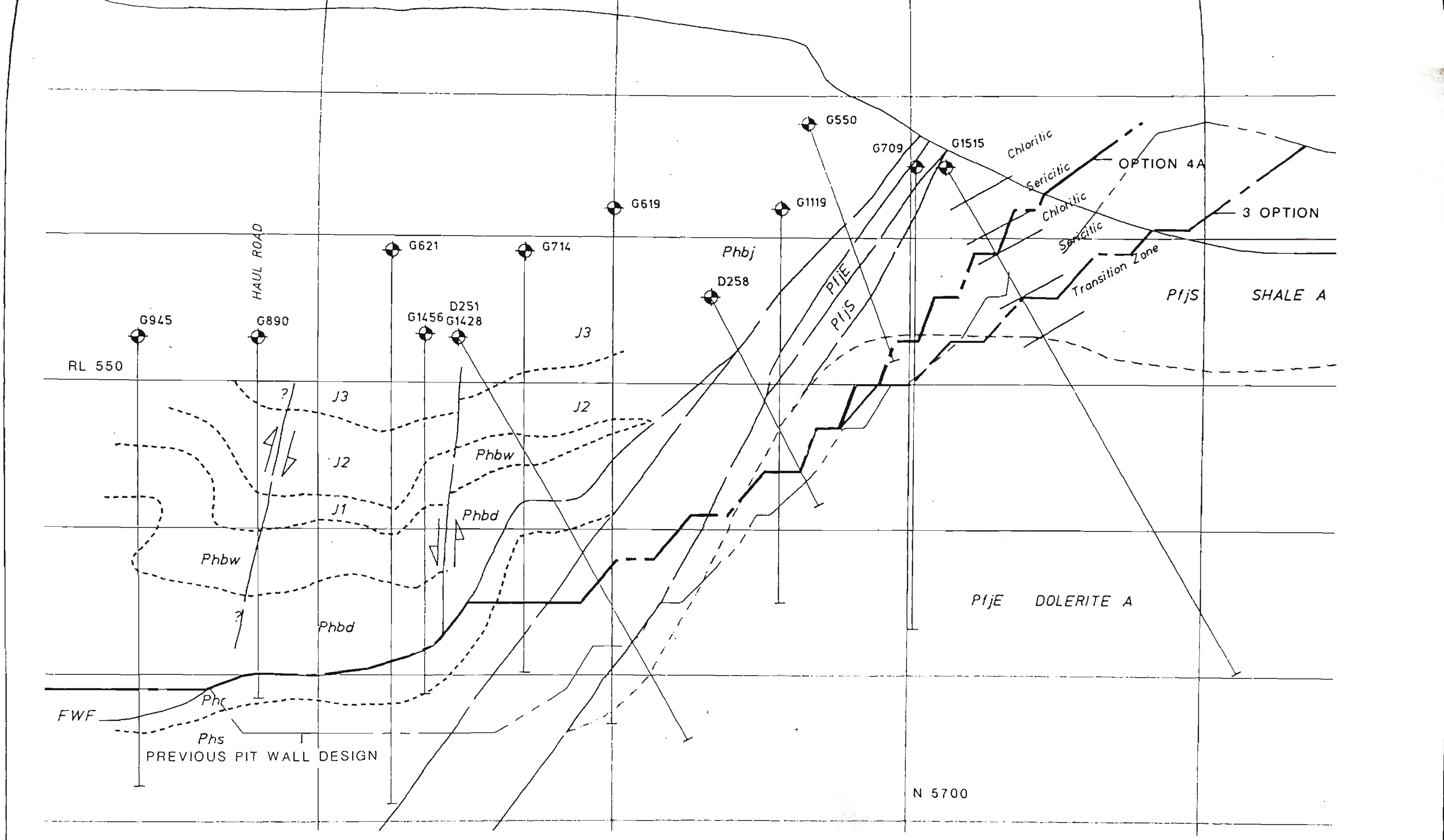
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7440 E	FIGURE A16
Drn		
Dwg No.		



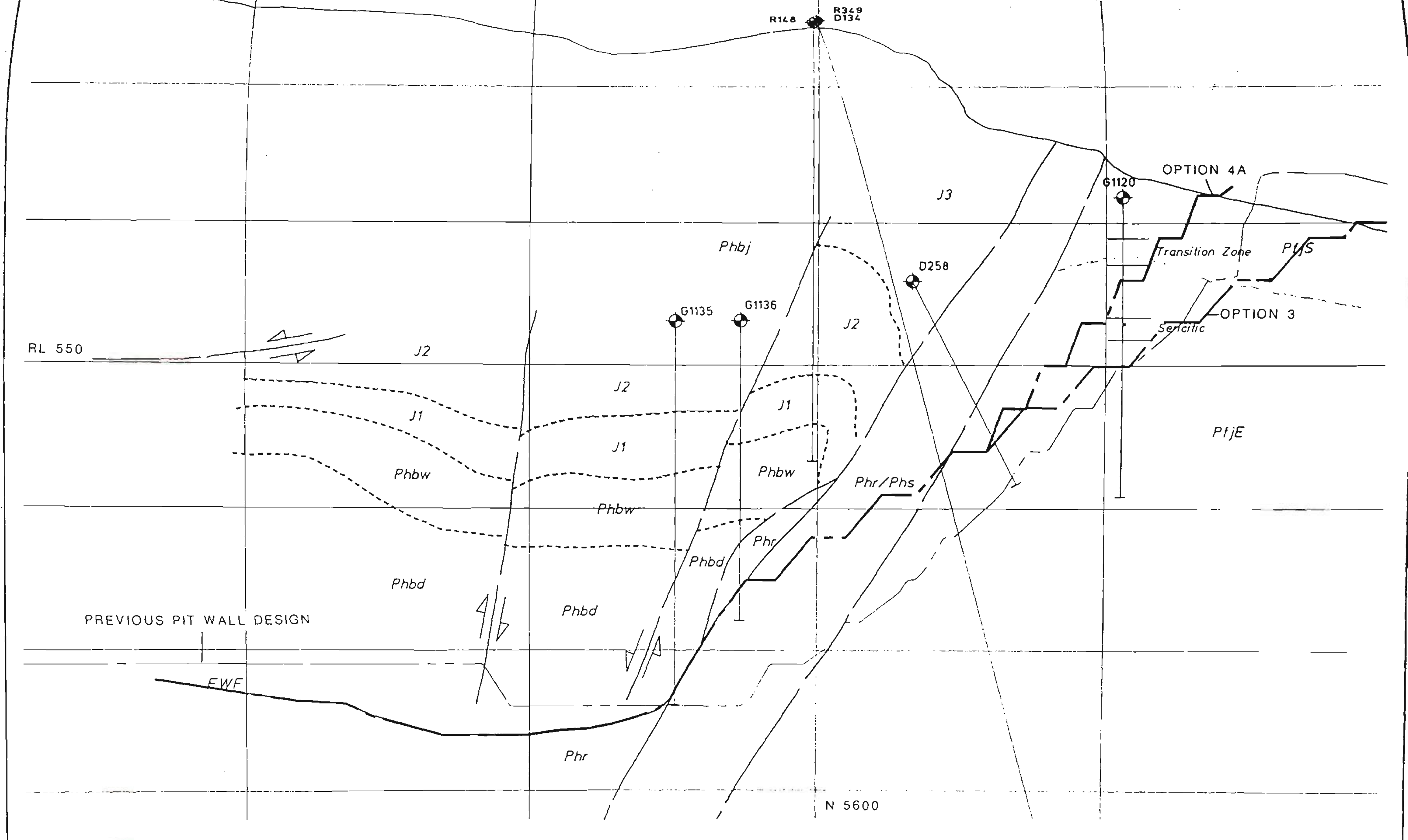
	Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7480 E	FIGURE A17
	Drn		
	Dwg No.		



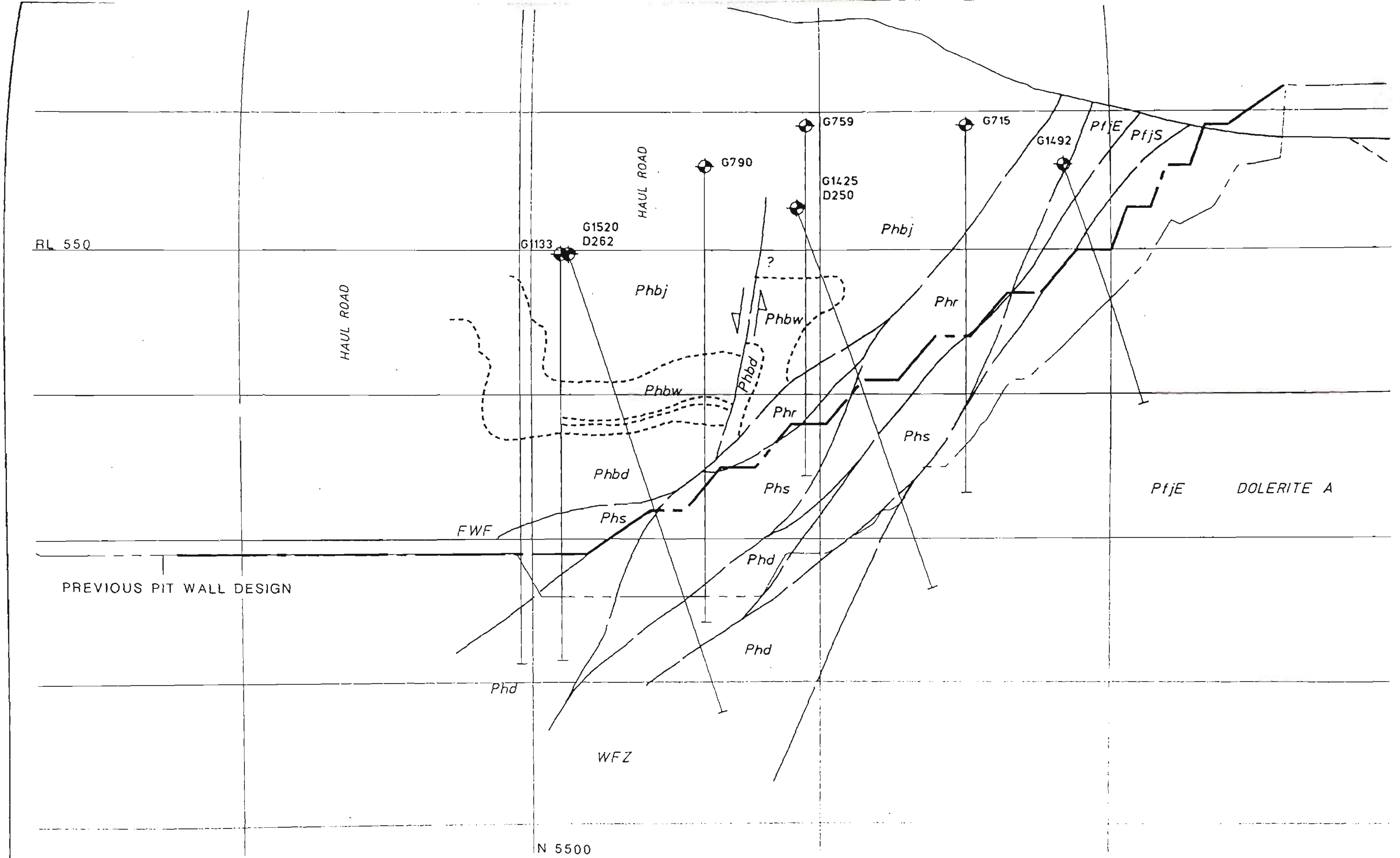
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7520 E	FIGURE A18
Drn		
Dwg No.		



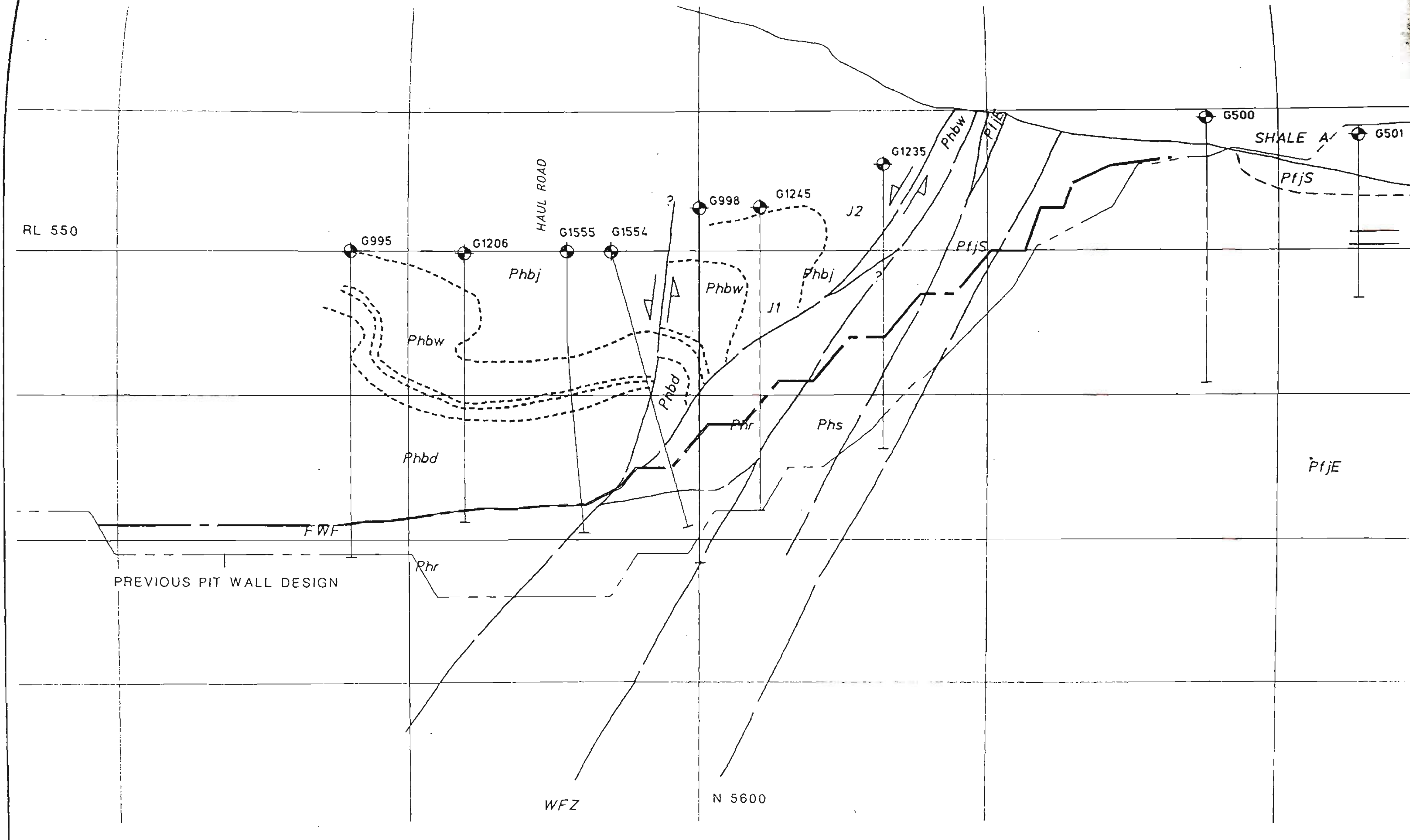
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7560 E	FIGURE A19
Drn		
Dwg No		



Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7600 E	FIGURE A20
Drn		
Dwg No.		



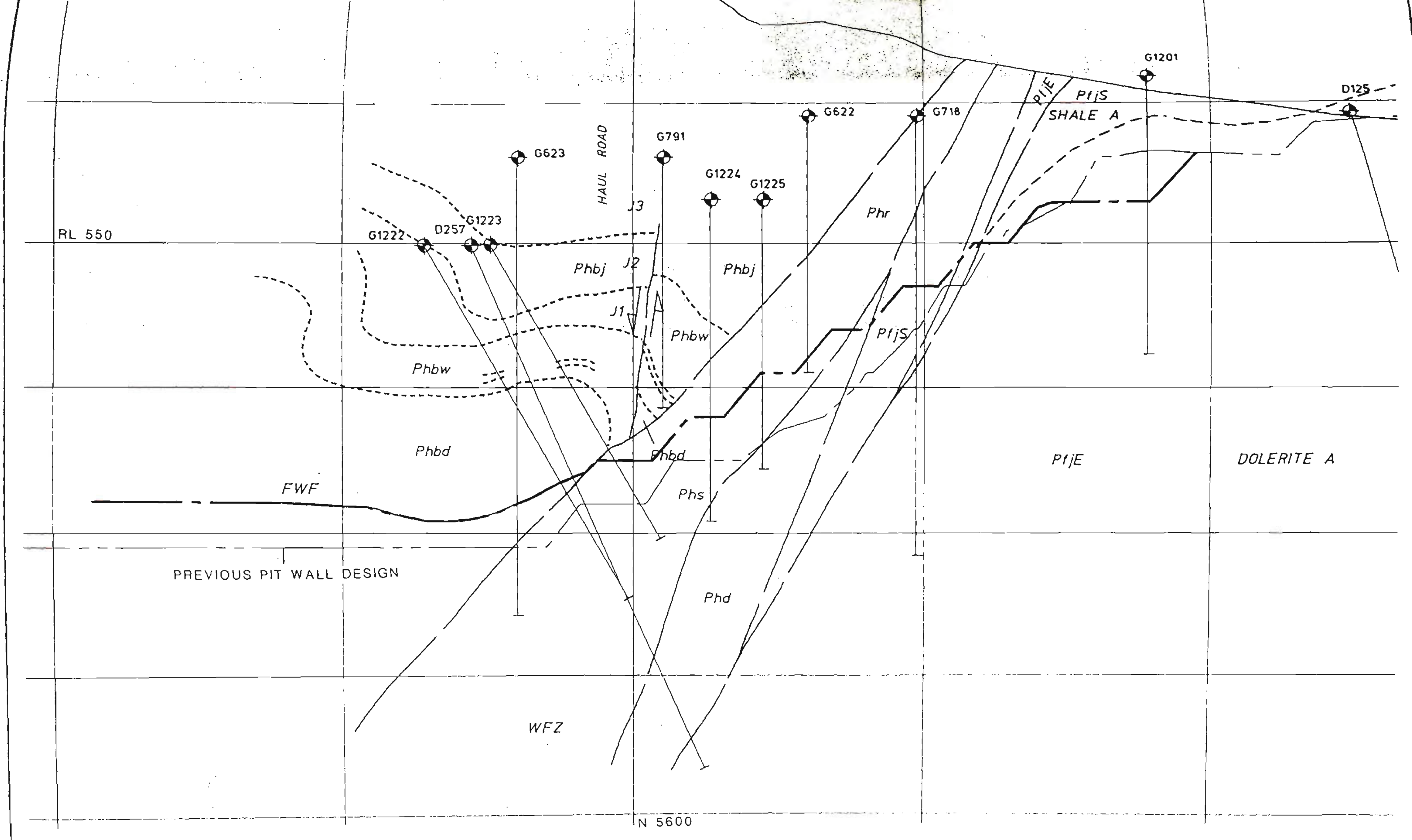
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7640 E	FIGURE A21
Drn		
Dwg No		



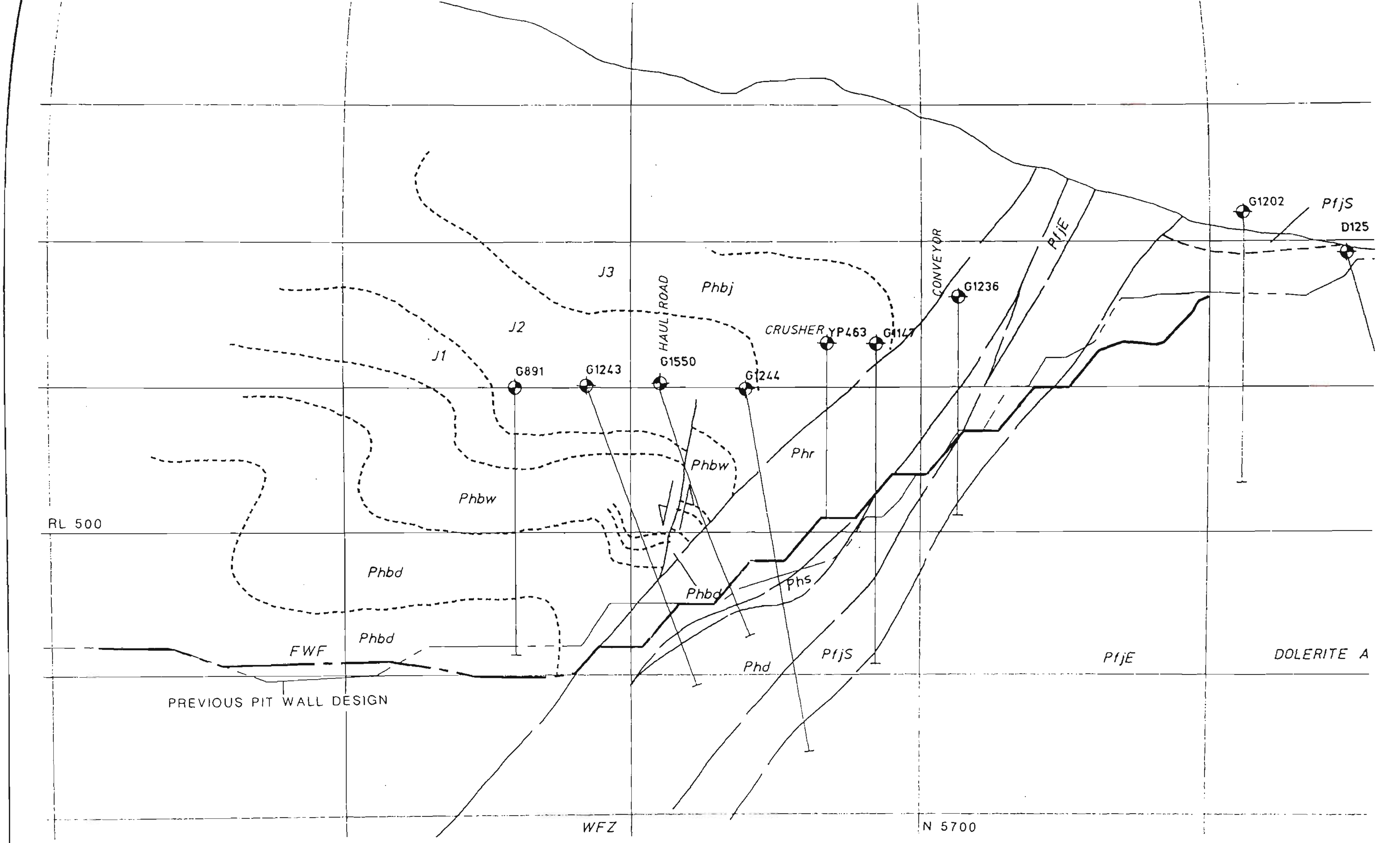
Scale	
Drn	
Dwg No.	

MT. WHALEBACK EAST PIT - NORTH WALL
CROSS SECTION 7680 E

FIGURE
A22



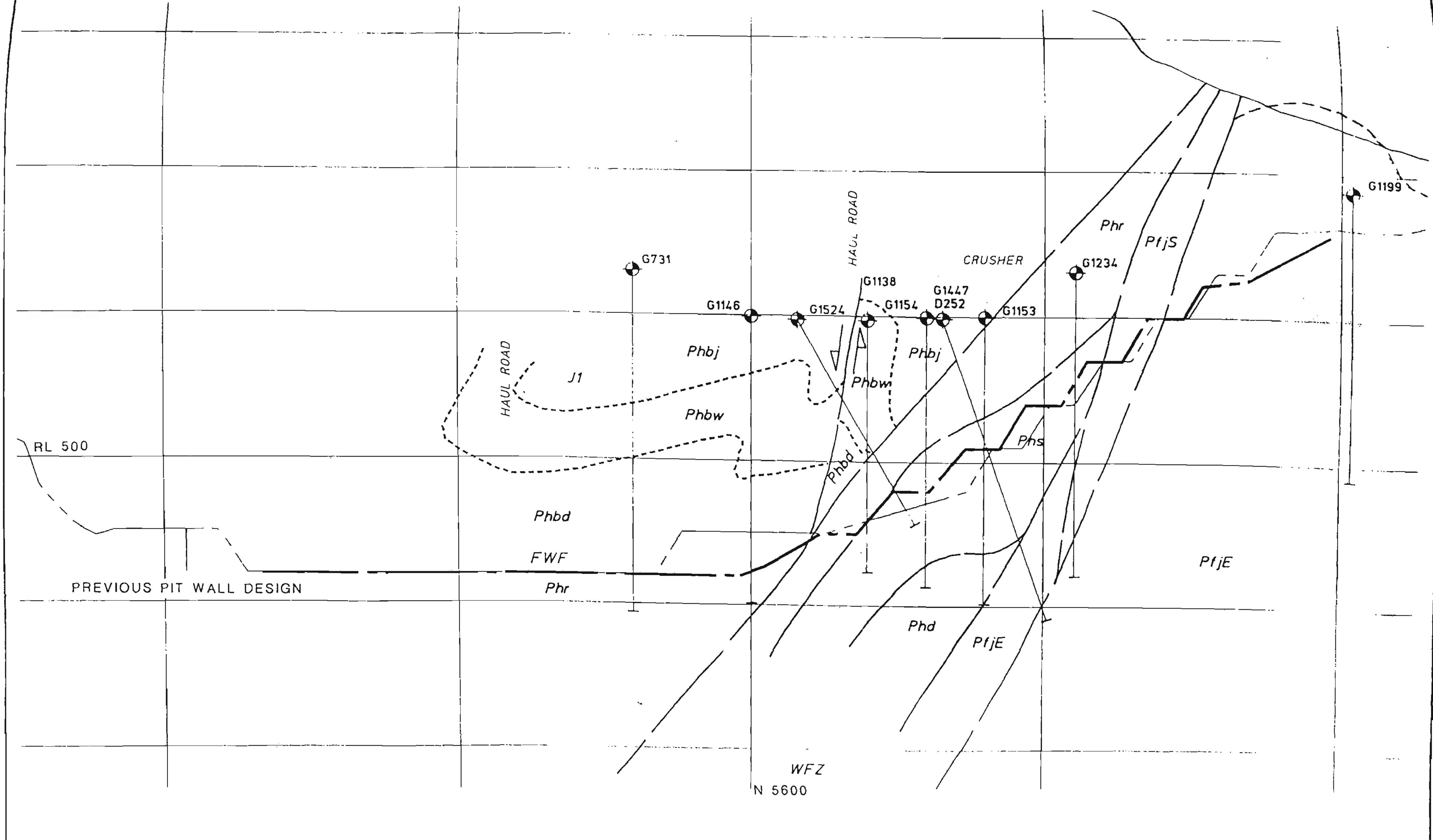
	Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7720 E	FIGURE A23
	Drn		
	Dwg No.		



Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7760 E
Drn	
Dwg No	

FIGURE

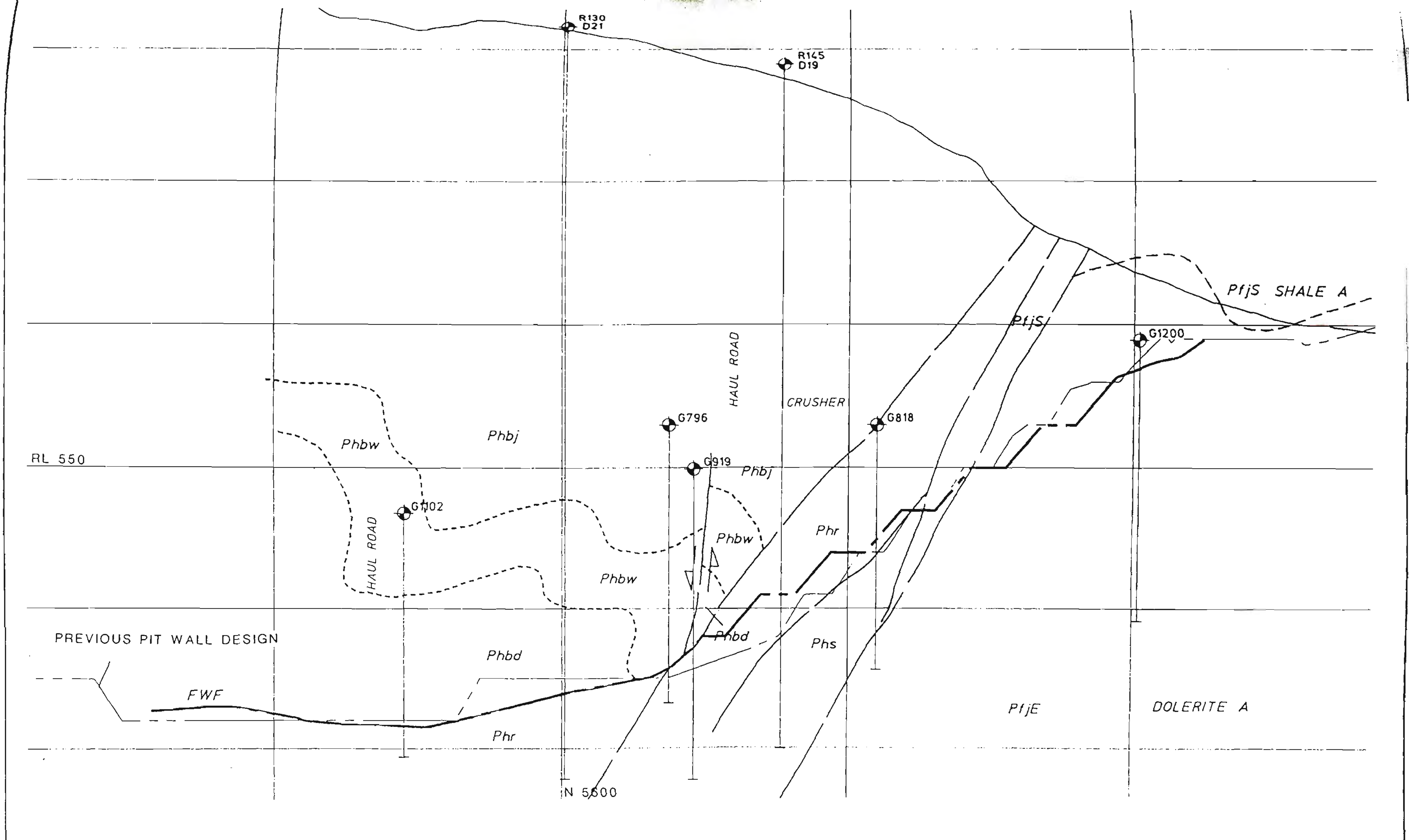
A24



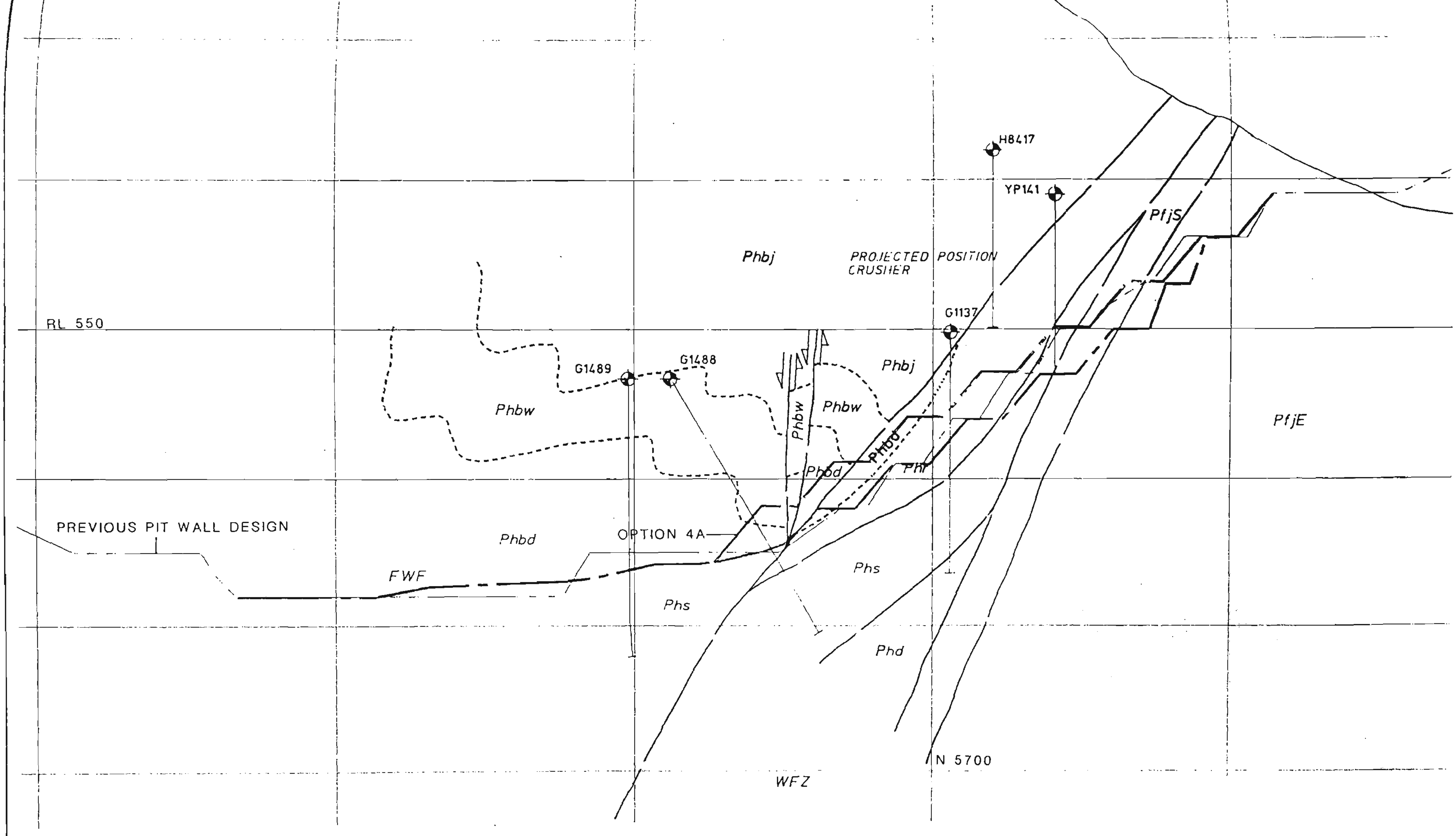
Scale	
Drn	
Dwg No.	

MT. WHALEBACK EAST PIT - NORTH WALL
CROSS SECTION 7800 E

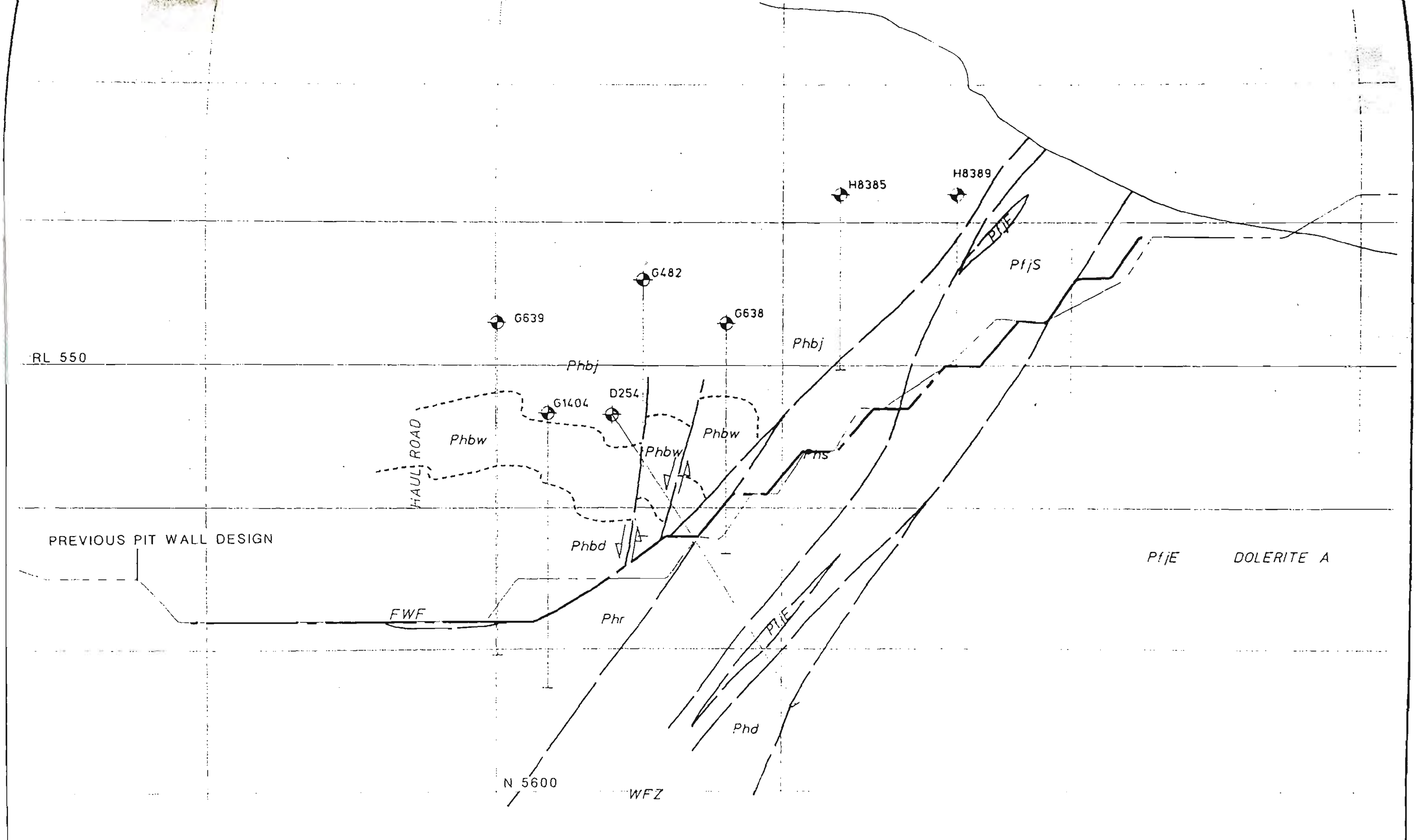
FIGURE
A25



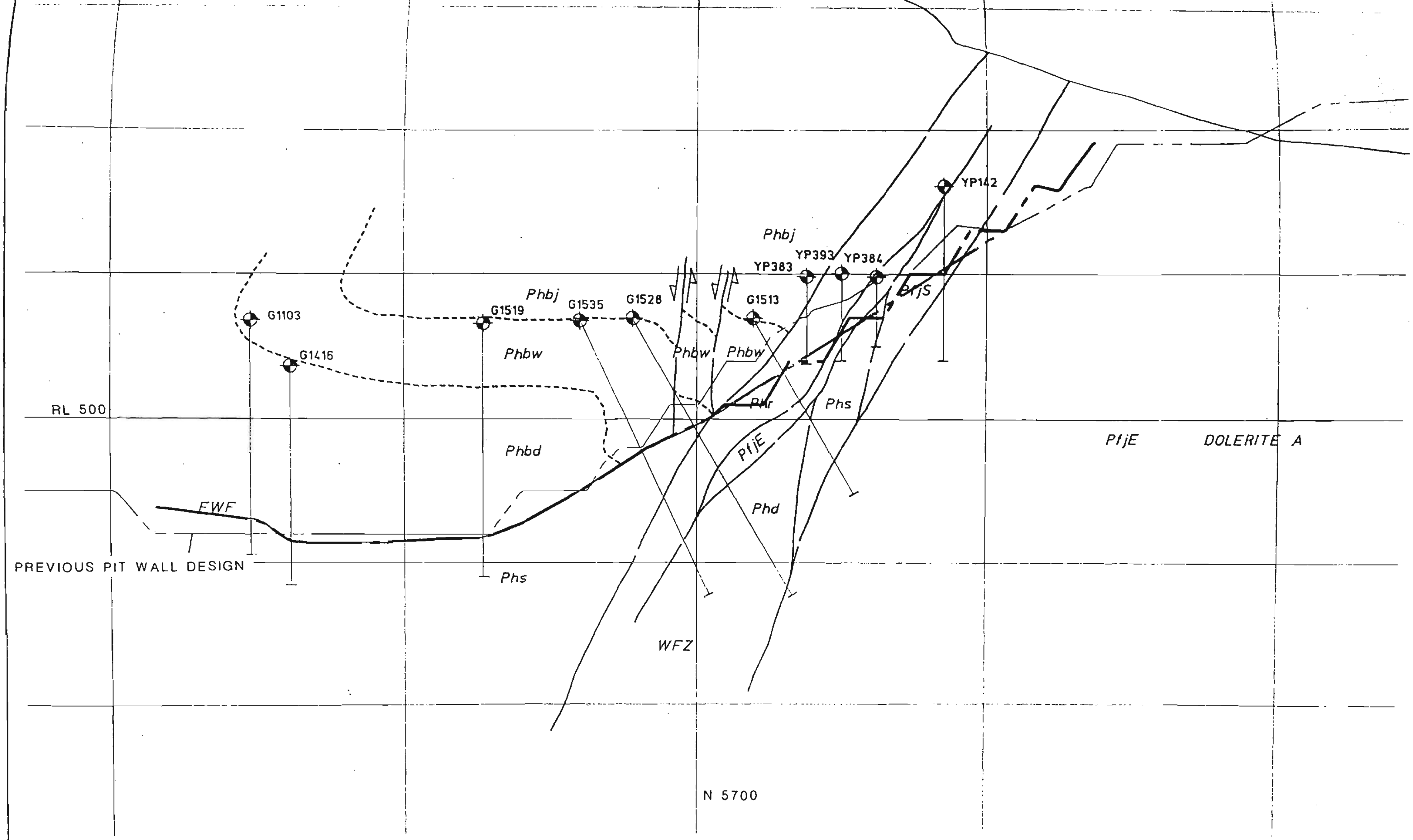
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7840 E	FIGURE A26
Drn		
Dwg No.		



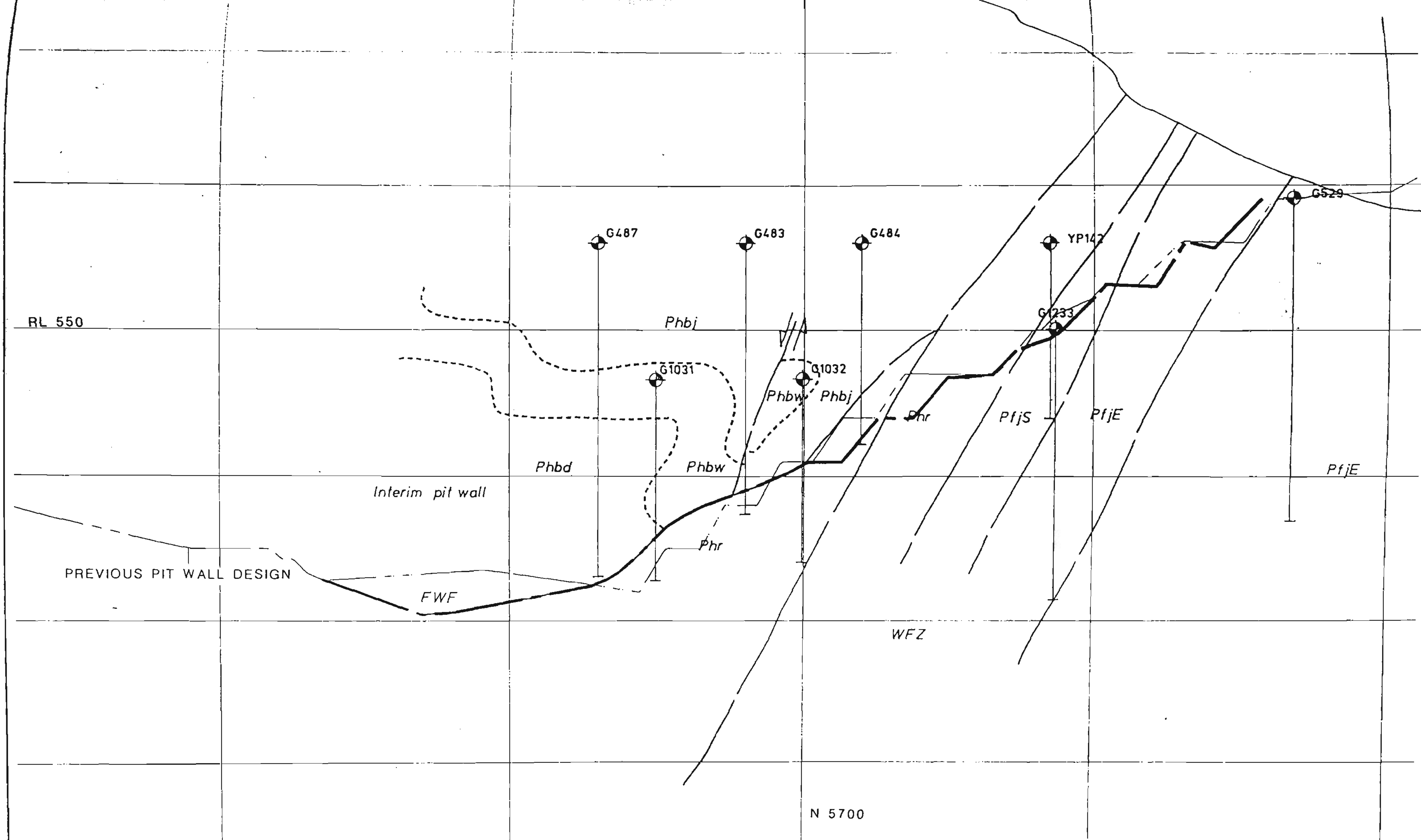
Scale	MT. WHALEBACK EAST PIT - NORTH WALL	FIGURE
Drn	CROSS SECTION 7880 E	A27
Dwg No		



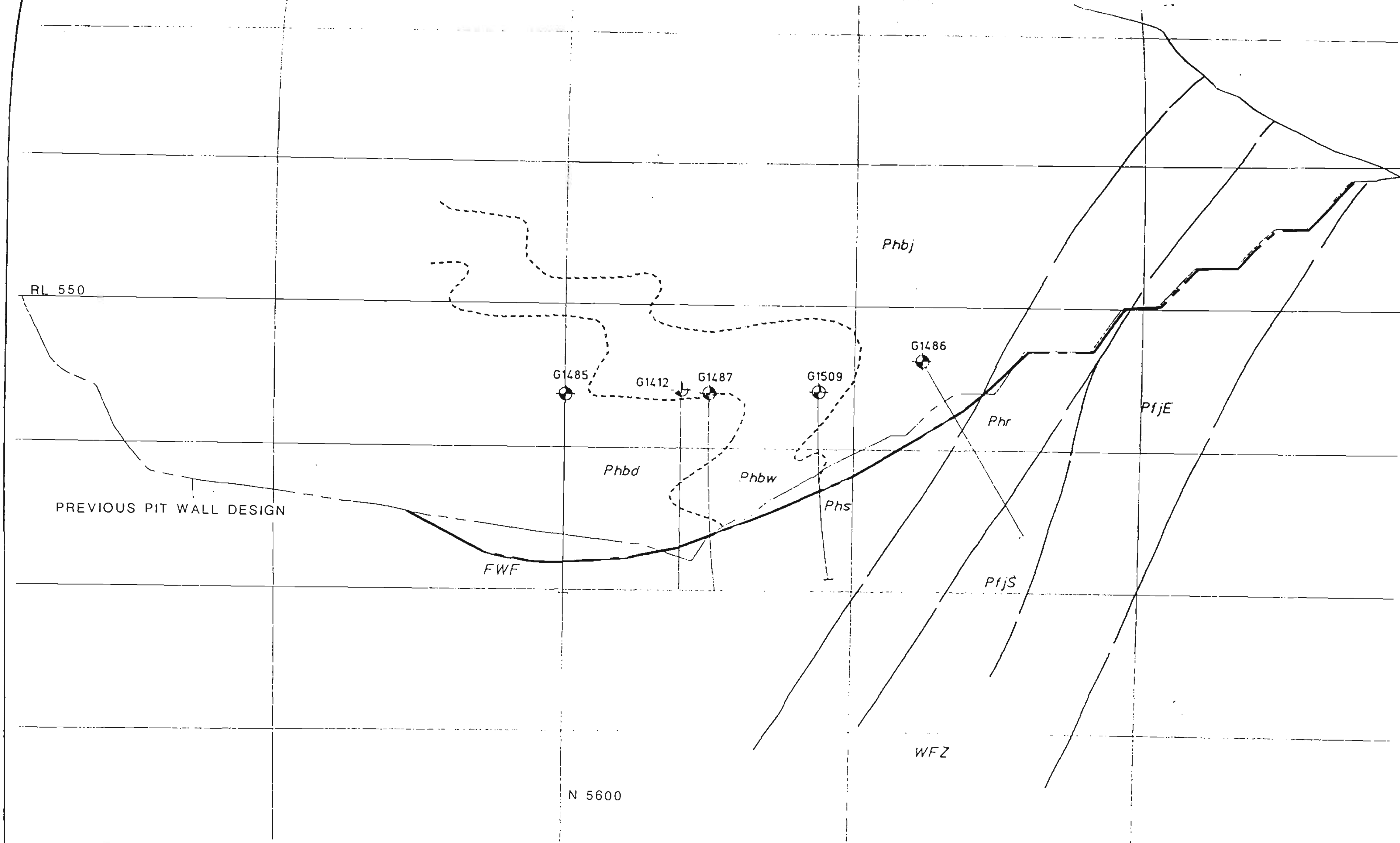
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7920 E	FIGURE A28
Drn		
Dwg No.		



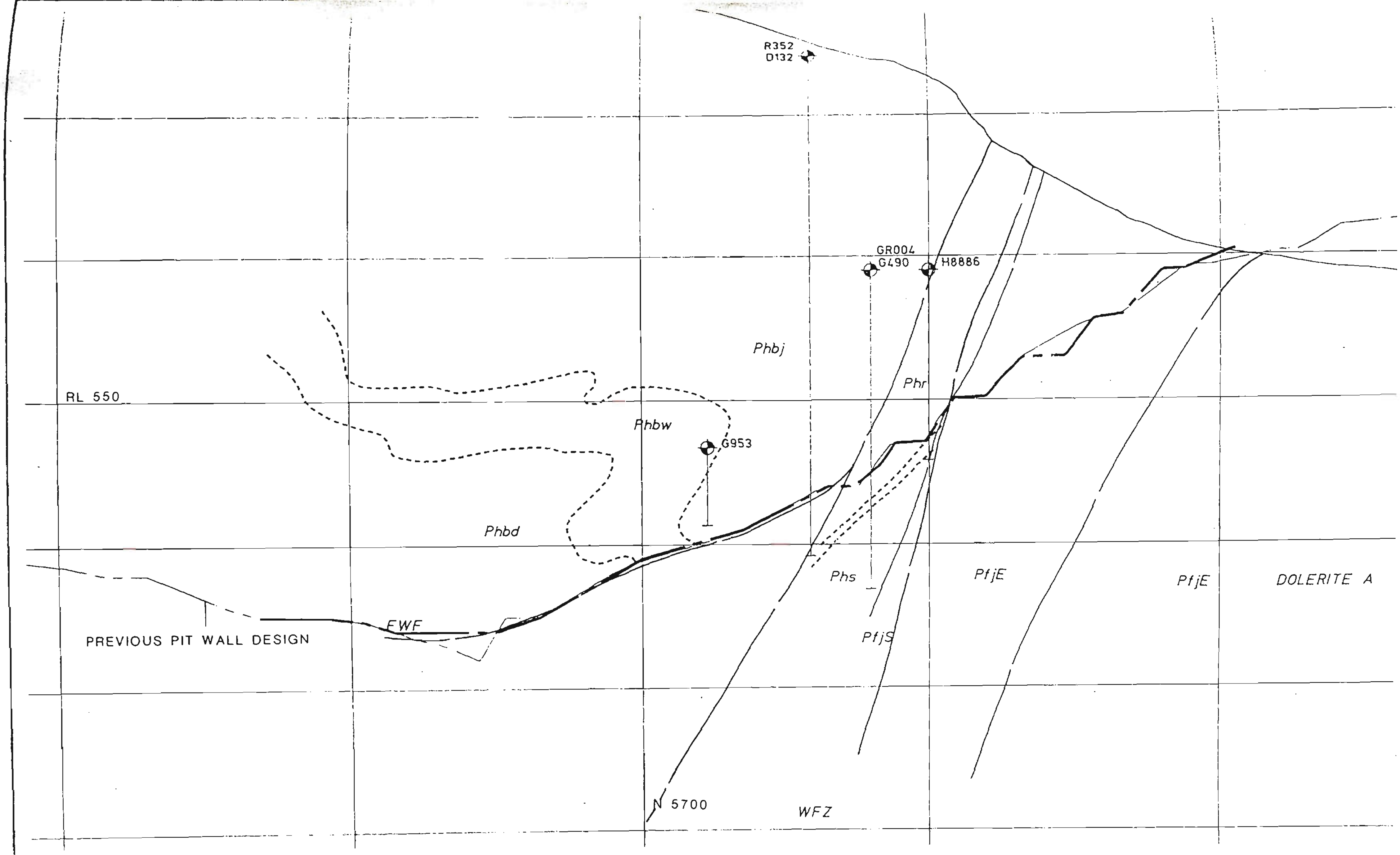
Scale		MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 7960 E	FIGURE A29
Drn			
Dwg No.			



	Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 8000 E	FIGURE A30
	Drn		
	Dwg No.		

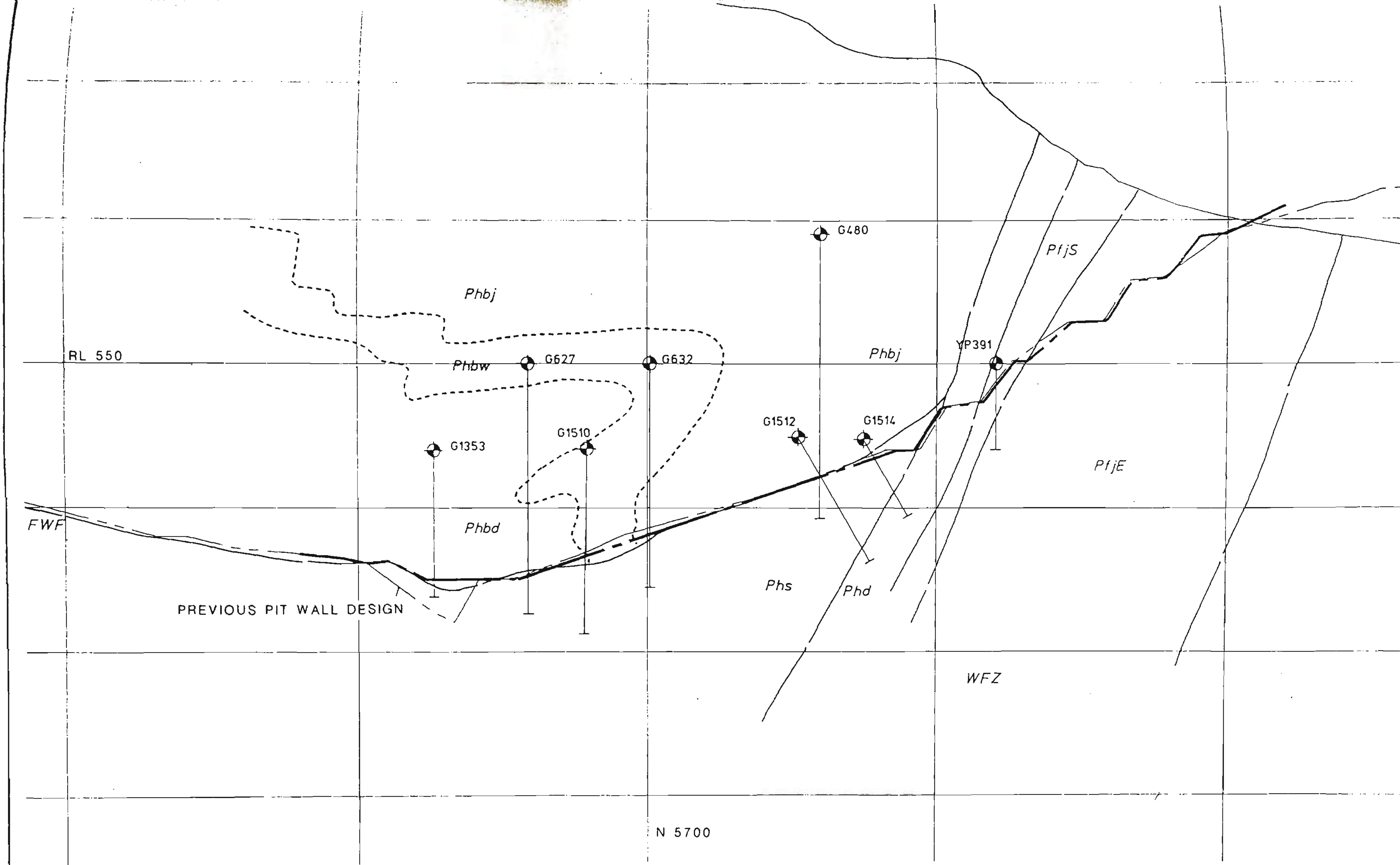


Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 8040 E	FIGURE A31
Drn		
Dwg No		

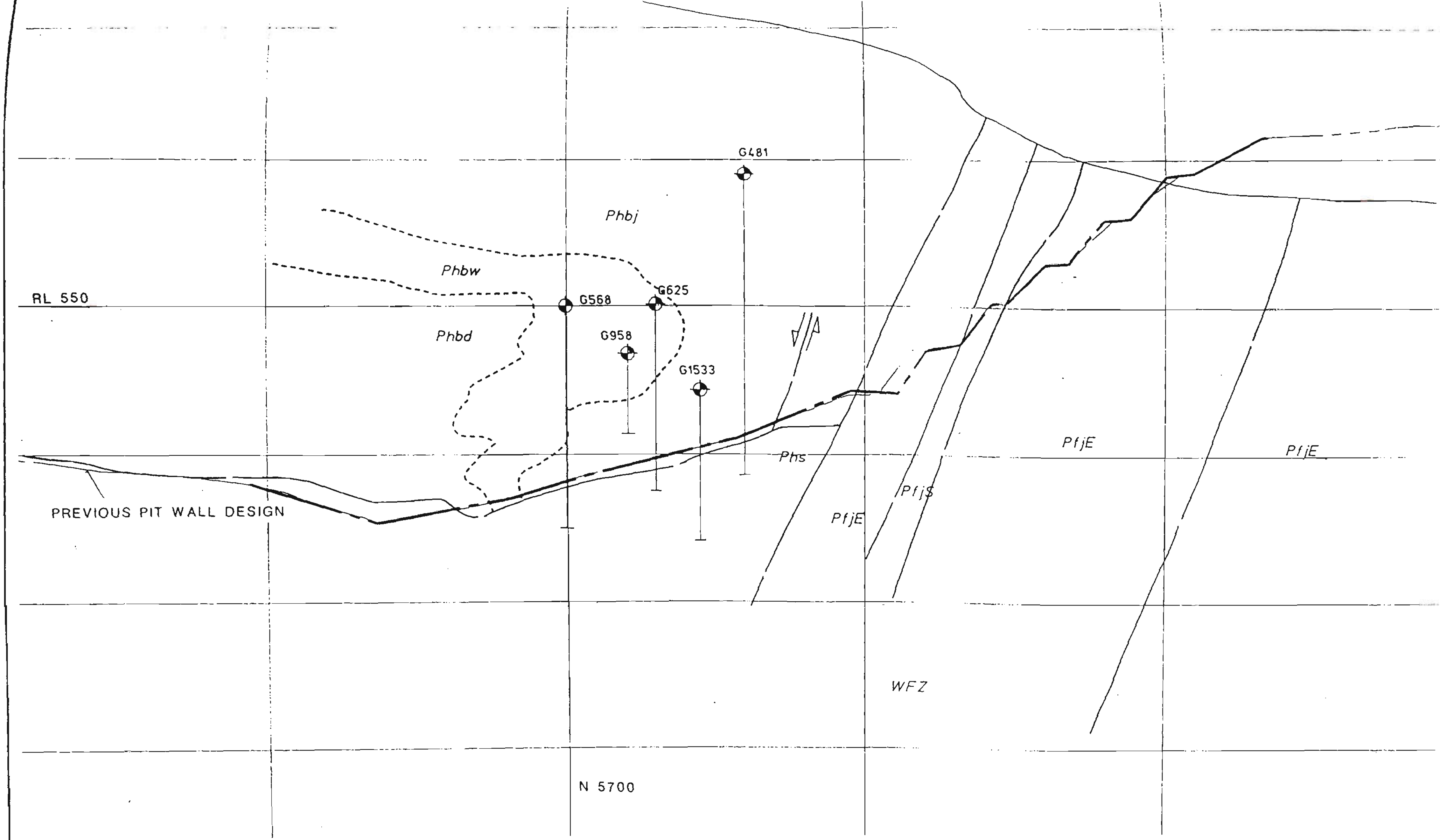


Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 8080 E
Drn	
Dwg No.	

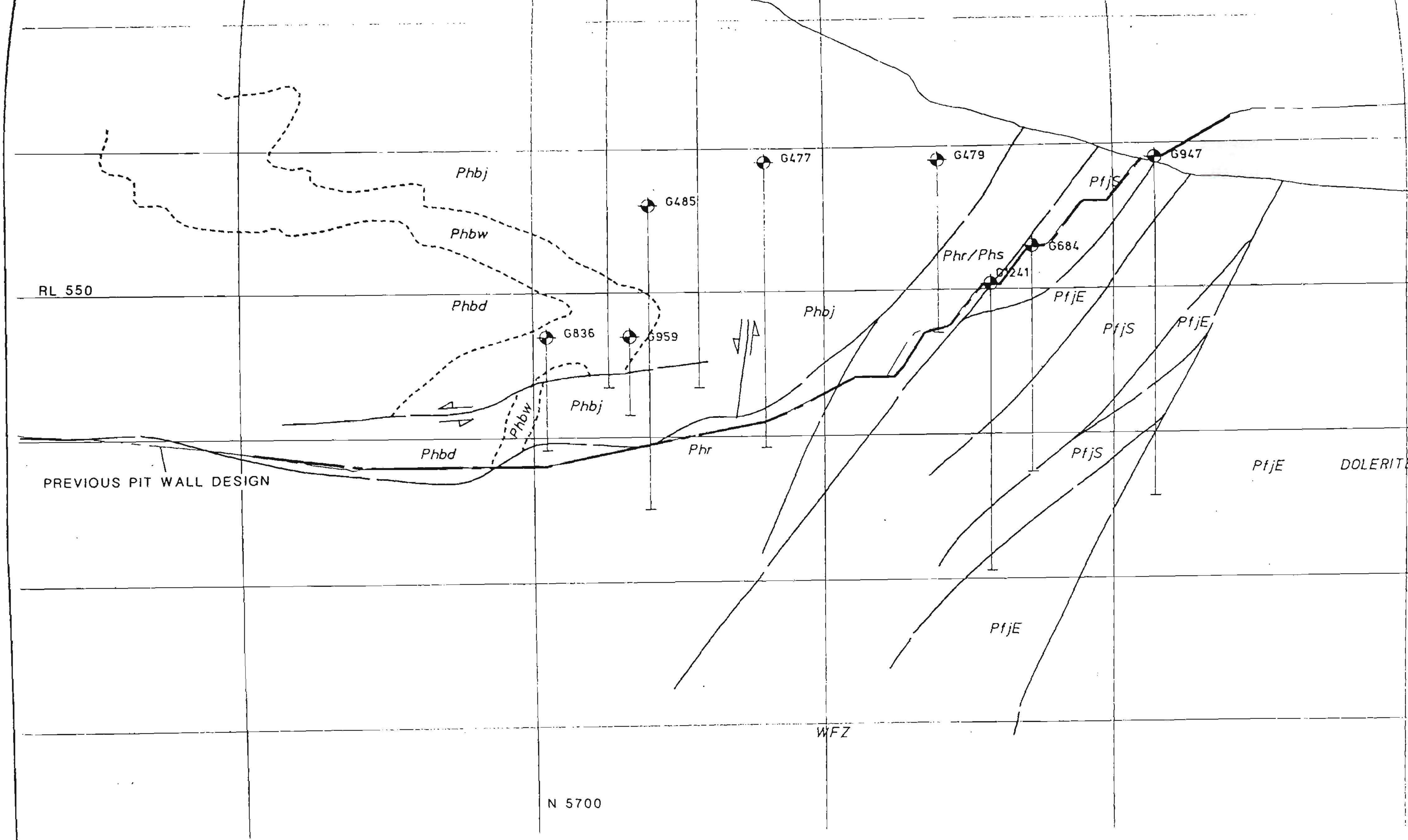
FIGURE
A32



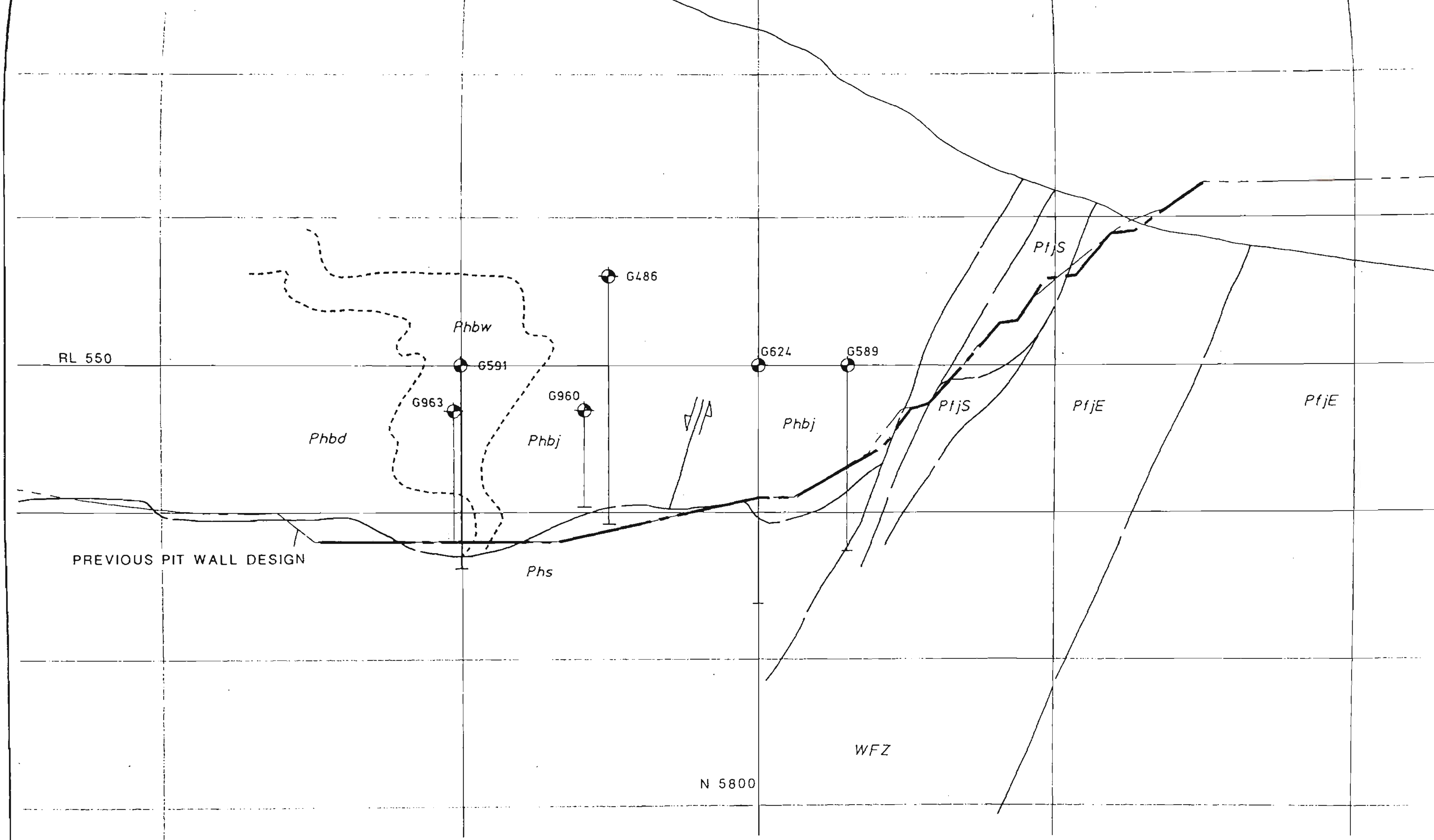
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 8120 E	FIGURE A33
Drn		
Dwg No.		



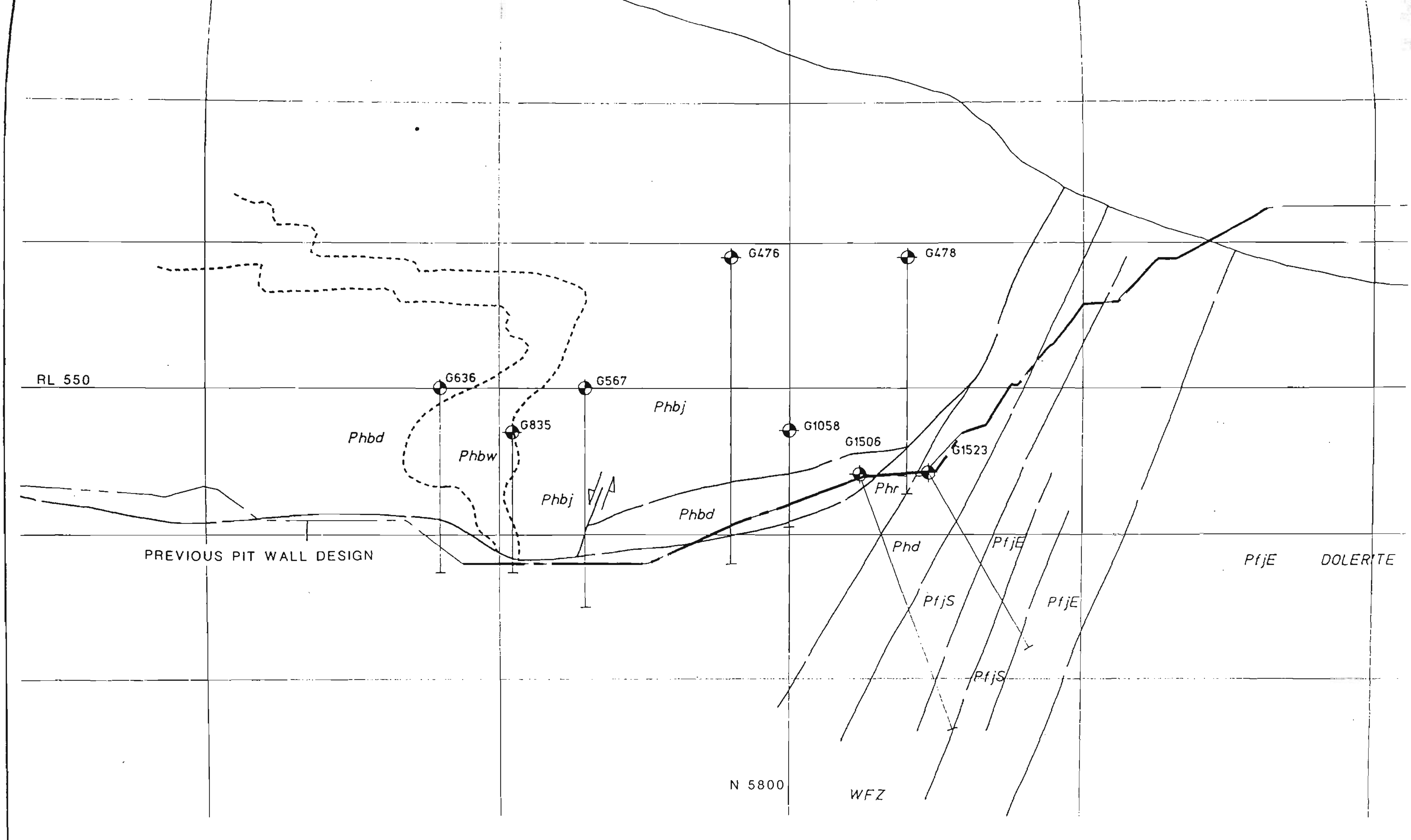
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 8160 E	FIGURE A34
Drn		
Dwg No		



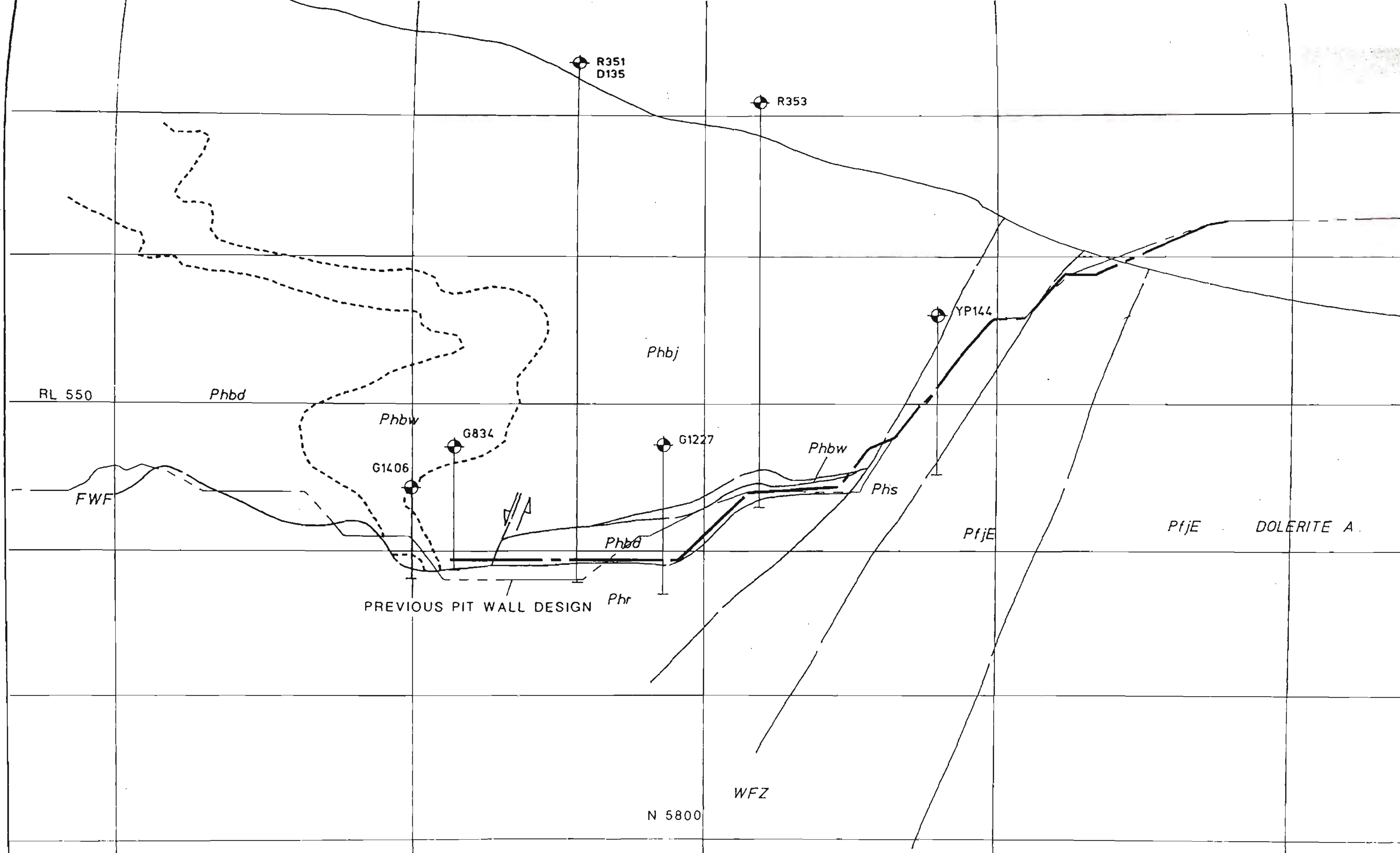
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 8200 E	FIGURE A35
Drn		
Dwg No		



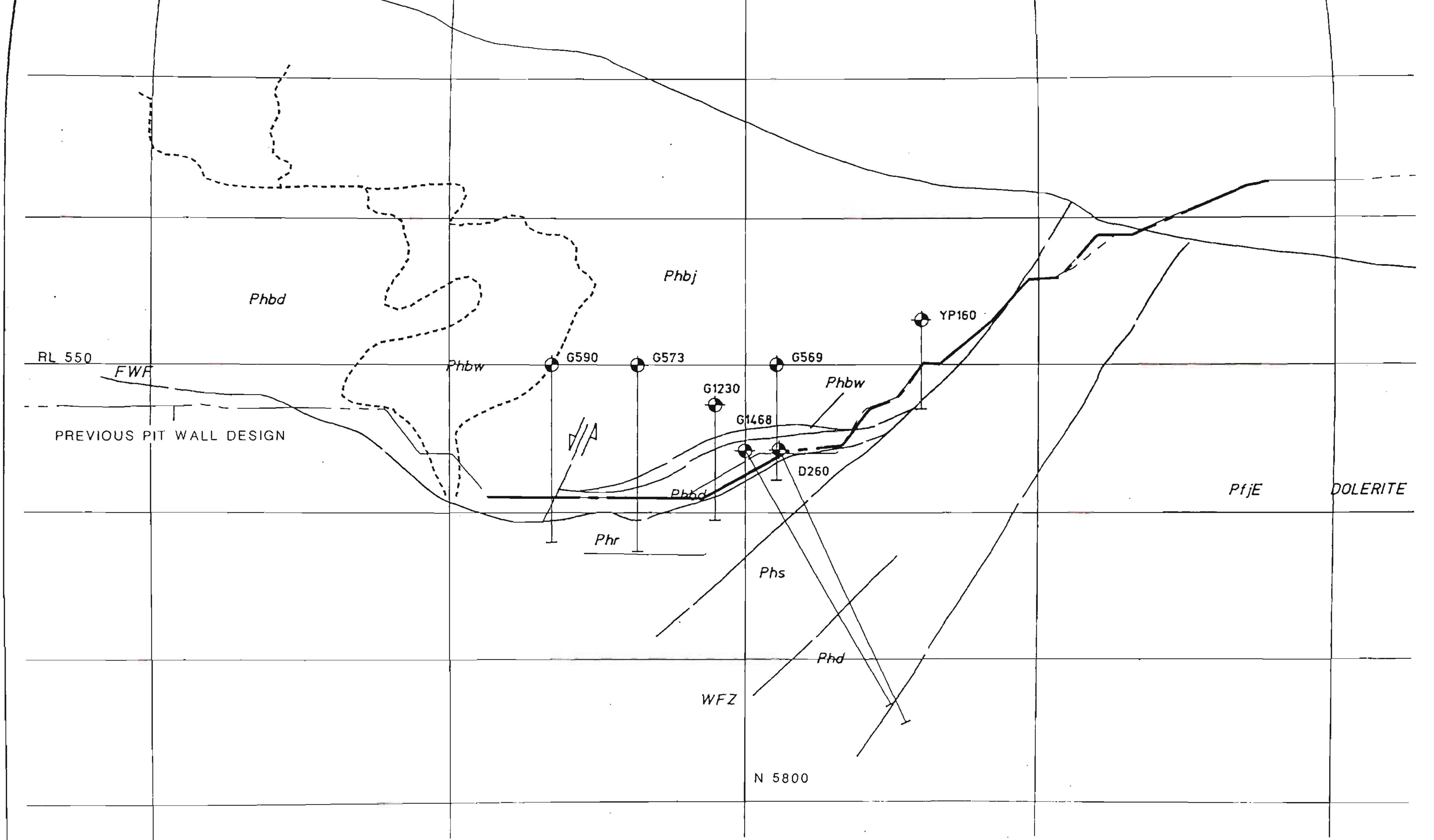
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 8240 E	FIGURE A36
Drn		
Dwg No		



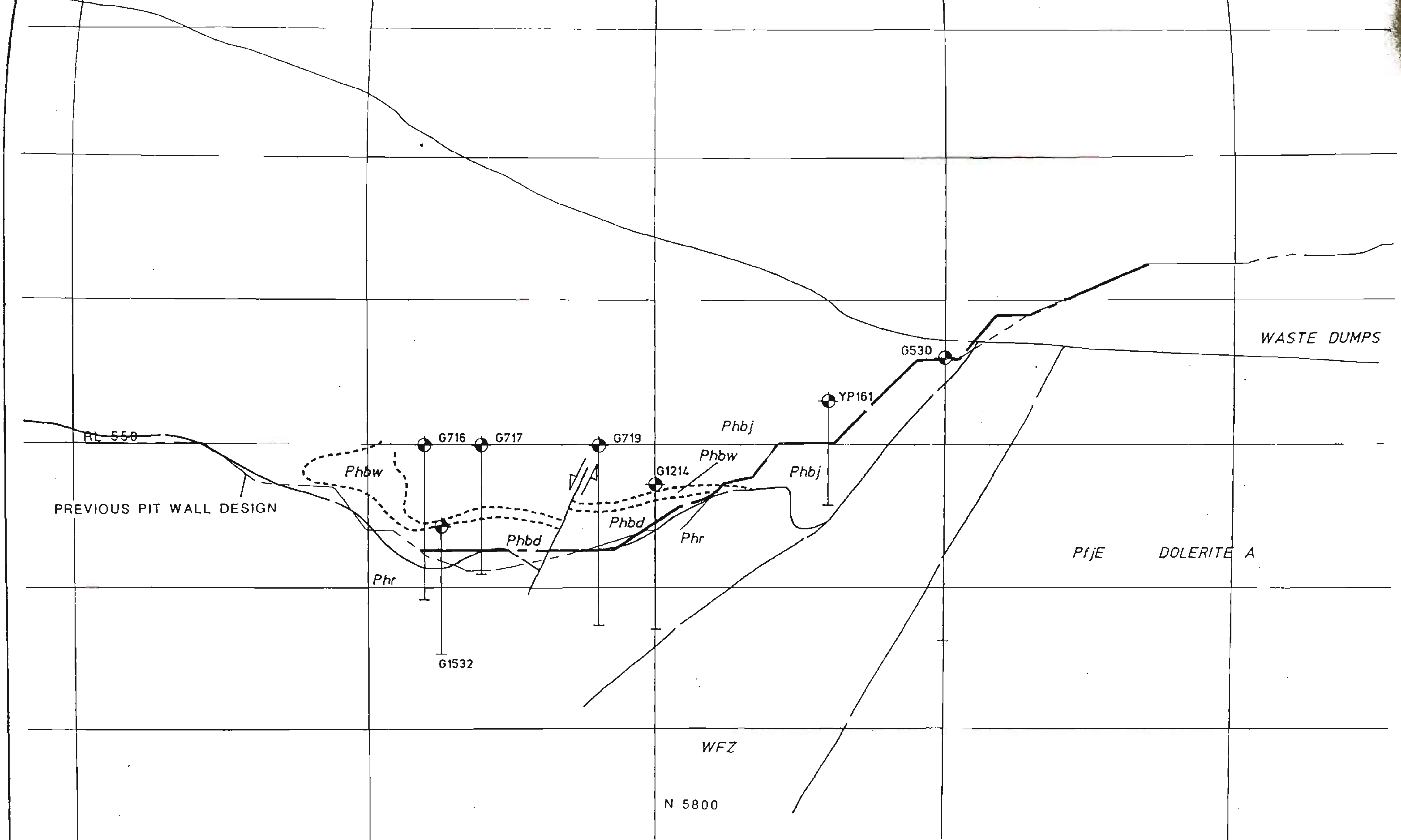
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 8280 E	FIGURE A37
Drn		
Dwg No		



Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 8320 E	FIGURE A38
Drn		
Dwg No		

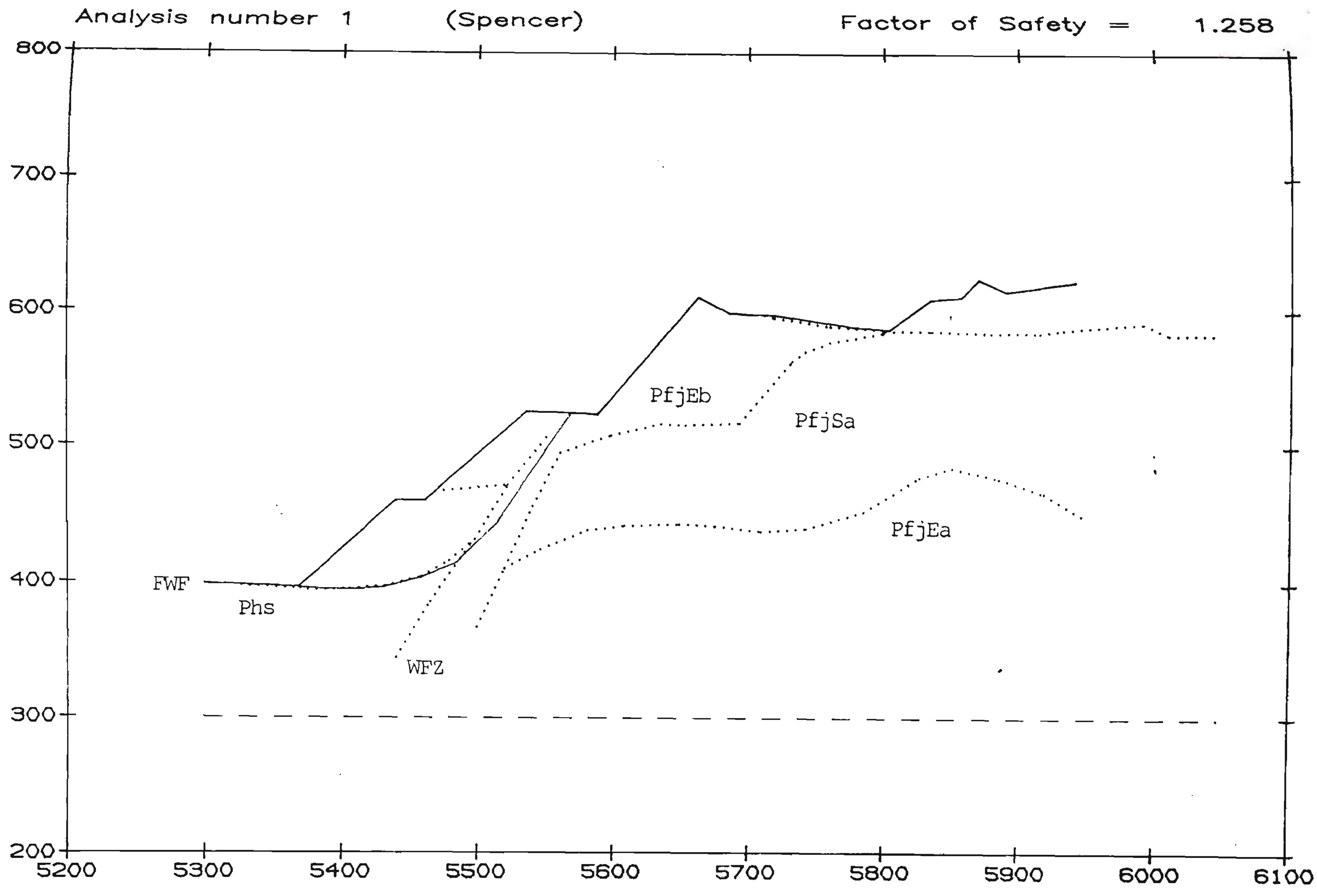


Scale	MT. WHALEBACK EAST PIT - NORTH WALL	FIGURE
Drn	CROSS SECTION 8360 E	A39
Dwg No.		



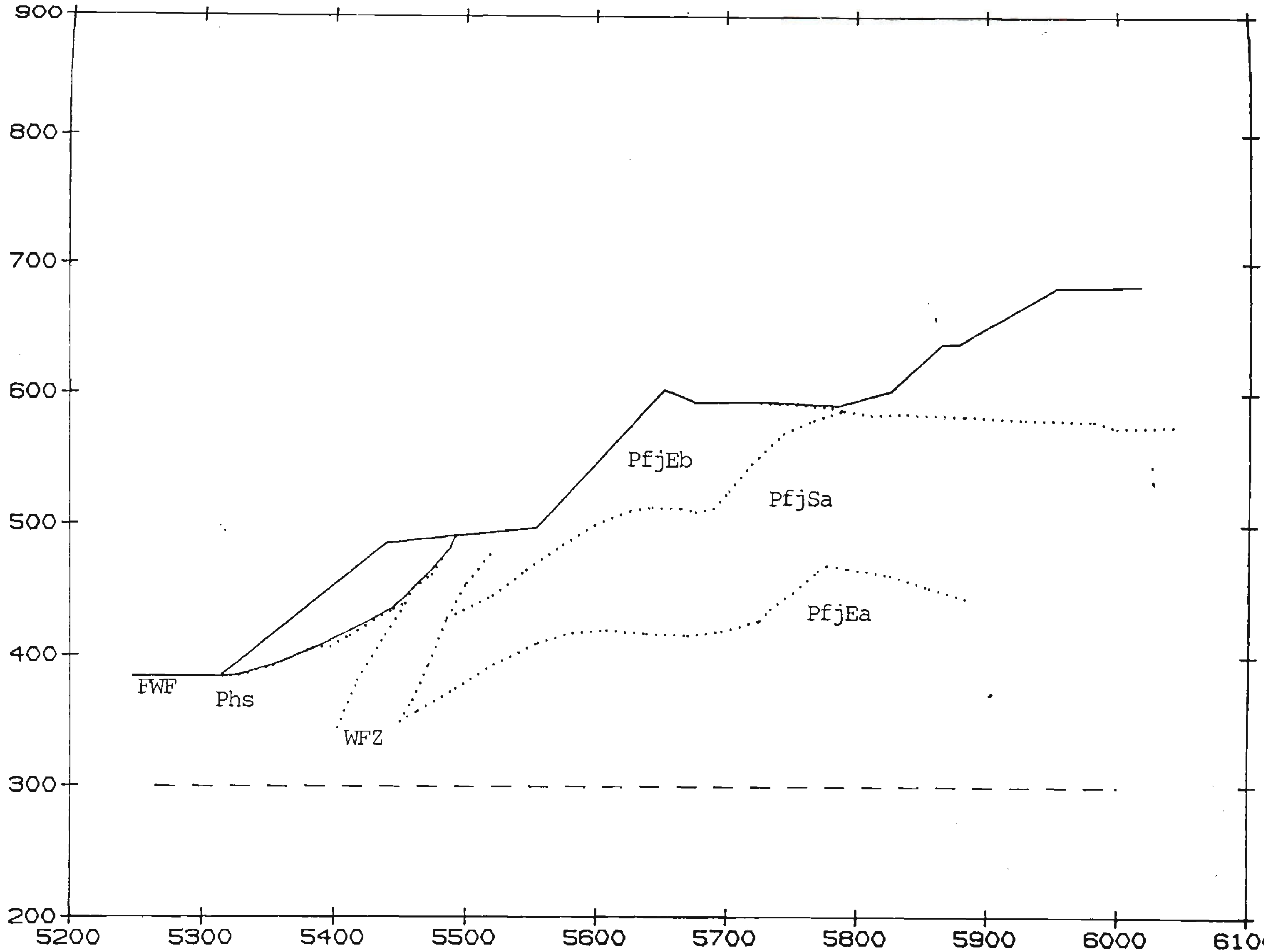
Scale	MT. WHALEBACK EAST PIT - NORTH WALL CROSS SECTION 8400 E	FIGURE A40
Drn		
Dwg No		

APPENDIX B



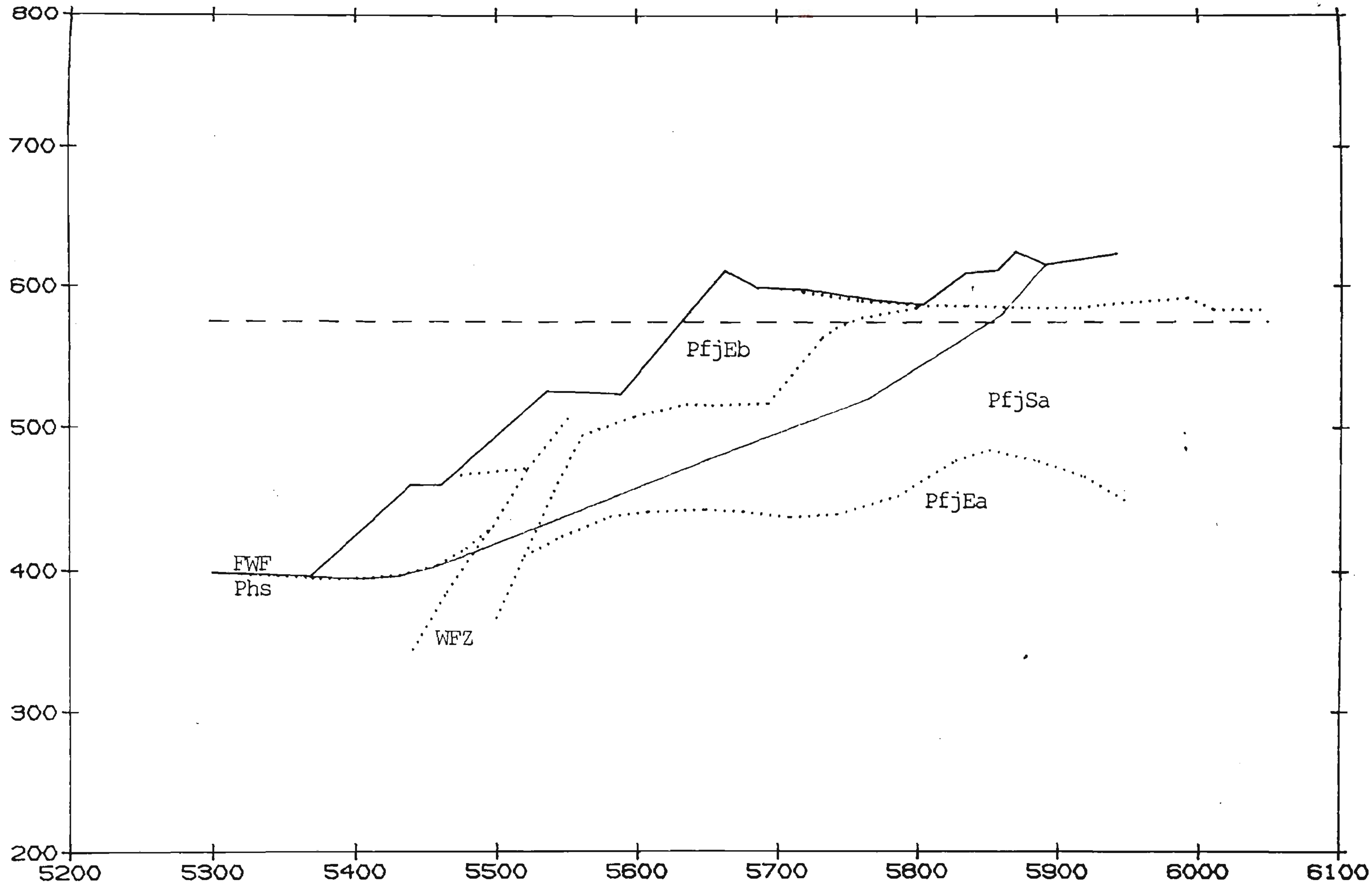
Scale	NEWMAN NORTH WALL 7200 NORMAL SECTION OPTION 4 BUTTRESS ZONE
Drn	FIGURE B1
Dwg No	

Analysis number 1 (Spencer) Factor of Safety = 1.038



Scale	NEWMAN NORTH WALL 7200 DOWN PLUNGE	FIGURE
Drn	SECTION OPTION 4 BUTTRESS ZONE	B2
Dwg No		

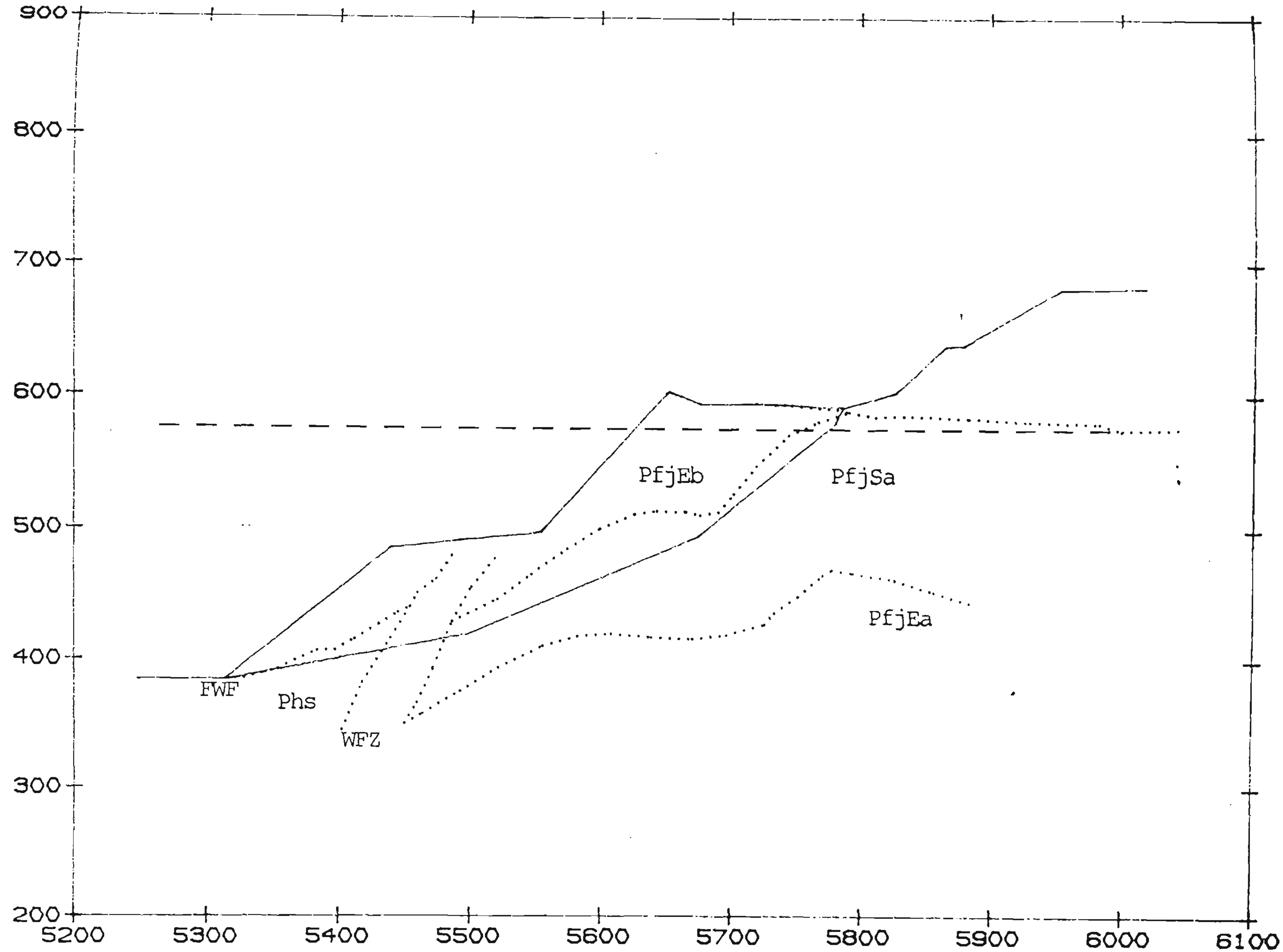
Analysis number 2 (Spencer) Factor of Safety = 2.481



Scale	NEWMAN NORTH WALL 7200 NORMAL
Drn	SECTION OPTION 4 OVERALL
Dwg No	

FIGURE B3

Analysis number 1 (Spencer) Factor of Safety = 2.408

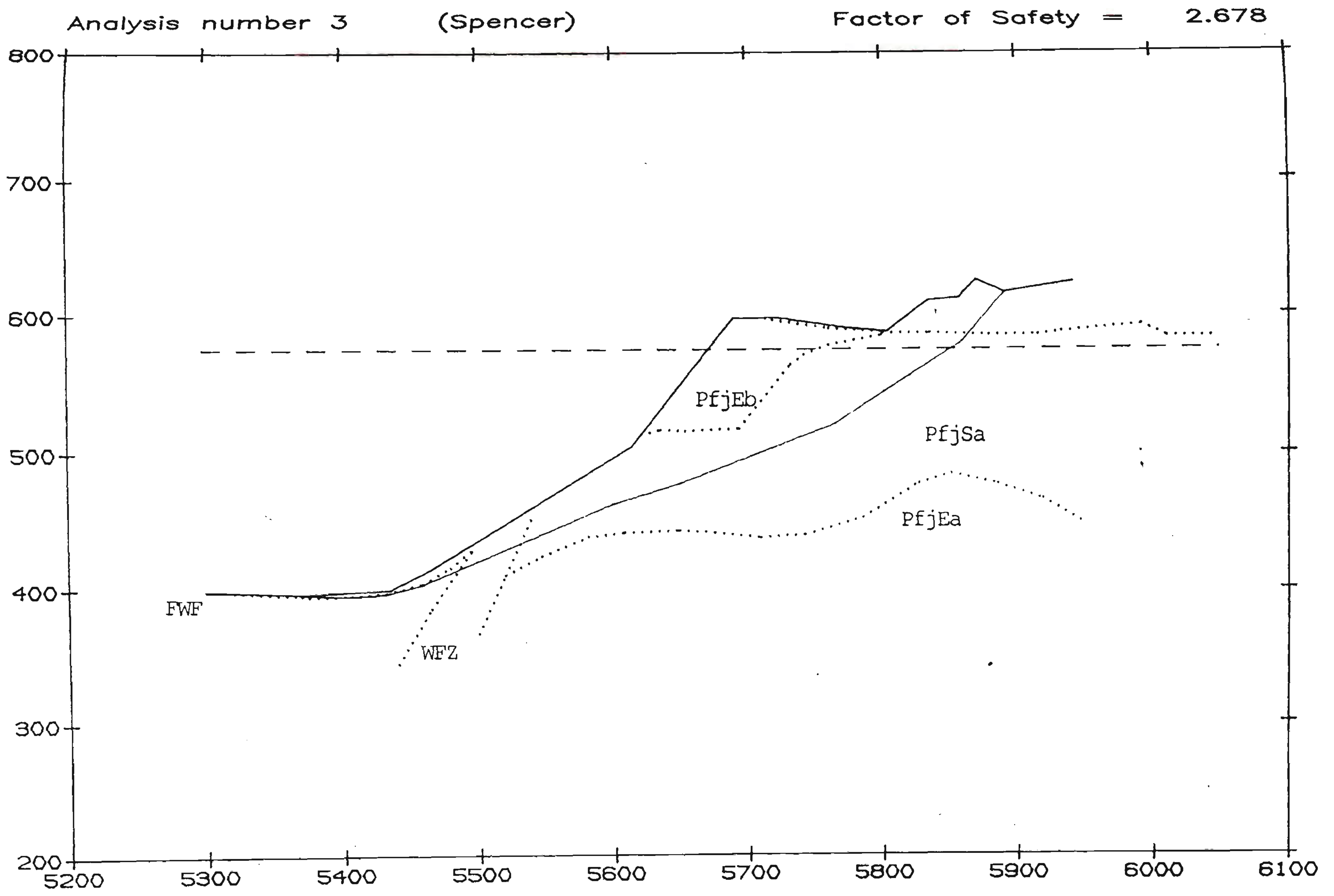


Scale	
Drn	
Dwg No	

NEWMAN NORTH WALL 7200 DOWN PLUNGE

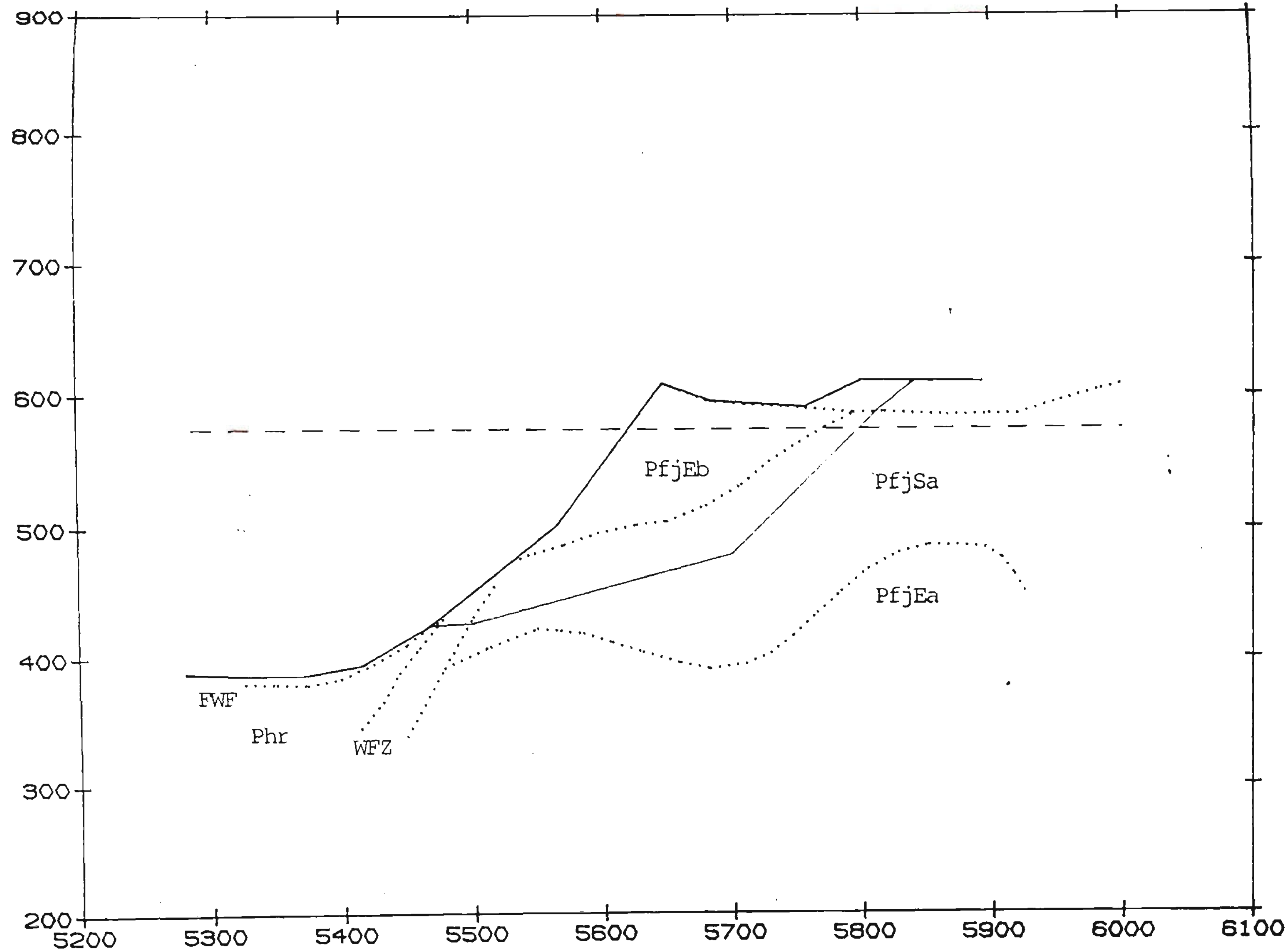
SECTION OPTION 4 OVERALL

FIGURE B4



Scale	NEWMAN NORTH WALL 7200 NORTH SOUTH SECTION OPTION 4A OVERALL	FIGURE B5
Drn		
Dwg No		

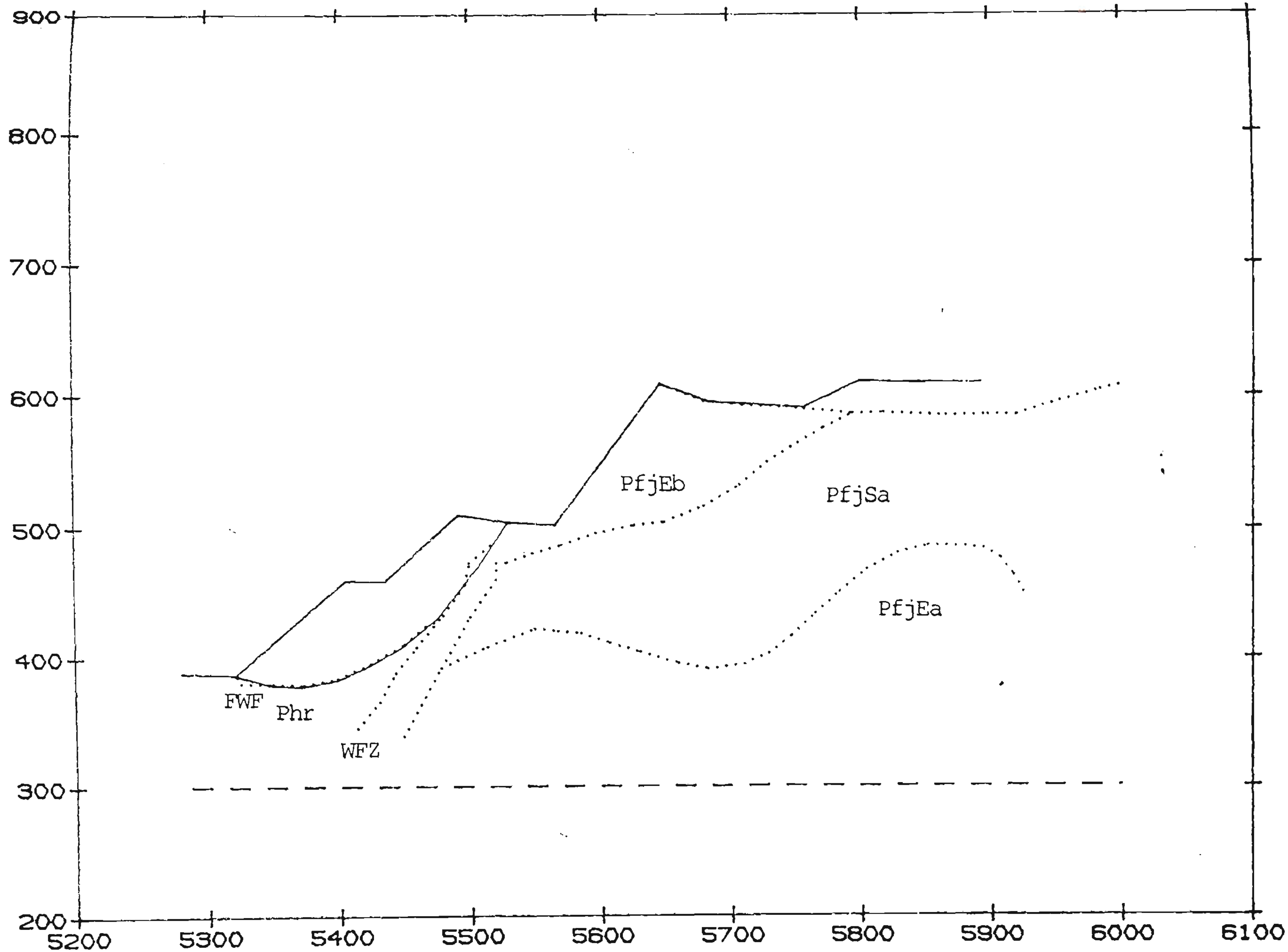
Analysis number 4 (Spencer) Factor of Safety = 2.788



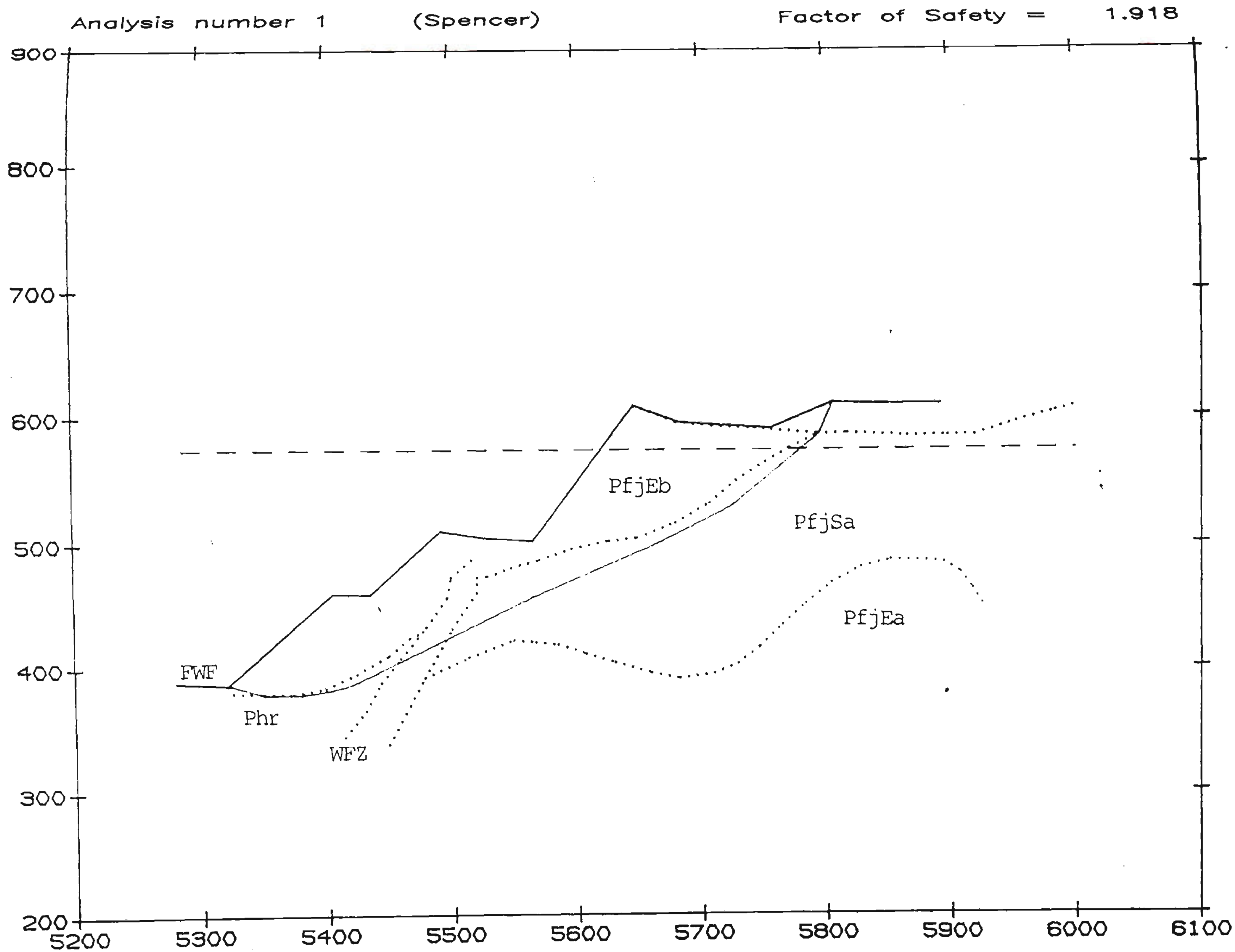
Scale	NEWMAN NORTH WALL 7160 NORTH SOUTH SECTION OPTION 4A OVERALL
Drn	
Dwg No	

FIGURE B6

Analysis number 3 (Spencer) Factor of Safety = 1.349



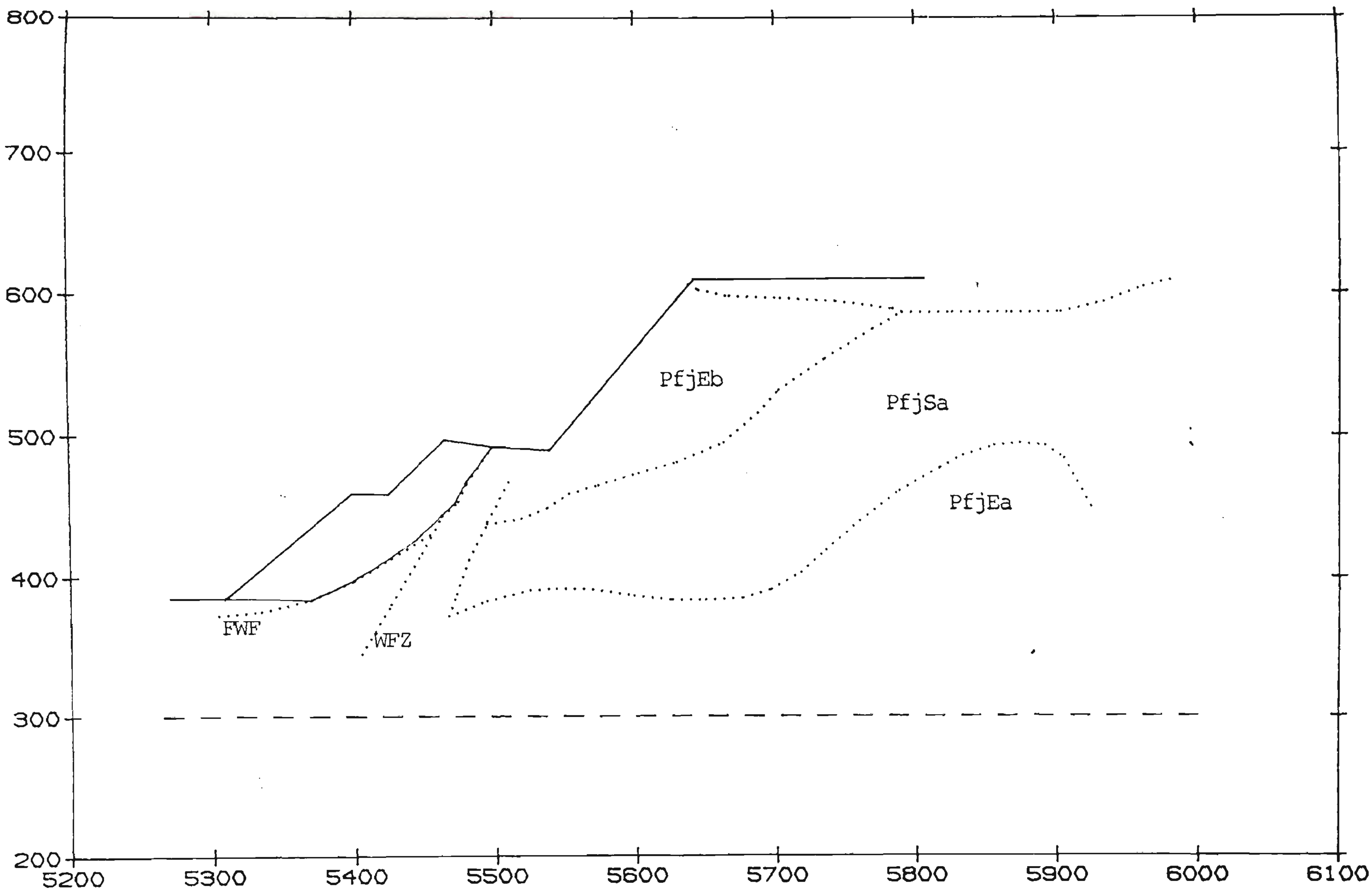
Scale	NEWMAN NORTH WALL 7120 NORMAL SECTION	FIGURE
Drn	OPTION 4 BUTTRESS ZONE	B7
Dwg No		



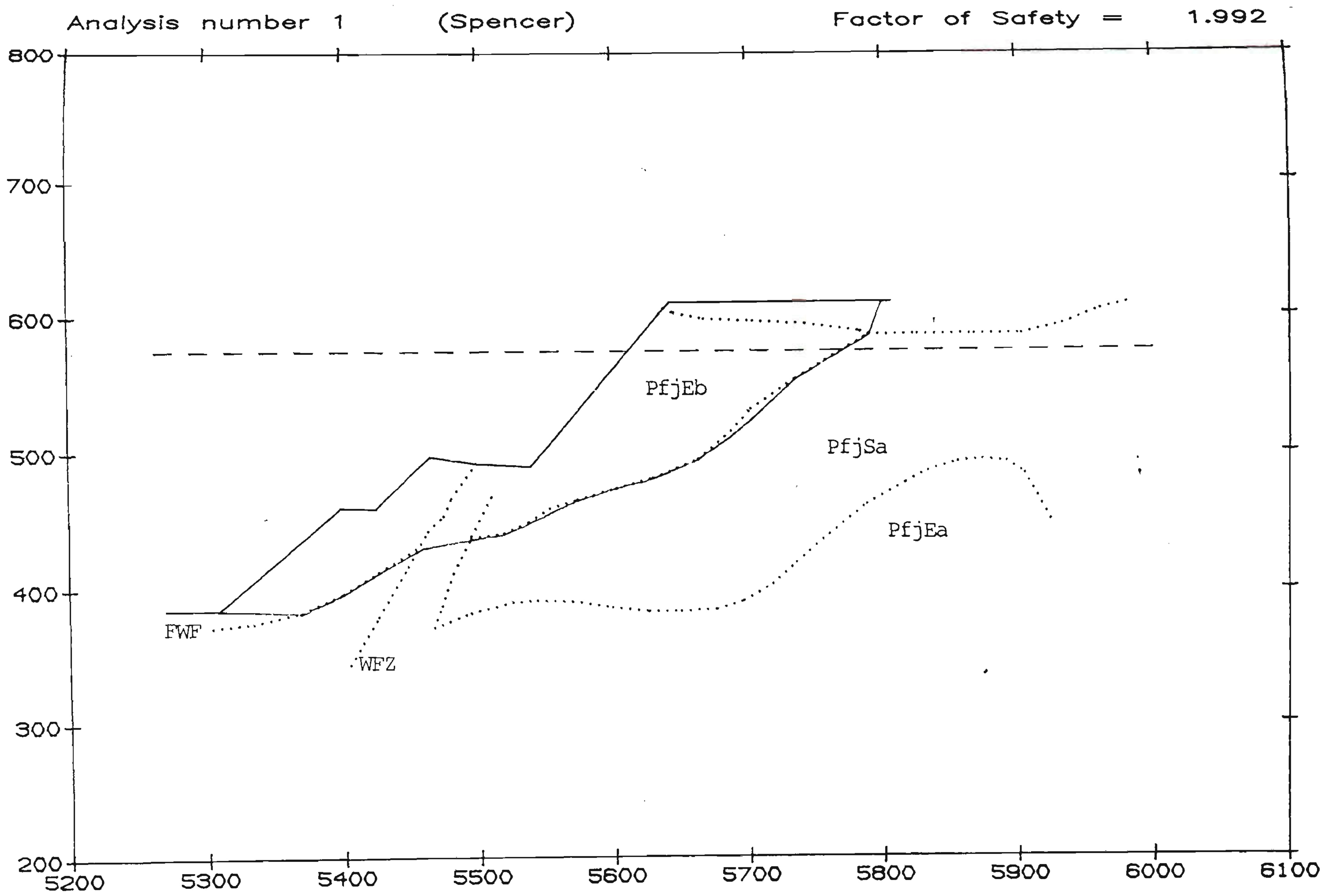
Scale	NEWMAN NORTH WALL 7120 NORMAL SECTION
Drn	OPTION 4 OVERALL
Dwg No	

FIGURE B8

Analysis number 2 (Spencer) Factor of Safety = 1.265

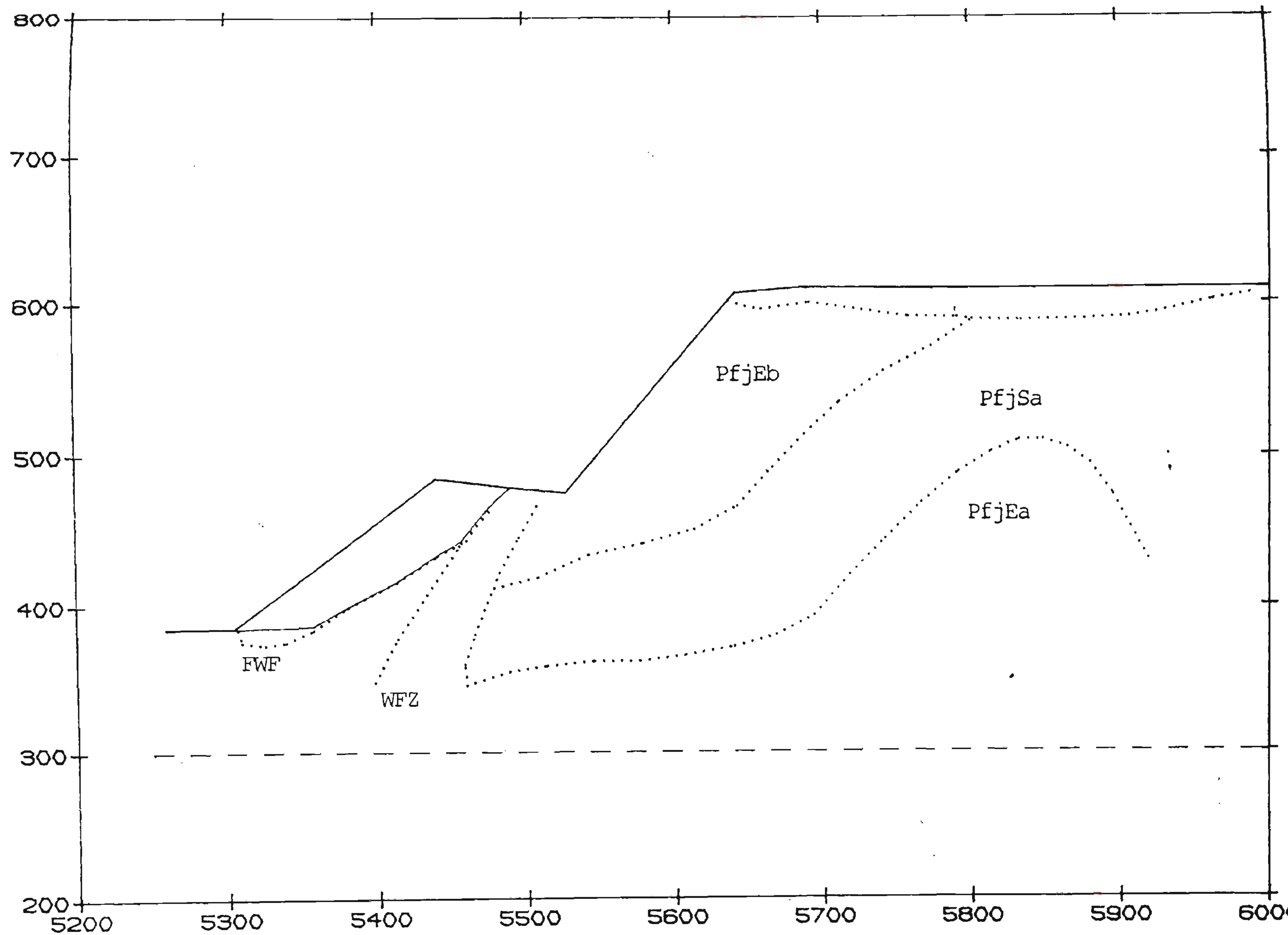


Scale	NEWMAN NORTH WALL 7080 NORMAL SECTION OPTION 4 BUTTRESS ZONE	FIGURE B9
Drn		
Dwg No		



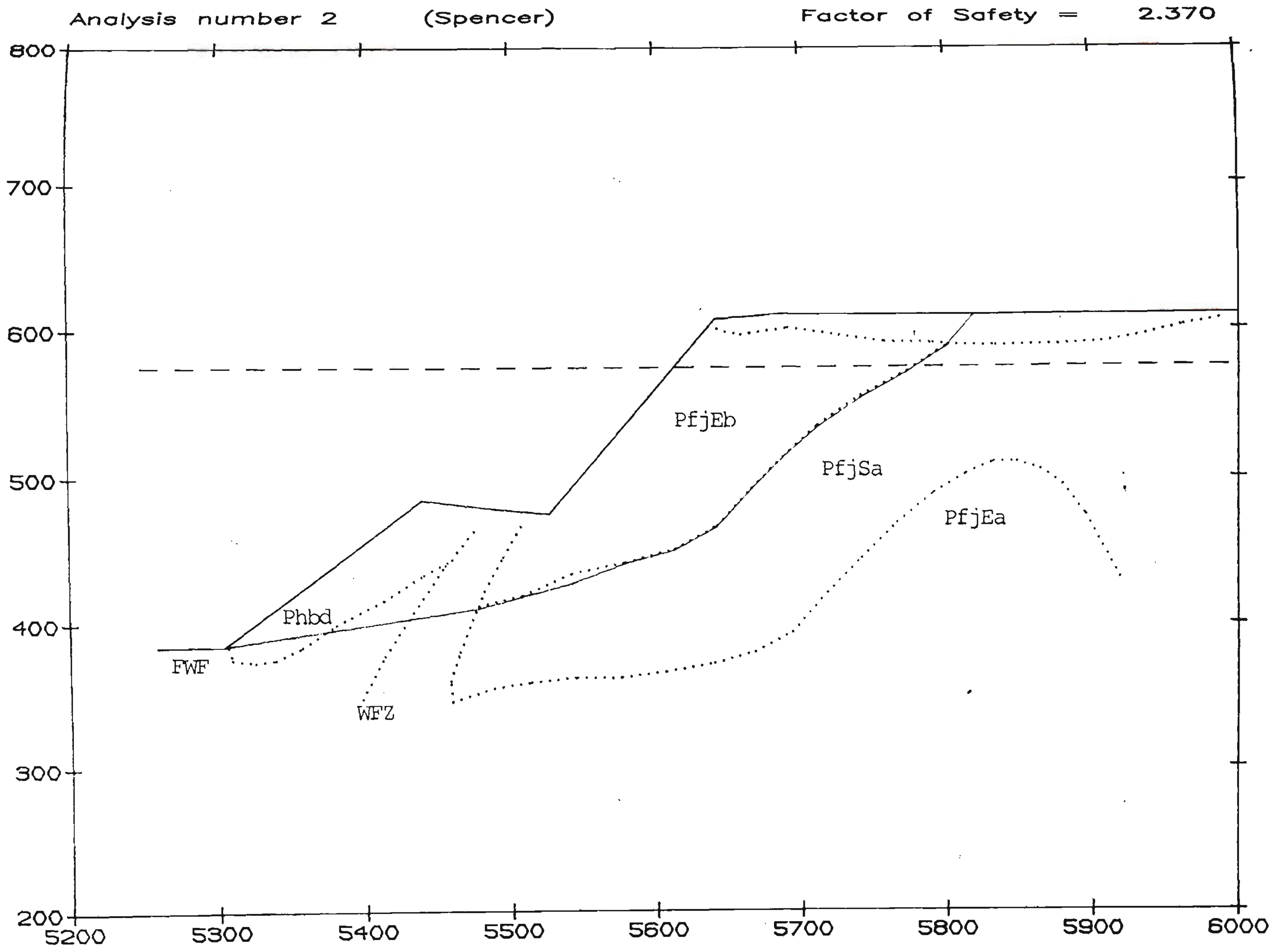
Scale	NEWMAN NORTH WALL 7080 NORMAL	FIGURE
Drn	SECTION OPTION 4 OVERALL	B10
Dwg No		

Analysis number 1 (Spencer) Factor of Safety = 1.202



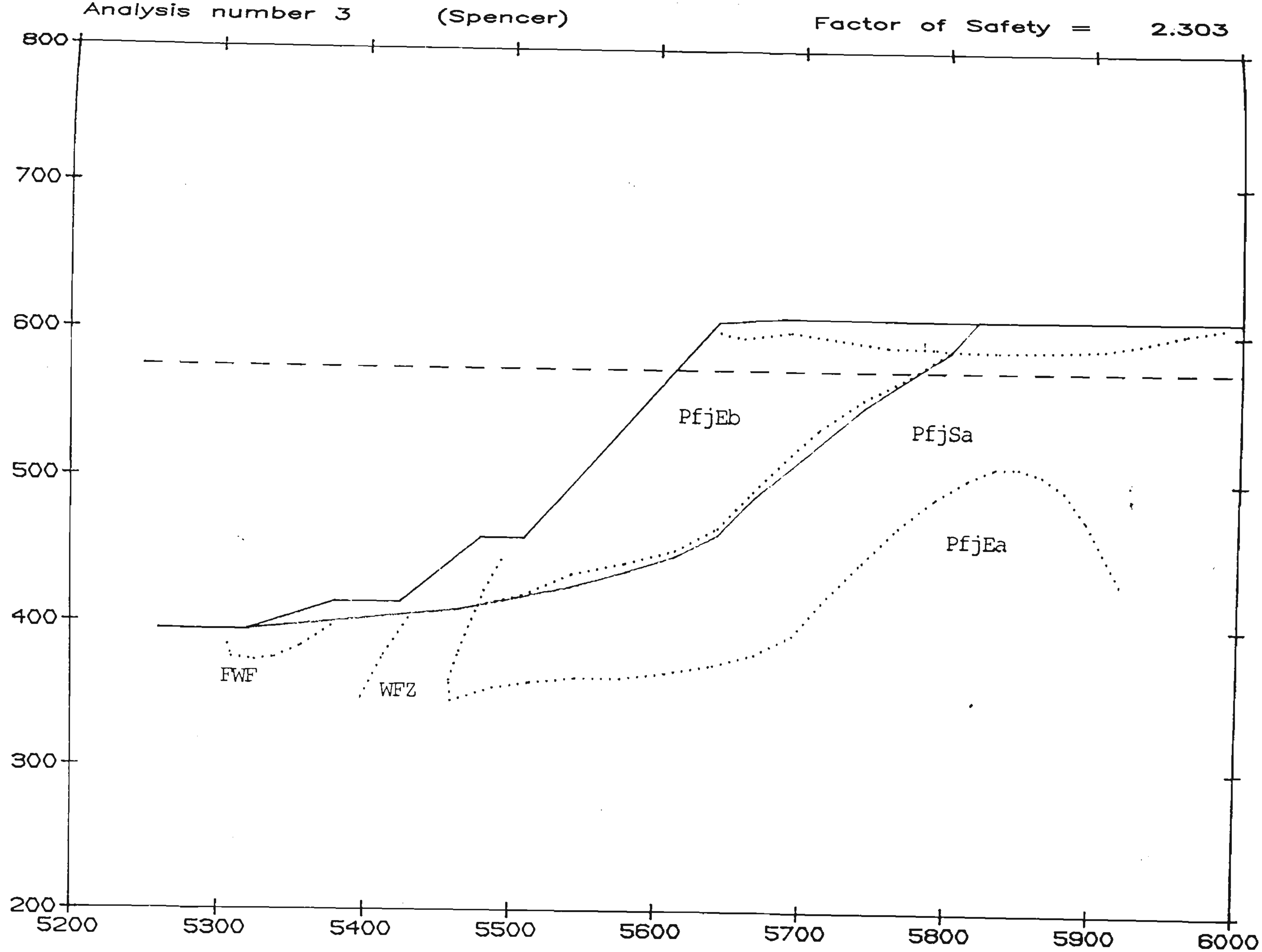
Scale	NEWMAN NORTH WALL 7040 NORMAL SECTION
Drn	OPTION 4 BUTTRESS ZONE
Dwg No	

FIGURE B11

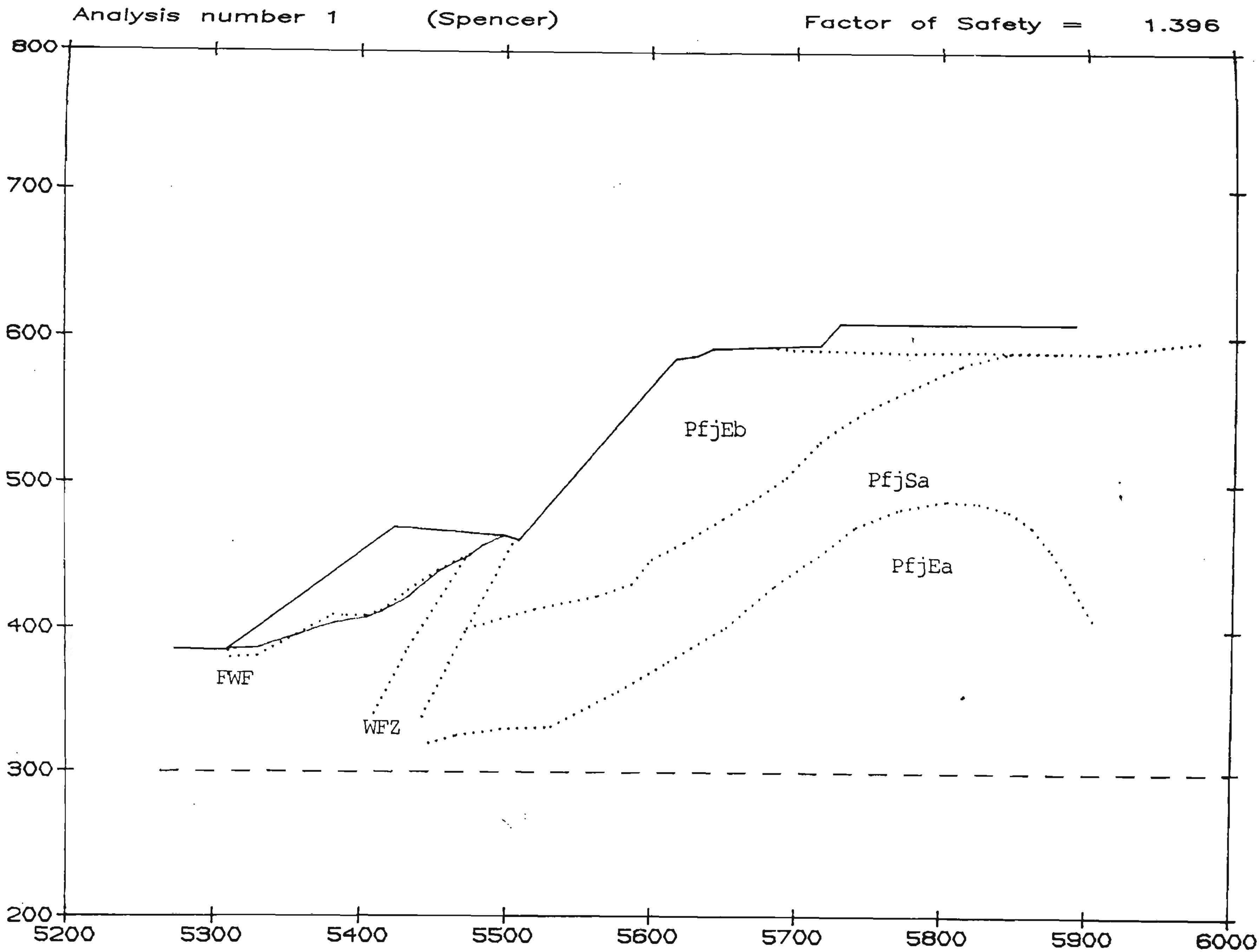


Scale	NEWMAN NORTH WALL 7040 NORMAL SECTION
Drn	OPTION 4 OVERALL
Dwg No	

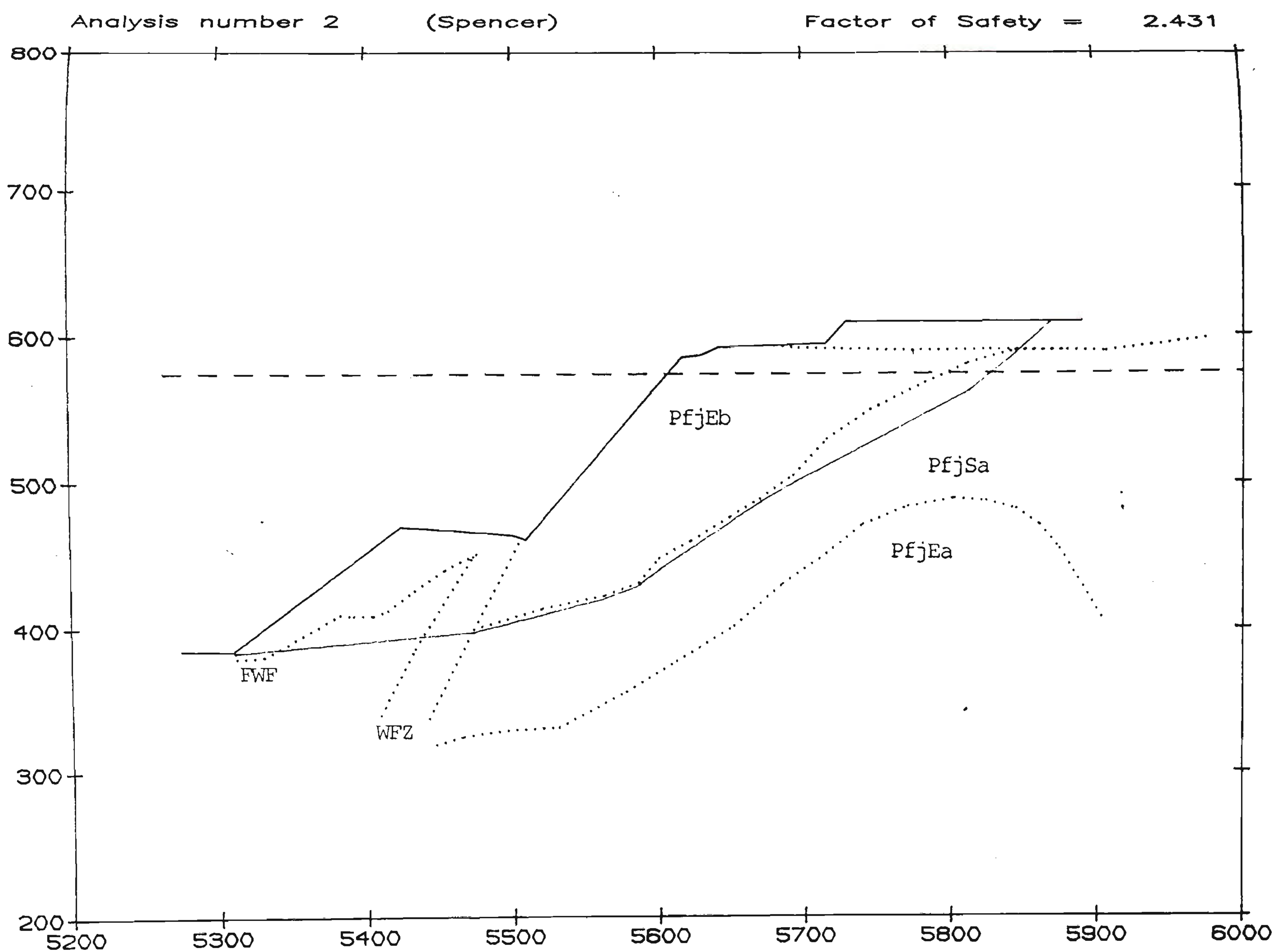
FIGURE B12



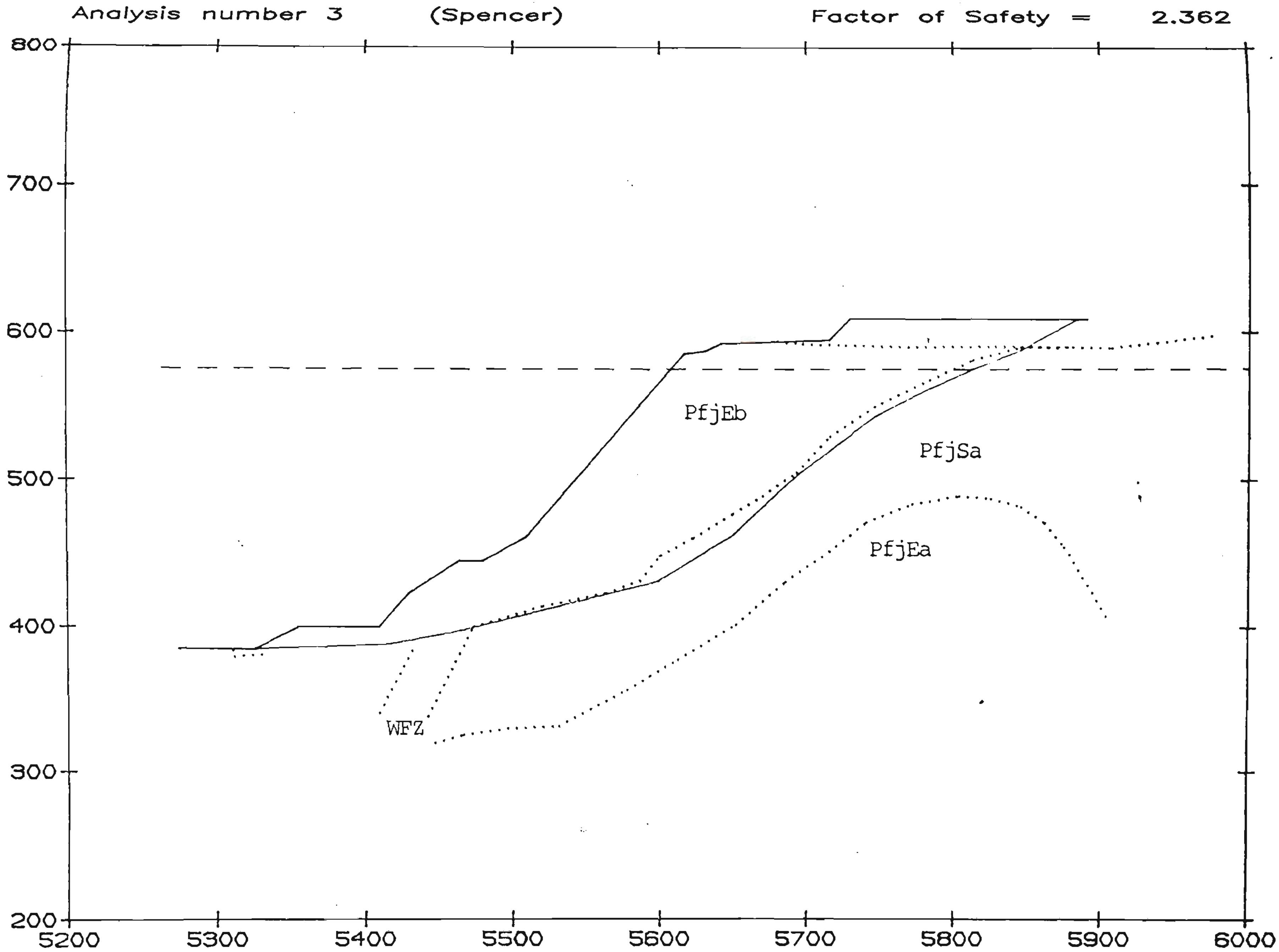
Scale	NEWMAN NORTH WALL 7040 NORTH SOUTH	FIGURE
Drn	SECTION OPTION 4A OVERALL	B13
Dwg No		



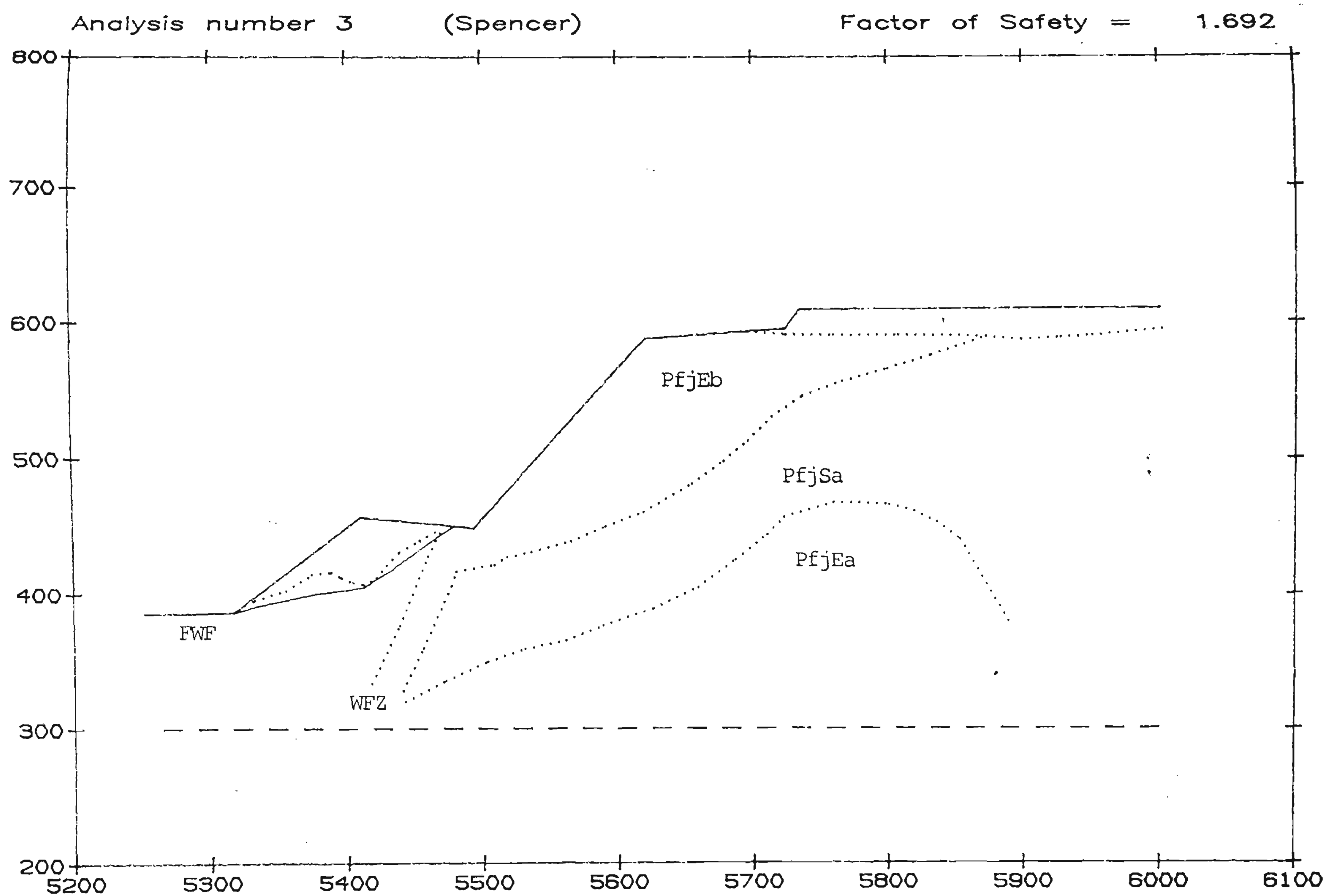
	Scale	NEWMAN NORTH WALL 7000 NORMAL SECTION	FIGURE
	Drn	OPTION 4 BUTTRESS ZONE	B14
	Dwg No		



Scale	NEWMAN NORTH WALL 7000 NORMAL SECTION	FIGURE
Drn	OPTION 4 OVERALL	B15
Dwg No		

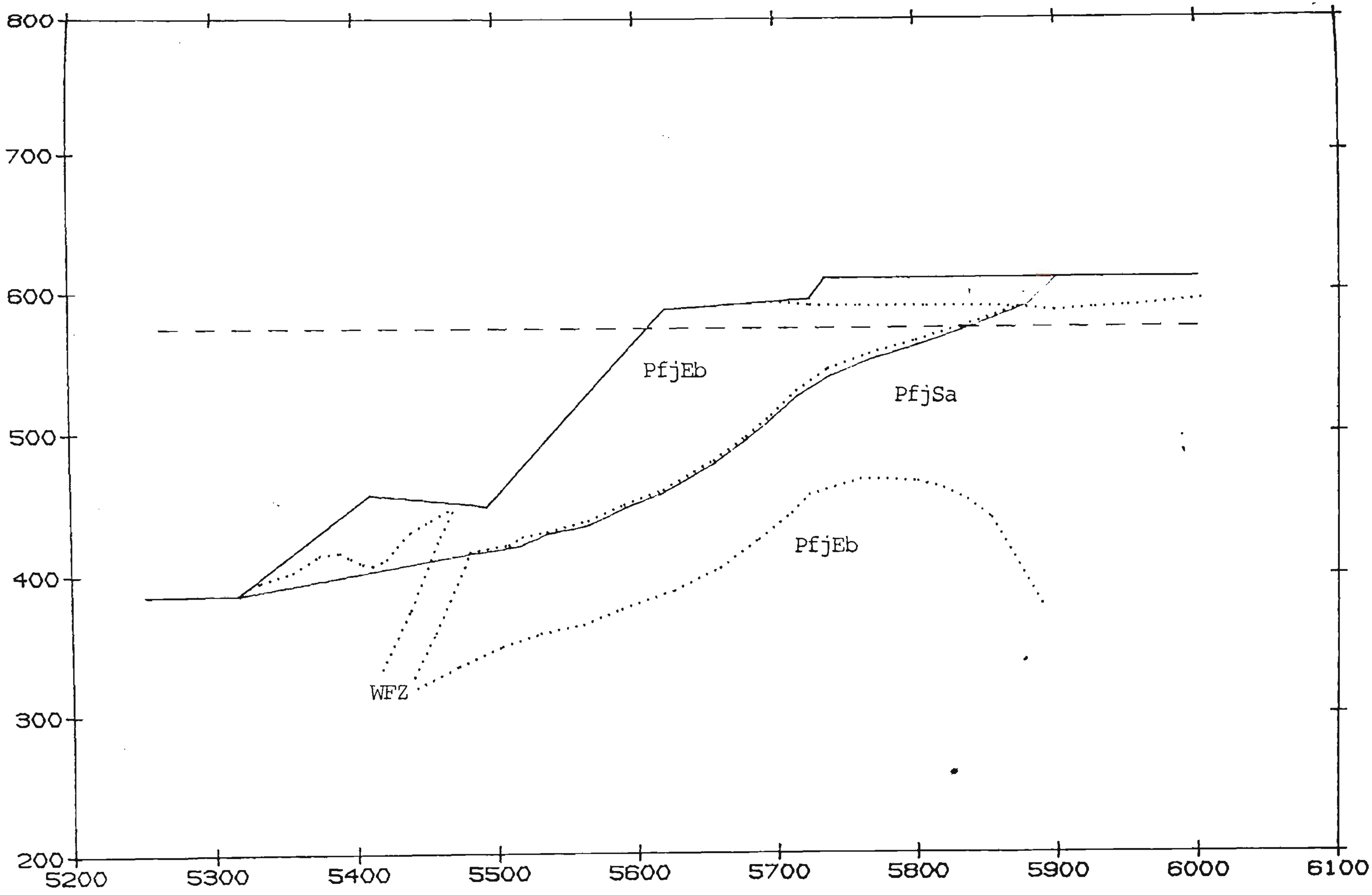


	Scale	NEWMAN NORTH WALL 7000 NORTH SOUTH	FIGURE
	Drn	SECTION OPTION 4A OVERALL	B16
	Dwg No		

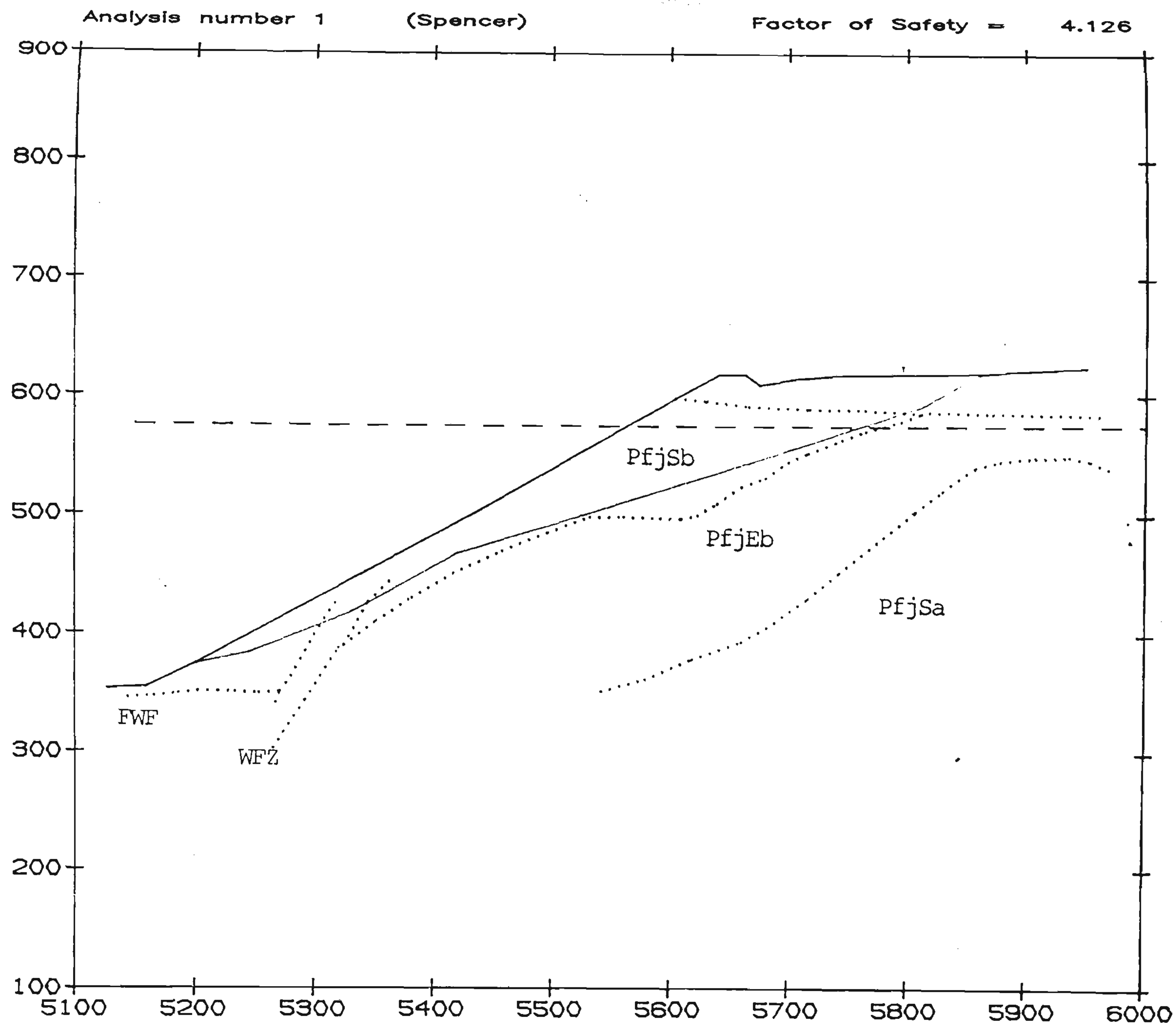


	Scale Drn Dwg No	NEWMAN NORTH WALL 6960 NORMAL SECTION OPTION 4 BUTTRESS ZONE	FIGURE B17

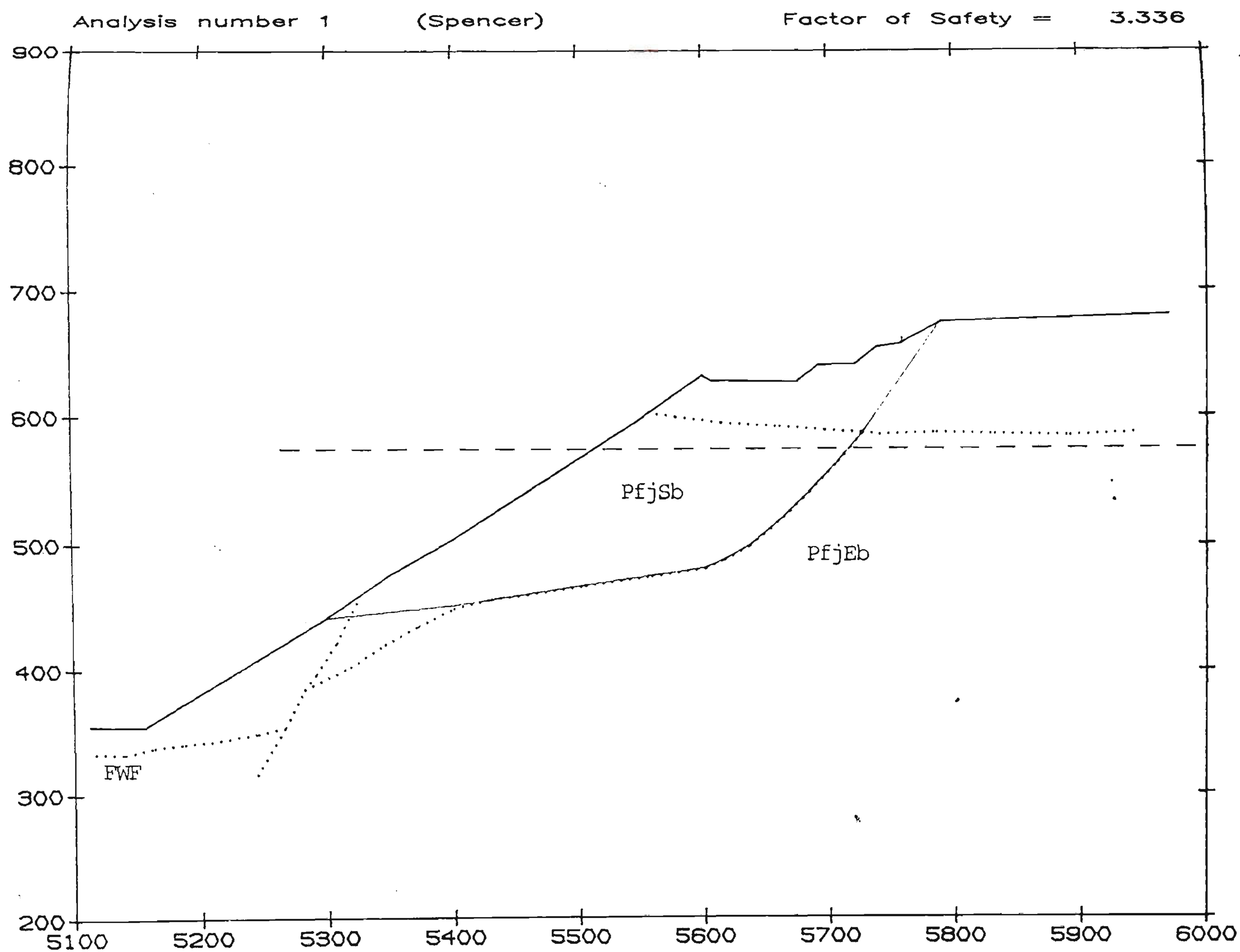
Analysis number 1 (Spencer) Factor of Safety = 2.804



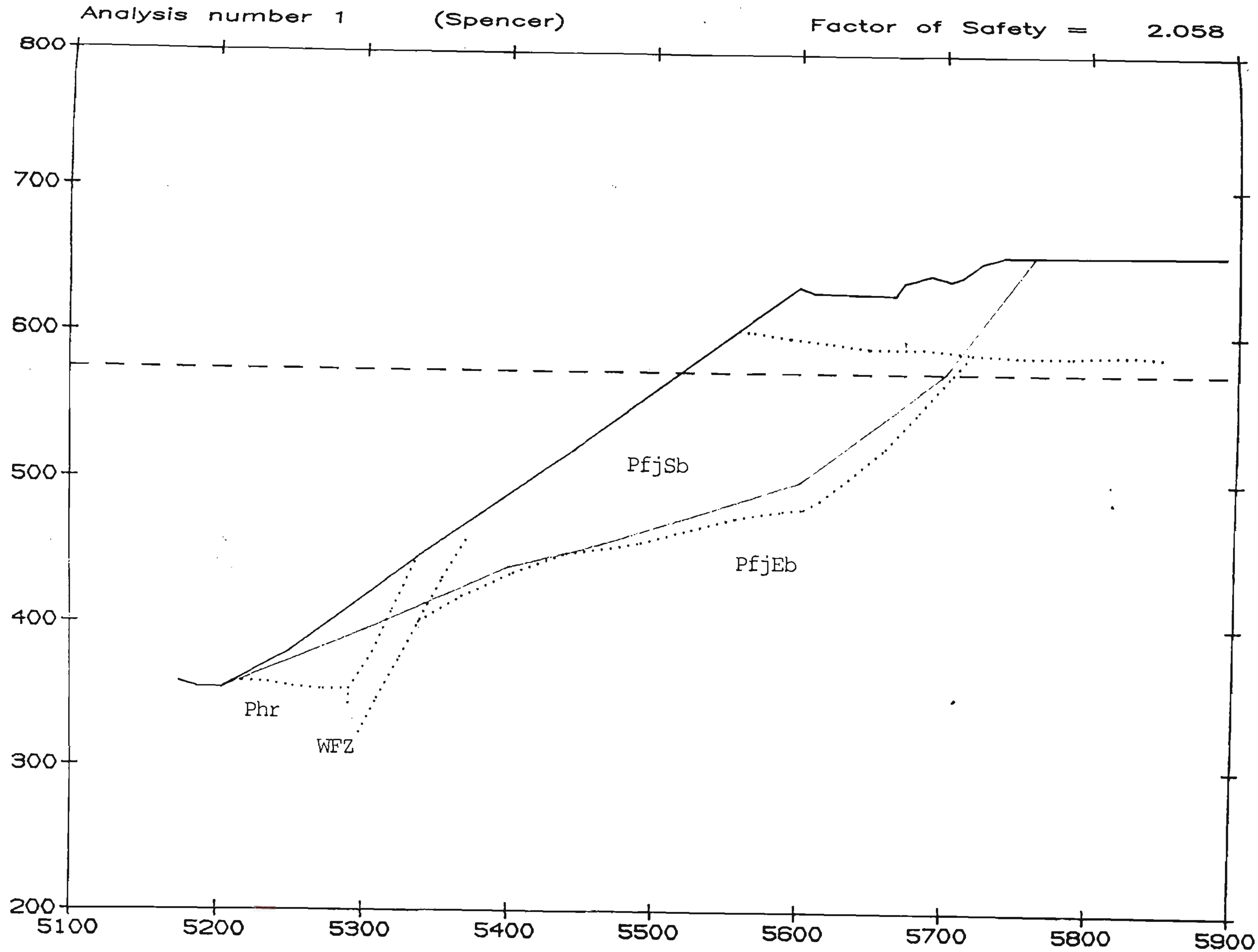
Scale	NEWMAN NORTH WALL 6960 NORMAL SECTION OPTION 4 OVERALL	FIGURE B18
Drn		
Dwg No		



Scale	NEWMAN NORTH WALL 6720 DOWN PLUNGE	FIGURE
Drn	SECTION OPTION 4 OVERALL	B19
Dwg No		

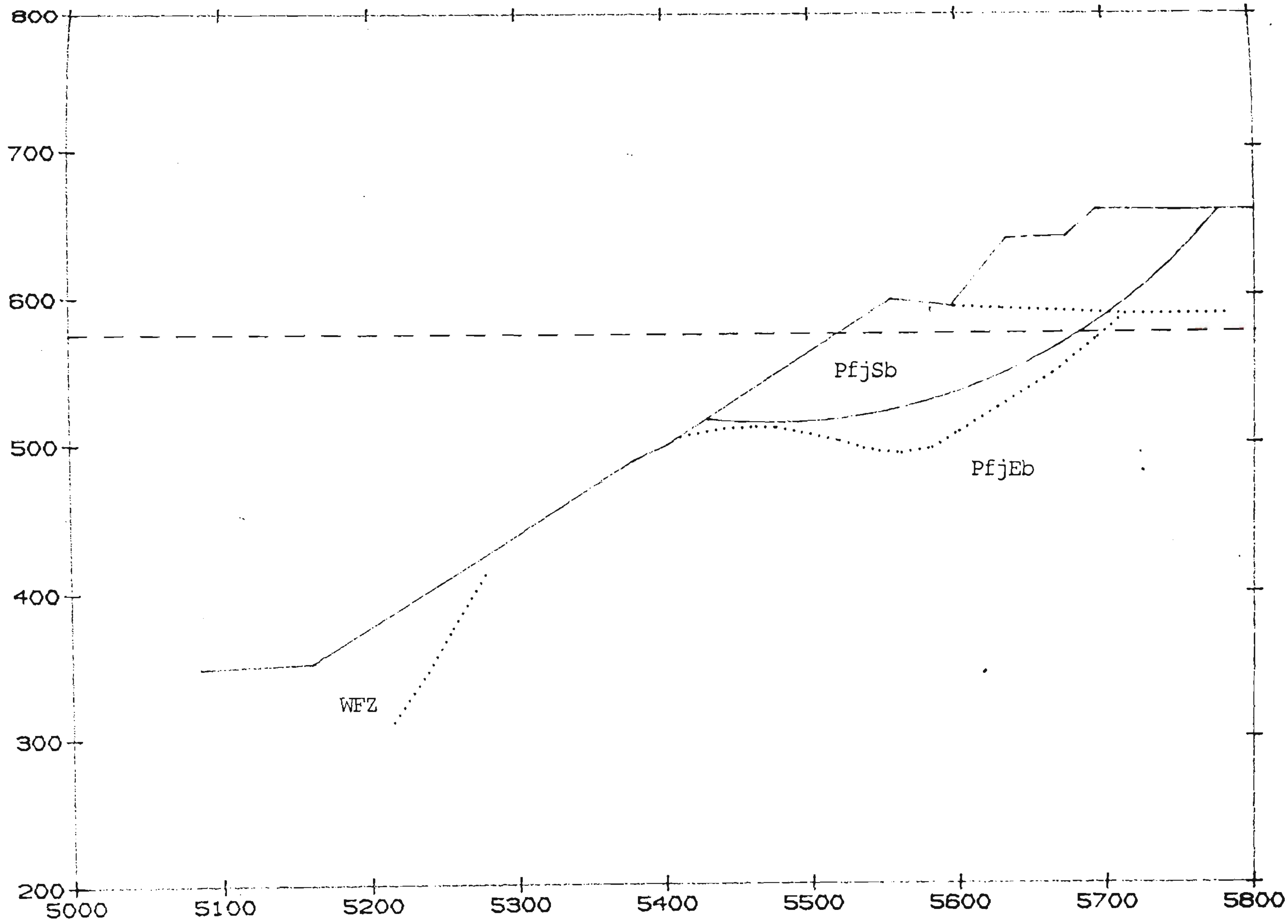


	Scale	NEWMAN NORTH WALL 6600 DOWN PLUNGE	FIGURE
	Drn		B20
	Dwg No	SECTION OPTION 4 OVERALL	



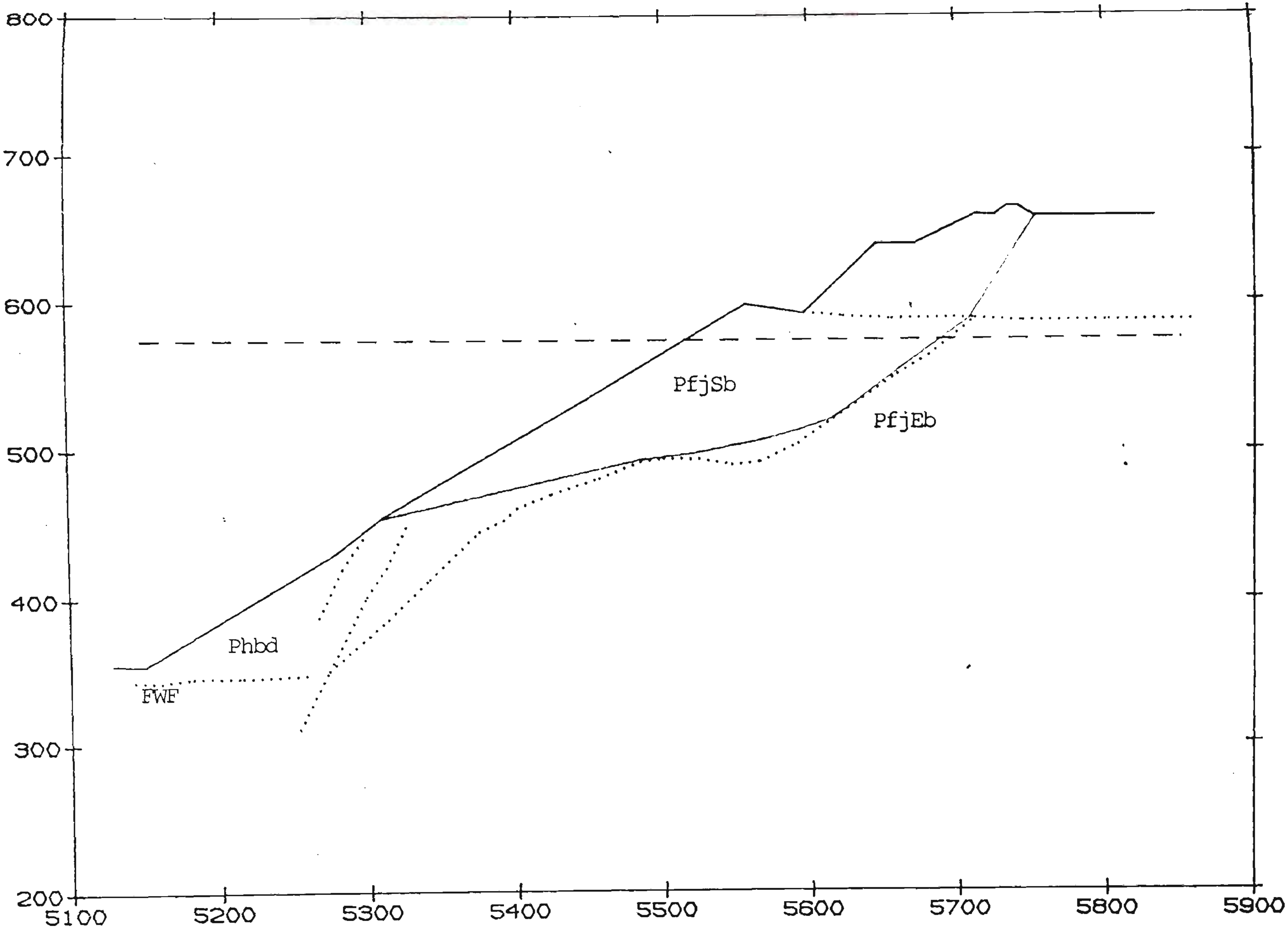
Scale	NEWMAN NORTH WALL 6480 NORMAL SECTION	FIGURE
Drn	OPTION 4 OVERALL	B21
Dwg No		

Analysis number 2 (Spencer) Factor of Safety = 4.246

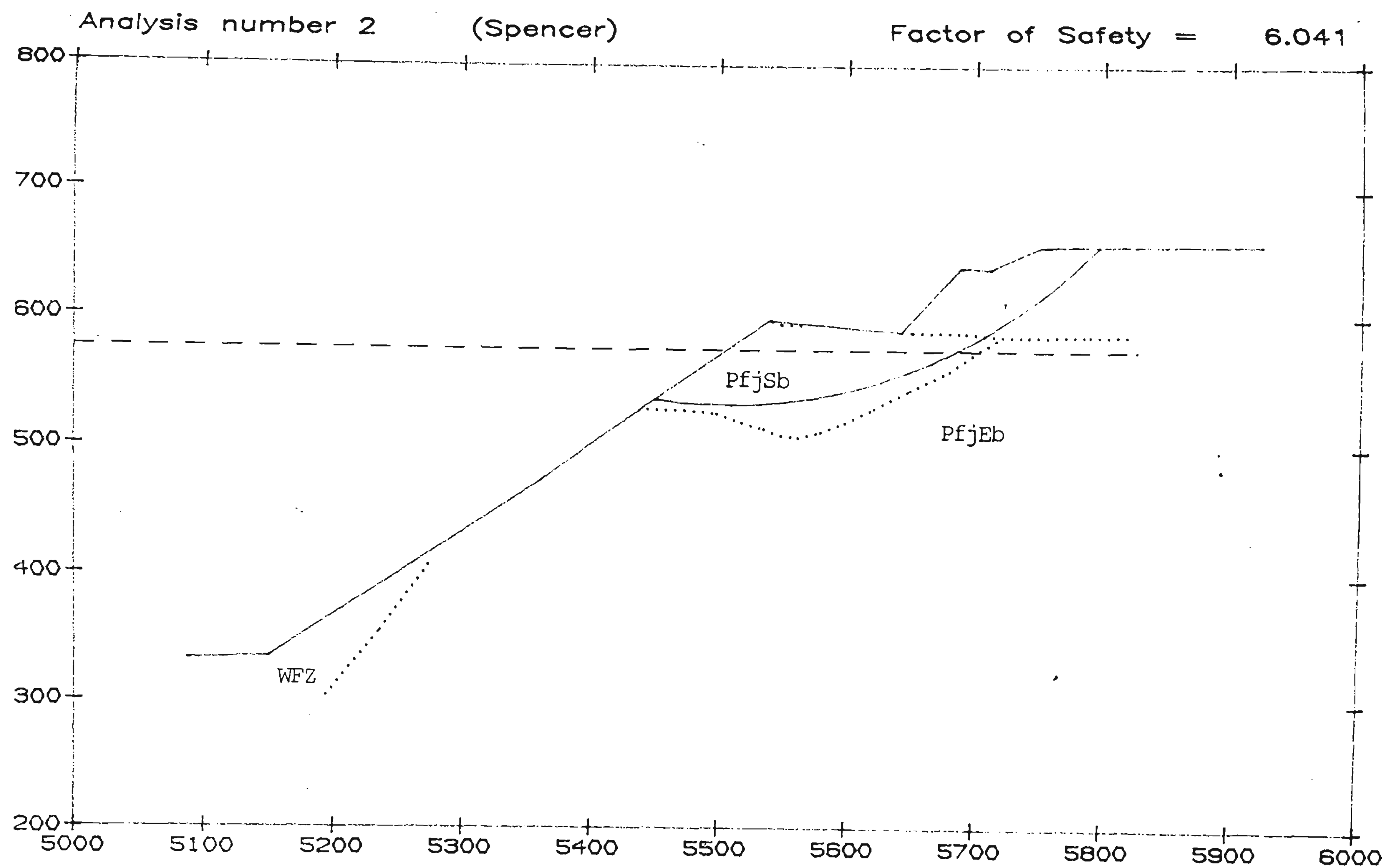


Scale	NEWMAN NORTH WALL 6480 DOWN PLUNGE	FIGURE
Drn	SECTION OPTION 4 OVERALL	B22
Dwg No		

Analysis number 1 (Spencer) Factor of Safety = 3.635



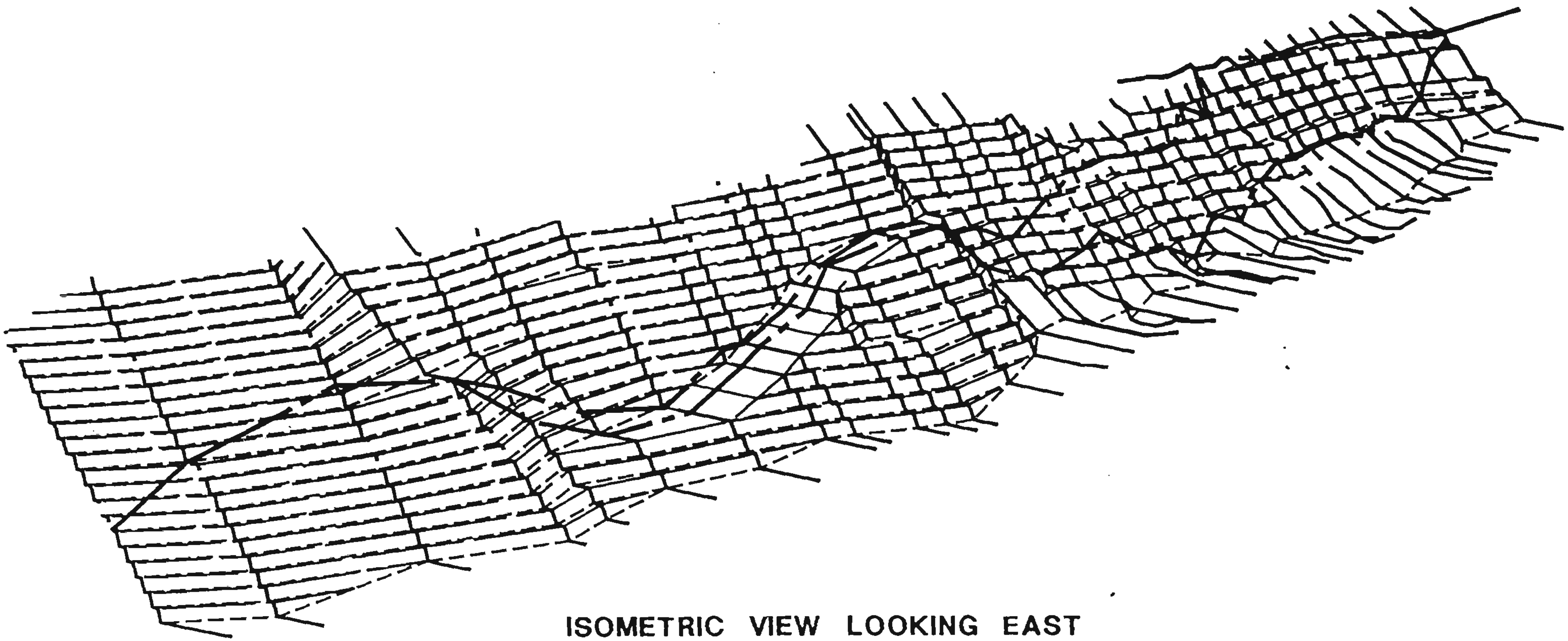
Scale	NEWMAN NORTH WALL 6360 NORMAL SECTION	FIGURE
Drn	OPTION 4 OVERALL	B23
Dwg No		



Scale	NEWMAN NORTH WALL 6360 DOWN PLUNGE
Drn	SECTION OPTION 4 OVERALL
Dwg No	

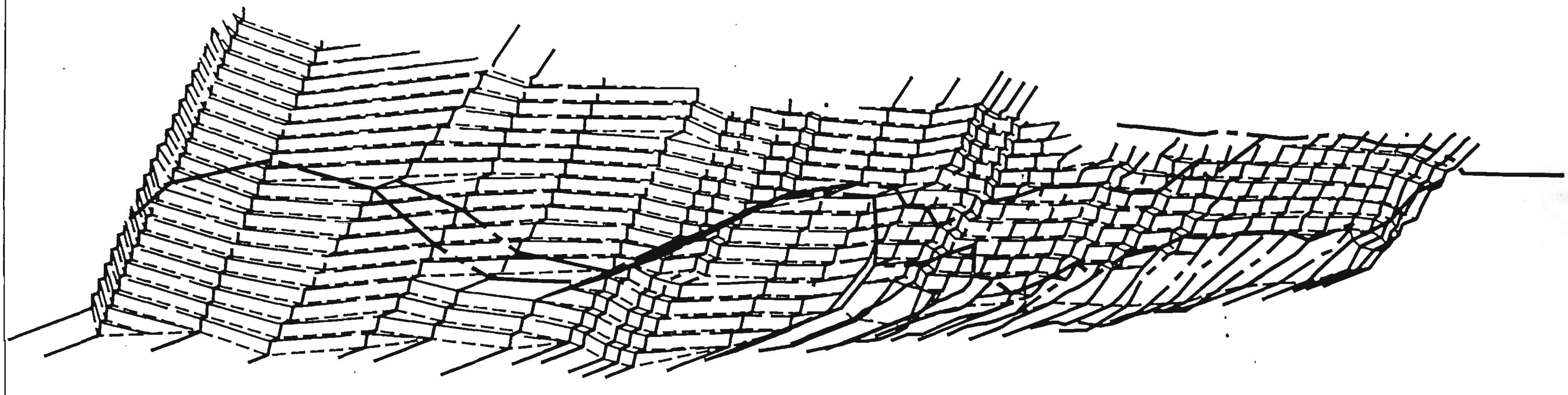
FIGURE B24

APPENDIX C



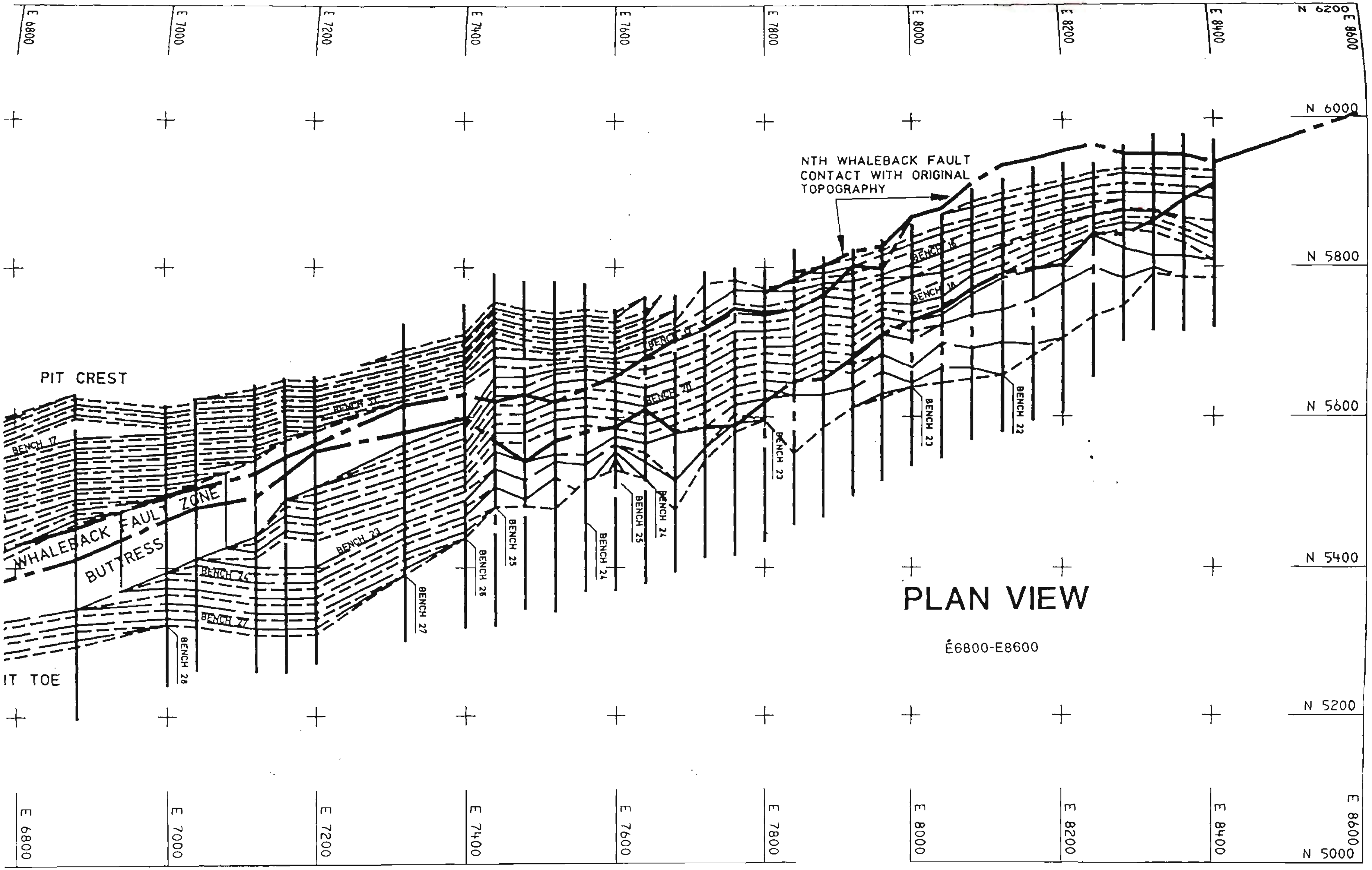
ISOMETRIC VIEW LOOKING EAST

Scale		MT. NEWMAN MINING CO. PTY. LTD.	FIGURE
Drn		MT. WHALEBACK EAST PIT - NORTH WALL	C1
Dwg No.		PROPOSED PIT WALL DESIGN OPTION 1	



ISOMETRIC VIEW LOOKING WEST

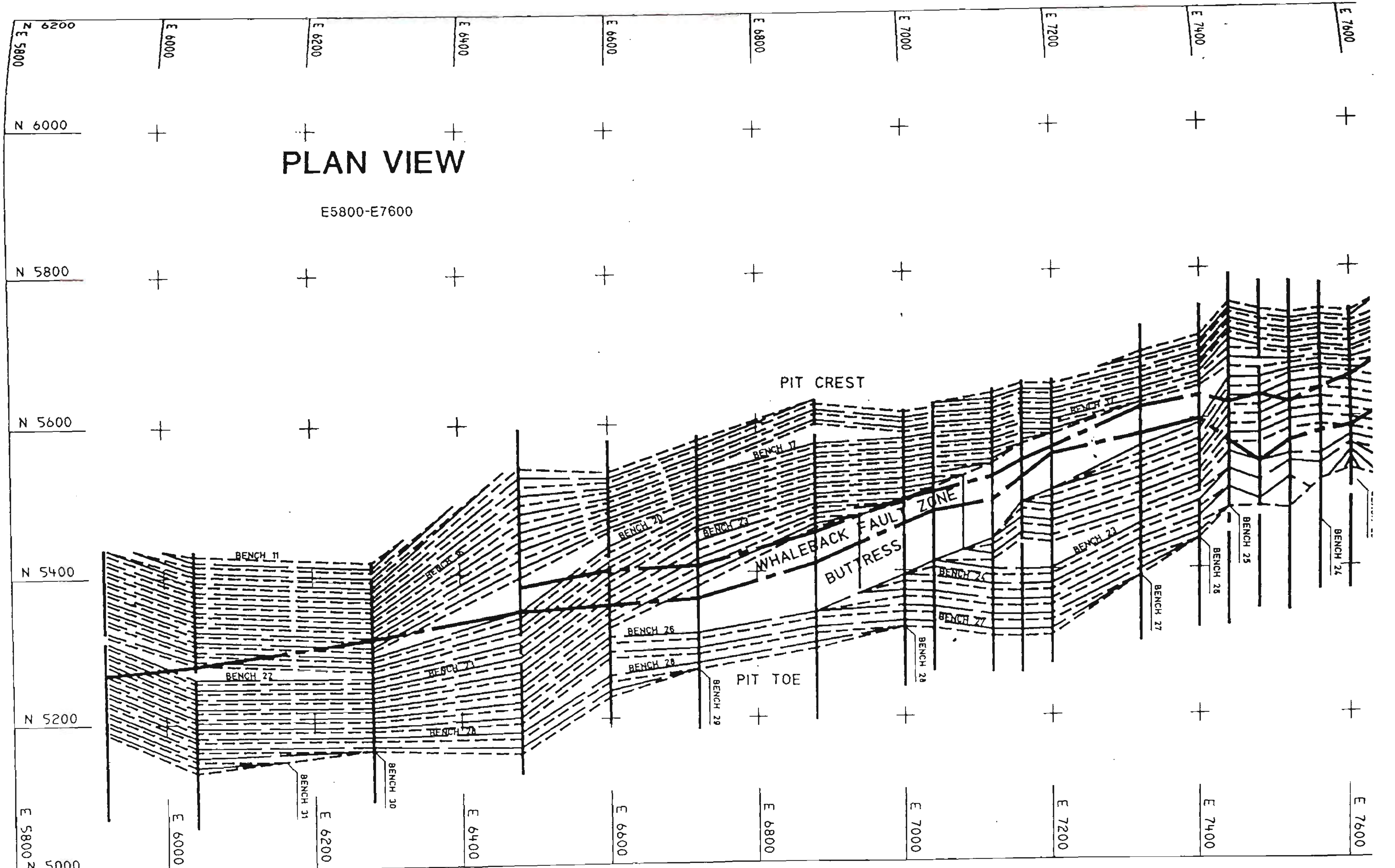
Scale	MT. NEWMAN MINING CO. PTY. LTD. MT. WHALEBACK EAST PIT - NORTH WALL PROPOSED PIT WALL DESIGN OPTION 1	FIGURE C2
Drn		
Dwg No		



Scale	**MT. NEWMAN MINING CO. PTY. LTD.**
Drn	**MT. WHALEBACK EAST PIT - NORTH WALL**
Dwg No	**PROPOSED PIT WALL DESIGN OPTION 1**

FIGURE

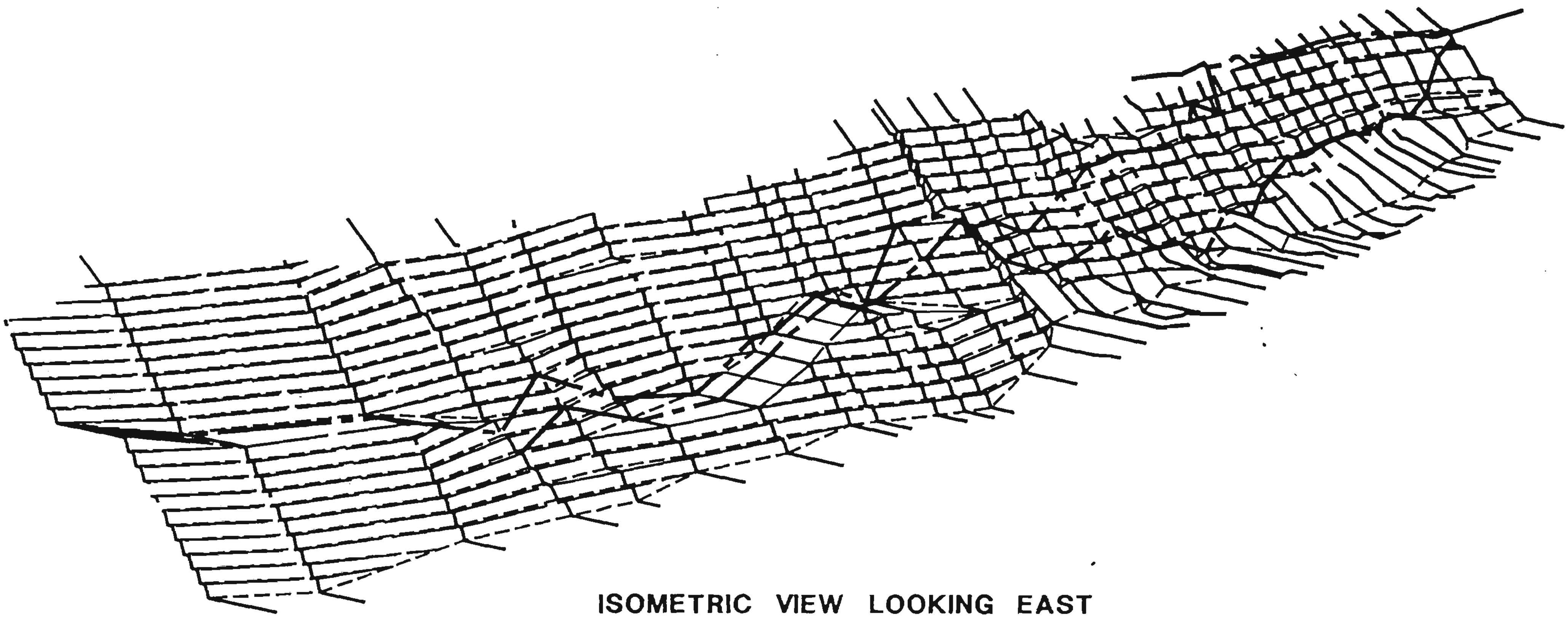
C3



Scale	MT. NEWMAN MINING CO. PTY. LTD.
Drn	MT. WHALEBACK EAST PIT - NORTH WALL
Dwg No	PROPOSED PIT WALL DESIGN OPTION 1

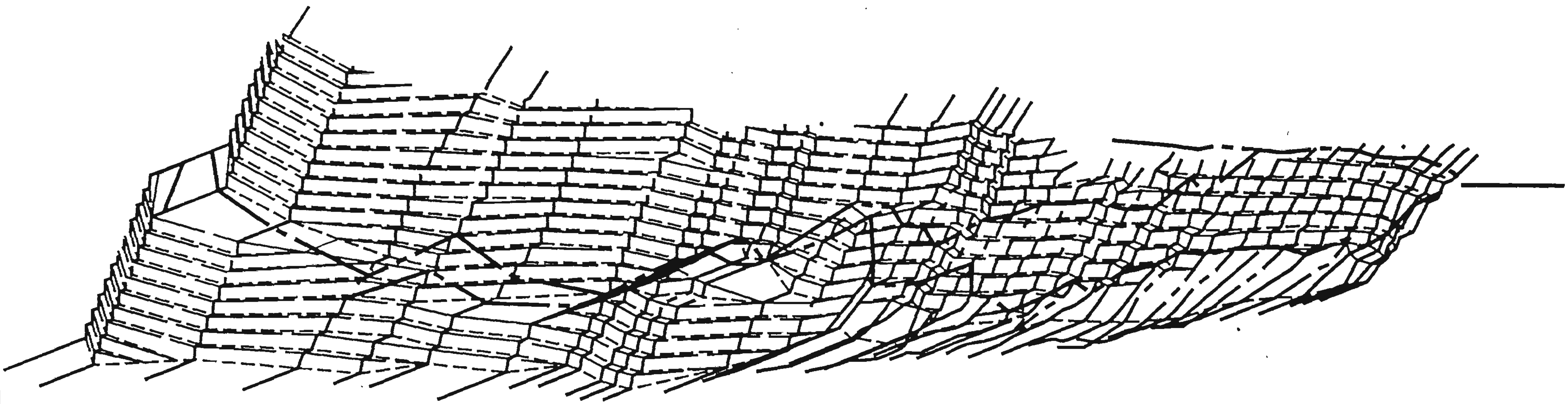
FIGURE

C4



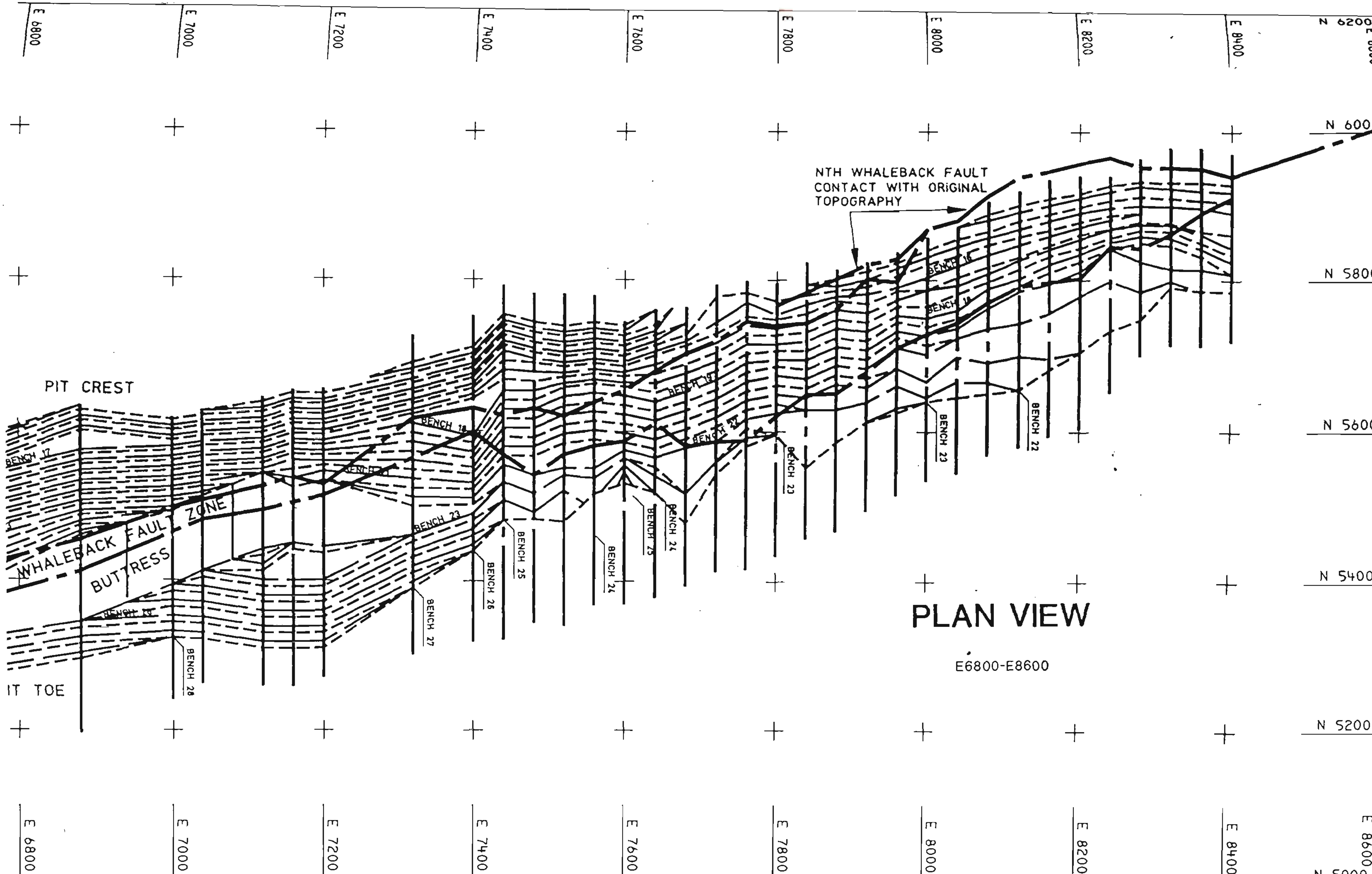
ISOMETRIC VIEW LOOKING EAST

Scale	MT. NEWMAN MINING CO. PTY. LTD.	FIGURE
Drn	MT. WHALEBACK EAST PIT - NORTH WALL	C5
Dwg No	PROPOSED PIT WALL DESIGN OPTION 2	

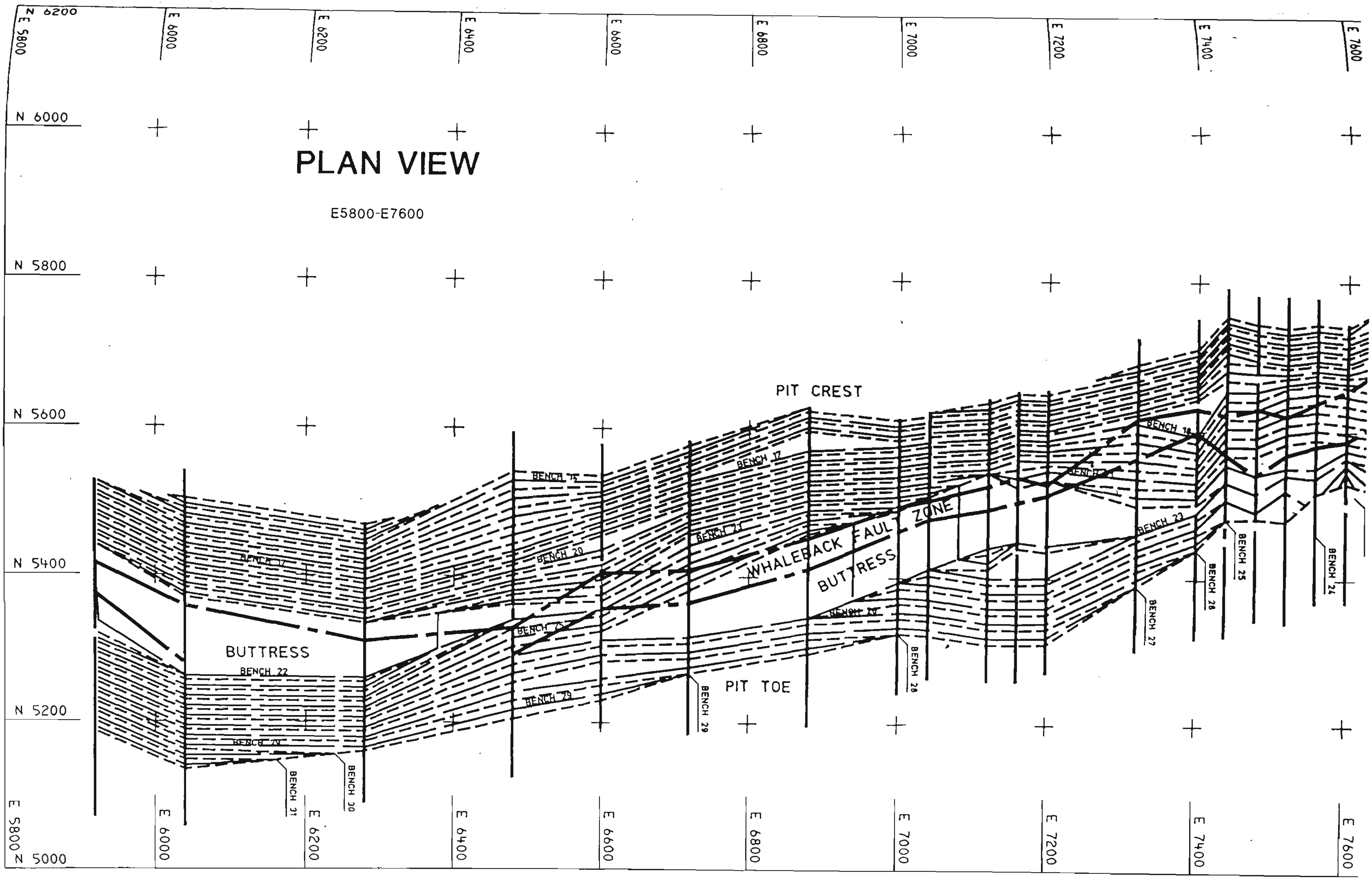


ISOMETRIC VIEW LOOKING WEST

Scale	MT. NEWMAN MINING CO. PTY. LTD.	FIGURE
Drn	MT. WHALEBACK EAST PIT - NORTH WALL	C6
Dwg No	PROPOSED PIT WALL DESIGN OPTION 2	

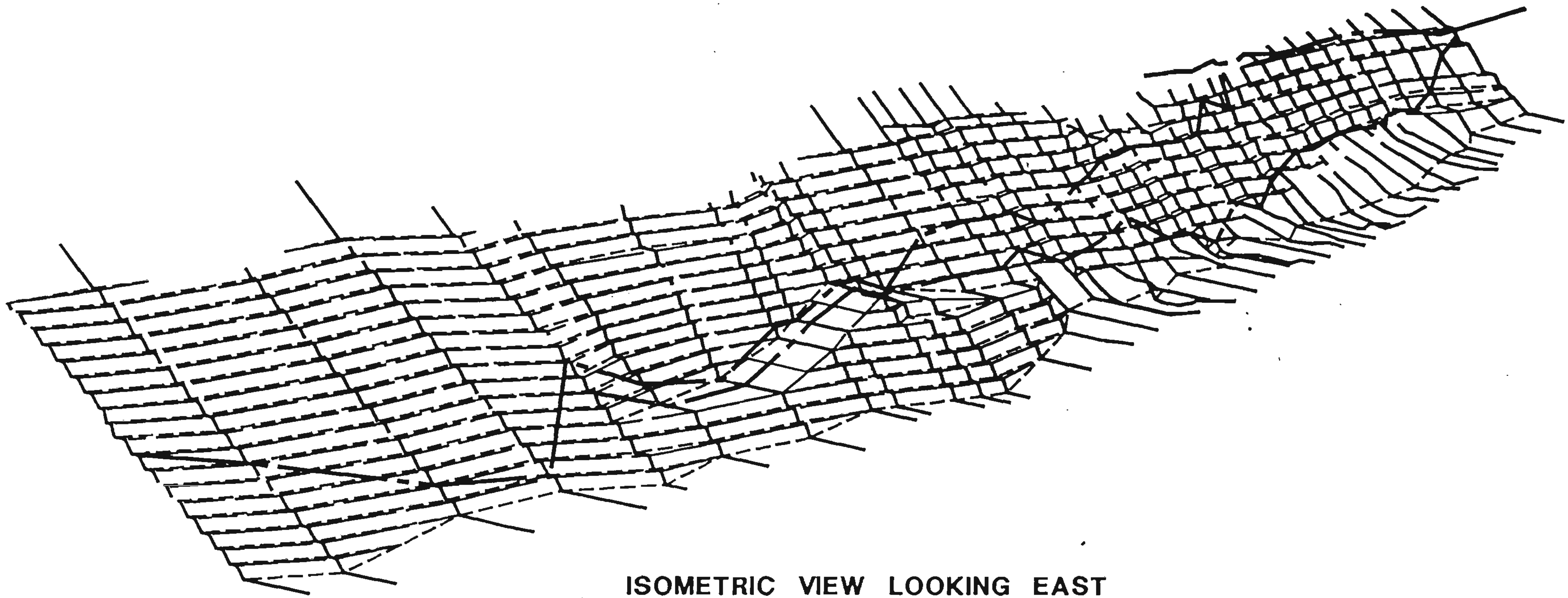


Scale	MT. NEWMAN MINING CO. PTY. LTD.	FIGURE
Drn	MT. WHALEBACK EAST PIT - NORTH WALL	C7
Dwg No	PROPOSED PIT WALL DESIGN OPTION 2	



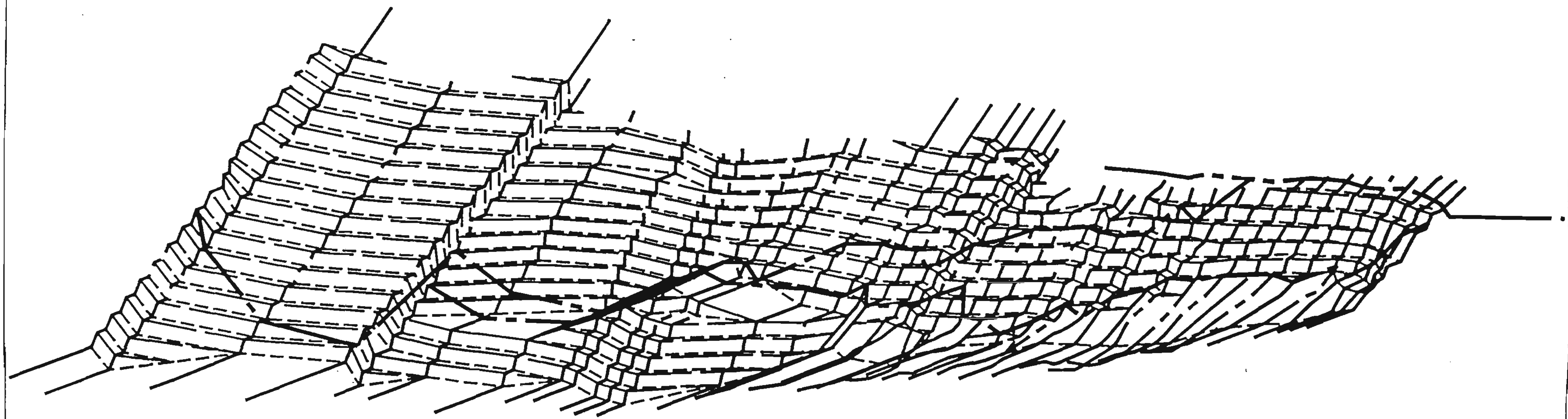
Scale	MT. NEWMAN MINING CO. PTY. LTD.
Drn	MT. WHALEBACK EAST PIT - NORTH WALL
Dwg No	PROPOSED PIT WALL DESIGN OPTION 2

FIGURE C8



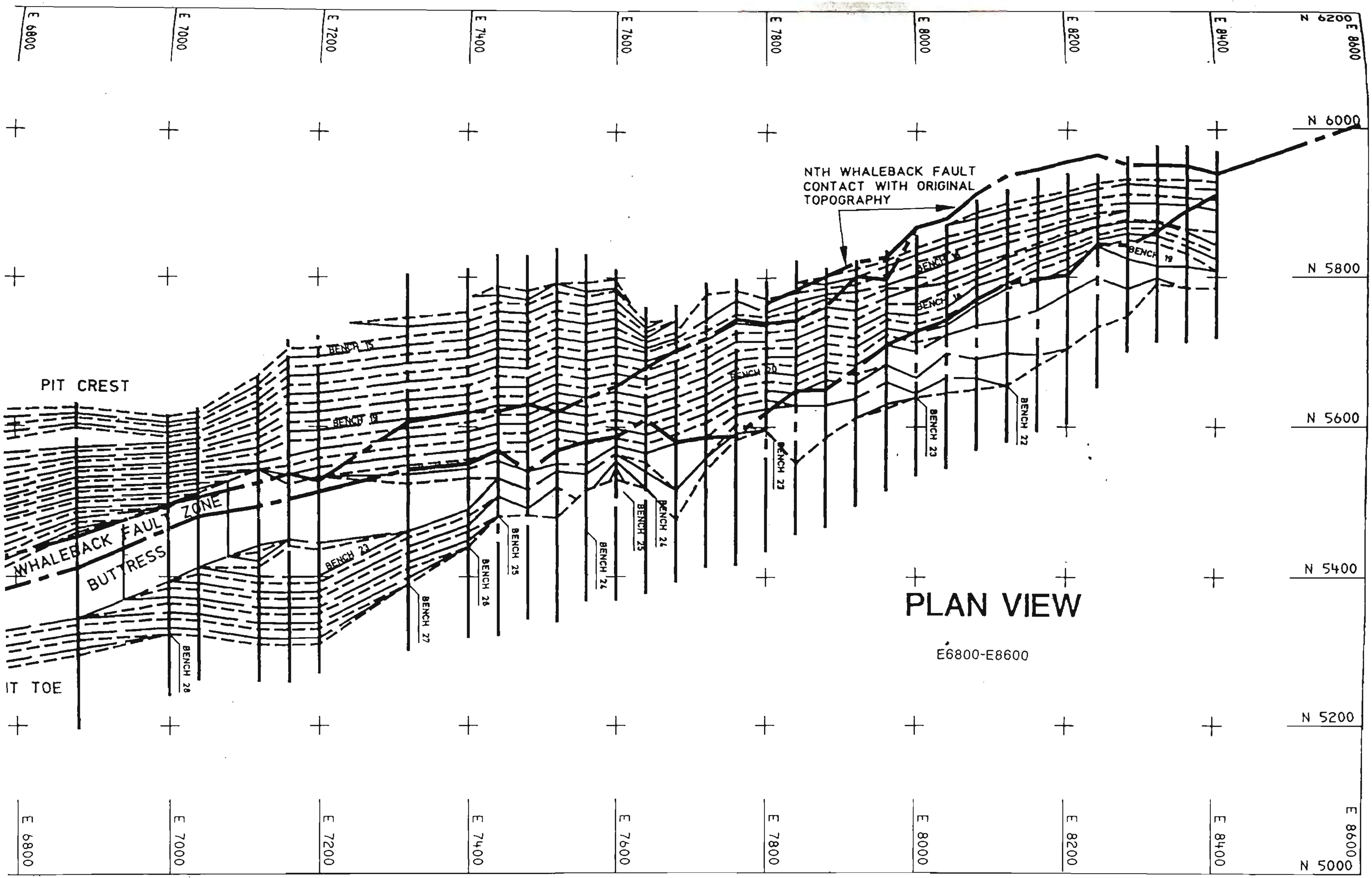
ISOMETRIC VIEW LOOKING EAST

Scale		MT. NEWMAN MINING CO. PTY. LTD.	FIGURE
Drn		MT. WHALEBACK EAST PIT - NORTH WALL	C9
Dwg No		PROPOSED PIT WALL DESIGN OPTION 3	

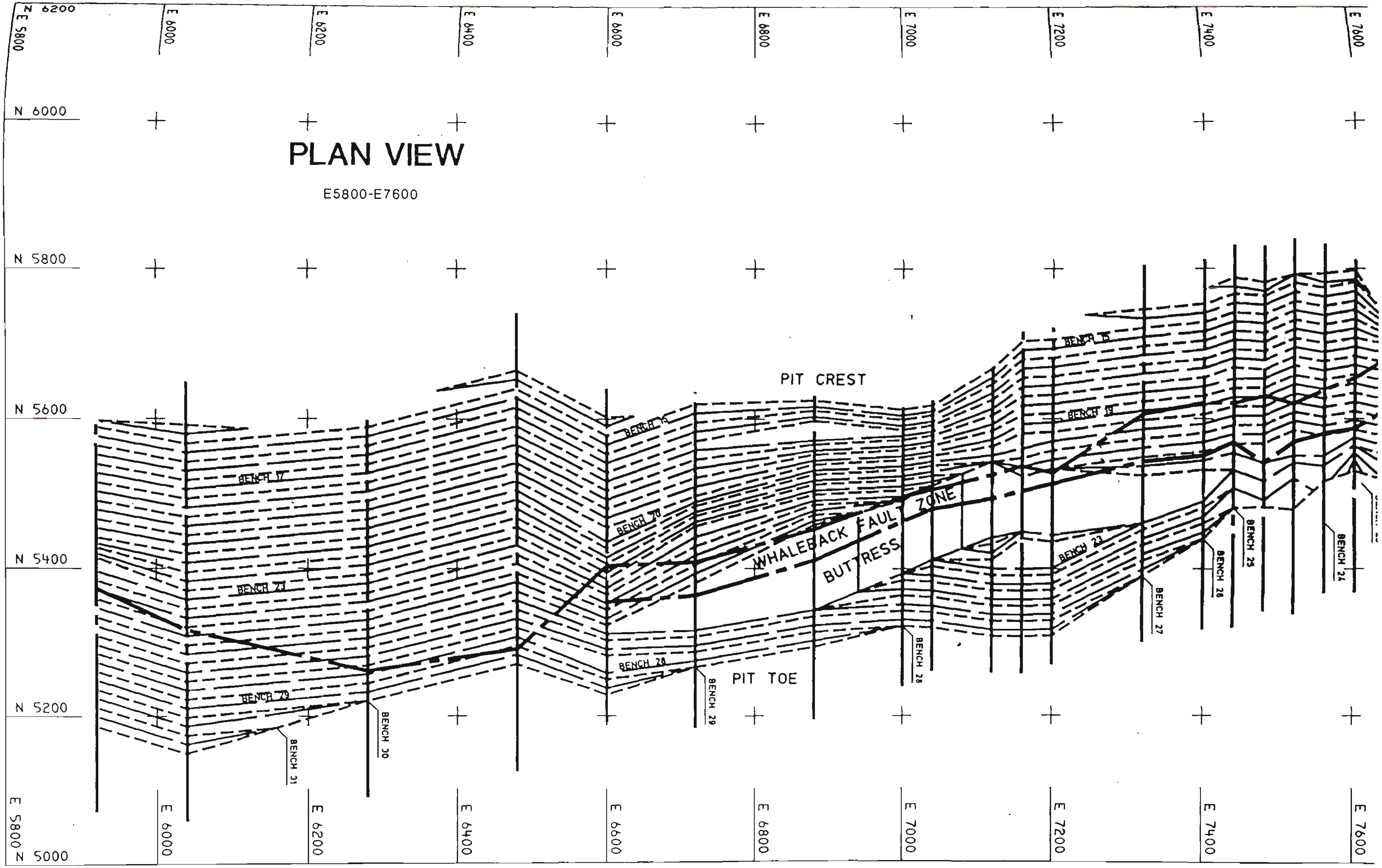


ISOMETRIC VIEW LOOKING WEST

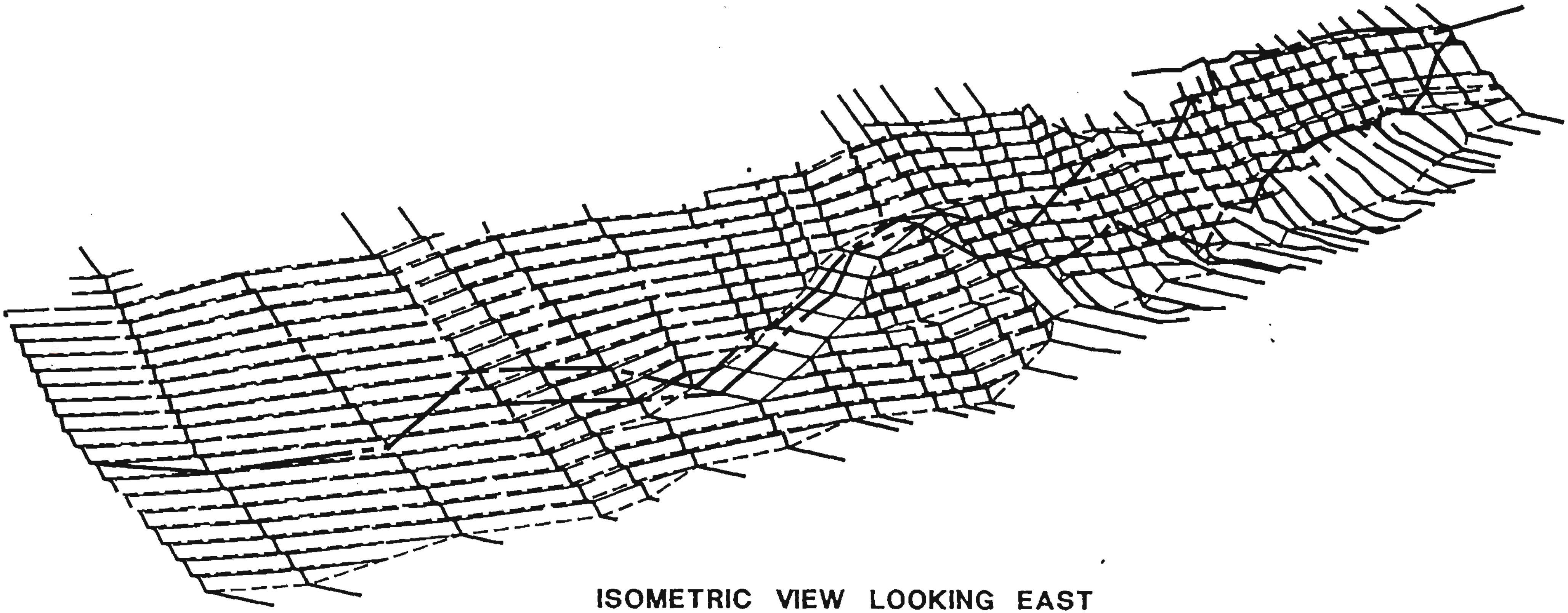
	Scale	MT. NEWMAN MINING CO. PTY. LTD.	FIGURE
	Drn	MT. WHALEBACK EAST PIT - NORTH WALL	C10
	Dwg No	PROPOSED PIT WALL DESIGN OPTION 3	



Scale	MT. NEWMAN MINING CO. PTY. LTD.	FIGURE
Drn	MT. WHALEBACK EAST PIT - NORTH WALL	C11
Dwg No	PROPOSED PIT WALL DESIGN OPTION 3	

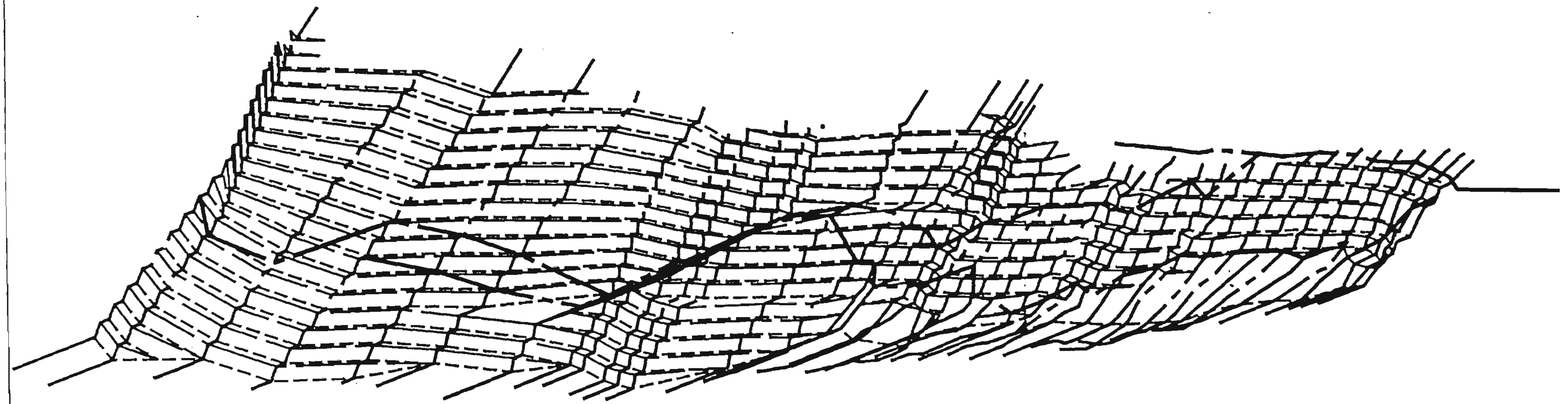


Scale	MT. NEWMAN MINING CO. PTY. LTD.	FIGURE
Drn	MT. WHALEBACK EAST PIT - NORTH WALL	C12
Dwg No.	PROPOSED PIT WALL DESIGN OPTION 3	



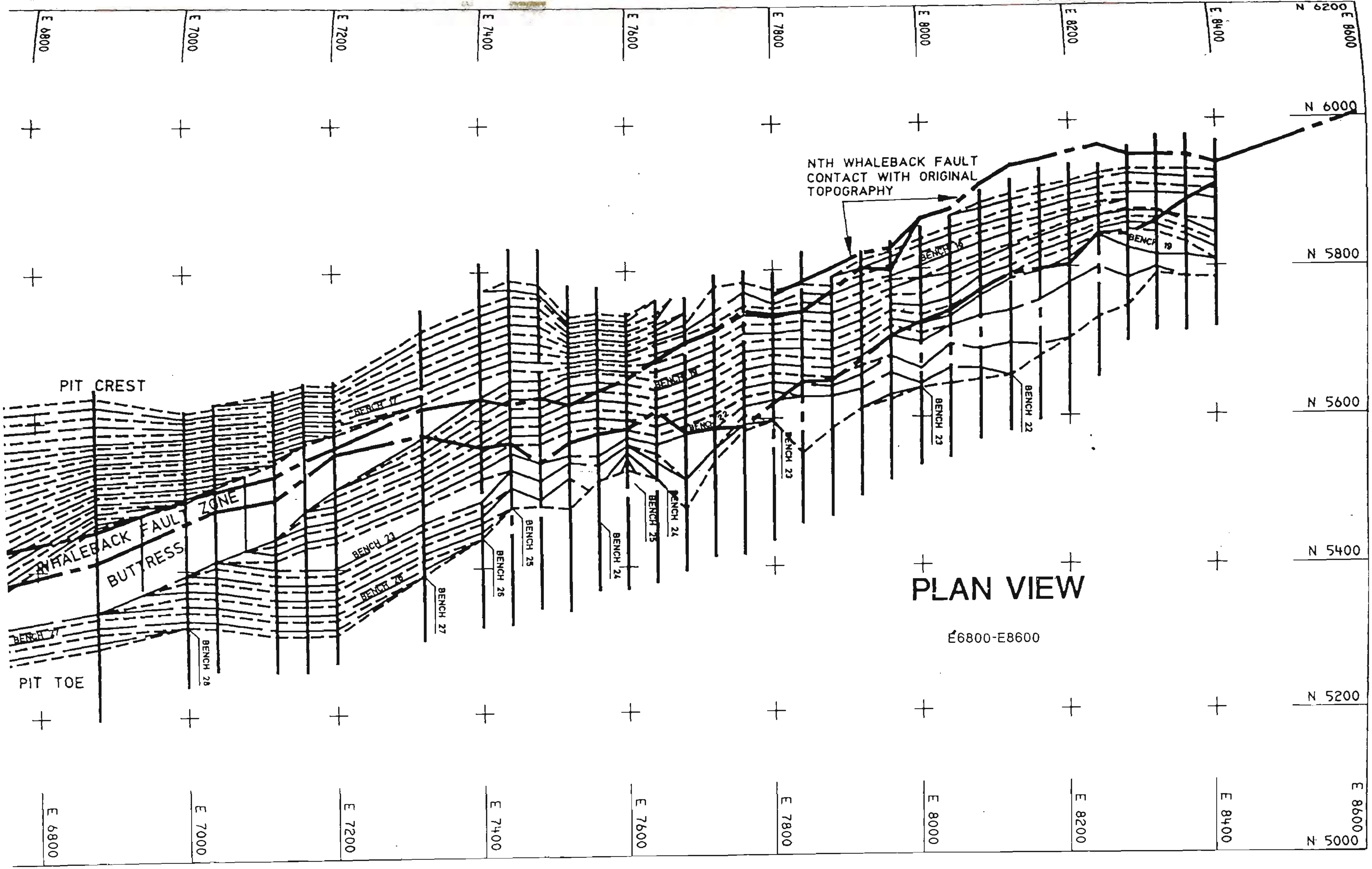
ISOMETRIC VIEW LOOKING EAST

	Scale	MT. NEWMAN MINING CO. PTY. LTD.	FIGURE
	Drn	MT. WHALEBACK EAST PIT - NORTH WALL	C13
	Dwg No	PROPOSED PIT WALL DESIGN OPTION 4	



ISOMETRIC VIEW LOOKING WEST

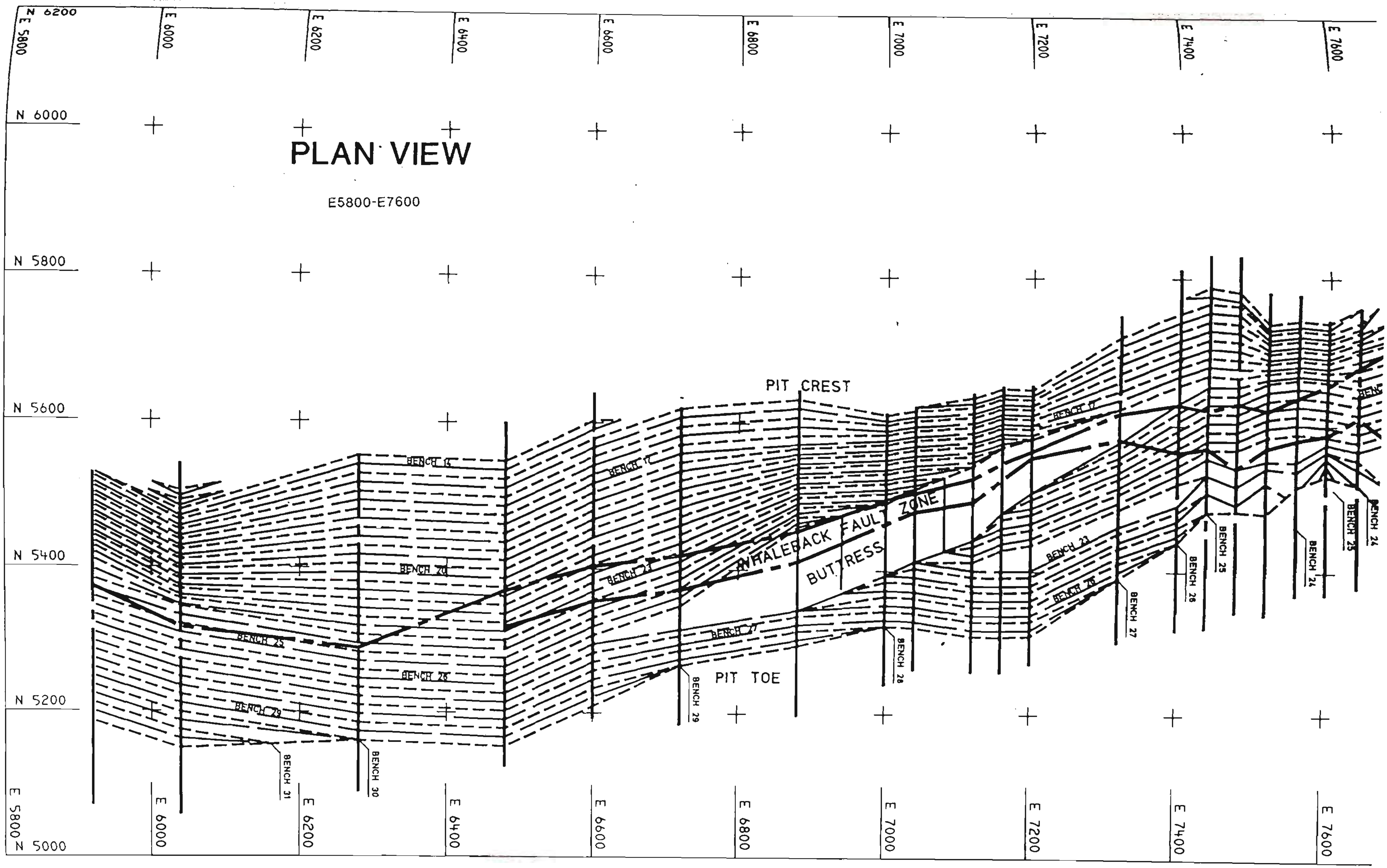
	Scale	MT. NEWMAN MINING CO. PTY. LTD.	FIGURE
	Drn	MT. WHALEBACK EAST PIT - NORTH WALL	C14
	Dwg No	PROPOSED PIT WALL DESIGN OPTION 4	



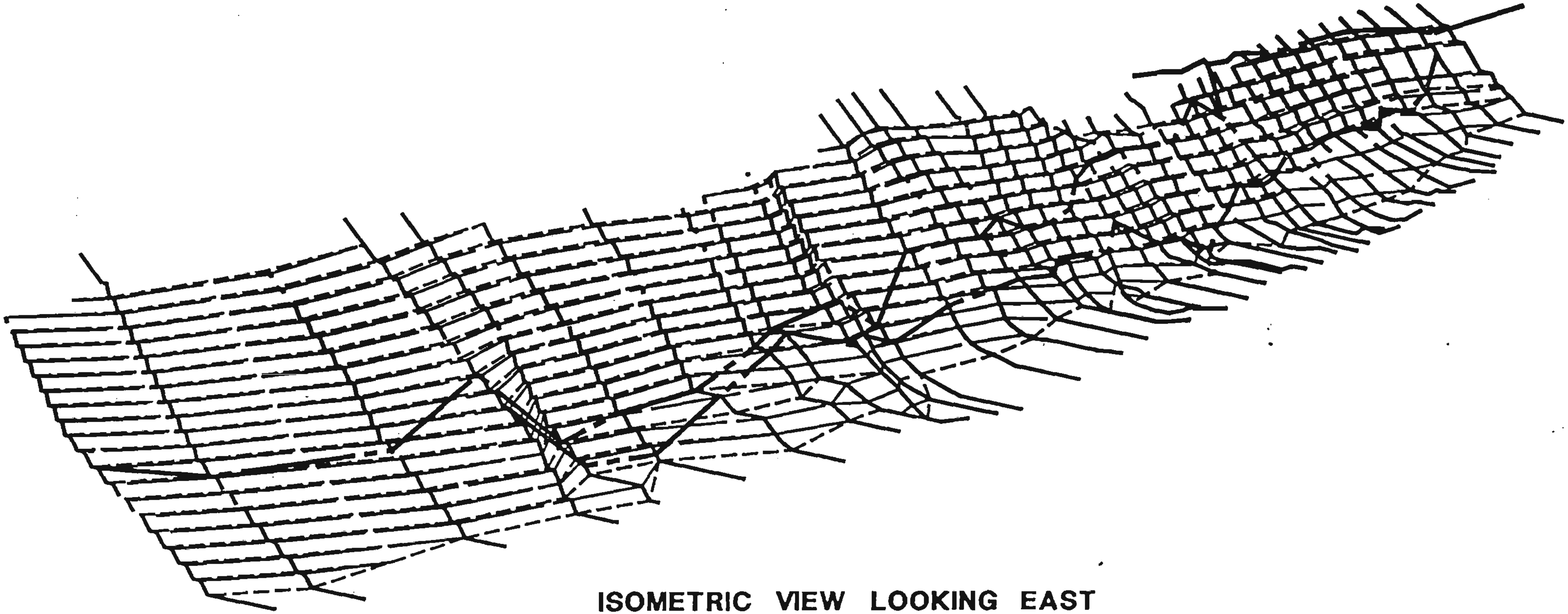
Scale	
Drn	
Dwg No	

MT. NEWMAN MINING CO. PTY. LTD.
MT. WHALEBACK EAST PIT - NORTH WALL
PROPOSED PIT WALL DESIGN OPTION 4

FIGURE
C15

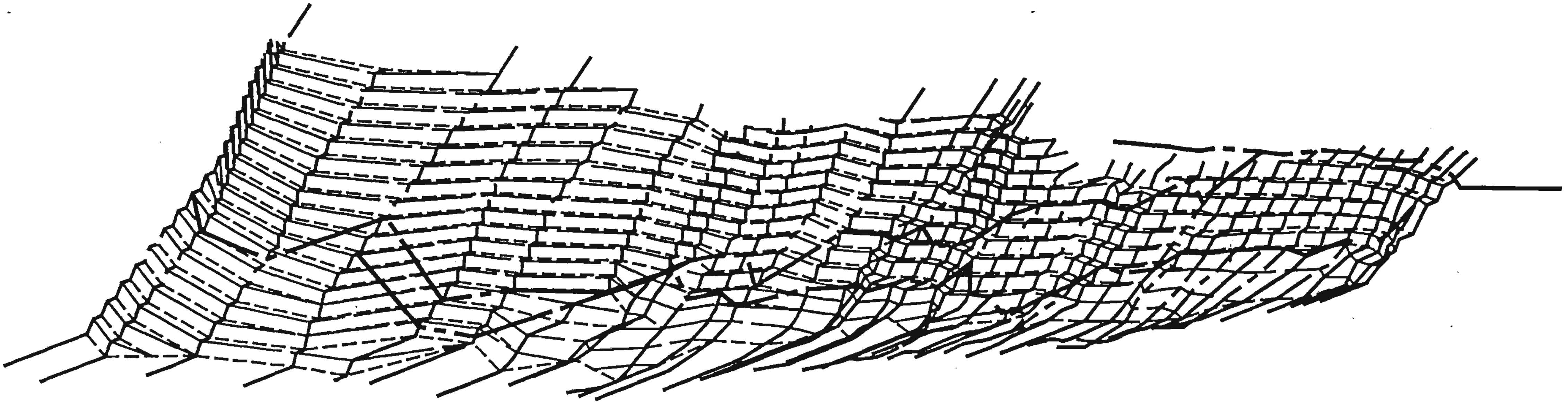


Scale	MT. NEWMAN MINING CO. PTY. LTD.	FIGURE
Drn	MT. WHALEBACK EAST PIT - NORTH WALL	C16
Dwg No	PROPOSED PIT WALL DESIGN OPTION 4	



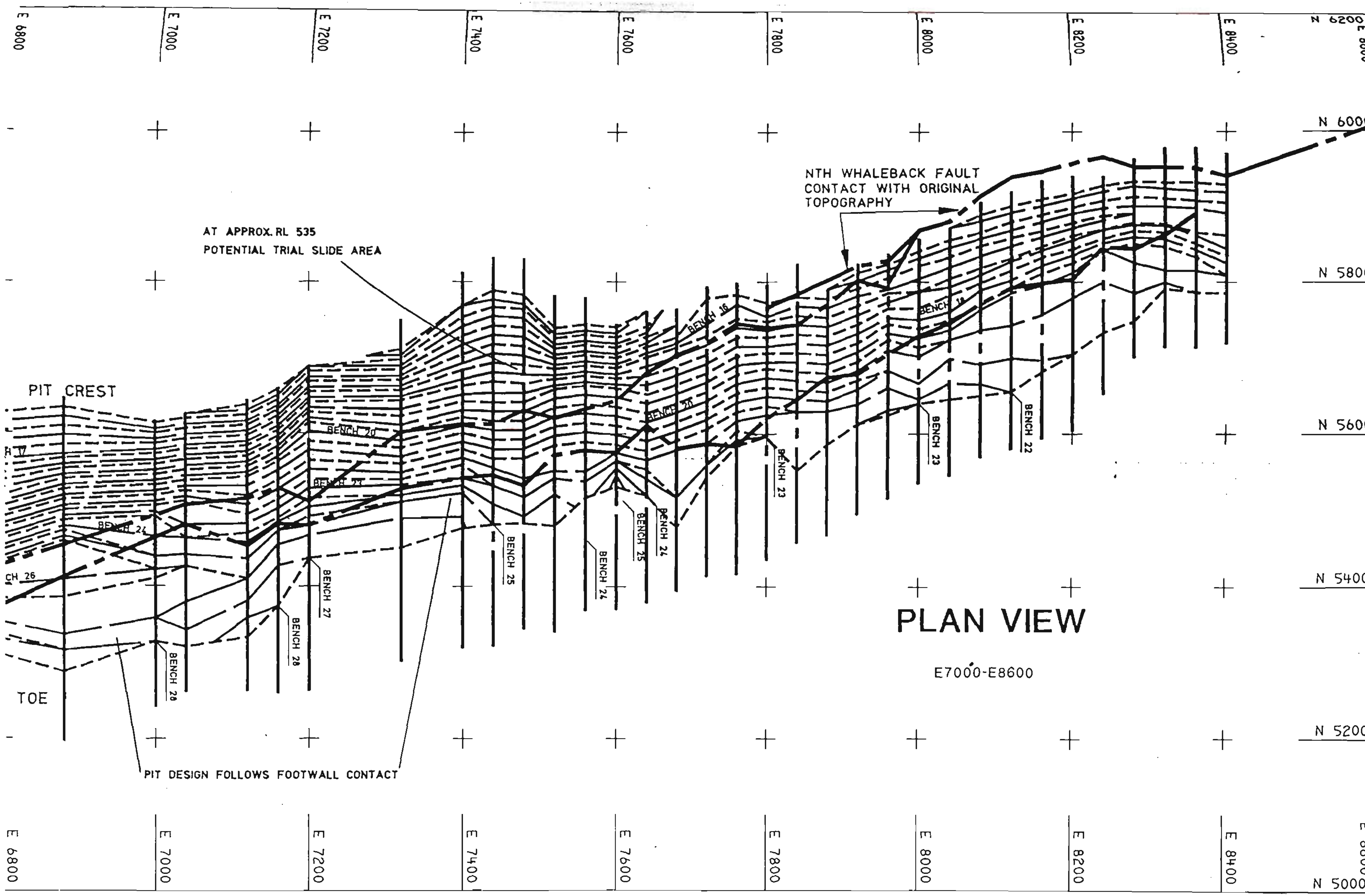
ISOMETRIC VIEW LOOKING EAST

	Scale	MT. NEWMAN MINING CO. PTY. LTD.	FIGURE
	Drn	MT. WHALEBACK EAST PIT - NORTH WALL	C17
	Dwg No	PROPOSED PIT WALL DESIGN OPTION 4A	



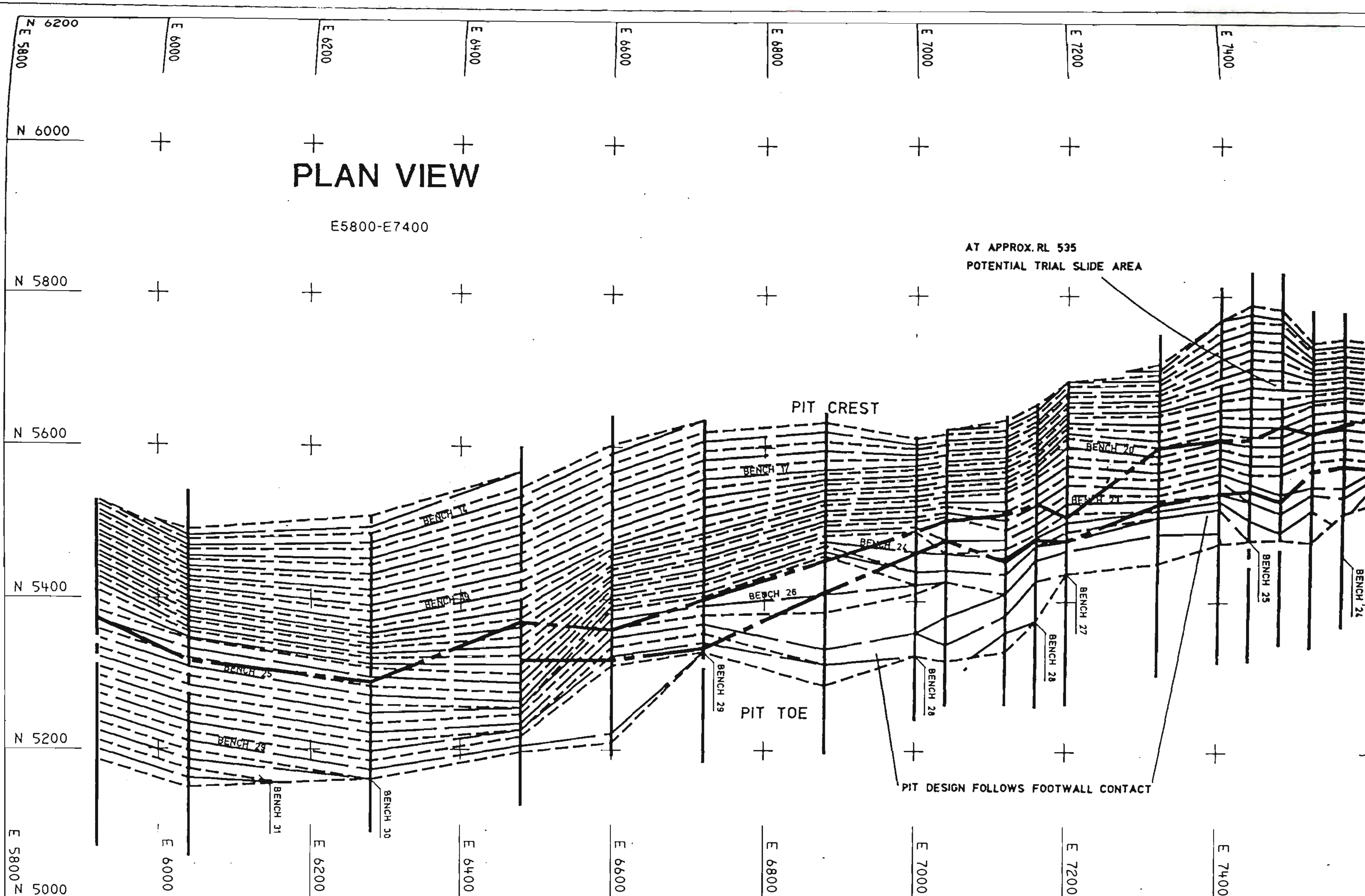
ISOMETRIC VIEW LOOKING WEST

	Scale	MT. NEWMAN MINING CO. PTY. LTD.	FIGURE
	Drn	MT. WHALEBACK EAST PIT - NORTH WALL	C18
	Dwg No	PROPOSED PIT WALL DESIGN OPTION 4A	



Scale	MT. NEWMAN MINING CO. PTY. LTD.
Drn	MT. WHALEBACK EAST PIT - NORTH WALL
Dwg No.	PROPOSED PIT WALL DESIGN OPTION 4A

FIGURE C19

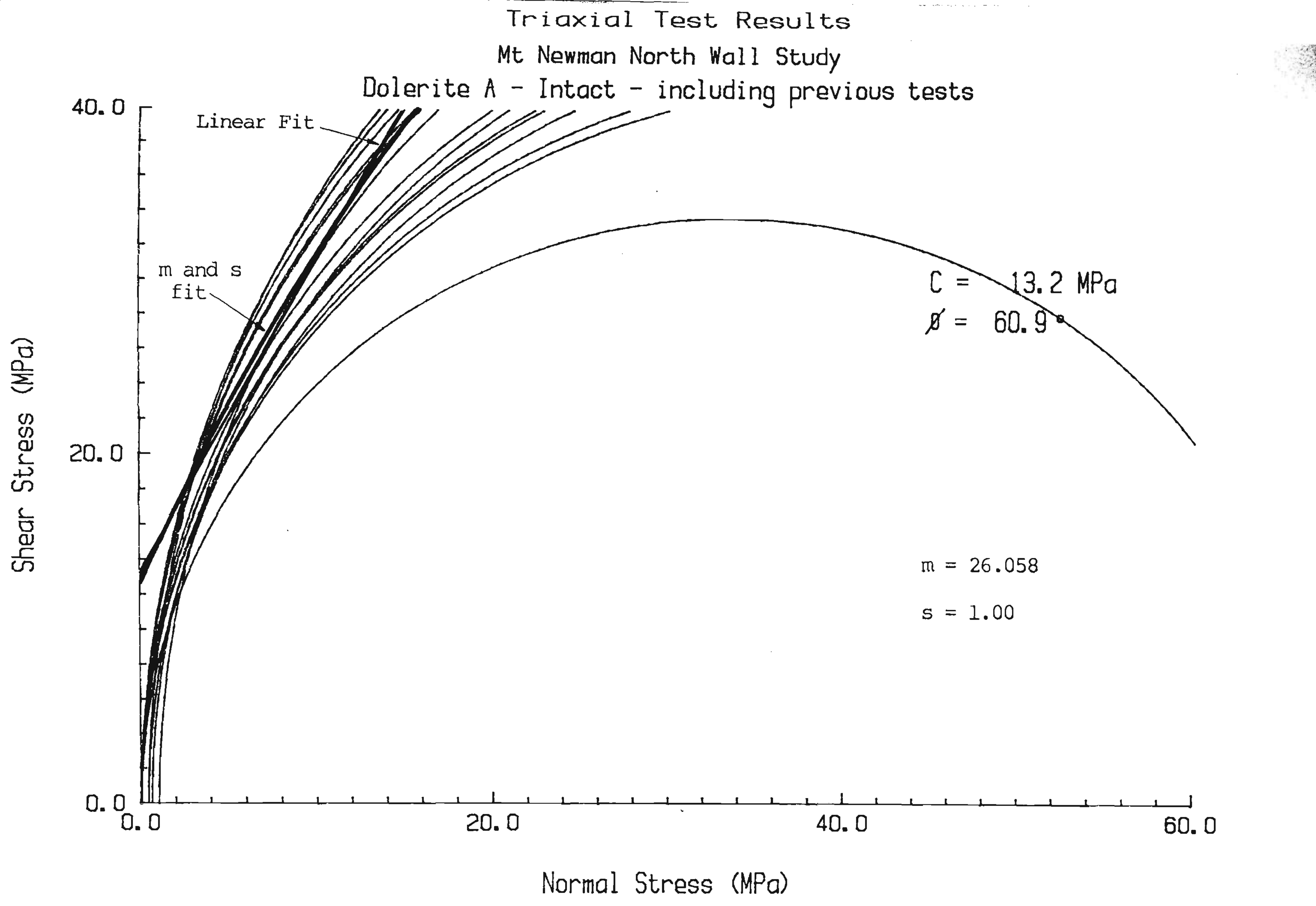


Scale	
Drn	
Dwg No	

MT. NEWMAN MINING CO. PTY. LTD.
MT. WHALEBACK EAST PIT - NORTH WALL
PROPOSED PIT WALL DESIGN OPTION 4A

FIGURE C20

APPENDIX D



Triaxial Test Results
Mt Newman North Wall Study
Dolerite A - Intact - including previous tests
Linear Fit
m and s fit
C = 13.2 MPa
Ø = 60.9°
m = 26.058
s = 1.00
Shear Stress (MPa)
Normal Stress (MPa)
0.0
20.0
40.0
60.0

Figure D1

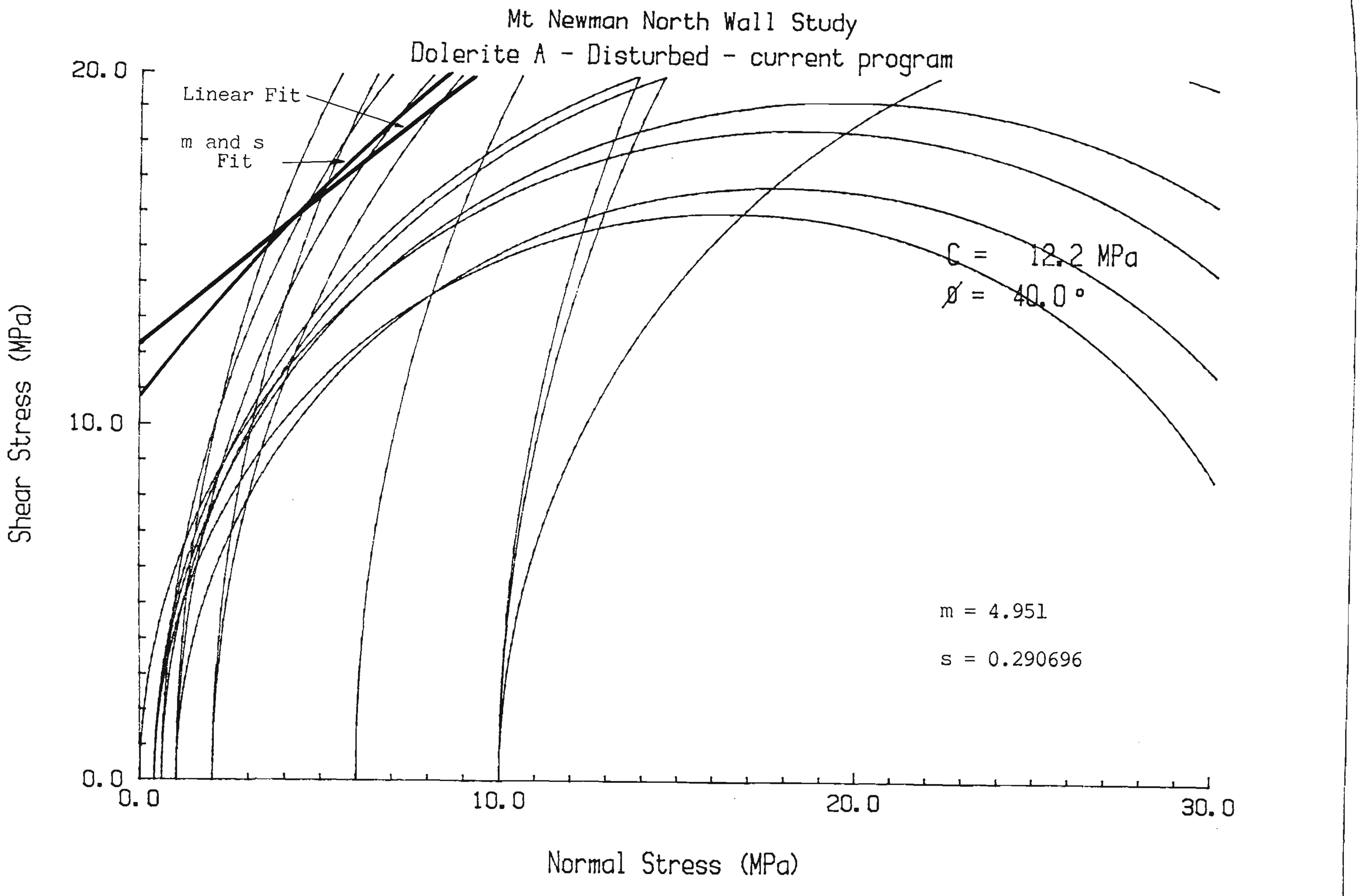

Mt Newman North Wall Study
Dolerite A - Disturbed - current program
Linear Fit
m and s Fit
C = 12.2 MPa
Ø = 40.0°
m = 4.951
s = 0.290696
Shear Stress (MPa)
Normal Stress (MPa)
20.0
10.0
0.0
0.0
10.0
20.0
30.0

Figure D2

Mt Newman North Wall Study

Dolerite B - Intact - including previous tests

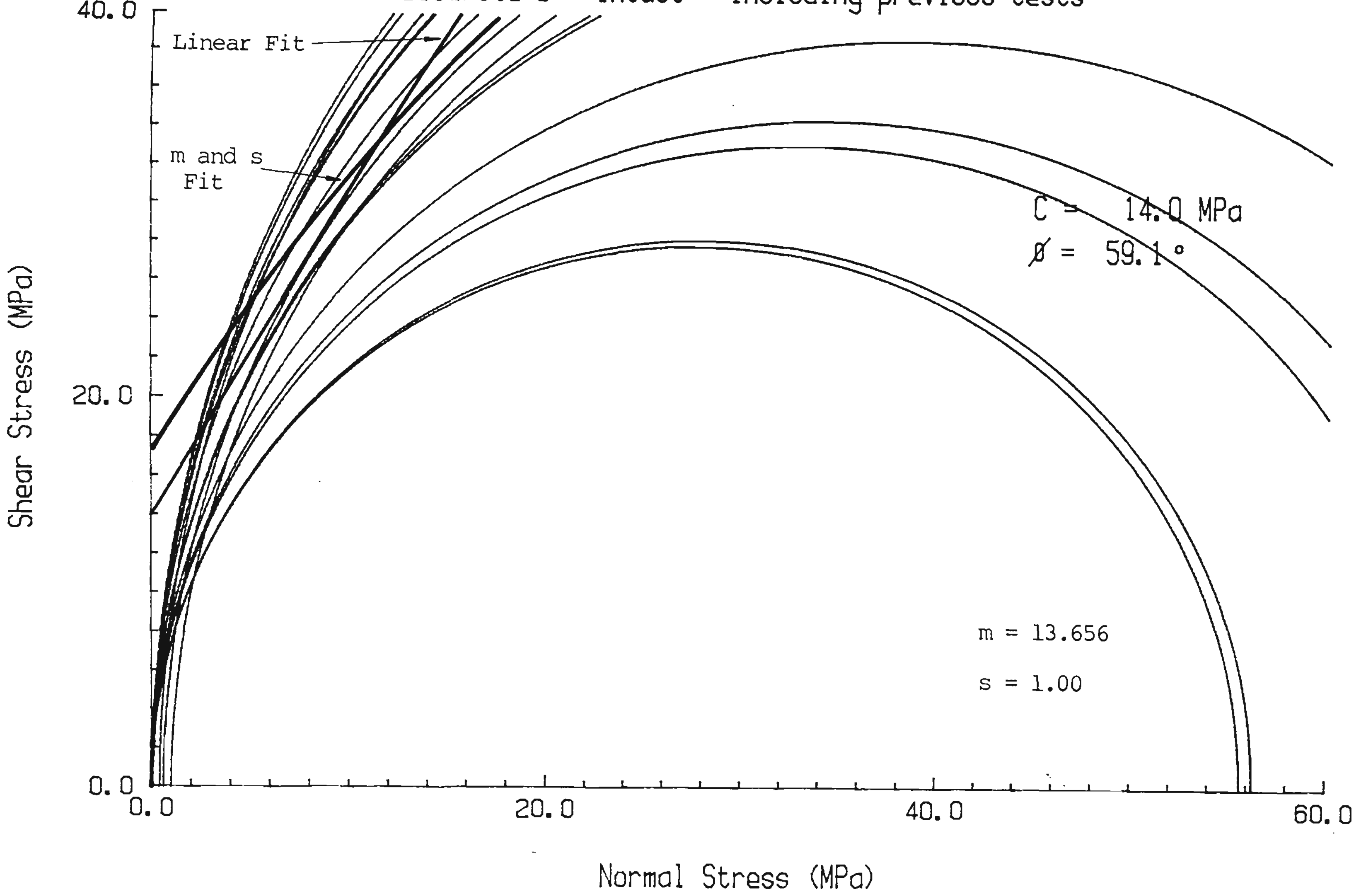


Figure D3

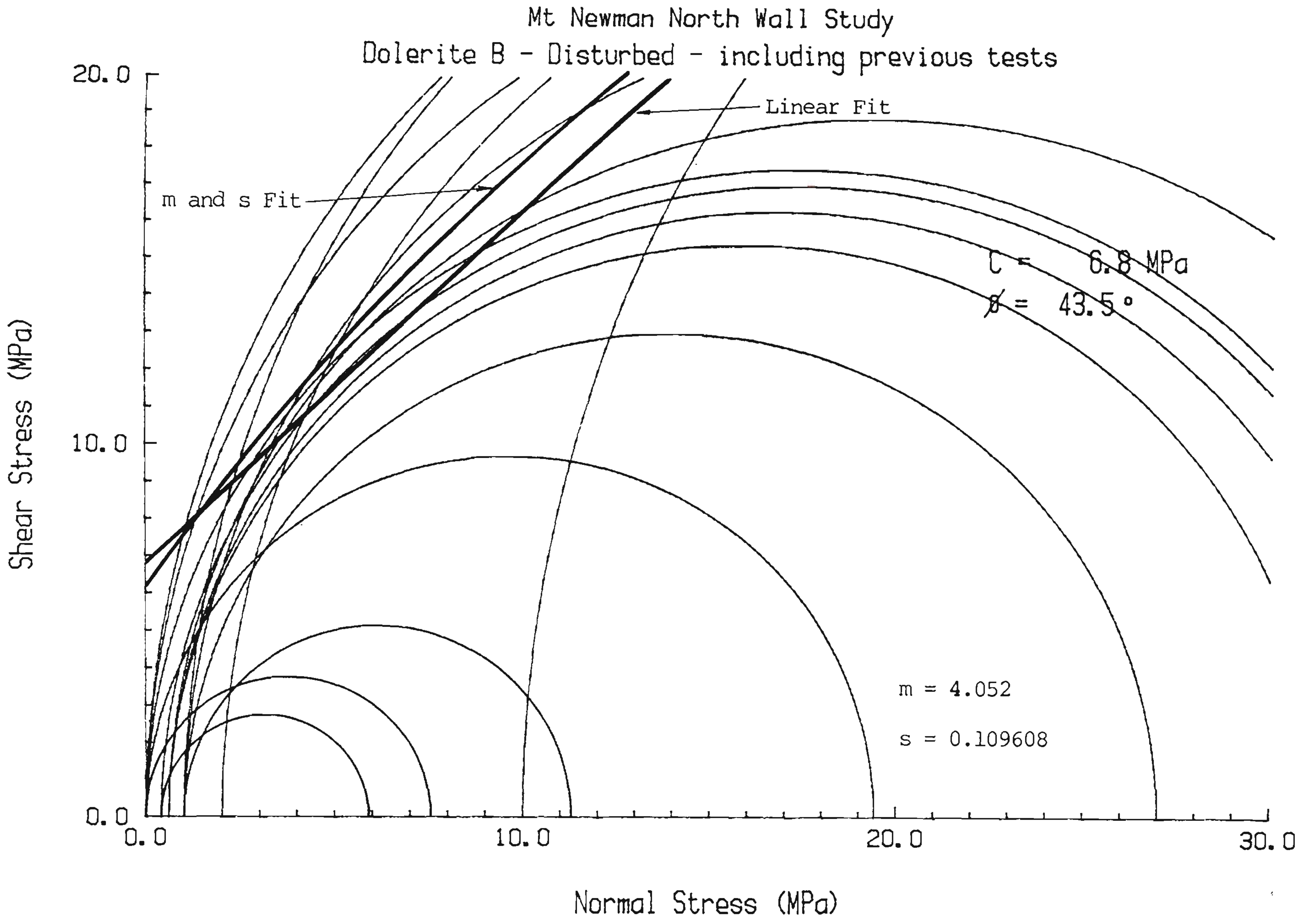
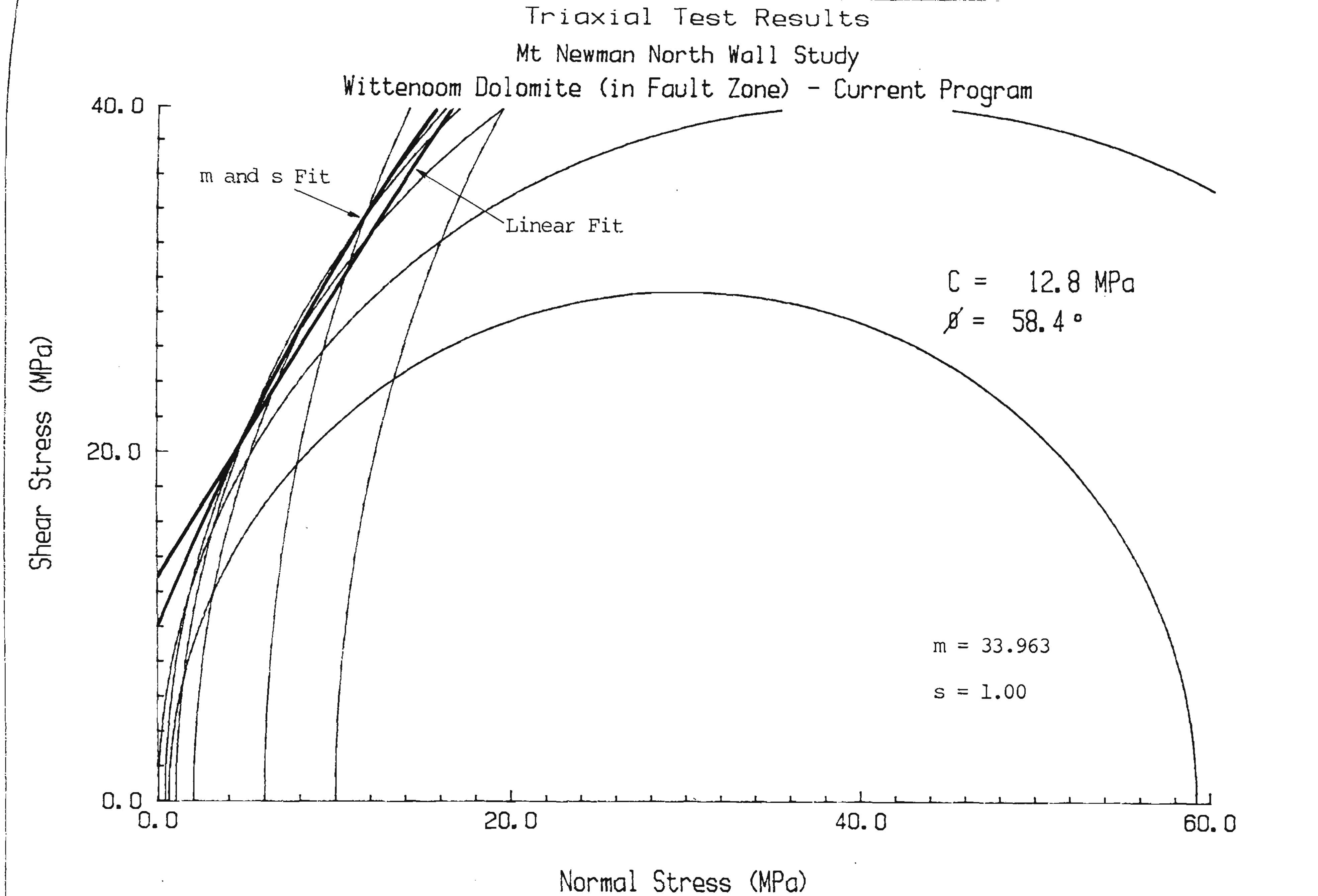

Mt Newman North Wall Study
Dolerite B - Disturbed - including previous tests
Linear Fit
m and s Fit
C = 6.8 MPa
ø = 43.5 °
m = 4.052
s = 0.109608
Shear Stress (MPa)
Normal Stress (MPa)
20.0
10.0
0.0
0.0
10.0
20.0
30.0

Figure D4



Triaxial Test Results
Mt Newman North Wall Study
Witteneoom Dolomite (in Fault Zone) - Current Program
40.0
20.0
0.0
Shear Stress (MPa)
m and s Fit
Linear Fit
C = 12.8 MPa
Ø = 58.4 °
m = 33.963
s = 1.00
0.0
20.0
40.0
60.0
Normal Stress (MPa)

Figure D5

Mt Newman North Wall Study

Jeerinah Shale A - Intact - All Results

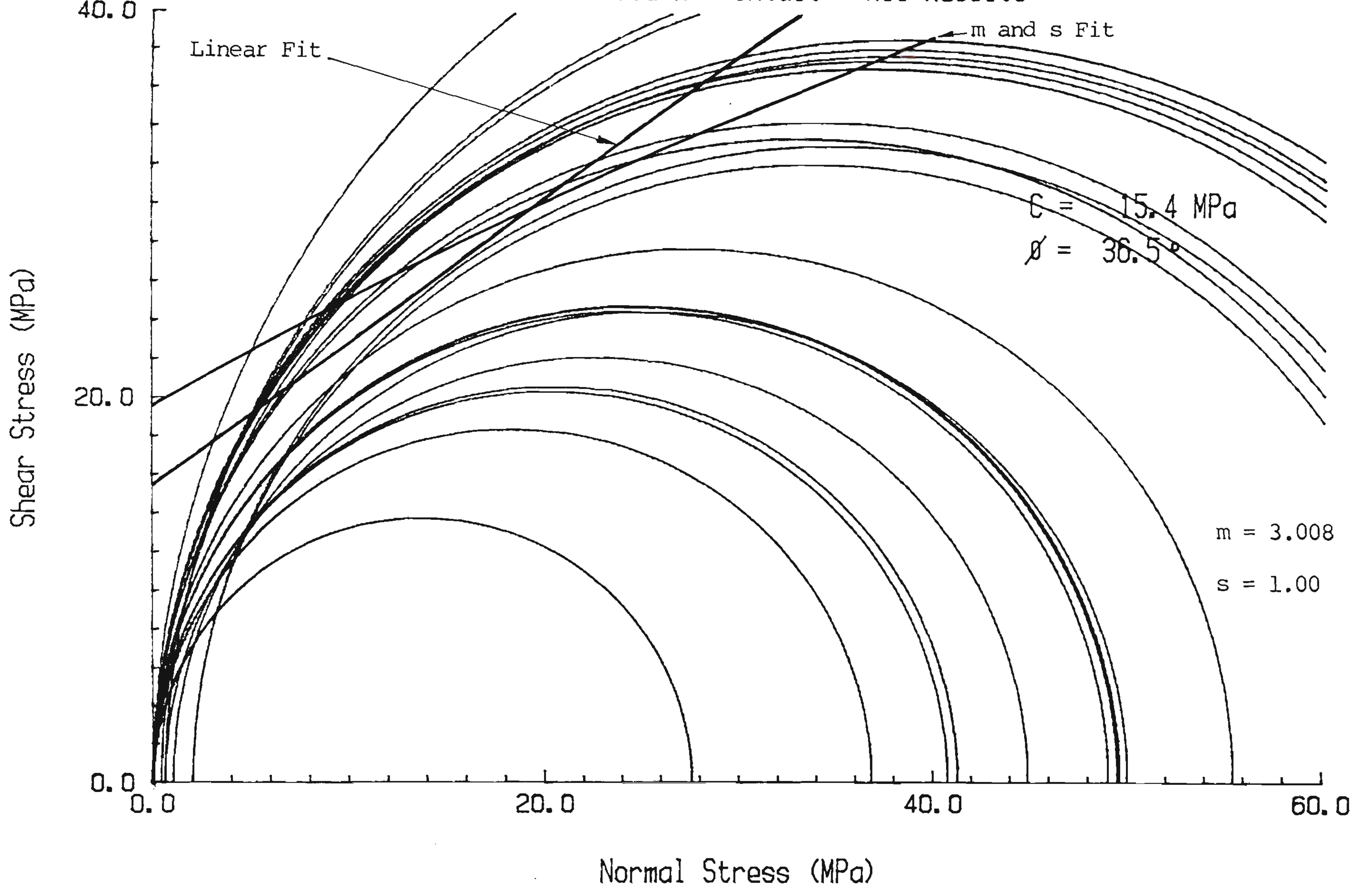


Figure D6

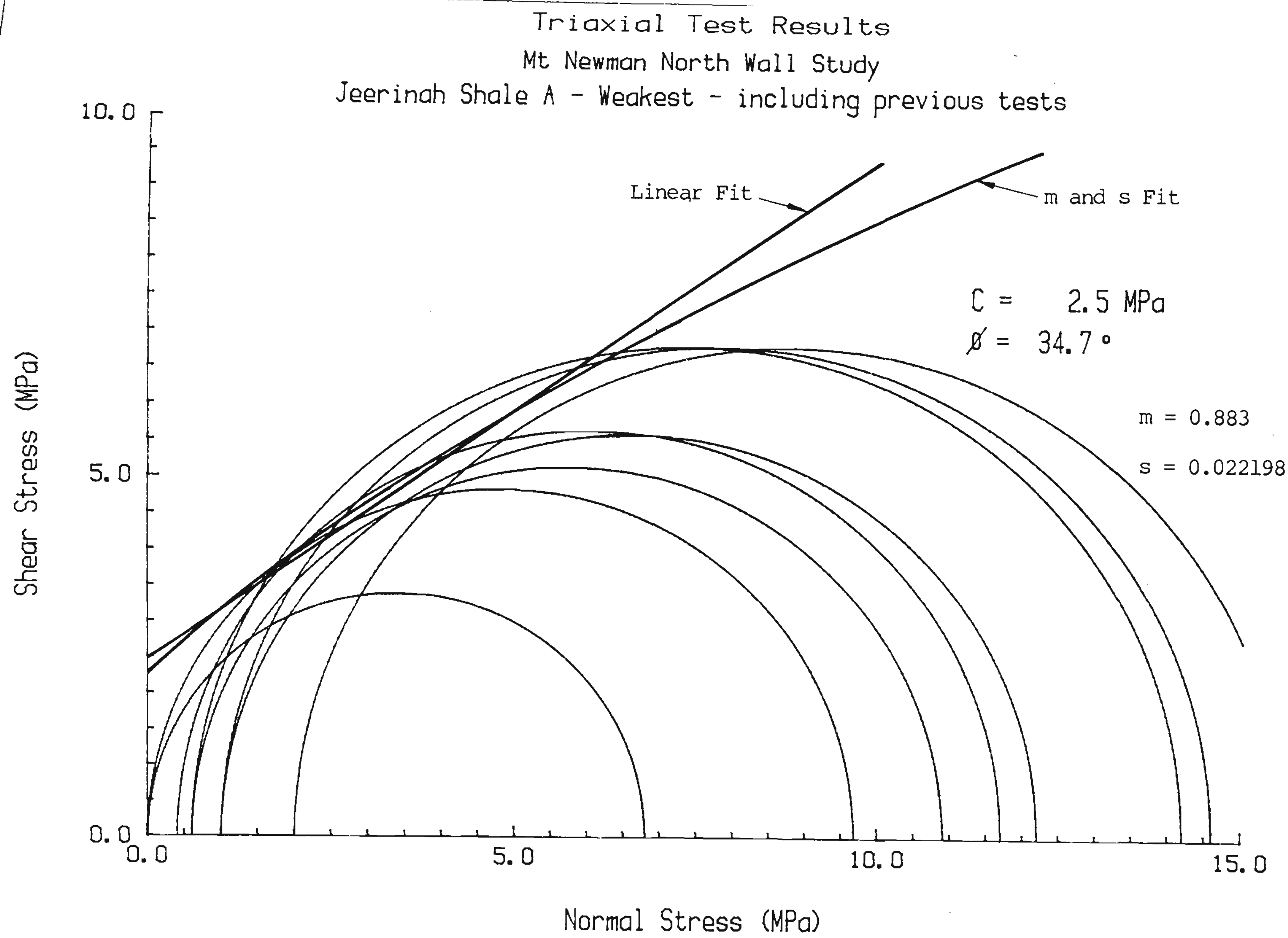
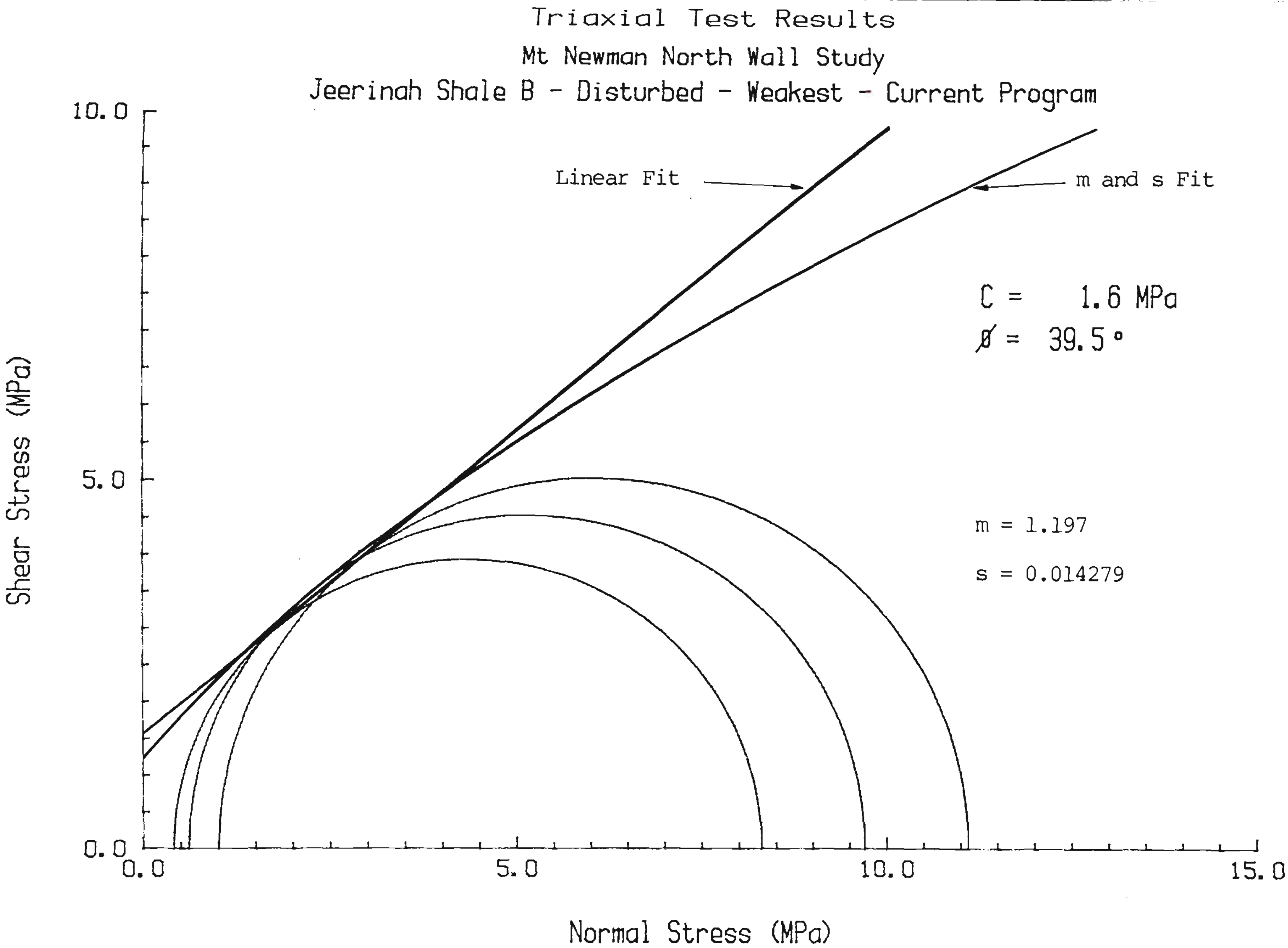

Triaxial Test Results
Mt Newman North Wall Study
Jeerinah Shale A - Weakest - including previous tests
Linear Fit
m and s Fit
C = 2.5 MPa
Ø = 34.7 °
m = 0.883
s = 0.022198
Shear Stress (MPa)
10.0
5.0
0.0
0.0
5.0
10.0
15.0
Normal Stress (MPa)

Figure D7



Triaxial Test Results
Mt Newman North Wall Study
Jeerinah Shale B - Disturbed - Weakest - Current Program
Linear Fit
m and s Fit
C = 1.6 MPa
ø = 39.5°
m = 1.197
s = 0.014279
Shear Stress (MPa)
Normal Stress (MPa)
10.0
5.0
0.0
0.0
5.0
10.0
15.0

Figure D8

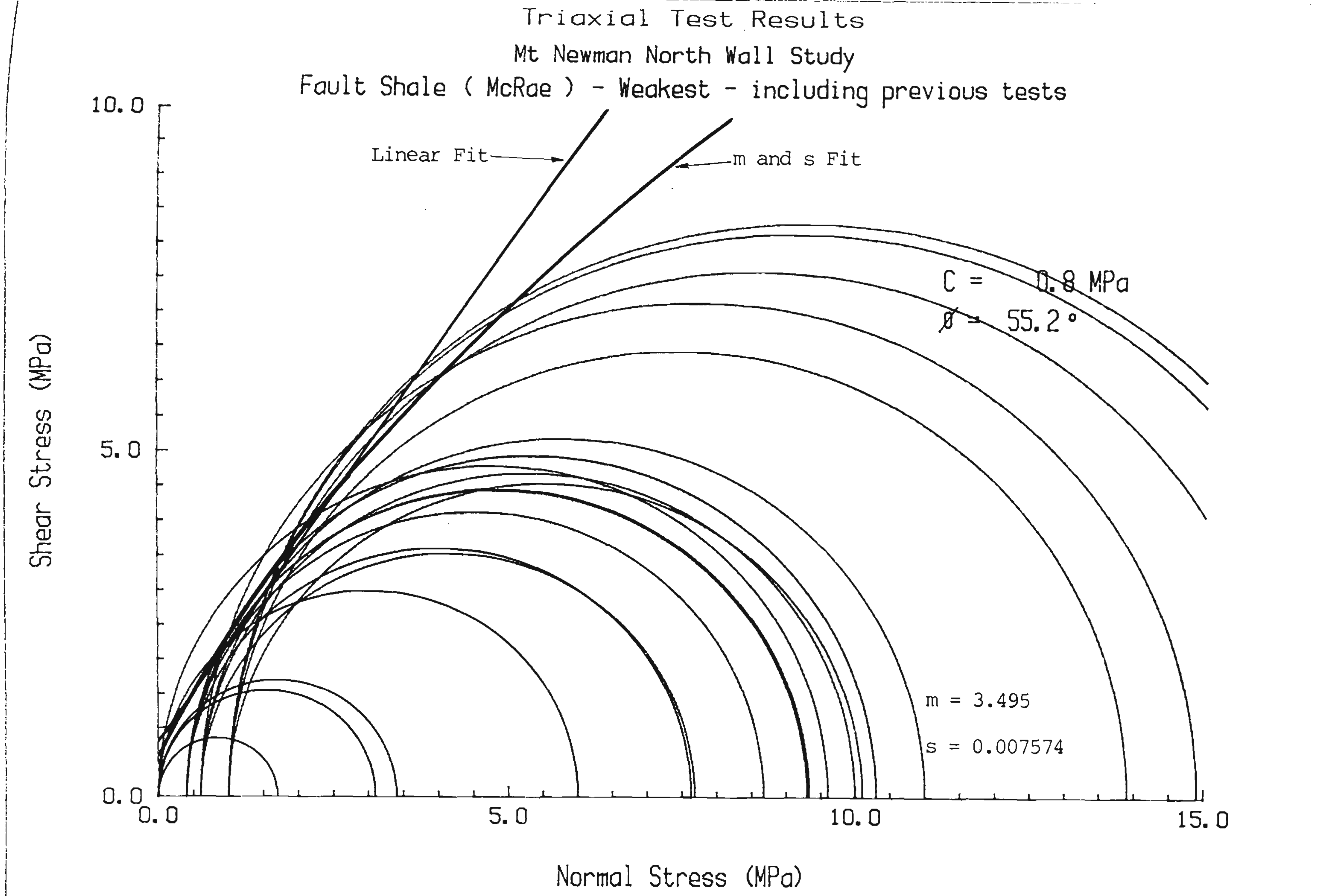

Triaxial Test Results
Mt Newman North Wall Study
Fault Shale (McRae) - Weakest - including previous tests
Linear Fit
m and s Fit
C = 0.8 MPa
ø = 55.2°
m = 3.495
s = 0.007574
Shear Stress (MPa)
Normal Stress (MPa)
10.0
5.0
0.0
0.0
5.0
10.0
15.0

Figure D9

Triaxial Test Results

Mt Newman North Wall Study

Fault Shale (Sylvia) - Weakest - including previous tests

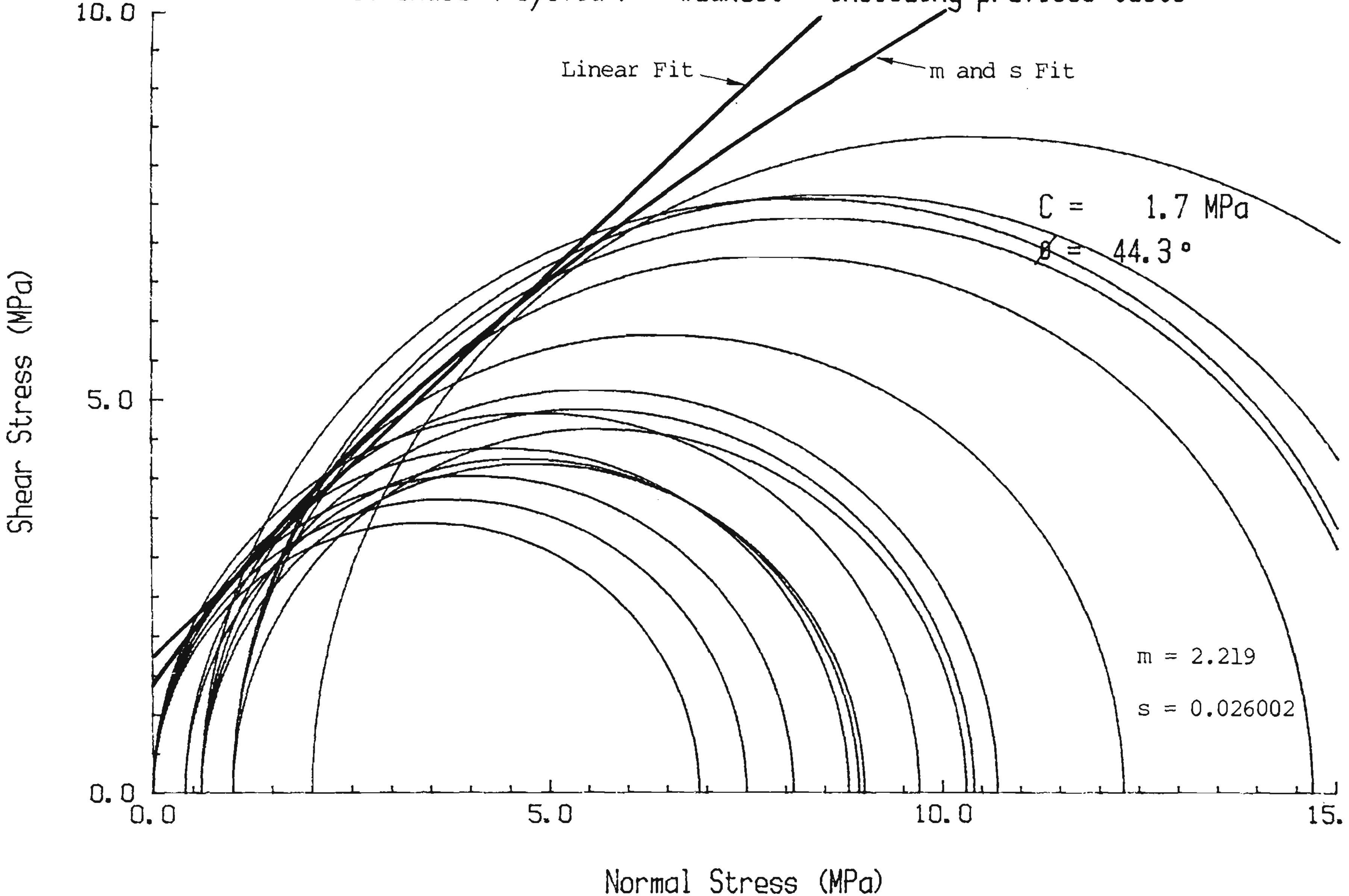


Figure D10

Triaxial Test Results

Mt Newman North Wall Study

Fault Shale (Jeerinah) - Weakest - including previous tests

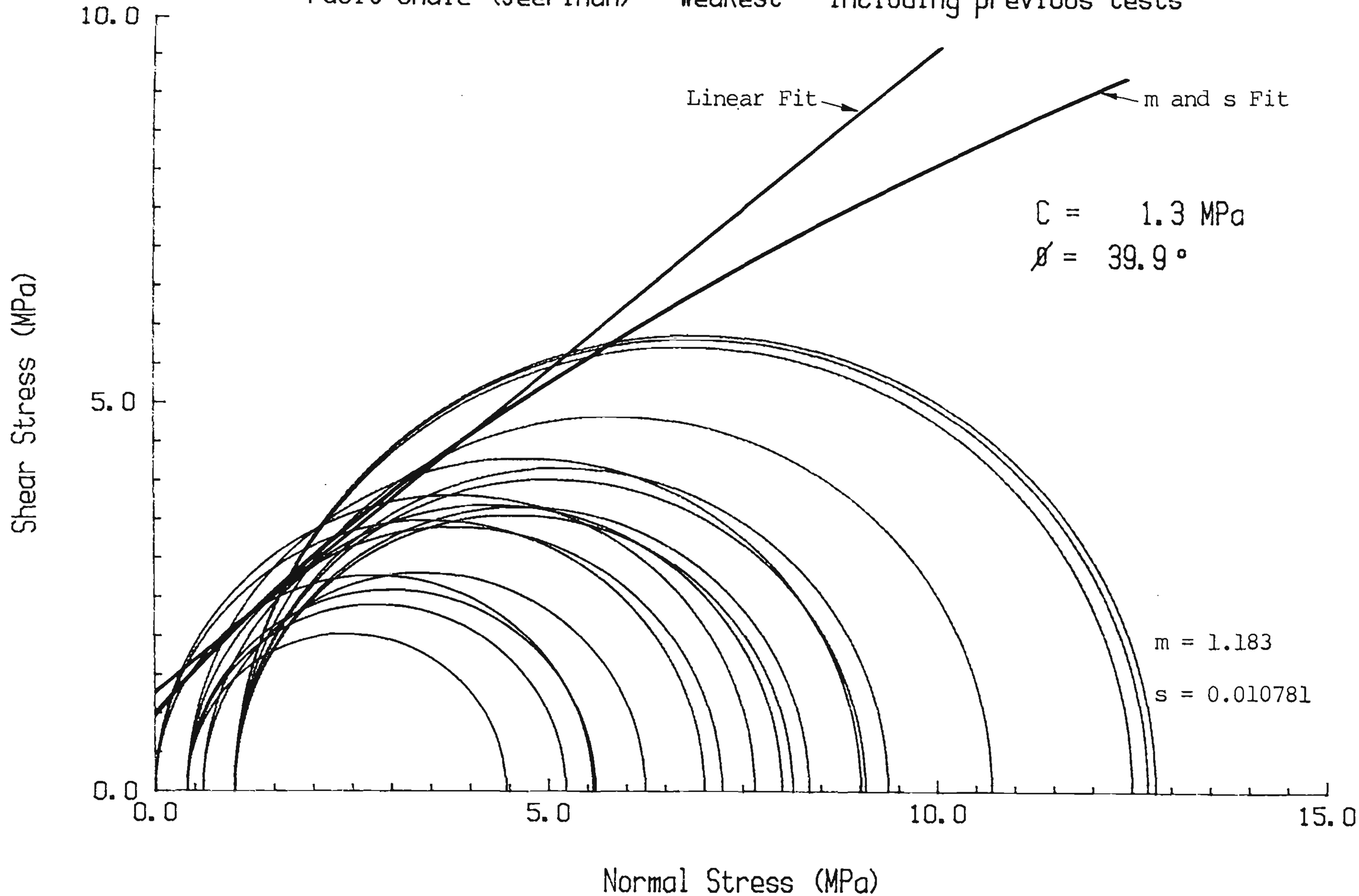


Figure D11

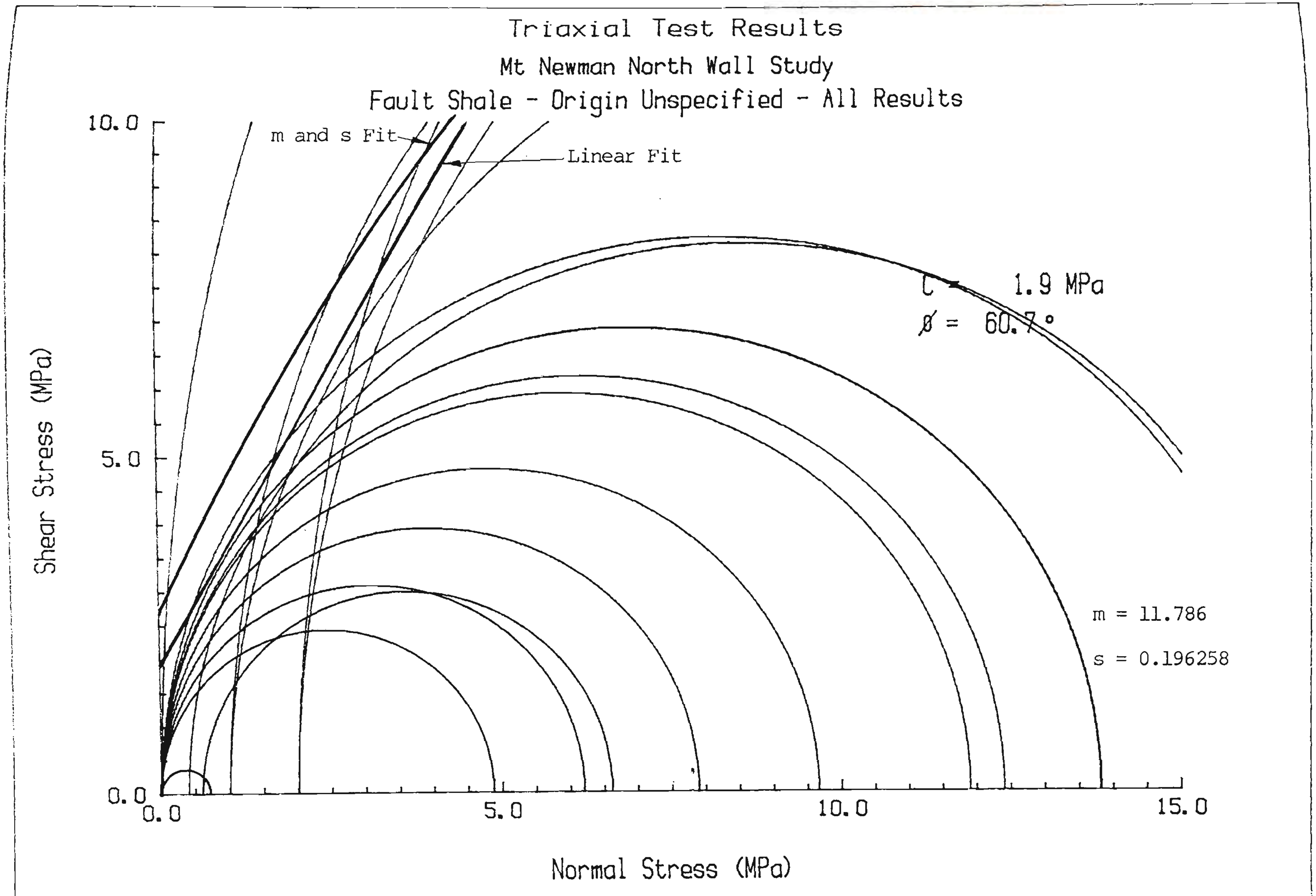

Triaxial Test Results
Mt Newman North Wall Study
Fault Shale - Origin Unspecified - All Results
m and s Fit
Linear Fit
C = 1.9 MPa
ø = 60.7°
m = 11.786
s = 0.196258
Shear Stress (MPa)
Normal Stress (MPa)
0.0
5.0
10.0
15.0

Figure D12

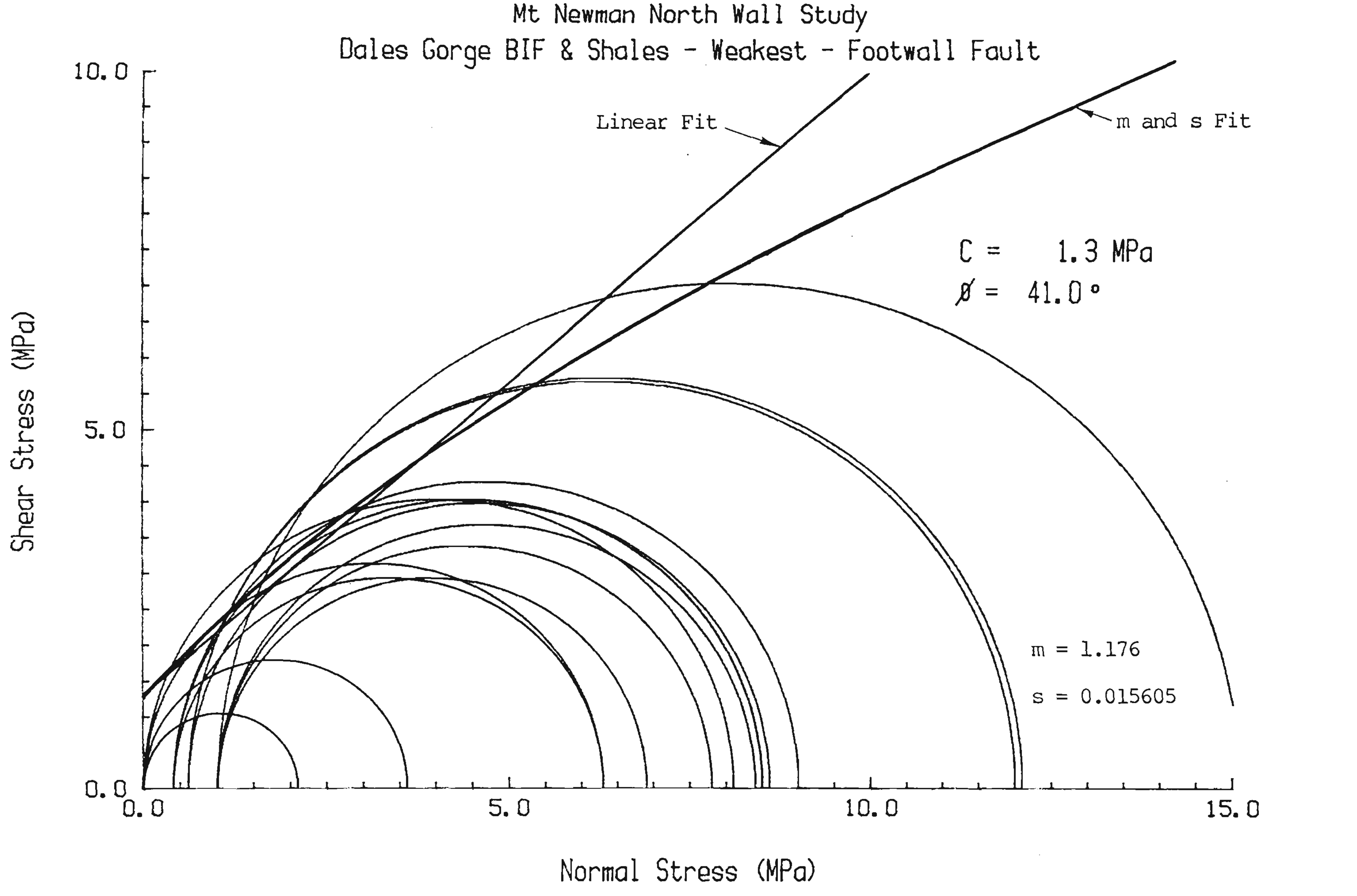

Mt Newman North Wall Study
Dales Gorge BIF & Shales - Weakest - Footwall Fault
Linear Fit
m and s Fit
C = 1.3 MPa
Ø = 41.0°
m = 1.176
s = 0.015605
Shear Stress (MPa)
Normal Stress (MPa)
10.0
5.0
0.0
0.0
5.0
10.0
15.0

Figure D13

APPENDIX E

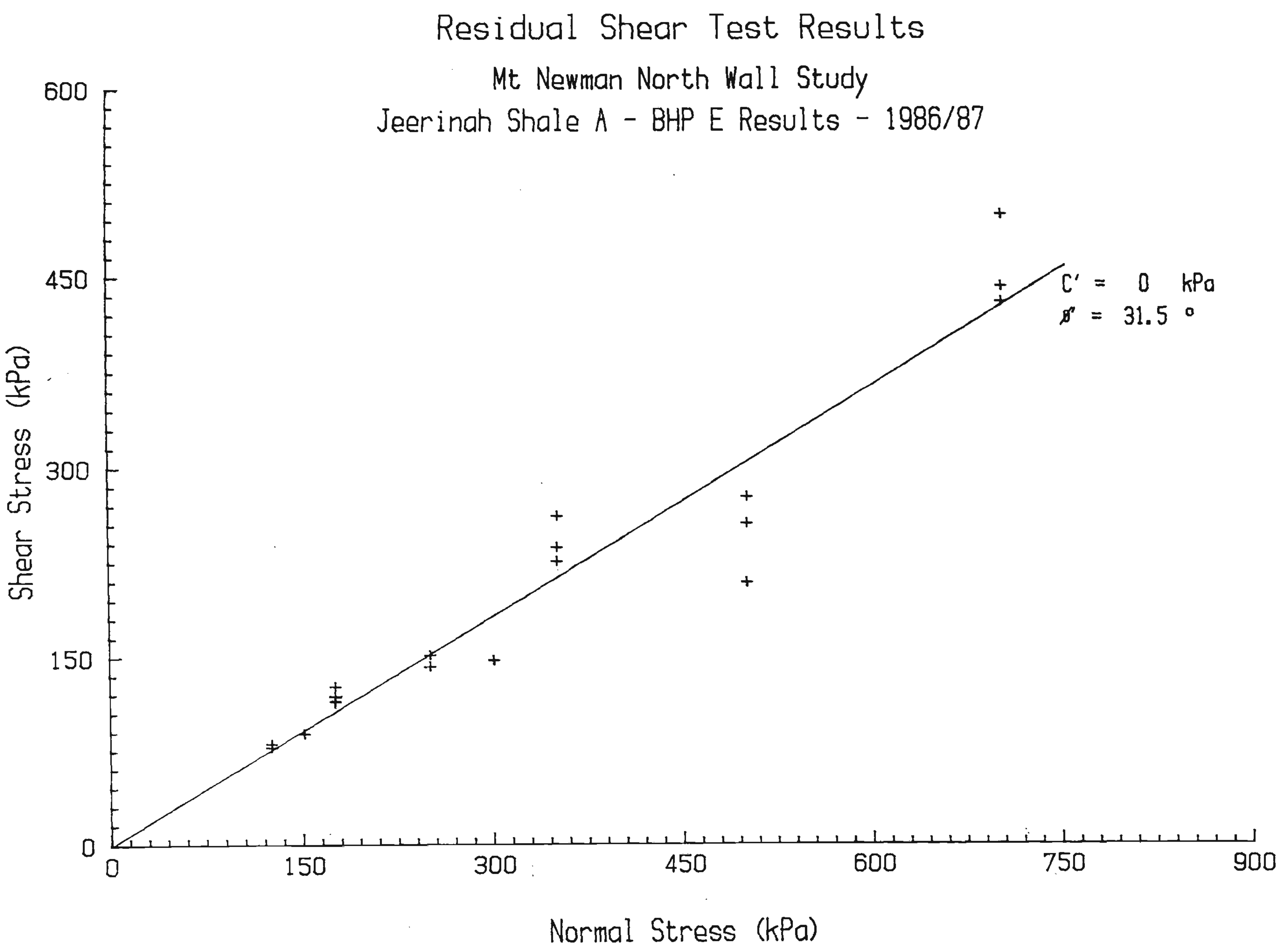

Residual Shear Test Results
Mt Newman North Wall Study
Jeerinah Shale A - BHP E Results - 1986/87
Shear Stress (kPa)
600
450
300
150
0
Normal Stress (kPa)
0
150
300
450
600
750
900
C' = 0 kPa
ø' = 31.5 °

Figure E1

Residual Shear Test Results

Mt Newman North Wall Study

Jeerinah Shale B - BHP E Results - 1986/87

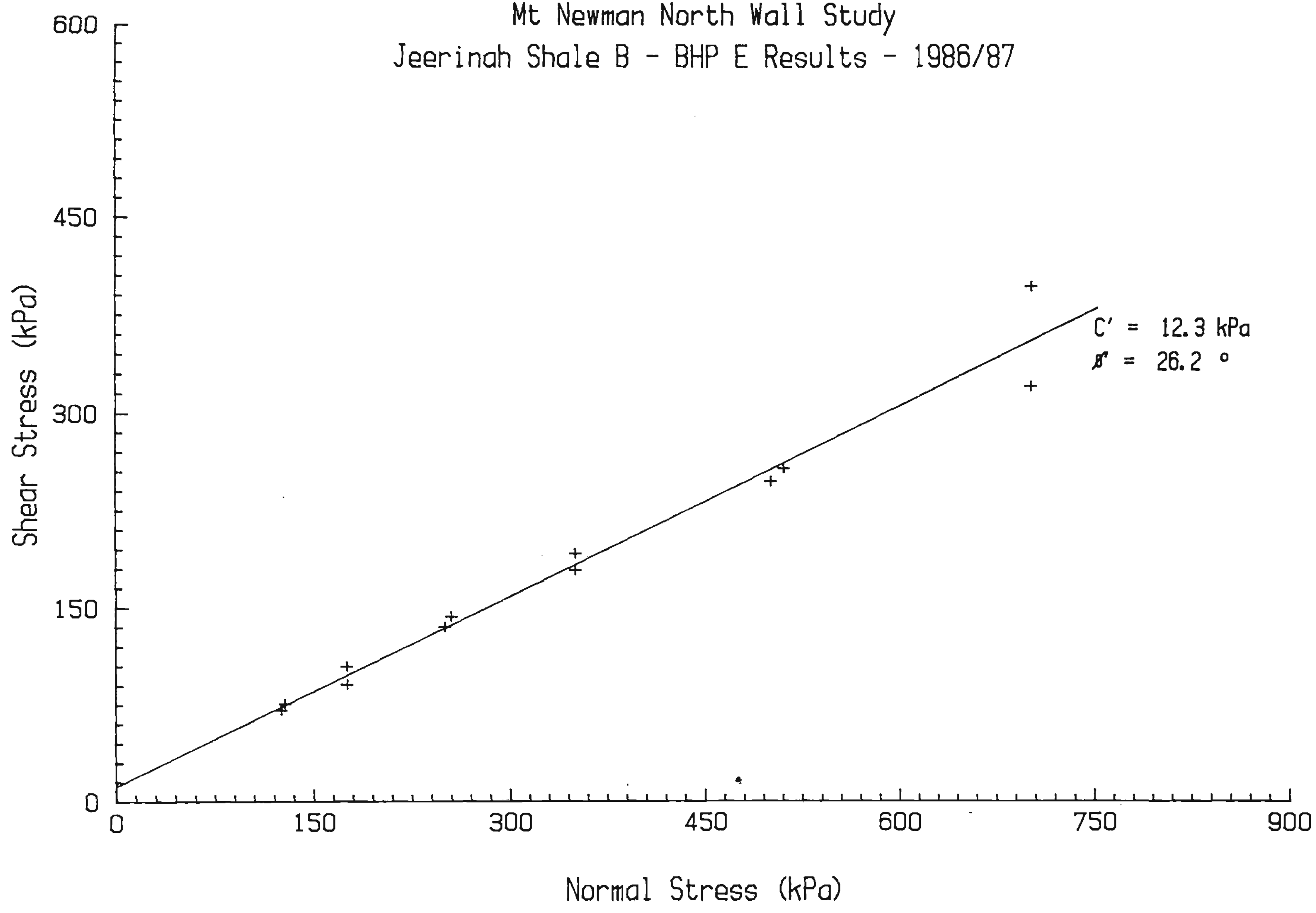


Figure E2

Residual Shear Test Results

Mt Newman North Wall Study

Fault Shale - Jeerinah (Crushed) - All Results

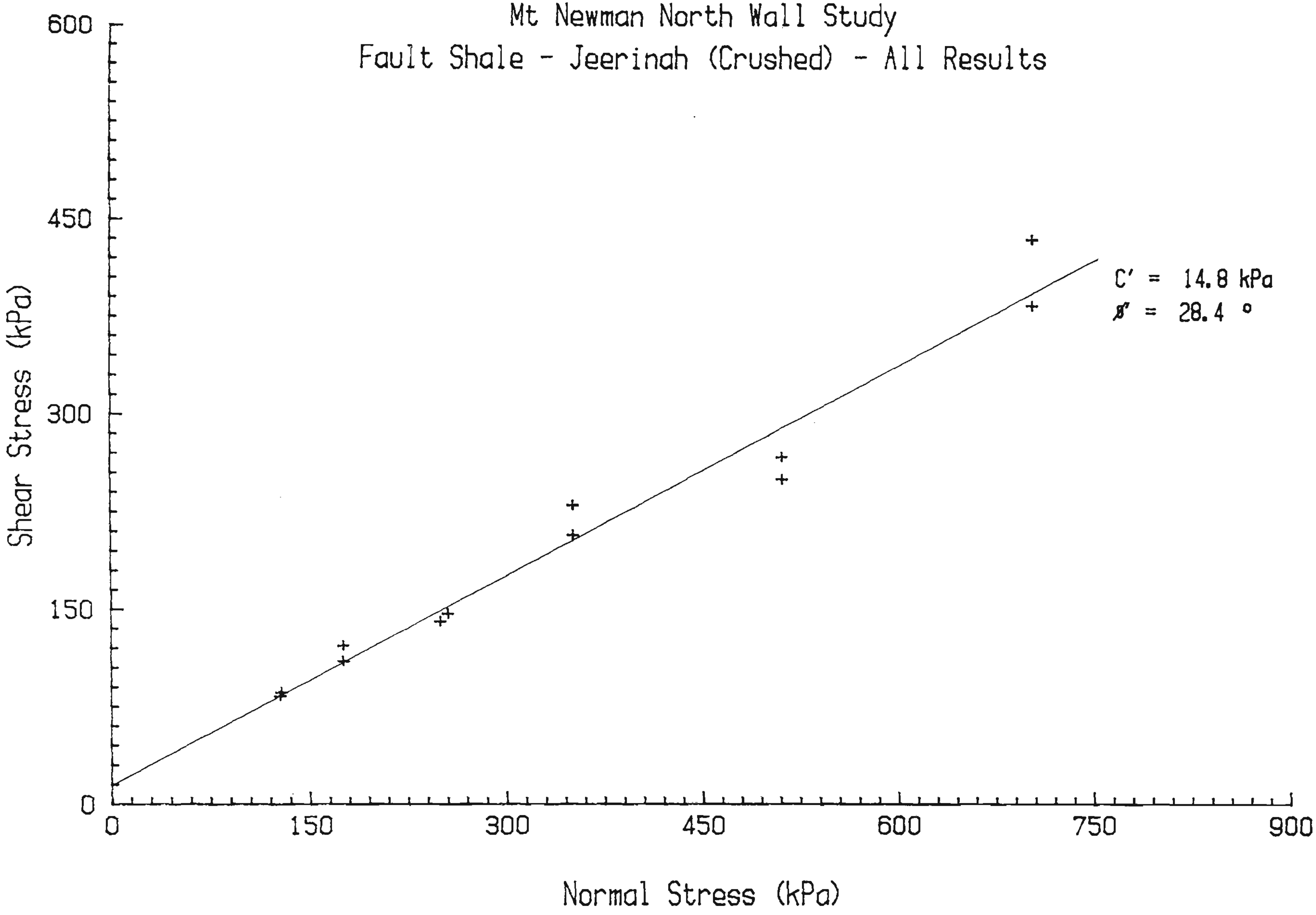


Figure E3

Residual Shear Test Results

Mt Newman North Wall Study

Fault Zone McRae Shale (Crushed) - 1986 Results

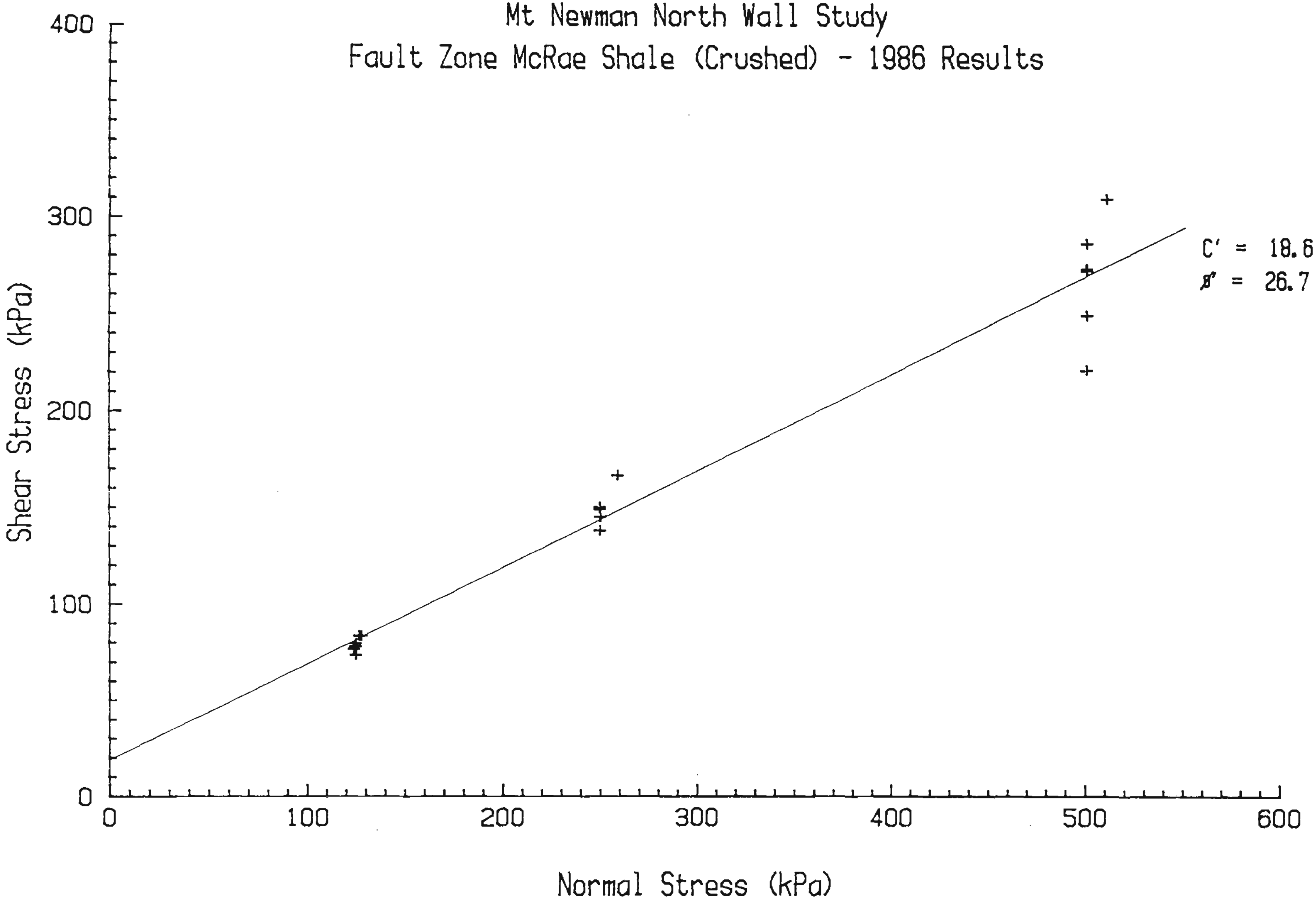


Figure E4

Residual Shear Test Results

Mt Newman North Wall Study

Fault Shale - Sylvia (Crushed) - All Results

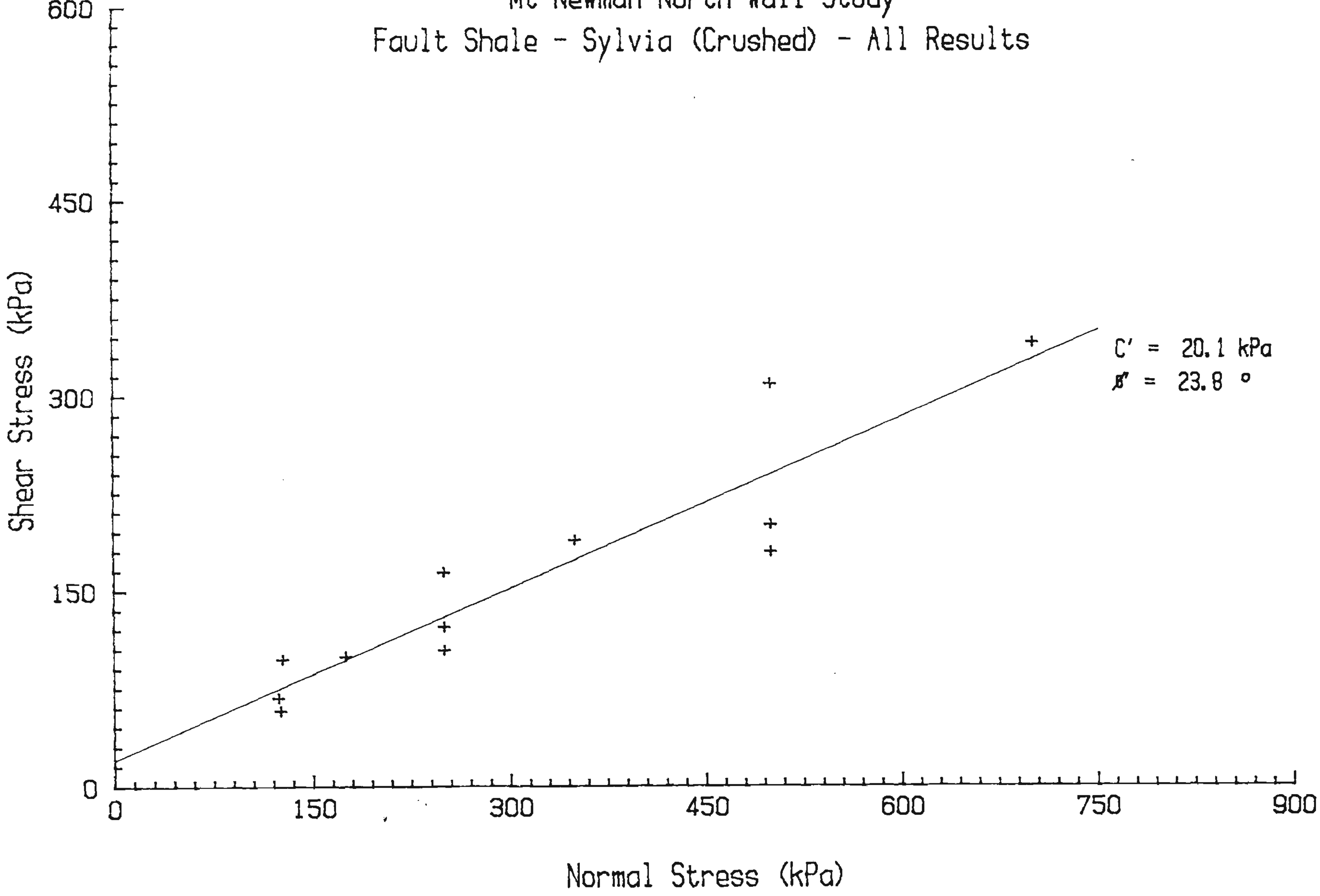


Figure E5

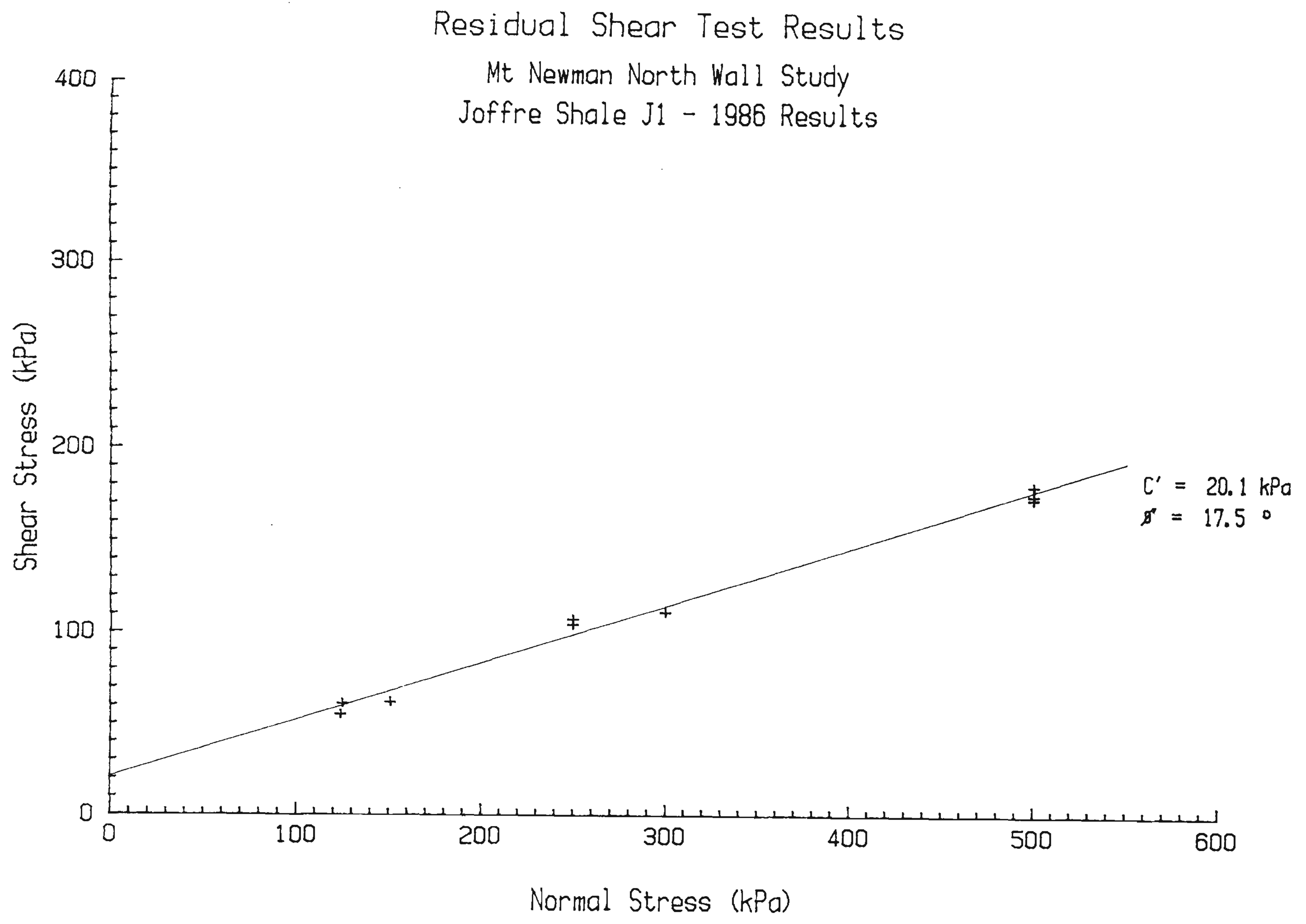

Residual Shear Test Results
Mt Newman North Wall Study
Joffre Shale J1 - 1986 Results
Shear Stress (kPa)
0
100
200
300
400
Normal Stress (kPa)
0
100
200
300
400
500
600
C' = 20.1 kPa
ø' = 17.5 °

Figure E6

Residual Shear Test Results

Mt Newman North Wall Study

Joffre Shale J2 - 1986 Results

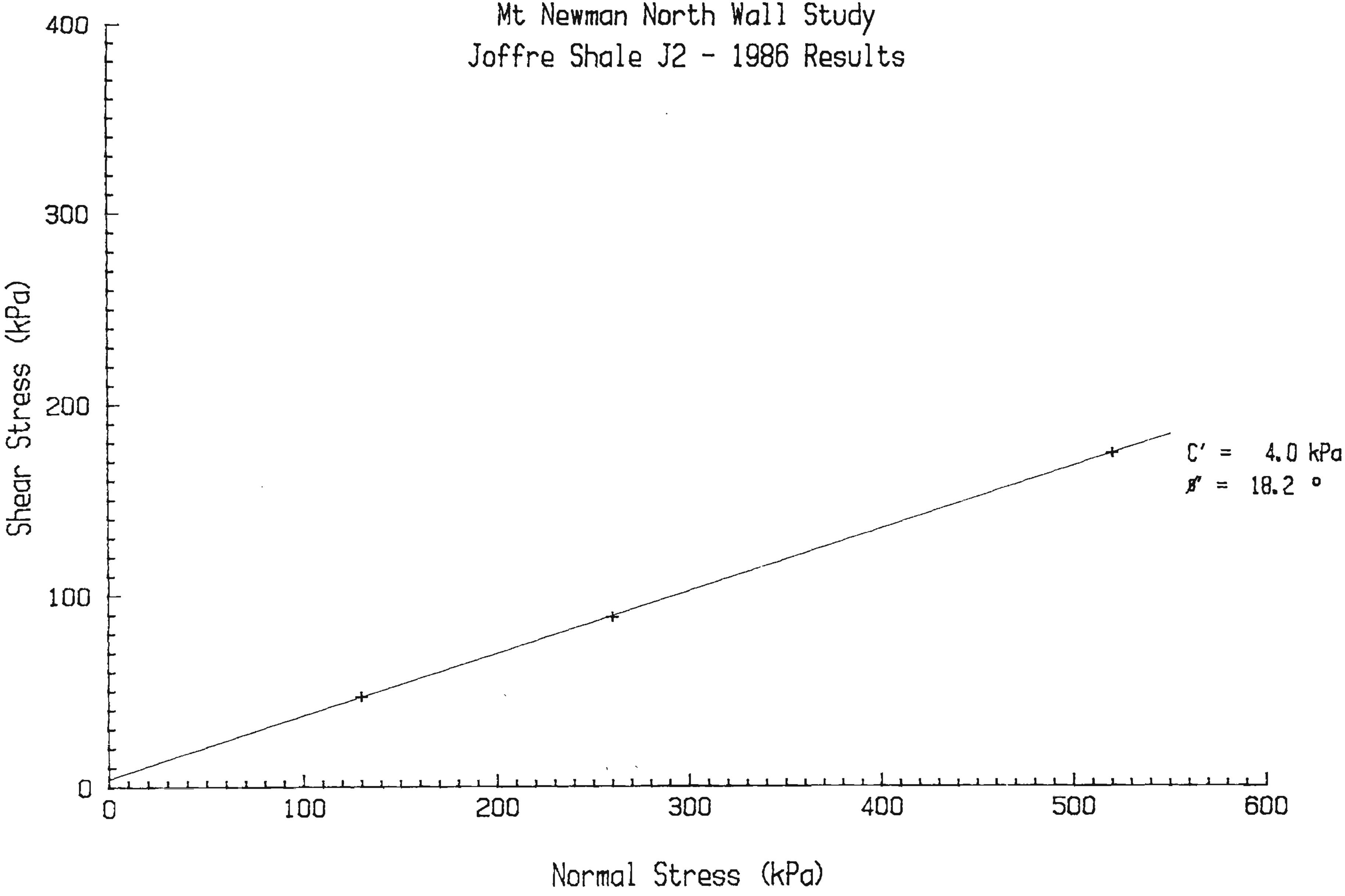


Figure E7

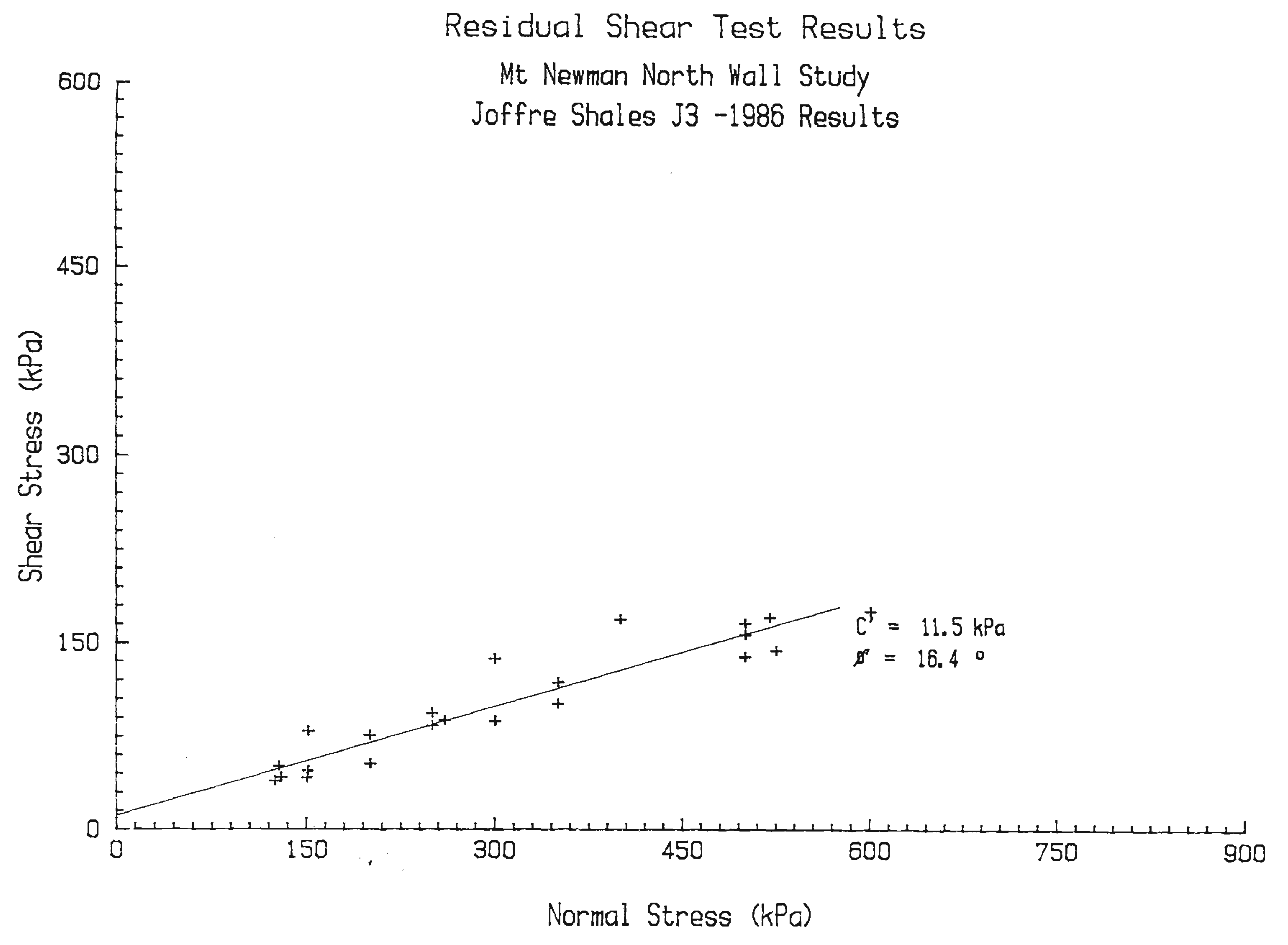
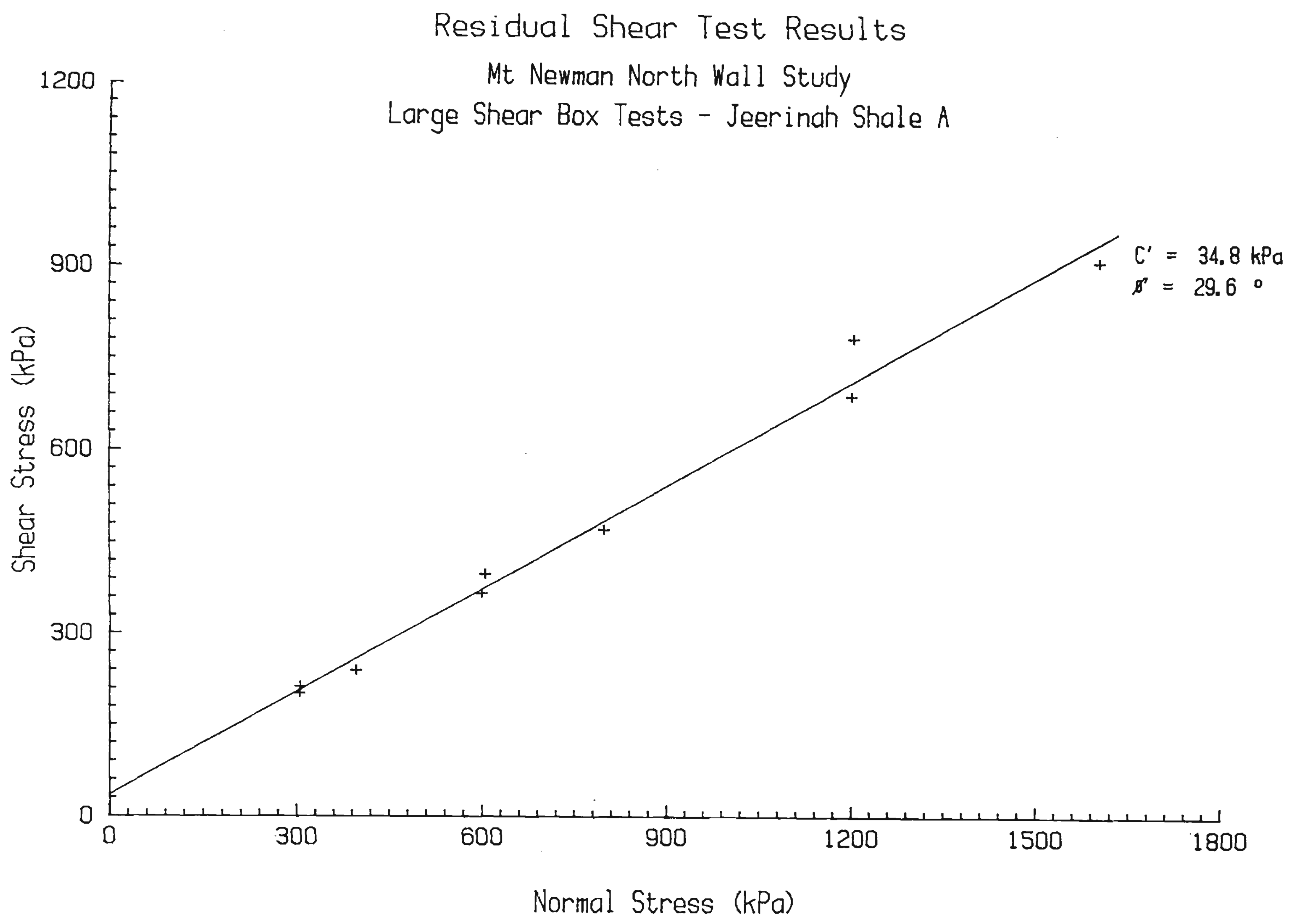

Residual Shear Test Results
Mt Newman North Wall Study
Joffre Shales J3 -1986 Results
Shear Stress (kPa)
600
450
300
150
0
0
150
300
450
600
750
900
Normal Stress (kPa)
C' = 11.5 kPa
ø' = 16.4 °

Figure E8



Residual Shear Test Results
Mt Newman North Wall Study
Large Shear Box Tests - Jeerinah Shale A
C' = 34.8 kPa
ø' = 29.6 °
Shear Stress (kPa)
Normal Stress (kPa)
0
300
600
900
1200
1500
1800

Figure E9

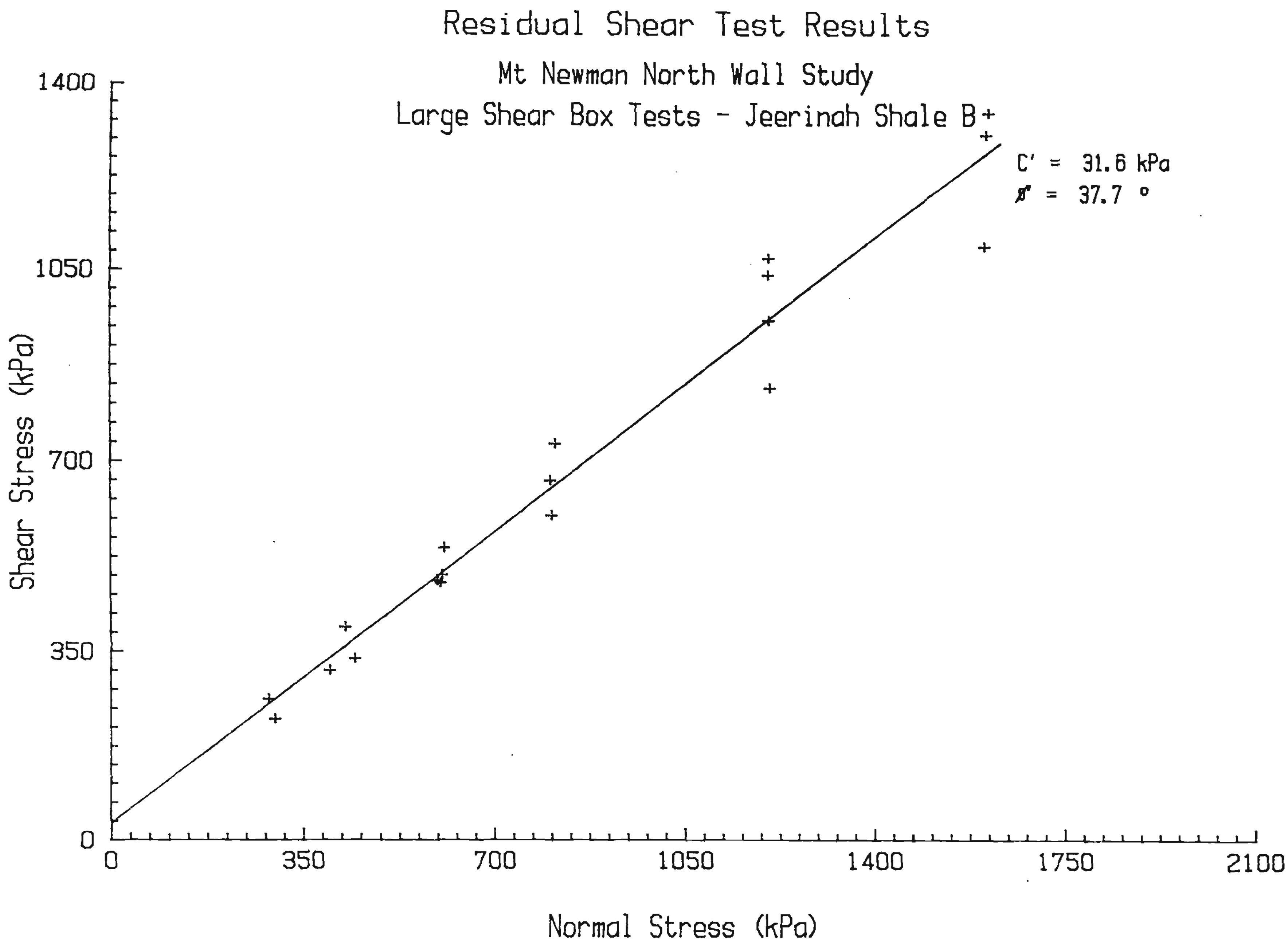

Residual Shear Test Results
Mt Newman North Wall Study
Large Shear Box Tests - Jeerinah Shale B
C' = 31.6 kPa
ø' = 37.7 °
Shear Stress (kPa)
1400
1050
700
350
0
Normal Stress (kPa)
0
350
700
1050
1400
1750
2100

Figure E10

APPENDIX F

BHP Engineering
Geotechnical Laboratory
Sample No.

RESIDUAL SHEAR TEST RESULT

Mt Newman North Wall Study
Clay from Jeerinah Shale A

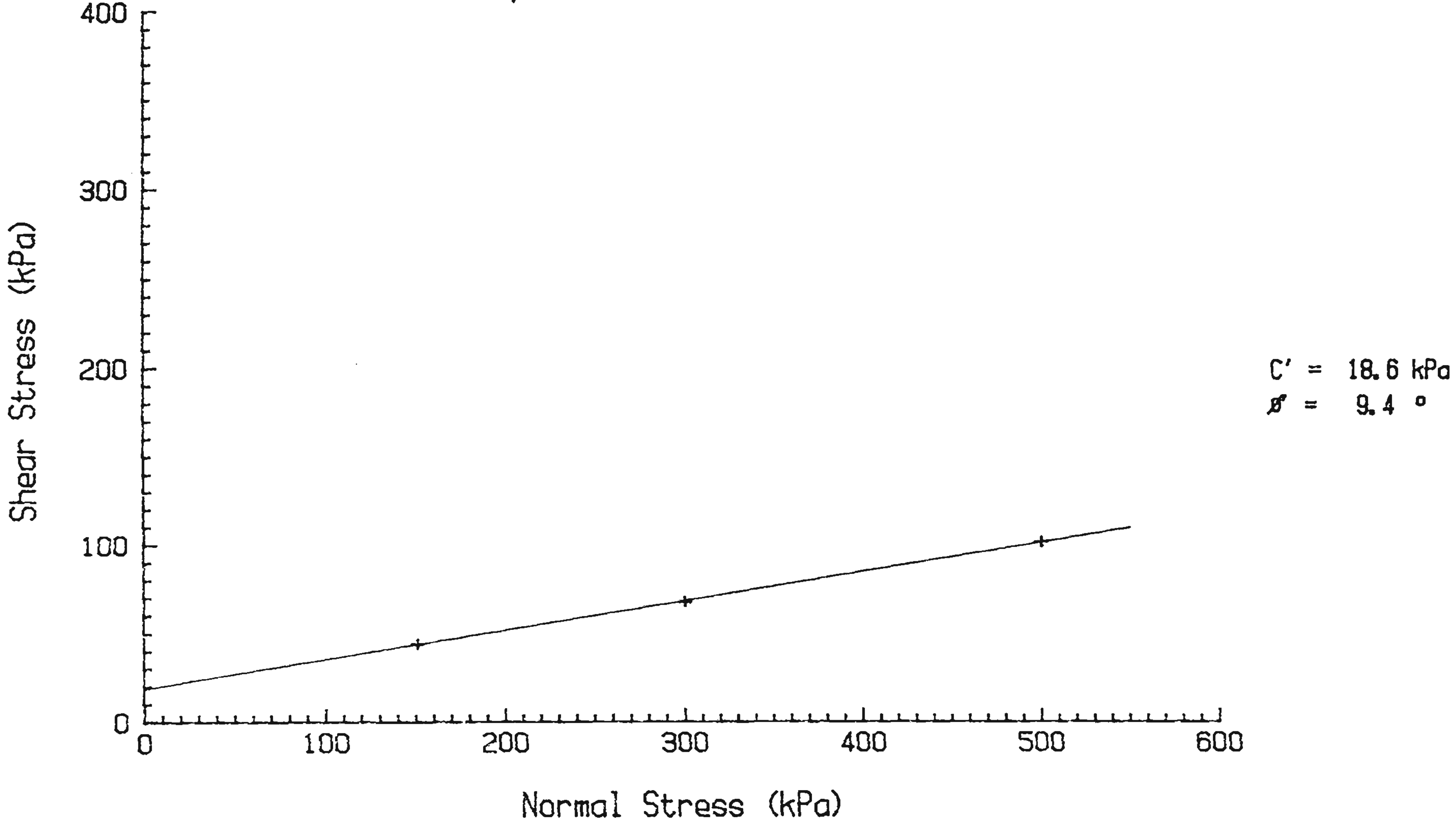


Figure F1

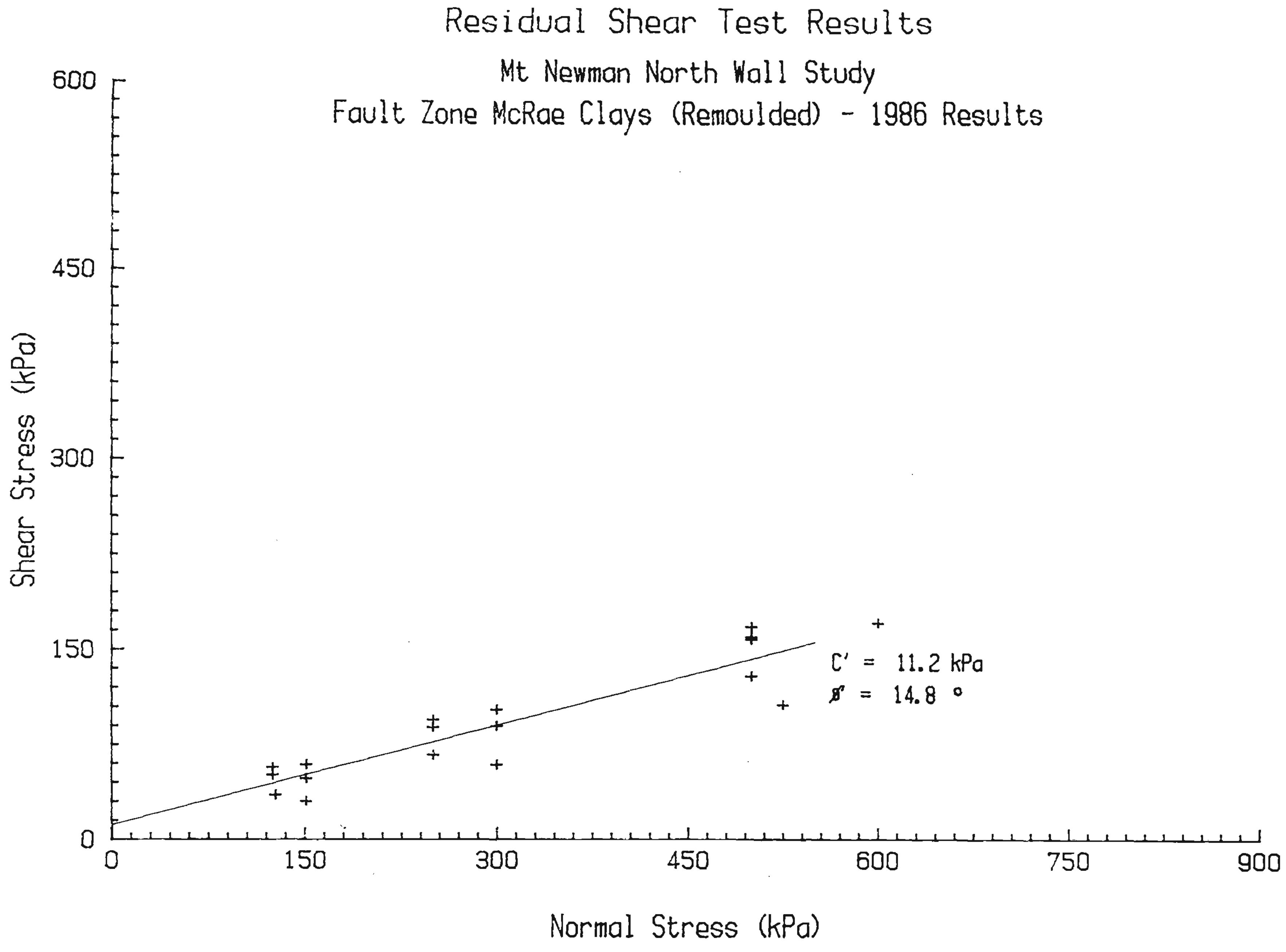

Residual Shear Test Results
Mt Newman North Wall Study
Fault Zone McRae Clays (Remoulded) - 1986 Results
600
450
300
150
0
Shear Stress (kPa)
0
150
300
450
600
750
900
Normal Stress (kPa)
C' = 11.2 kPa
ø' = 14.8 °

Figure F2

Residual Shear Test Results

Mt Newman North Wall Study

Fault Clay - Sylvia - Remoulded

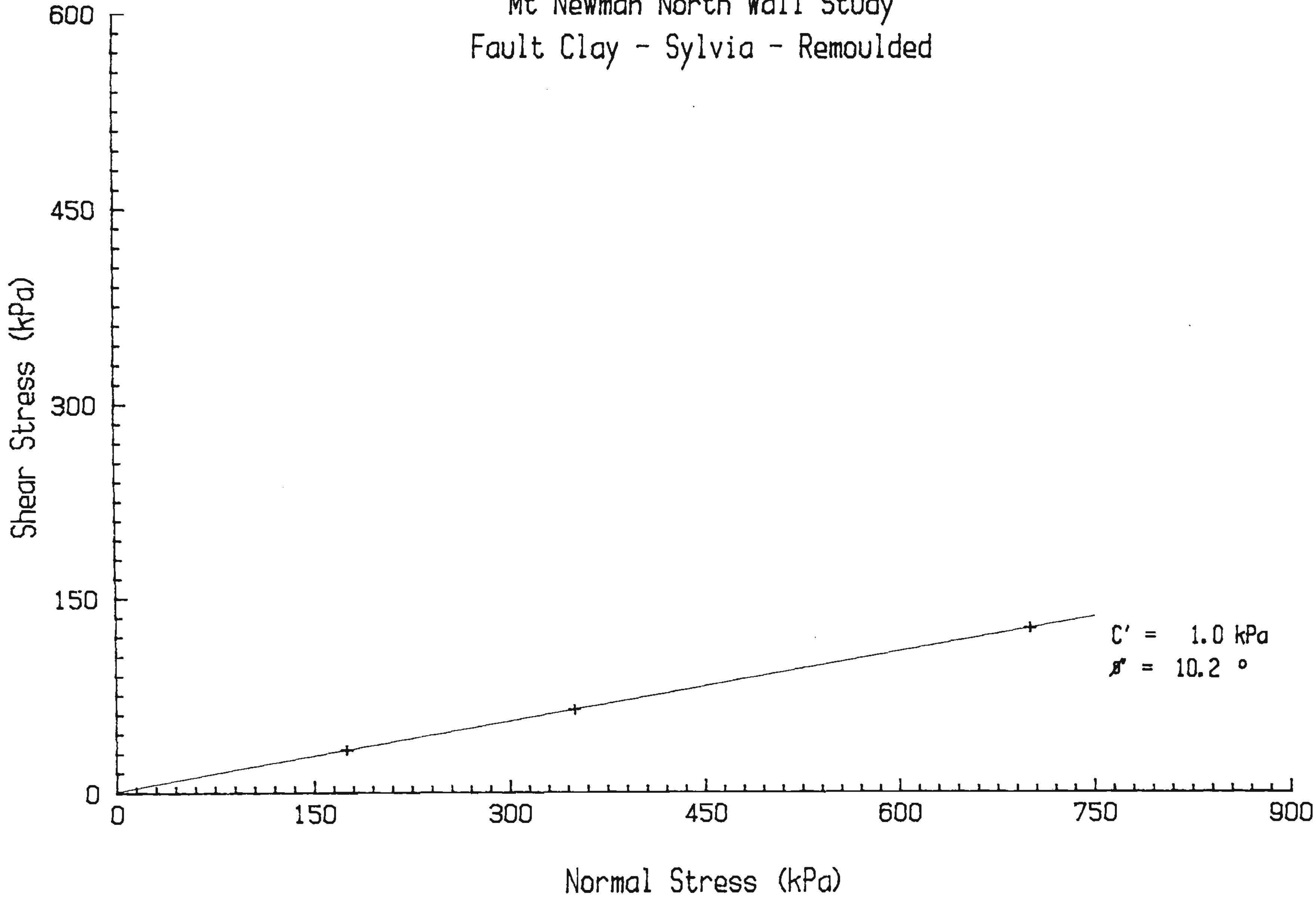


Figure F3

Residual Shear Test Results

Mt Newman North Wall Study

Fault Clay - Origin Unspecified - Remoulded

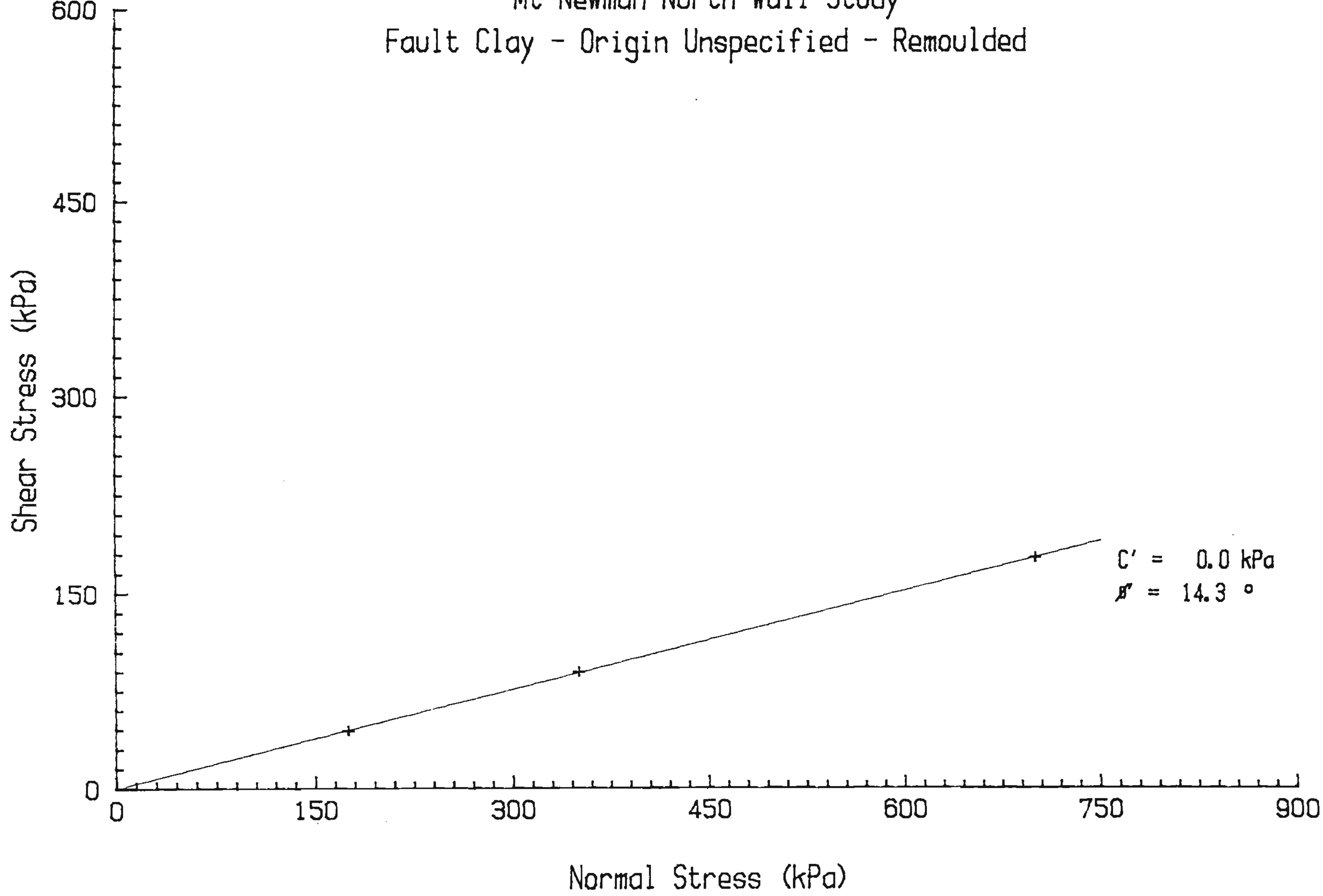


Figure F4

Residual Shear Test Results

Mt Newman North Wall Study

Clays from Dales Gorge Shales - 1987 Results

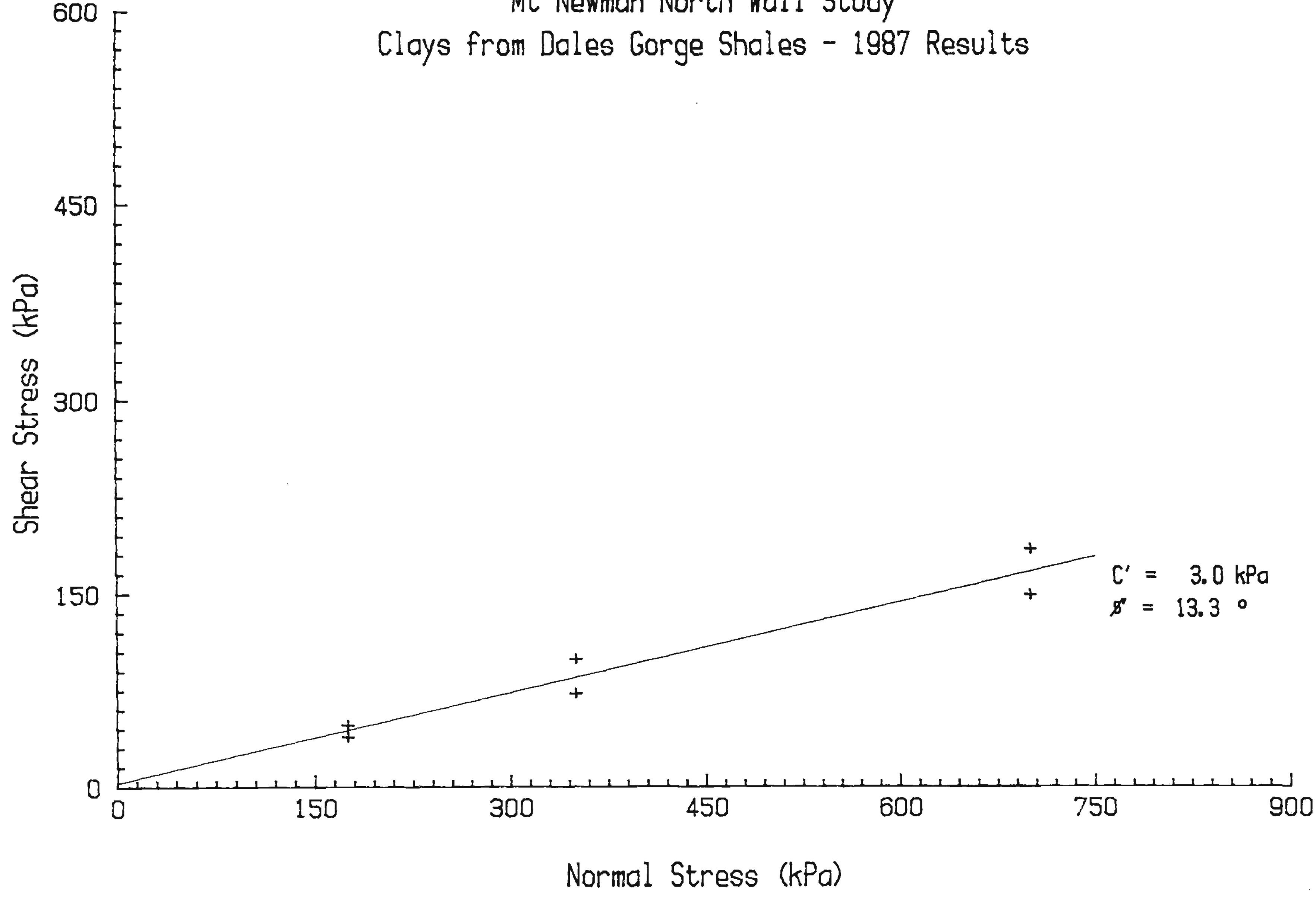


Figure F5

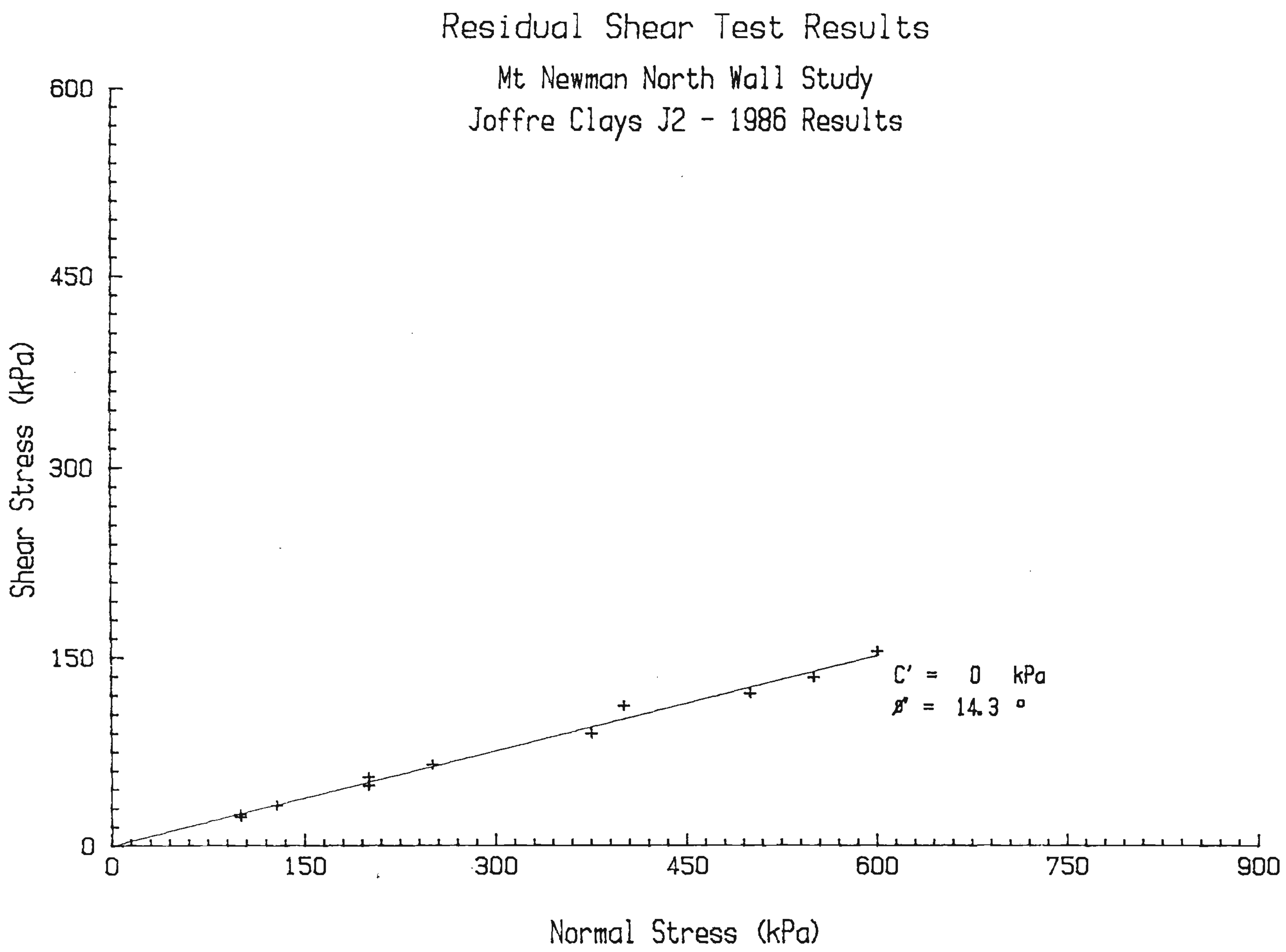

Residual Shear Test Results
Mt Newman North Wall Study
Joffre Clays J2 - 1986 Results
Shear Stress (kPa)
600
450
300
150
0
0
150
300
450
600
750
900
Normal Stress (kPa)
C' = 0 kPa
ø' = 14.3 °

Figure F6

Residual Shear Test Results

Mt Newman North Wall Study

Joffre Clays J3 - 1986 Results

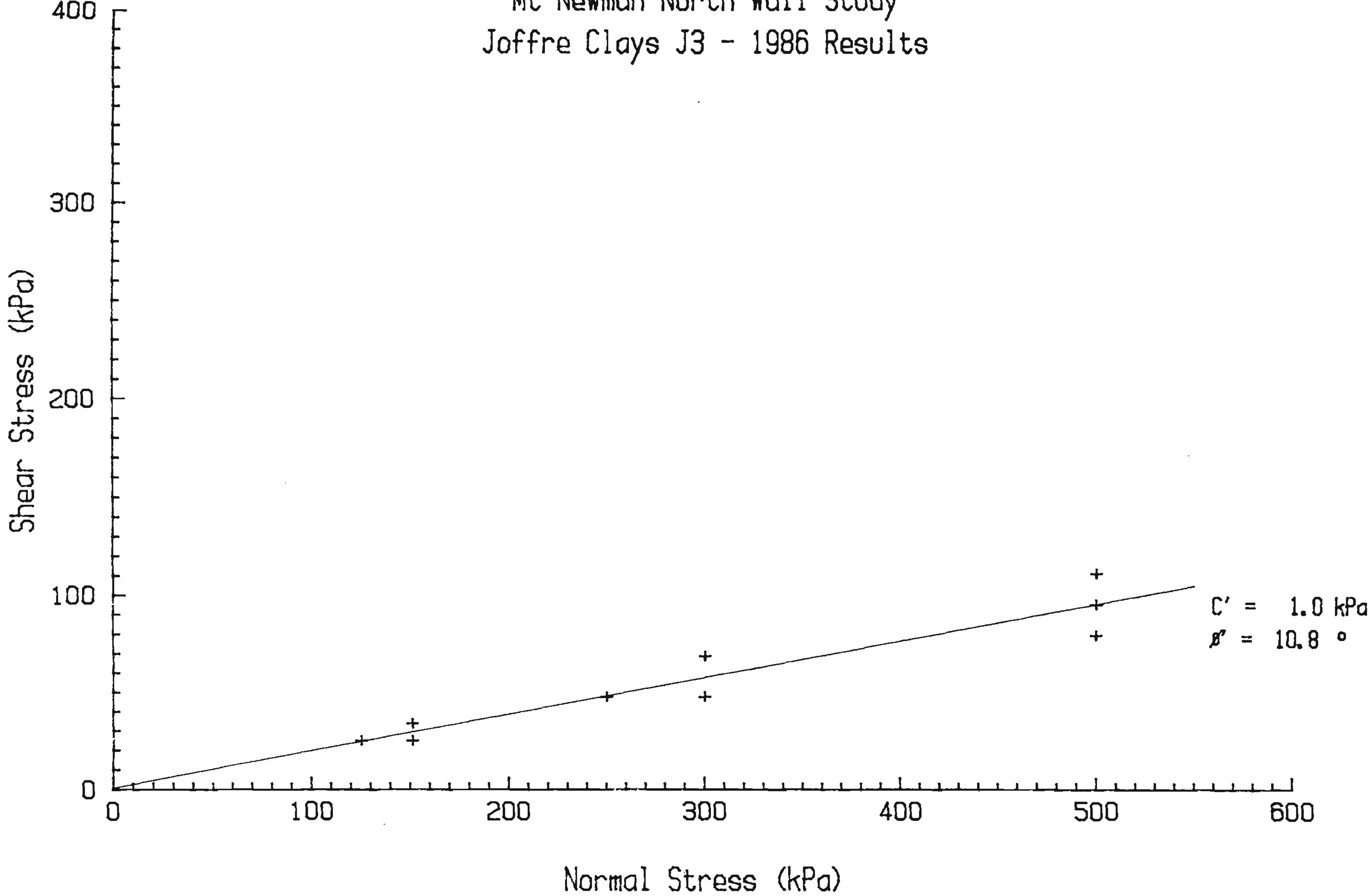


Figure F7