ROCK MASS RATING AND SLOPE STABILITY ANALYSIS OF QUARRY FACES WITHIN THE DYWKA TILLITE OF KWAZULU-NATAL.

by

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ABSTRACT

Dwyka tillite quarries in the Province of KwaZulu–Natal have shown remarkable stable slopes faces even though some of them were quarried over 30 years ago. This can be attributed to their resistance to weathering, the high degree of joint surface roughness, the general lack of any joint infill and the limonitic staining found on most weathered joint surfaces. The latter appears to increase joint roughness. The high percentages of joints terminating within the rock mass or against other discontinuities as well as their low persistence results in a high degree of joint interlocking. These are shown to be very important factors contributing to the overall slope stability.

Detailed discontinuity surveys were carried out at five different quarries located throughout the KwaZulu-Natal region. Only three of these quarries are presently being quarried. This allowed the study and comparison of joint and slope stability characteristics for both the older, more weathered rock faces and those of the recently quarried, and thus fairly unweathered rock faces. Joint orientation data from the various sites show that two to three sets of high angle joints and one low angle joint set are common. The potential of wedge and planar failure is therefore very high. The steeply dipping discontinuities also promote the potential for flexural toppling failure and this was noted in several of the quarry faces. Recognised geotechnical techniques and computer models were used to establish potential modes of failure and to estimate factors of safety. Wedge failure, at partially saturated and saturated conditions, was identified as being the main source of potential slope instability on the quarry rock faces.

The quality of the rock mass of each slope was also classified according to various rock mass classification systems. The rock mass quality generally was rated as being 'fair' to 'good', meaning that slopes are partially stable to stable. The results of each rating system were also compared.

Table of Contents

1. Introduction.. 1
2. Local and Regional Geology 6
2.1 Local Geology 6
2.2 Regional Geology 6
2.2.2 General 7
2.2.2 Structure 8
3. Discontinuity survey 10
3.1 Description of discontinuities 10
3.2 Discontinuity Survey 12
3.3. Discontinuity Analysis 12
3.3.1 Orientation Data 12
3.3.2 Joint spacing and persistence 17
3.3.3 Joint surface weathering and infill 17
3.3.4 Joint roughness 22
3.3.5 Other structural features 23
4. Shear testing of discontinuities 27
4.1 Determination of dry density 27
4.2 Shear box testing 28
4.2.1 Testing procedure 29
4.2.2 Results and Discussion 30
5. Empirical determination of shear strength 35
5.1 Joint Roughness Coefficient 36
5.2 Joint Wall Compressive Strength 37
5.2.1 Unconfined Compression Strength Test 37
5.2.2 Schmidt Hammer Rebound Test 39
5.3 Basic Friction Angle 42
5.3.1 Stimpson's Tilt Test 42
5.3.2 Residual Tilt Test 43
5.4 Back analysis Calculation 45

5.4.1 JRC 45
5.4.2 Peak Shear Strength 46
6. Rock Mass Classification of the Dwyka tillite 48
6.1 Rock Tunnelling Quality Index, Q. 49
6.2 Geomechanics classification 50
6.3 Laubschers Rock Mass Rating System 51
6.4 Romano's Slope Mass Rating System 52
6.5 Discussion 54
6.5.1 Correlation between Bieniawski's RMR and Barton's Q-index 55
7. Slope Stability Analysis 58
7.1 Factors affecting slope stability 58
7.2 Factor of Safety of a slope 58
7.3 Type of slope failures 59
7.4 Wedge failure 62
7.5 Plane failure 67
7.6 Toppling failure 71
8. Discussion 74
9. Acknowledgements 77
10. References 78

List of Tables

Table 3.1	Summary of orientation data	13
Table 3.2	Joint spacing and average FF/m for each joint set	18
Table 3.3	Average joint persistence	19
Table 3.4	Termination Indices	20
Table 4.1	Dry density determination	28
Table 4.2	Shear test results	31
Table 5.1	Unconfined compressive strength results	38
Table 5.2	UCS values of Dwyka tillite in Natal	38
Table 5.3	Schmidt hammer correction factors	41
Table 5.4	Average joint wall compressive strength values per weathering grade	41
Table 5.5	Results of the Stimpson Tilt Test	43
Table 5.6	Results of the Residual Tilt Test	44
Table 5.7	Summary of the results from the back analysis calculations	46
Table 5.8	Peak shear strength (MPa) for varying roughness profiles and normal stresses	47
Table 6.1	Results of the Rock Mass Classification Systems	53
Table 7.1	List of joints to produce potential wedge failure	65
Table 7.2	Summary of wedge analysis results	66
Table 7.3	FOS for plane failure analysis using the Deterministic method	69
Table 7.4	Probabilistic slope analysis for plane failure	70
Table 7.5	Slopes showing potential toppling failure	72

List of Figures

Figure 1.1	Locality map	2
Figure 1.2	Rough outline of the Zululand Quarry	3
Figure 1.3	Rough outline of the Westville Quarry	3
Figure 1.4	Rough outline of the Ridge View Quarry	4

Figure 1.5	Rough outline of the Umkomaas Quarry	4
Figure 1.6	Rough outline of the Margate Quarry	5
Figure 3.1	Stereonet plot of Face 3 – Margate Quarry	16
Figure 3.2	Scattered pole plot of all quarries	16
Figure 3.3	Stereonet plot of Faults and dykes – Zululand Quarry	25
Figure 4.1	Plot of Estimated JRC value vs Peak shear strength	32
Figure 4.2	Sample 15d – Shear stress (MPa) vs Horizontal displacement	32
Figure 4.3	Sample 15d – Vertical displacement (mm) vs Horizontal displacement (mm)	33
Figure 5.1	Joint roughness profiles with JRC values	36
Figure 6.1	Line graphs comparing the Rock Mass Ratingss of Bieniawski, Romano and Loubscher	55
Figure 6.2	Various relationships between RMR and Q	56
Figure 6.3	Relationship between N and RCR	57
Figure 7.1	Planar failure in rock with highly ordered structure	59
Figure 7.2	Wedge failure on two intersecting discontinuities	60
Figure 7.3	Circular failure in waste rock or heavily fractured rock with no identifiable structural pattern	60
Figure 7.4	Toppling failure in rock with steeply dipping discontinuities	61
Figure 7.5	Stereonet plot showing a potential wedge failure	63
Figure 7.6	Stereonet plot showing a potential plane failure	67
Figure 7.7	Stereonet plot showing a potential toppling failure	71

List of Plates

Plate 2.1	Unsorted, massive tillite	8
Plate 3.1	Joint wall alteration and coarse grained, angular tillite joint infill	21
Plate 3.2	Calcite staining on joint surfaces at the Margate Quarry	22
Plate 3.3	Quartz veining at Westville Quarry, Face 4	23
Plate 3.4	Curved joint planes	24
Plate 3.5	Fault 1 – Zululand Quarry	25

Plate 3.6 Fault 3 – Zululand Quarry.. 26
Plate 4.1 Golder Shear Box.. 30
Plate 5.1 Schmidt Hammer... 39
Plate 7.1 Example of wedge failure at the Ridge View Quarry – Face 4......... 62
Plate 7.2 Possible wedge failure at the Ridge View Quarry – Face 2............. 63
Plate 7.3 Example of plane failure at Margate Quarry – Face 2.................... 68
Plate 7.4 Potential toppling failure at Ridge View Quarry – Face 3.............. 73

List of Appendices

Appendix A Quantitative descriptions of discontinuities
Appendix B Rock Mass Classification Data
Appendix C Stereonet plots
Appendix D Shear test – Description of joint surfaces.
Appendix E Plots of shear test results.
Appendix F Classification parameters and their Ratings.
Appendix G Photographic Summary

1. Introduction

The stability of the rock slopes found in the old (up to 30 years) Dwyka tillite quarries in the Durban area of KwaZulu-Natal is remarkable. Understanding the nature of discontinuities within the rock mass of these quarries will provide some clue to the rock face stability. For this purpose detailed discontinuity surveys were carried out on five quarries within the coastal region of KwaZulu-Natal (Figure 1.1). The study involved looking at some old and weathered rock slopes as well as some fresh and newly mined rock faces. The quarries visited, of which not much information is known, were as follows (Figure 1.2):

- Zululand Quarries (ZQ) situated in Zululand approximately 10 km outside Mandini along the banks of the Tugela River. This working quarry has been mined since early 1960's and therefore hosts old and fresh mined rock faces (see Photographic Summary in Appendix G).
- Westville Quarry (WQ) situated below the University of Westville along the Jan Hofmeyer and University Roads. The quarry was last mined in 1976 and now forms part of an Office Park.
- Ridge View Quarry (RVQ) situated in Cato Ridge, approximately 4 km west of the University of Durban, Natal. The quarry is presently being mined by RMM (Ready Mix Material) for material used in road construction.
- Umkomaas Quarry (UQ) situated approximately 5 km outside the town of Umkomaas along the Umkomaas River. This quarry is no longer active.
- Margate Quarry (MQ) situated approximately 10 km northwest of the town of Margate adjacent to the N2 freeway. The quarry is presently being worked and only relatively recent mined benches were available for study. The quarry is also owned by RMM.

The information obtained from the surveys was used to carry out basic slope stability analysis as well as providing input for various rock mass classification systems. In addition a number of shear tests were carried out on joints from core samples drilled in the Inanda area, which were supplied by Drennan Maud and Partners. The core samples were fairly weathered and the results from this work were therefore not used in the slope stability analysis calculations.

Previous investigations on the structure and geotechnical properties of the Dwyka tillite were carried out by various authors such as Maud (1962), Brink (1983) and Paige-Green (1975).

Figure 1.1: Locality Map and outline of the Dwyka Tillite in KwaZulu-Natal.

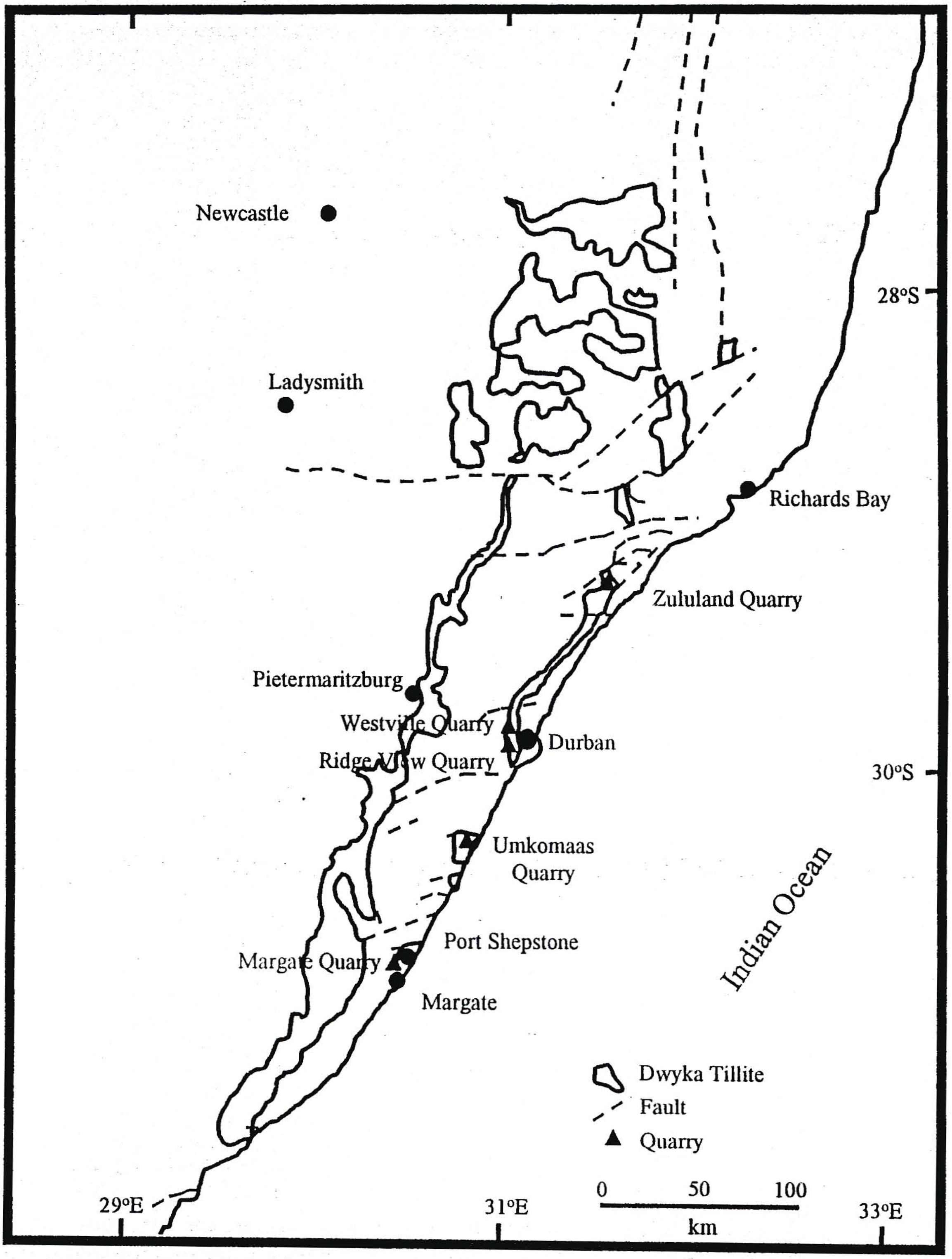


Figure 1.2: Rough outline of the Zululand Quarry with Line Survey Sites

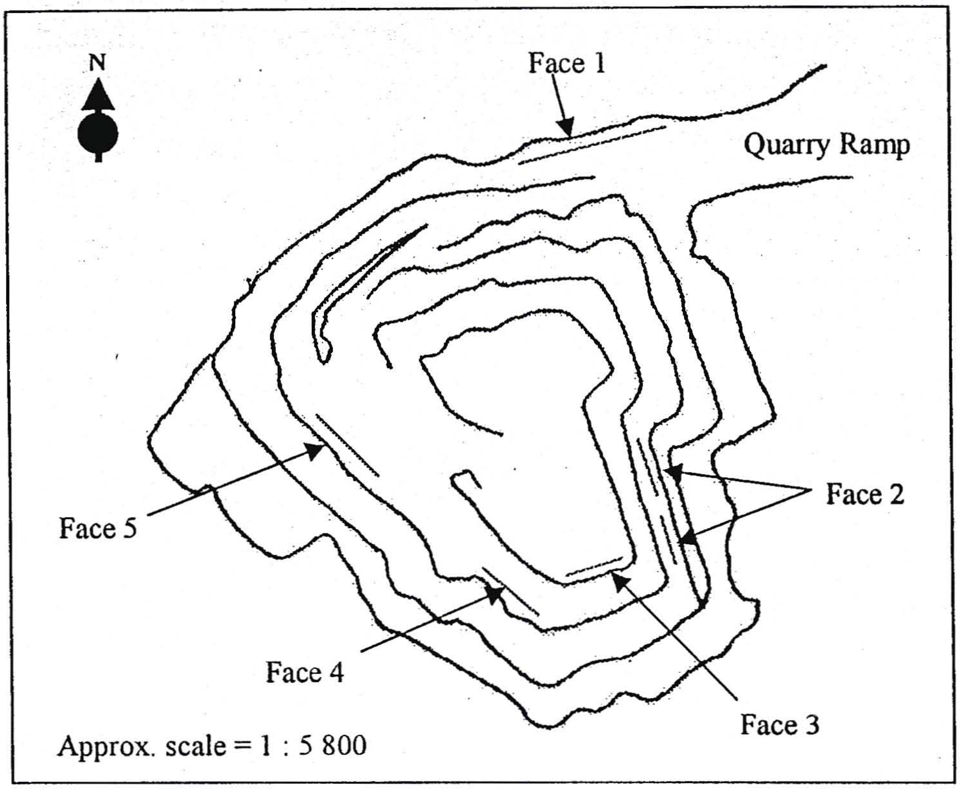


Figure 1.3: Rough plan of the Westville Quarry with Line Survey Sites.

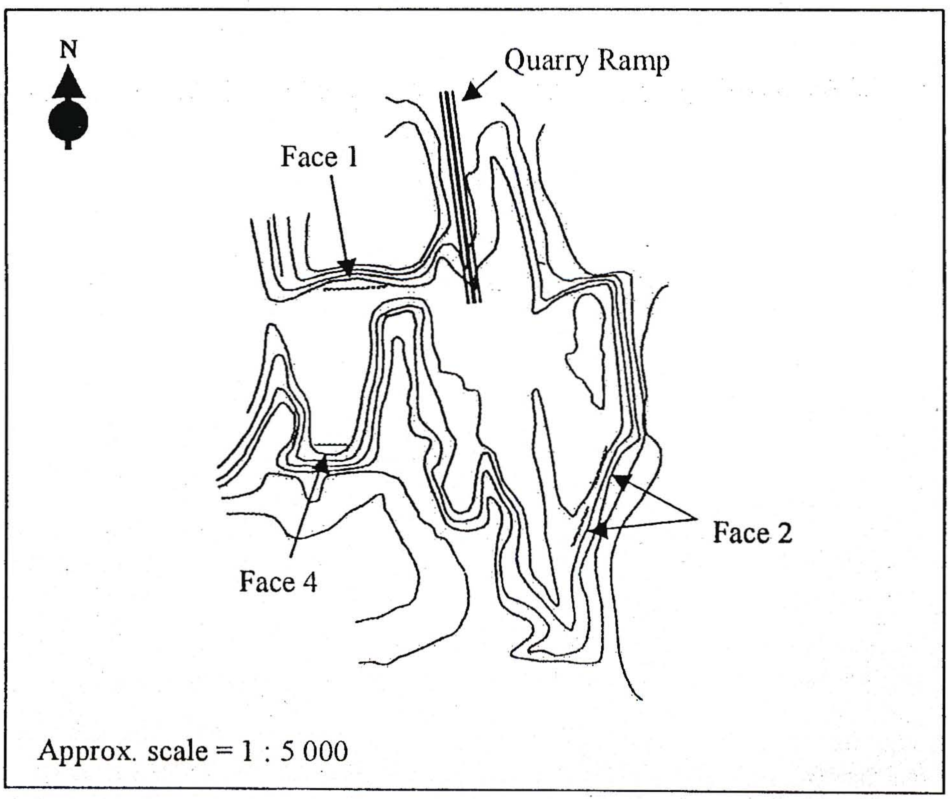
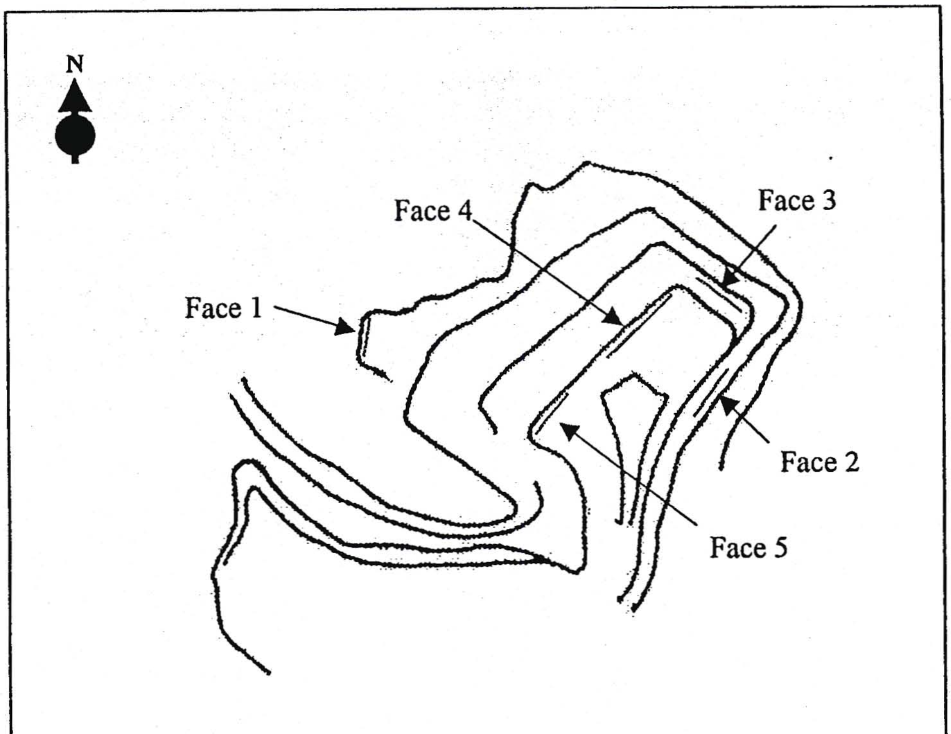


Figure 1.4: Rough outline of the Ridge View Quarry with Line Survey Sites



Approx. scale = 1 : 14 000

Figure 1.5: Rough outline of the Umkomaas Quarry with Line Survey Sites.

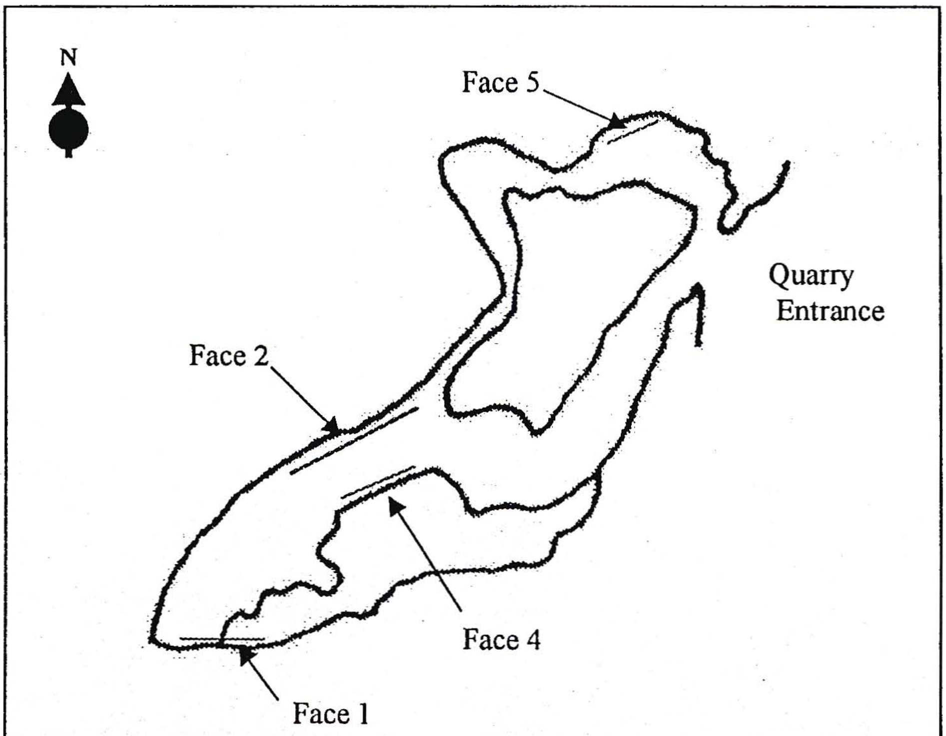


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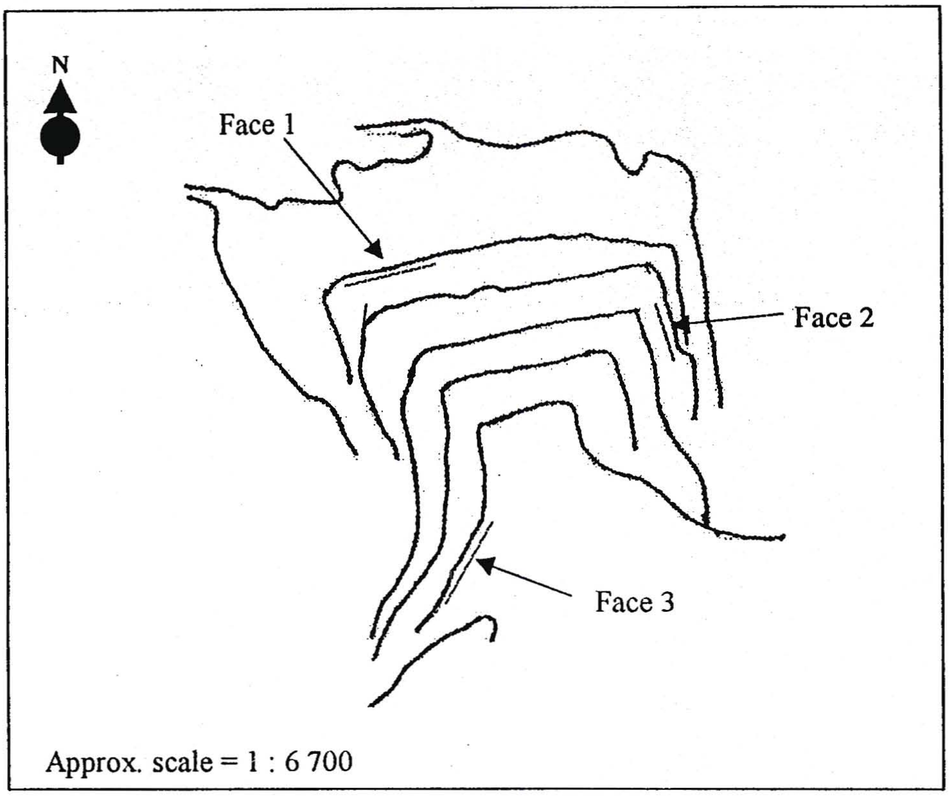
Figure 1.6: Rough outline of the Margate Quarry with Line Survey Sites.



2. Local and Regional Geology

2.1 Local Geology

Not much variation in the local geology of the Dwyka tillite has been recorded in the five quarries. The tillite can in all cases be described as an unsorted, massive diamictite. Depending on the degree of weathering, the colour of the fine-grained matrix may vary from yellow - brown for weathered rock to light or dark grey to dark bluish-grey for fresh rock. The degree of weathering of most faces logged varied between fresh (Grade I) (Barton et al, 1978) to slightly weathered (Grade II). A moderately weathered face (Grade III) was logged at the Ridge View Quarry in Durban. A bimodal clast size distribution was noted in the field and the shape of these clasts varied from angular to sub-angular in the pebble range, to sub-rounded to rounded in the large pebble and boulder size fraction. An increase in the maximum clast size towards the north was observed where the Zululand Quarry showed a higher abundance of larger sized boulders (over 1 m in diameter) as compared to the Margate Quarry in the south. The decrease in clast size from north to south does tie in with the southerly movement of ice sheets during the Permo-Carboniferous glaciation.

The coarser fraction consists essentially of sandstone, conglomerates, argillaceous rocks, shale, quartz and chert. None of these clasts show any degree of weathering. In the study areas the tillite is rarely covered by more than 1 m of gravely residual soil, which is underlain by blocky weathered tillite to an estimated depth of 3 – 6 m.

The tillite is extensively jointed and is characterized by a number of steeply dipping joint sets as well as a low angle joint set (see Photographic Summary in Appendix G). The nature and characteristics of discontinuities and joint sets within each of the quarries are described in detail in Chapter 5.

Plate 2.1: Unsorted massive tillite.

2.2 Regional Geology

2.2.1 General

A diverse assemblage of basement and cover rocks underlies the KwaZulu-Natal region. The geological foundation consists of basement granite and greenstone of the Kaapvaal Craton (3000 million years ago) (Tankard et al, 1982). Erosion of these rocks resulted in the deposition of sediments in shallow basins forming the sandstone and shale of the Pongola Supergroup. Subduction and collision of the Kaapvaal Craton approximately

1000 million years ago produced complex deformed metamorphic rocks. The Palaeozoic sedimentary succession of the overlying Natal Group, Dwyka Formation and Ecca Group is volumetrically the most important. Marine sediments of the Cretaceous Zululand Group and Quaternary coastal and riverine deposits complete the geological succession.

The Karoo sedimentation in Southern Africa was initiated by the Permo-Carboniferous glaciation, depositing tillites in a glacial environment by retreating ice sheets 300 million years ago (Tankard et al, 1982). In KwaZulu-Natal the Dwyka Group unconformably overlies the sedimentary rocks of the Natal Group, made up primarily of arkose and quartz arenites. Deposition of the Dwyka in this area took place on a highly dissected surface resulting in variable thickness estimated between 80 m to 165 m (Brink, 1983, Von Brunn et al, 1989). Shale and siltstone of the lower Ecca Group generally overlap the Dwyka succession. In places the tillite is unconformably overlain by Berea Formation of late Tertiary age.

2.2.2 Structure

The Natal coast is characterized by numerous faults, with the regional pattern comprising a series of arcs trending east-west along the north coast, but swinging southwards from south-west to north-south (von Veh et al, 1990). In the northern parts of the region the faults tend to have downthrows to the south and bound half-grabens and grabens are tilted to the north or northwest. In the south the faults generally have downthrows to the north. Here the fault blocks are rotated to the southeast or south.

Joints are present to a greater or lesser extent in all the rocks found in the region. Von Veh et al (1990) identified up to five joint sets in the basement:

A set parallel to the bedding;

A set parallel to the strike of the bedding;

A conjugate pair with the strike of the bedding bisecting the acute angle;

A single set perpendicular to the strike of the bedding;

A conjugate pair with an obtuse angle bisected by the strike of bedding.

The Dwyka tillite in the region is well jointed, which probably resulted from stress relief (Paige-Green, 1975). The tillite along the Natal coastline dips seawards at an approximate angle of 10° - 30° (Brink, 1983). Bedding planes in the tillite are seldom visible.

3. Discontinuity Survey and Analysis

A discontinuity is a plane of weakness within a rock mass, across which the rock is structurally discontinuous. They can occur in the form of joints, faults, bedding planes, fissures, cleavage or schistosity. The most common discontinuities are joints and bedding planes. Their presence strongly influence the mechanical and hydrological behaviour of the rock mass in terms of its strength, deformability, porosity, transmissivity and water storage capacity. The mechanical behaviour is strongly influenced by the number of joint sets whereas the shear strength and deformability of the rock mass is influenced by the joint pattern, its geometry and how well it is developed. The nature of discontinuities and rock masses can be described by employing established field practices.

3.1 Description of discontinuities

The parameters used to describe discontinuities for this project are those recommended by the International Society for Rock Mechanics (ISRM) (Barton et al, 1978). These are briefly described below and the quantitative description listed in Appendix A.

3.1.1 Type of discontinuity

Determines whether the discontinuity is a joint, bedding plane, fault etc.

3.1.2. Weathering grade of the rock mass

3.1.3 Hardness of the rock mass.

3.1.4 Orientation

This describes the attitude of the discontinuity in space and is measured by recording the dip direction and the dip (261°/80°).

3.1.5 Joint spacing

This is the perpendicular distance between adjacent joints and refers to the mean or modal spacing of a joint set.

3.1.6 Persistence

Length of discontinuity, measured in metres, as observed in an exposure.

3.1.7 Termination data

Termination data for each end of each joint was recorded in addition to the length. Discontinuities, which extended beyond the exposure (x), were differentiated from those terminating in the rock (r) and from those terminating against other joints in the exposure (d). This made it possible to calculate the termination index (T_r) for the rock mass as a whole. The termination index is defined as the percentage of joint ends terminating in the rock (Σr) compared to the total number of terminations ($\Sigma r + \Sigma d + \Sigma x$) (Barton et al, 1978).

3.1.8 Aperture

The average perpendicular distance measured in millimeters, between the adjacent joint walls.

3.1.9 Roughness and JRC value

The inherent small scale surface irregularities relative to the mean plane of a discontinuity. Barton and Choubey's (1977) descriptive terms for joint roughness and JRC roughness profiles were used. The roughness profiles were estimated using a Carpenter's comb.

3.1.10 Schmidt Hammer Test

The Schmidt hammer test was used to obtain estimates of wall rock strength for subsequent calculation of shear strength. The orientation of the Schmidt hammer was also recorded in order to apply a correction factor for reducing Schmidt hammer rebound (r) values when the hammer was not used vertically downwards.

3.1.11 Joint surface weathering

The degree of weathering of the joint surface determines the wall strength and therefore the strength of the discontinuity.

3.1.12 Joint waviness amplitude and length

The waviness of discontinuities, measured in metres, describe the large scale undulations of joint surfaces, which affects the initial direction of shear displacement relative to the mean discontinuity plane, and which if interlocked can cause dilation (Barton et al, 1978).

3.1.13 Joint filling

Filling is the material separating the adjacent rock walls of discontinuities. The following factors were recorded: Nature of the filling, type of filling, particle size, hardness and previous displacement.

3.1.14 Seepage

This records the flow of water through the rock mass along discontinuities.

3.2 Discontinuity Survey

In the quarries line scans were used to collect discontinuity data. This involved extending a metric tape across the quarry face ensuring that it was level and securely attached. Within the same quarry several line scans at different orientations (as perpendicular to one another as possible) were carried out. Due to the near vertical nature of the quarry walls, no vertical line scan could be carried out. The distance along the tape at which each joint intersected was recorded. The distance of a line scan along which measurements are taken should ideally be have been 30 m (Bell, 1992) in order to obtain a representative survey. In the quarries studied this was not always possible either due to badly exposed rock faces or due to the small extent of most quarry faces. An average of 200 readings per locality was attempted in order to ensure statistical reliability.

3.3 Discontinuity Analysis

The discontinuity data collected for each line survey of the five quarries has been summarised and can be found in Appendix B. This information is also used for the Rock Mass Classification.

3.3.1 Orientation Data

The DIPS V3.0 program was used for the plotting, analysis and presentation of the discontinuity data using spherical projection techniques. For each individual line survey a contoured, scatter pole data stereonet showing joint set windows and slope was constructed (e.g. Figure 3.1). All the stereonet plots are found in Appendix C. The bias angle, used by the Terzaghi correction, was set at 15° (Diederichs et al, 1969), the default setting. Changing the Terzaghi weighting did not affect the data distribution.

The mean, minimum and maximum orientation data for each quarry is listed in Table 3.1 below.

Table 3.1: Summary of the orientation data

Margate Quarry

	Dip direction (°)			Dip angle (°)		
Joint set	**Mean**	**Min.**	**Max.**	**Mean**	**Min.**	**Max.**
JS1a	243	234	255	89	82	90
		054	075		83	90
JS1b	278	262	291	86	74	90
		082	111		80	90
JS2a	131	119	142	79	71	87
JS2b	170	339	360	83	81	90
		159	179		70	90
JS3	353	303	062	07	03	17
JS7	066	055	077	60	48	71

Umkomaas Quarry

	Dip direction (°)			Dip angle (°)		
Joint set	**Mean**	**Min.**	**Max.**	**Mean**	**Min.**	**Max.**
JS1	262	240	279	59	48	71
JS2a	326	321	330	85	81	90
JS3	085	061	117	19	05	30
JS4	194	184	203	53	46	61
JS6	108	093	124	60	48	74

Ridge View Quarry

	Dip direction (°)			Dip angle (°)		
Joint set	**Mean**	**Min.**	**Max.**	**Mean**	**Min.**	**Max.**
JS1	258	230	286	87	66	90
		055	107		73	90
JS2a	316	304	325	80	69	90
	149	134	161	77	69	90
JS3	063	351	126	11	00	26
JS4	195	185	200	74	63	85
JS8	002	348	014	87	80	90

Westville Quarry

	Dip direction (°)			Dip angle (°)		
Joint set	**Mean**	**Min.**	**Max.**	**Mean**	**Min.**	**Max.**
JS1	261	232	283	80	61	90
JS2a	334	326	344	85	75	90
JS3	065	000	108	09	00	21

Zululand Quarry

	Dip direction (°)			Dip angle (°)		
Joint set	**Mean**	**Min.**	**Max.**	**Mean**	**Min.**	**Max.**
JS2a	307	292 117	317 136	84	74 88	90 90
JS2b	338	329	349	77	65	86
JS3	138	194	194	19	07	33
JS4a	183	170	195	70	54	84
JS4b	209	200	218	74	65	81
JS5	134	118	150	59	46	72

When comparing the discontinuity data from the various locations the following can be concluded:

Joint set 1: Joint set 1 has been identified to occupy a range from approximately 050°/230° to 110°/290° for dip direction and between 65° and 90° for dip angle. JS1 is present in all quarries with the exception of Zululand, where it is absent, and Umkomaas where the set is possibly present but dipping at shallower angles. At the Margate quarry JS1 can be sub-divided into two distinct joint sets.

Joint set 2: Joint set 2 has been recorded in all the quarries and occupies a range from approximately 125°/300° to 180°/320° for dip direction and 65° and 90° for dip angle. It can be sub-divided into two sets i.e. JS2a and JS2b. JS2a is found at all quarries with the exception of Westville. At Margate JS2a it is present but with dip directions ranging between 300° and 320°. JS2b has been recorded at Zululand, Westville and Margate Quarries. JS2 is most poorly developed at Umkomaas (perhaps a function of low sampling).

Joint set 3: Joint set 3 is present in all the quarries and because it has a low range in dip angle (0° to 30°) it shows high variation in the dip orientation (62° to 350°). As no vertical joint surveys were possible JS3 is poorly presented in the orientation data. JS3 possibly represents the poorly developed bedding within the Dwyka tillite.

Joint set 4: Joint set 4 is best developed at the Zululand Quarry were it has been sub-divided into JS4a and JS4b. (dip direction between 170° to 220° and dip angle ranging from 54° to 81°). It is also found at Umkomaas, although at steeper dip angles (46° to 61°), and at the Ridge View Quarry.

Joint set 5: JS5 was only recorded at the Zululand Quarry and occupies the similar dip direction as JS2a but at much higher dip angles (46° to 72°).

Joint set 6: Joint set 6 was only recorded at the Umkomaas Quarry with a minor concentration of poles at Zululand.

Joint set 7: Joint set 7 is poorly defined and was noted only at the Margate Quarry.

Joint set 8: This ill-defined joint set was only recorded at the Ridge View Quarry.

Joint set 9: JS9 has only been recorded at the Margate Quarry.

The data for all the quarries was combined and plotted per site (Figure 3.2). This gives a good indication of the differences in the joint systems between the various quarries. It can be seen that there is, as expected, a great spread in joint orientation data between the five quarries. The variation is as a result of the following:

- Differences in local structural geology between the various sites;
- Distances between the quarries;
- Blasting of the rock mass will have resulted in the shifting of rock blocks leading to a greater variation of the joint orientation data.

The following conclusions can also be drawn from Figure 3.2:

- Each quarry has its own particular joint system.
- Zululand and Umkomaas Quarries exhibit different joint patterns as compared to the other three sites.
- The majority of joints are high angle joints i.e. >50°.

Figure 3.1: Stereonet plot of Face 3 - Margate Quarry

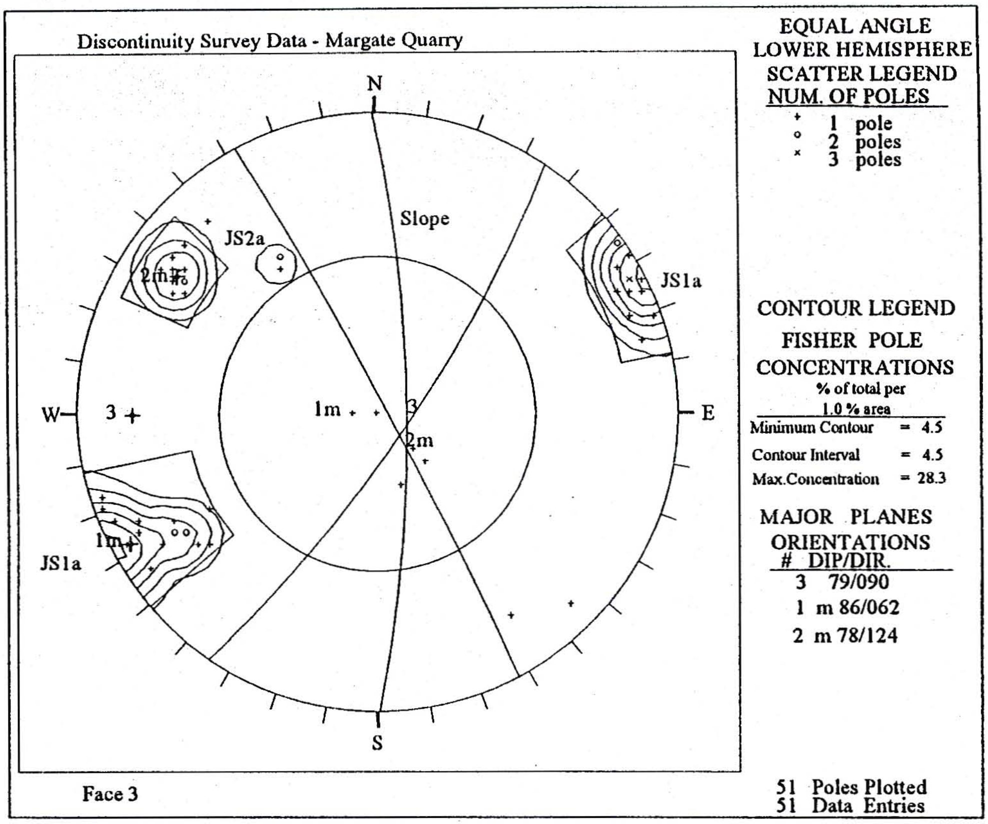
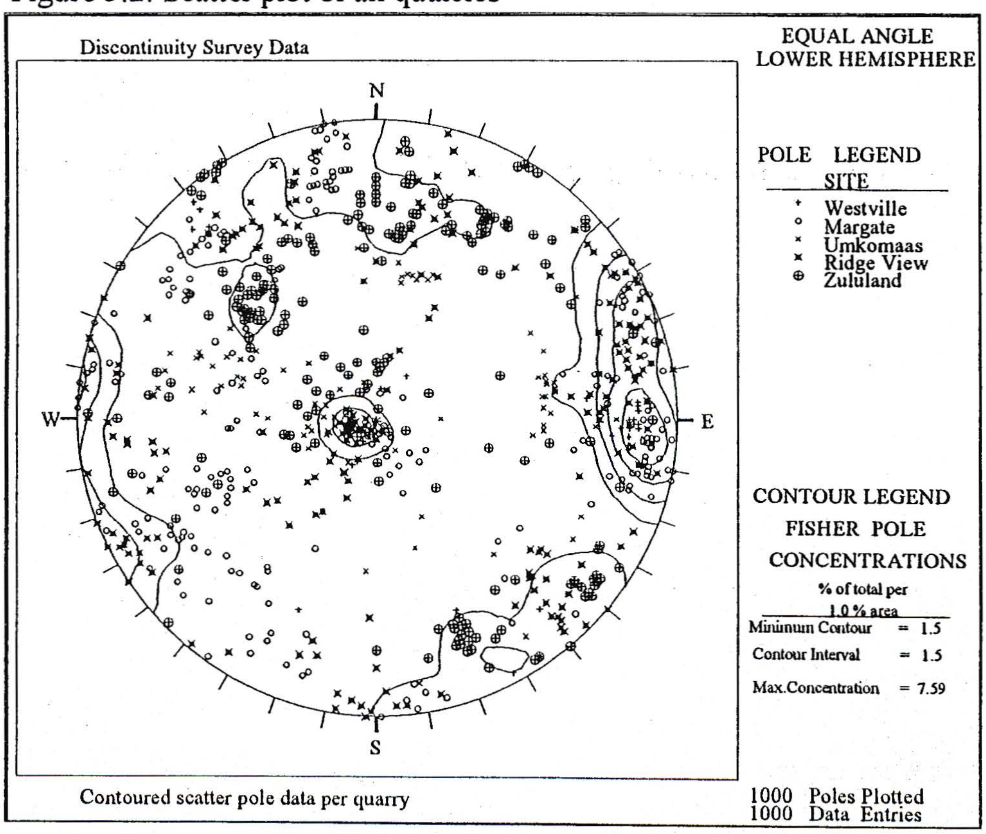


Figure 3.2: Scatter plot of all quarries



3.3.2 Joint spacing and persistence

The joint spacing is important as it controls the size of individual blocks within the rock mass. The more widely spaced the joints are, the better the interlocking conditions (Barton et. al., 1978). More closely spaced joints tend to give conditions of low mass cohesion. In order to avoid directional bias, true spacing was directly measured in the field. From the data collected the modal spacing was then obtained for each joint set per slope (Table 3.2). The average fracture frequency per meter (FF/m) is also recorded. It is obvious that the joint spacing for each joint set varies dramatically, not only between the various quarries but also within each quarry.

It is important to assess the degree of persistence of those joints that are likely to cause slope instability. The average joint length for each set was calculated (Table 3.3). These figures show that the average persistence of the joints, compared to the slope height, is not that great. This is better illustrated by calculating the Termination Indices (T_r, T_x and T_d), where T_x represents the percentage of joints, which extend throughout the exposure (Table 3.4). Joint sets JS1a, JS1b, JS2a and JS2b are relatively persistent compared to the remaining joint sets. These are therefore the dominant joint sets and will thus play an important role in slope instability.

3.3.3 Joint surface weathering and infill

The tillite in the rock slopes of the quarries studied can mostly be classified as either fresh (recently mined) or slightly to moderately weathered (older quarries). The latter can be identified in the field by the discontinuity planes being stained yellow-brown through alteration processes such as oxidation and hydration. This limonitic staining gives the joint surfaces a rough appearance, which must play a factor in slope stability. This alteration does not affect the strength of the joint wall.

Table 3.2: Joint spacing and average FF/m for each joint set.

Line Survey	JS1a	JS1b	JS2a	JS2b	JS3	JS4	JS5	JS6	JS7	JS8	JS9	Average FF/m
Mar – 1		0.25		0.19					0.19			0.33
Mar – 2	0.11		0.11	0.16	0.04				0.31		0.16	0.27
Mar – 3	0.12		0.21									0.43
Umk – 1			---		---			0.20				
Umk – 2	0.15				1.05	1.00		0.17				
Umk – 4					---			0.55				
Umk – 5	0.31				---							1.09
WQ – 1	0.33			0.29	0.78							0.25
WQ – 2/1	0.60			0.60	0.15							0.41
WQ – 2/23	0.32			0.58	0.45							0.35
WQ – 4	0.27		0.37									0.28
RVQ – 1	1.00		0.40		0.21							0.27
RVQ – 2	0.53		---		0.36	2.15				1.38		0.50
RVQ – 3	0.33		0.27		0.24					0.43		0.54
RVQ – 4	0.33		0.31	0.13	0.23	1.10						0.34
RVQ – 5	0.29		0.31		0.18							0.63
ZQ – 1				1.10	0.31							0.56
ZQ – 2					0.32	0.13	0.32					0.58
ZQ – 3			0.16			0.33	0.23					0.59
ZQ – 4			0.44	0.18			0.44					0.61
ZQ – 5			0.11			0.14						0.38

Note: Insufficient data to calculate the joint spacing is denoted by ---.

Table 3.3: Average Joint Persistence (m).

Line Survey	JS1a	JS1b	JS2a	JS2b	JS3	JS4	JS5	JS6	JS7	JS8	JS9	Slope height (m)
Mar – 1		6.2		10.0					1.5			10
Mar – 2	3.0		6.1	5.1	3.1				1.8		1.6	8
Mar – 3	2.0		5.2									10
Umk – 1			0.7		18.5			3.6				30
Umk – 2	16.6				---	14.3		2.3				40
Umk – 4					5.4			2.4				20
Umk – 5	5.4				6.3							17
WQ – 1	7.2			4.0	3.3							32
WQ – 2/1	6.3			1.9	0.9	2.4						30
WQ – 2/23	3.9			3.4	7.3							25
WQ – 4	7.1		3.6									32
RVQ – 1	8.8		4.8		3.1							20
RVQ – 2	3.3		0.7		10.1	4.1				3.8		10
RVQ – 3	2.8		5.7		5.0					3.7		7
RVQ – 4	4.6		2.1	3.8	3.5	6.6						10
RVQ – 5	6.6		5.1		5.6							12
ZQ – 1				10.7	5.7							17
ZQ – 2					3.4	0.8	2.6					10
ZQ – 3			2.8			2.5	3.9					12
ZQ – 4			4.5	1.4			6.1					12
ZQ – 5			4.6			1.2						10

Table 3.4: Termination Indices (T_r, T_x and T_d)

Line Survey	JS1a $T_r/T_x/T_d$	JS1b $T_r/T_x/T_d$	JS2a $T_r/T_x/T_d$	JS2b $T_r/T_x/T_d$	JS3 $T_r/T_x/T_d$	JS4 $T_r/T_x/T_d$	JS5 $T_r/T_x/T_d$	JS6 $T_r/T_x/T_d$	JS7 $T_r/T_x/T_d$	JS8 $T_r/T_x/T_d$	JS9 $T_r/T_x/T_d$
Mar – 1		5/67/28		0/100/0					0/25/75		
Mar – 2	5/65/30		0/85/15	7/67/26	10/0/90				19/0/81		25/19/56
Mar – 3	13/75/12		5/80/15								
Umk – 1			17/33/50		0/75/25			14/36/50			
Umk – 2	0/70/30				---	0/83/17		36/43/21			
Umk – 4					20/50/30			0/83/17			
Umk – 5	29/50/21				0/50/50						
WQ – 1	19/63/18			33/22/45	20/10/70						
WQ – 2/1	10/67/23			31/69/0	0/25/75	25/75/0					
WQ – 2/23	23/77/0			17/42/41	0/25/75						
WQ – 4	14/70/16		25/63/12								
RVQ – 1	0/90/10		3/53/44		8/42/50						
RVQ – 2	0/56/44		10/30/60		38/19/43	25/25/50				0/30/70	
RVQ – 3	29/63/8		13/87/0		36/14/50					10/80/10	
RVQ – 4	10/66/24		8/92/0	17/25/58	15/10/75	0/80/20					
RVQ – 5	0/72/28		28/75/0		0/0/100						
ZQ – 1				5/75/20	42/33/25						
ZQ – 2					0/25/75	31/12/56	8/88/4				
ZQ – 3			30/30/40			38/38/24	23/27/50				
ZQ – 4			0/80/20	41/36/23			7/64/29				
ZQ – 5			23/58/19			0/71/29					

Plate 3.1: Joint wall alteration and coarse grained, angular tillite joint infill.

Joint infill in the Dwyka tillite is not very common and joints are predominately clean. The type and abundance of infill varies from site to site and consists mainly of calcite staining (e.g. Margate Quarry, Plate 3.2), fine grained to coarse grained, angular tillite fragments (Plate 3.1) and to a lesser extent of quartz (e.g. Westville Quarry – Face 4, Plate 3.3). Soft clay infill was rarely observed and was only found to be associated with the low angle joint set (J3) in areas where water seepage occurred.

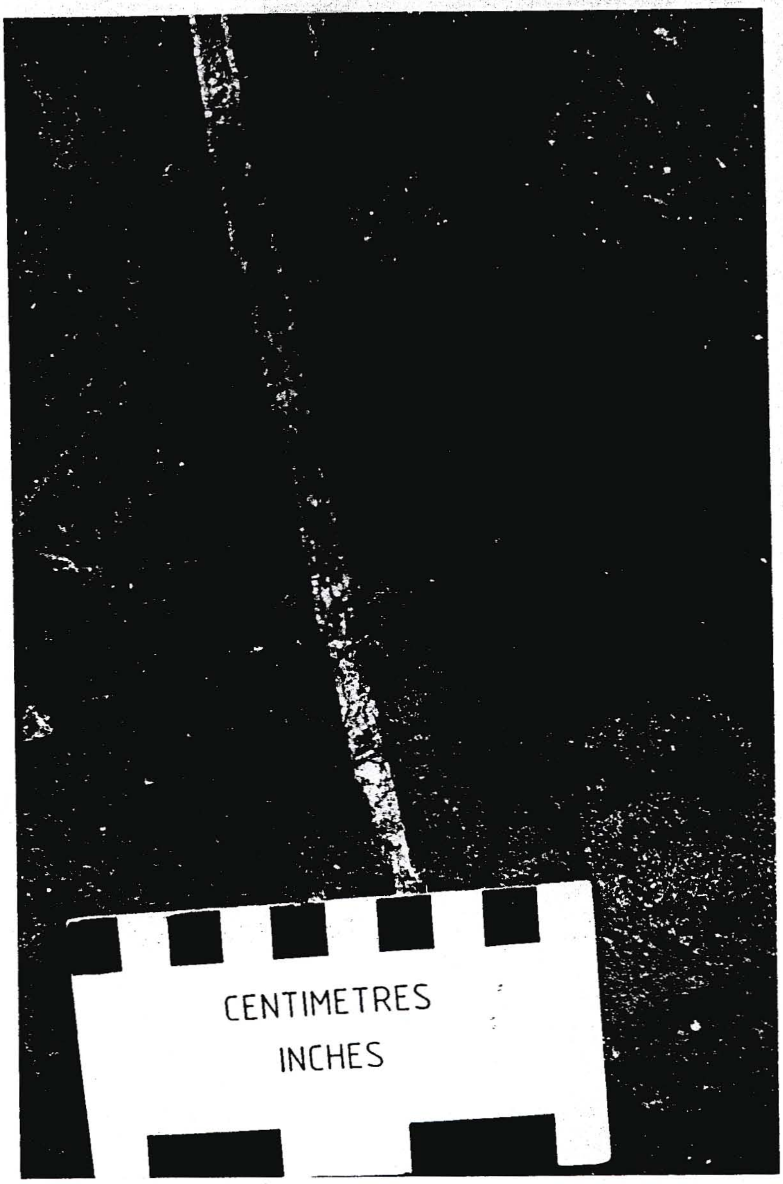
Plate 3.2: Calcite staining on joints surfaces at the Margate Quarry.

3.3.4 Joint roughness

Joint roughness is an important factor contributing towards slope stability as it determines the shear strength of the discontinuity. Overall the joint roughness in the Dwyka tillite is very variable and generally ranges between rough planar, smooth undulating to rough undulating (see Appendix A for description of the roughness profiles). Roughness profiles are similar for all the different joint sets. Field observation does however show that, compared to the other quarries, joints in the Zululand Quarry generally are more planar in nature and fall within the rough planar category.

On a macro scale joints are generally planar but occasional curved joint profiles were noted (e.g. Ridge View Quarry – Face 6, Plate 3.4).

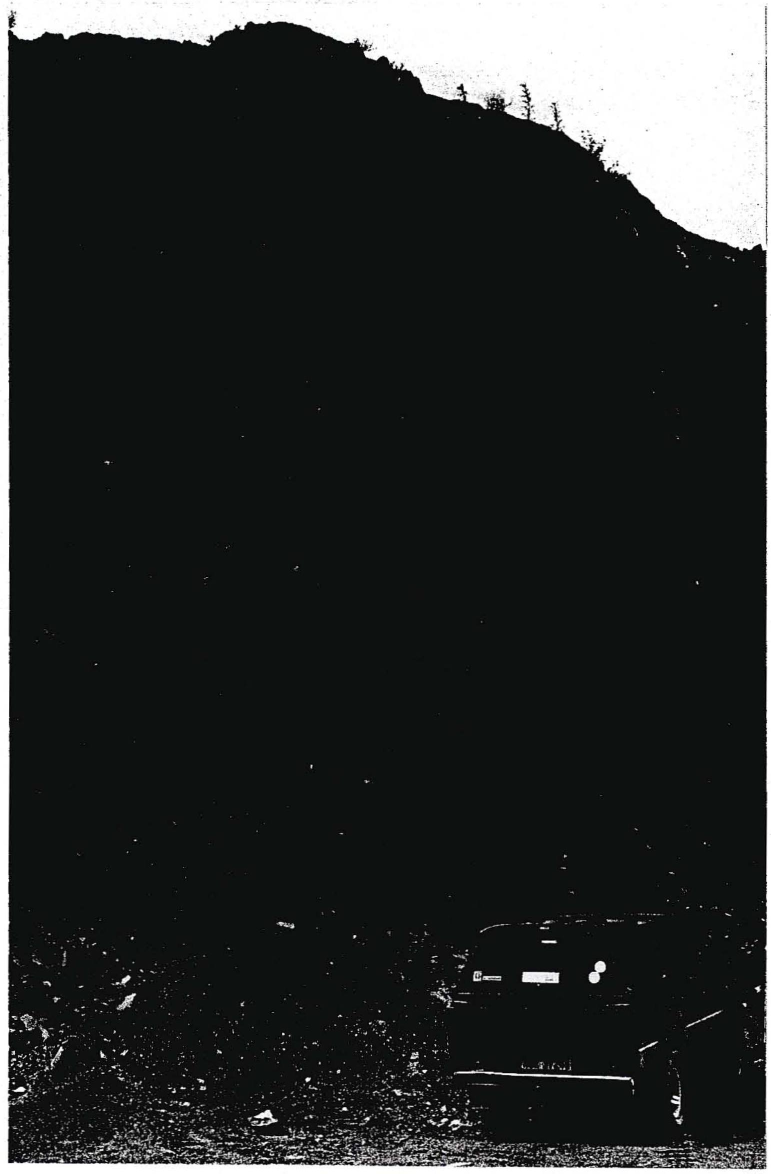
Plate 3.3: Quartz veining at Westville Quarry, Face 4.



3.3.5 Other structural features

No other structural features, such as faults and shear zones, were observed in the rock faces covered by the discontinuity survey lines. Three major faults were however observed in the north-western part of the Zululand Quarry (see Photographic Summary in Appendix G as well as Plate 3.5 and Plate 3.6). Two dolerite dykes, up to 3 m wide and

Plate 3.4: Curved joint planes.



trending in a NE-SW direction, were also noted. Two of the faults and the dykes are plotted on a stereonet in Figure 3.3.

Figure 3.3: Stereonet plot of Faults and Dykes – Zululand Quarry.

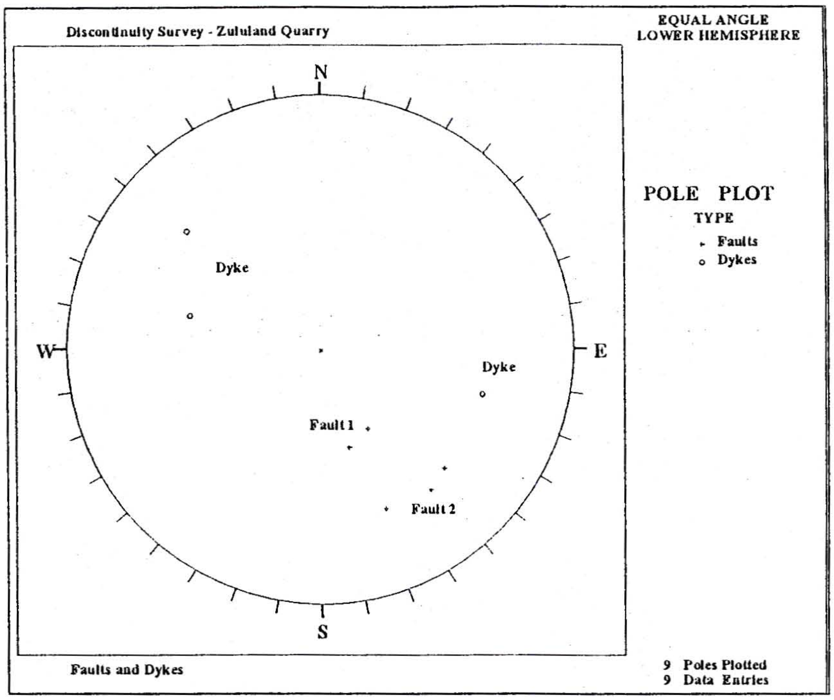


Plate 3.5: Fault 1 – Zululand Quarry. Note slickensided stepped fault plane.

Plate 3.6: Fault 3 – Zululand Quarry. Note slope failure on right and top centre of photo.

The faults are a major contributor towards slope instability of the north-western section of the quarry. Continuous slope failure along Fault 3 (Plate 3.6) made the area unsafe to work in.

4. Shear Testing of discontinuities

Apart from the geometry of the rock mass, the next most important factor when analysing the stability of a rock slope, is the shear strength of the potential failure surfaces. Determination of reliable shear strength data is critical as small changes in shear strength can result in significant changes in the safe height or angle of the slope. It is important to not only obtain accurate test data but also to modify these results to account for the differences between the laboratory shearing process and that anticipated in the rock mass. Differences in the shear strength of joint surfaces can occur due to the influence of weathering, surface roughness, nature of the joint infilling and the presence of water.

A total of 13 shear test where carried out on joint surfaces obtained from core samples, which were supplied by Drennan Maud and Partners, Durban. The core was recovered during the 1986 exploratory diamond-drilling programme carried out in Dwyka tillite for the planning of the proposed Inanda, Northern and Newlands Expressways, Natal.

The joint surface of each sample was described in detail in terms of weathering grade, roughness, aperture, hardness, nature of joint filling and estimated JRC value (Appendix D).

4.1 Determination of dry density

For the calculation of shear strength values, the dry density of each core sample was determined (Table 4.1). Small sub-samples were weighed, immersed in water for volume determination and then placed in an oven for seven days. The weight of each sample was measured after 48 hours and at the end of the seven-day period. The weight loss was then determined and the dry density calculated. In order to obtain an average dry density for each weathering grade, the samples were grouped together based on the degree of weathering of the rock.

Table 4.1: Dry density determination

Sample No.	Weath. Grade	Vol. (ml)	Bulk wt.(g)	After 48 hrs Dry wt. (g)	Wt. Loss (g)	Density (g/cm^3)	After 7 days Dry wt. (g)	Wt. Loss (g)	Density (g/cm^3)
WQ/1/2	1	21	59.0	58.6	0.4	2.79	58.6	0.4	2.79
MAR/3/1	1	40	114.4	113.9	0.5	2.85	113.8	0.6	2.85
BH4/5	1	103	288.7	288.0	0.7	2.80	287.9	0.8	2.80
BH4/10	1	112	303.9	300.2	3.7	2.68	300.2	3.7	2.68
BH3/4	1	73	167.8	167.0	0.8	2.29	166.9	0.9	2.29
BH1/3	2	43	111.4	110.5	0.9	2.57	110.4	1.0	2.57
Average						**2.66**			**2.66**
BH2/16	3	93	243.6	240.1	3.5	2.58	239.4	4.2	2.57
BH2/15	3	85	217.9	214.4	3.5	2.52	213.8	4.1	2.52
BH4/7	3	17	41.1	40.5	0.6	2.38	40.4	0.7	2.38
BH2/14	3	35	80.2	78.9	1.4	2.25	78.6	1.6	2.25
BH2/13	3	26	73.3	71.6	1.7	2.75	71.5	1.8	2.75
BH4/8	3	46	107.4	105.9	1.5	2.30	105.6	1.8	2.3
Average						**2.47**			**2.46**

The difference in the average dry densities between the two time periods is negligible. The average dry densities used in subsequent calculations are 2.66 g/cm^3 and 2.46 g/cm^3. These values are slightly lower than those determined by Paige-Green (1975) who obtained 2.74 g/cm^3 and 2.63 g/cm^3 for weathering grade I & II and grade III respectively.

4.2 Shear box testing

The Golder Associates shear box as described by Henscher and Richards (1989) and Hingston (1997) was used for the shear testing. Samples were prepared by wiring together and casting joint pairs in gypsum plaster, using moulds for the Golder shear box. Prior to pouring the plaster, the moulds were generously lined with LM grease and by using a specially designed jig the cast could be easily removed from the moulds.

The joint angle relative to the core axis differed from sample to sample, resulting in a different surface area for the joint. Each sample was therefore sheared with a constant weight of 20 kg on the hanging system of the shear box resulting in normal loads ranging between 0.78 MPa and 1.69 MPa. The normal loads were calculated using the following formula (Hingston, 1997):

$$y = 0.112207x + 0.156859 \qquad \text{Equation 4.1}$$

Where y = normal load and

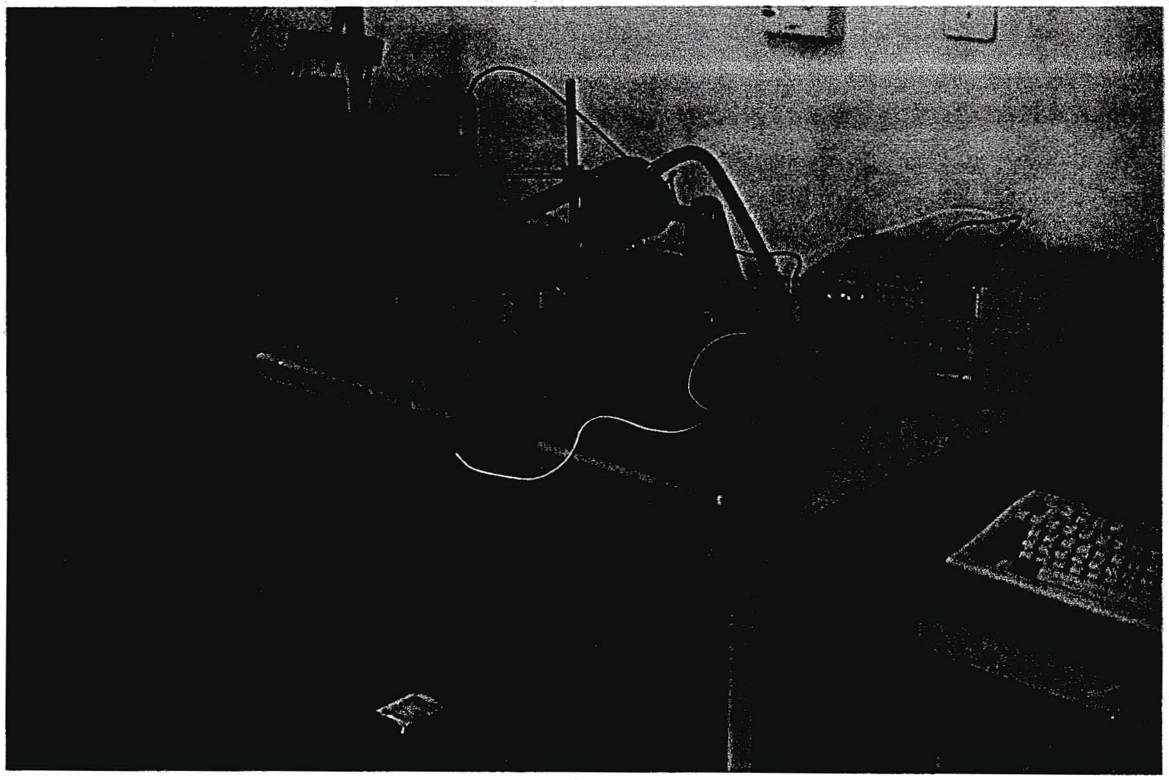
x = weight on hanger

The normal stress was than calculated by dividing the shear load by the surface area of the joint.

4.2.1 Testing procedure

The two moulds were placed in the shear box in such a way so that the plane of the discontinuity coincided with the shear box movement. The lower box was placed on a low friction bearing plate. A ball bearing was placed on the upper mould before lowering the lever into position. The wires that were used to hold the joint pairs in position during preparation were then cut. Great care was taken to ensure that the ends of the wires did not interfere with the actual testing. The pre-determined weight of 20 kg was then placed on the hanger system. Any slack in the system was eliminated by tightening a wing-nut attached to the top mould and the back of the shear box apparatus. The load cell and the vertical and horizontal displacement transducers were then installed and zeroed. The vertical displacement transducer was positioned on the loading arm such that the measured displacement was amplified by a factor of eight.

Plate 4.1: Golder Shear Box



Shearing commenced by slowly pumping a hydraulic jack, which sheared the lower mould relative to the upper mould. The computer, using Windmill software, automatically recorded the reading from the various transducers and the load cell. The test data was transferred to a spreadsheet, which allowed for further analysis. Results are summarized in Table 4.2. In order to obtain better residual strength values each sample was re-sheared in the same direction at the same normal stress.

4.2.2 Results and Discussion

The test data and the plots of shear stress versus horizontal displacement and horizontal displacement versus vertical displacement are found in Appendix E. Peak and residual shear strengths were determined from the shear stress versus horizontal displacement graphs.

Table 4.2: Shear test results

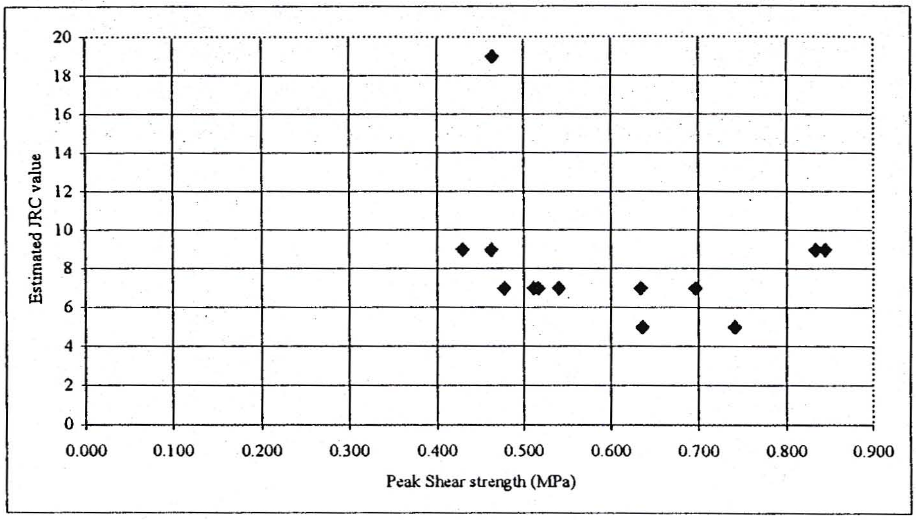
			Test 1		Test 2	
Sample No.	Normal stress (MPa)	Estimated JRC value	Shear strength (MPa)		Shear strength (MPa)	
			Peak	Residual	Peak	Residual
3	1.03	5	0.636	0.400	0.584	0.450
4	0.82	19	0.464	0.230	0.691	0.620
16	0.78	7	0.634	0.329	0.379	0.350
15a	0.82	9	0.429	0.285	0.400	0.310
15b	0.98	7	0.466	0.430	0.479	0.371
15c	0.96	7	0.477	0.403	0.466	0.450
15d	0.83	5	0.741	0.400	0.370	0.350
13	1.69	7	0.696	0.530	0.840	0.693
14	1.01	9	0.463	0.300	0.470	0.400
5a	0.98	9	0.844		0.838	0.530
10b	0.93	7	0.517		0.440	0.477
7	0.82	7	0.511	0.345		0.345
8	0.96	9	0.833	0.700	0.741	0.690

Note: JRC values above have been estimated using Barton and Choubey (1977) figures. The estimates are very subjective due to the small joint surface area derived from the core.

In some instances no residual values for the first test were recorded and this can either be attributed to a too short shearing distance, which was controlled by the surface area of the sample, or by the continual decrease in shear stress even after extended horizontal displacement.

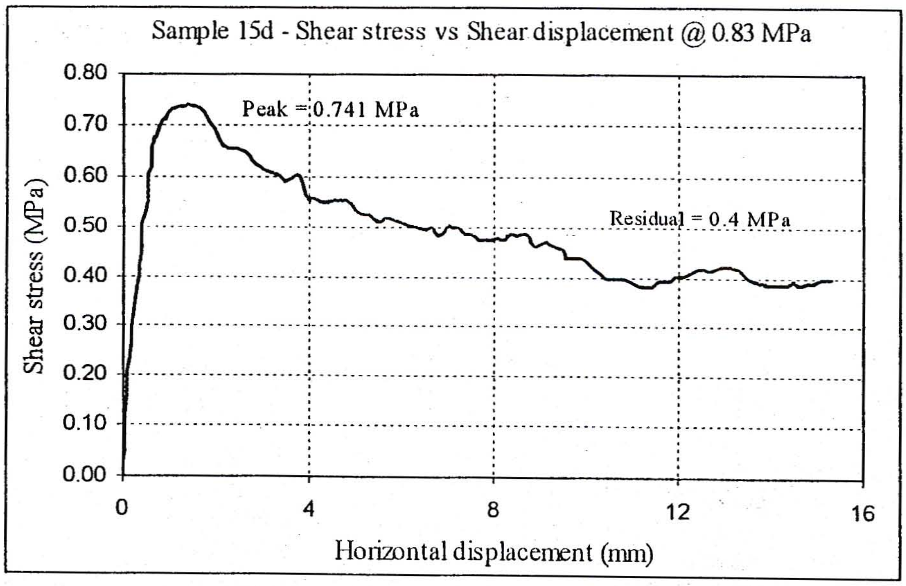
The roughness (estimated JRC value) of each joint surface that was tested was compared to the peak shear strength obtained from the first test (Figure 4.1). The plot shows that there is no correlation between the two. This could be as a result of the different normal stresses used and perhaps due to the weathered nature of the joint surfaces.

Figure 4.1 Plot of the estimated JRC value vs Peak shear strength.



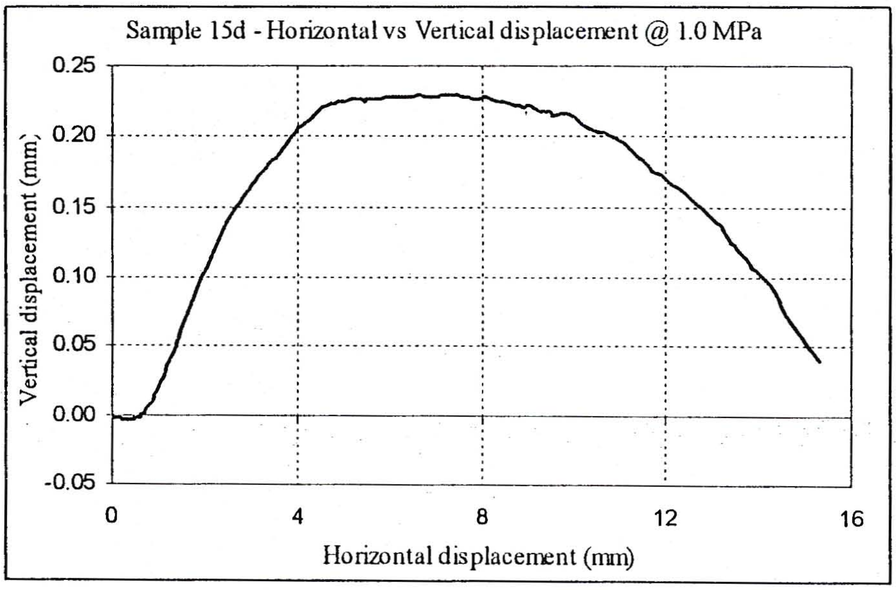
The shear stress versus horizontal displacement graphs show, with the exception of samples 5a, 15b and 4ar (re-shear), a rapid rise in shear strength at relatively small horizontal displacements.

Figure 4.2: Sample 15d – Shear stress (MPa) vs Horizontal displacement (mm).



Peak shear strength was in most instances reached after 1mm to 3mm of horizontal displacement. This was followed by a decrease in shear strength with continued horizontal displacement, until residual shear strength was reached. In some cases secondary, but smaller peaks were recorded (e.g. samples 3, 4a and 15a) and this can be attributed to additional interlocking of asperities. In other cases (e.g. samples 5a and 10b) shear strength continued to decrease with continued horizontal displacement until the end of the test. This could be due to the weathered nature of samples resulting in the partial disintegration of the joint surface with continued shearing. The vertical displacement versus horizontal displacement graphs showed varied profiles.

Figure 4.3: Sample 15d – Vertical displacement (mm) vs Horizontal displacement (mm).



In some cases (e.g. samples 15d, 16, 7) an initial small decrease in vertical displacement was noted, followed by a sudden and continued increase in dilation with horizontal displacement. This change coincides in all instances with peak shear strength. Continued horizontal displacement finally resulted once more in a gradual decrease in vertical displacement. Samples 14, 14r, 7r and 15dr displayed a continued decrease in vertical displacement, possibly due to the weathered nature of the samples, throughout the tests. Samples 8 and 4r showed no change in vertical displacement until peak shear strength was reached after which positive dilation took place throughout the remainder of the shear process. These two samples both had high estimated JRC values. There appears to be no correlation between the roughness of the joint profiles and the amount of dilation during shearing.

The main disadvantage for having carried out each shear test at single normal loads is that no value for cohesion could be determined. For the slope stability analysis (Chapter 7) an initial estimated cohesion value of 100 kPa is used.

For the purpose of this project the shear test results will not be used for the slope stability analysis of the quarries. Firstly, the core samples tested are not considered to be representative of the quarry areas investigated. Secondly, the core will also have been subjected to some degree of weathering during storage. This may have affected the shear strength of the tested joint surfaces.

5. Empirical determination of shear strength

A number of parabolic empirical strength criteria to calculate peak shear strength have been reported in the literature e.g. Barton and Choubey, 1977; Hoek and Bray, 1977. For the purpose of this report the empirical relationship first described by Barton and Choubey (1977) will be used to back analyse the shear strength of joint surfaces and JRC values.

In 1973 Barton first described the following empirical relationship:

$$\tau = \sigma_n \text{ x } \tan\left(\text{JRC x } \log_{10}\left(\frac{\text{JCS}}{\sigma_n}\right) + \phi_b\right) \qquad \text{Equation 5.1}$$

where τ = peak shear strength

σ_n = effective normal strength

JRC = joint roughness coefficient

JCS = joint wall compressive strength

ϕ_b = basic friction angle

The above equation can also be re-written to back analyse for JRC:

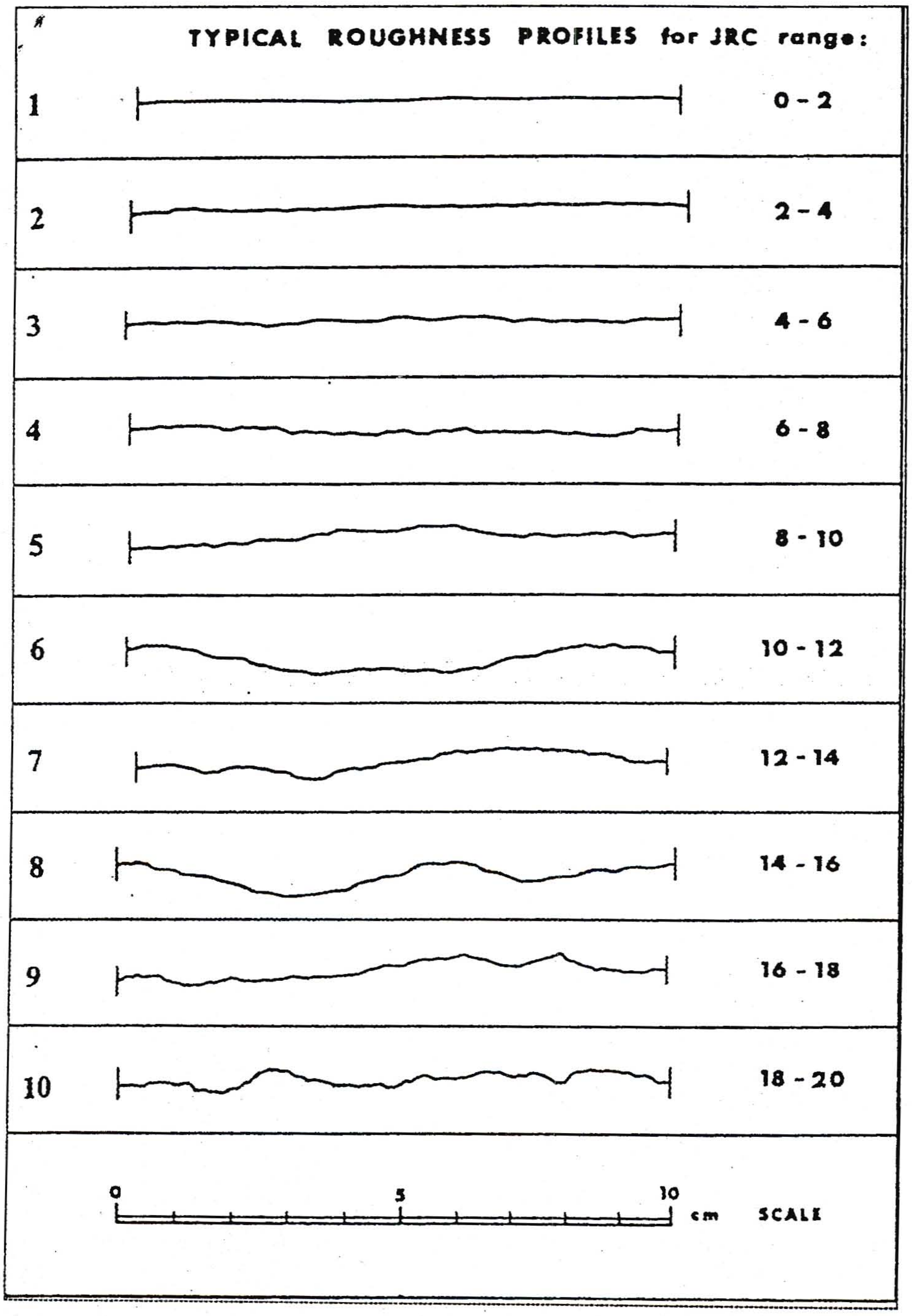
$$\text{JRC} = \left[\left(\arctan\frac{\tau}{\sigma_n}\right) - \phi_b\right] \div \left[\log\left(\frac{\text{JCS}}{\sigma_n}\right)\right] \qquad \text{Equation 5.2}$$

In order to solve for the peak shear strength it is necessary to determine the three index parameters namely JRC, JCS and ϕ_b. These index values can be measured in the laboratory.

5.1 Joint Roughness coefficient

Barton and Choubey (1977) used back-calculated JRC values from numerous joint specimens to compile 10 different profiles of increasing roughness. Each profile was assigned a specific JRC value and these are reproduced in Figure 5.1.

Figure 5.1: Joint roughness profiles with JRC values (Barton and Choubey, 1977).



By comparing the roughness profiles of joints studied with the above standard profiles it is possible to estimate the JRC values for those joint surfaces. The method of visually estimating the closest profile is very subjective and unreliable. To partially overcome this problem, a carpenter's comb was used to obtain, where possible, three profiles for each joint. Ideally each profile should have been drawn and than digitised to determine the JRC value using a program called JRCTSEC (pers. comm. C. Jermy, 1998). Due to the vast number of joints surveyed during this study, this was not practical and an average JRC value (corresponding to Barton and Choubey roughness profiles) was estimated from the three profiles. Appendix B lists the modal JRC value for each joint set for each face from the different quarries. JRC values have also been estimated on the thirteen samples used for the shear box test (Appendix D).

5.2 Joint Wall Compressive Strength

The strength of the rock mass as a whole is largely determined by the strength of the rock adjacent to the joint wall. If the joints are completely unweathered then JCS will be equal to the unconfined compressive strength (σ_c) of the unweathered rock. The compressive strength can be determined from laboratory tests such as the UCS test or point load test. In most cases joint walls are weathered to some extent, in which case the JCS will be lower than σ_c. JCS values can than be measured by using a Schmidt Hammer Rebound Test.

5.2.1 Unconfined Compression Strength Test

Five core samples, derived from the same boreholes as the shear box samples, were used for UCS testing. The recommendations given by ASTM (1979) were used for these tests. The results of the tests are tabulated below.

Table 5.1: Unconfined compressive strength results

Sample Number	**UCS (MPa)**	**Weathering Grade**	**Specimen Description**
BH1 – B	59.9	Grade 1	Strong, unweathered, blue tillite.
BH1 – C	52.1	Grade 1	Strong, unweathered, blue tillite.
BH2 – A	28.6	Grade 2	Medium strong, slightly weathered, blue to yellow brown tillite.
BH2 – B	11.9	Grade 3	Soft to medium strong, moderately weathered, yellow brown tillite.
BH4 – A	83.0	Grade 1	Very strong, unweathered, light pinkish brown tillite.

The average UCS value obtained for the Grade 1 Dwyka tillite is 65 MPa. This value and the UCS values obtained for the Grade 2 and Grade 3 rock are very low when compared to average UCS values of the same grade of Dwyka tillite taken at various inland localities in KwaZulu - Natal by Brink, 1983 (Table 5.2).

Table 5.2 UCS values of Dwyka tillite in Natal (Brink, 1983).

Weathering Grade	**Maximum (MPa)**	**Minimum (MPa)**	**Mean (MPa)**
Grade 1	222	56	146.1
Grade 2	130	80	107
Grade 3	82	59	73.5

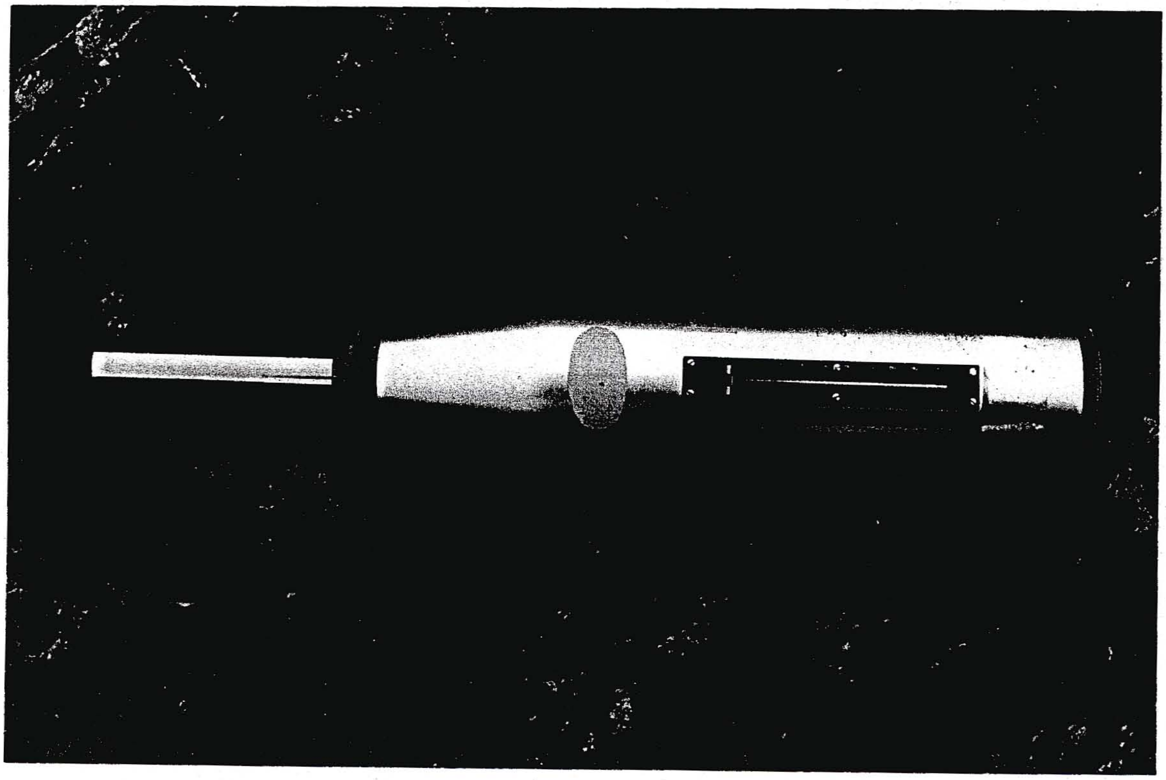
The low UCS values obtained from the Inanda Express Way core is to be expected as the core has been in storage since 1986 and has therefore been subjected to a fair amount of weathering. During the study it was mostly dealt with very strong to strong (50 – 250 MPa; Barton et.al., 1978), unweathered to slightly weathered rock (Grade 2) and a mean UCS value of 107 MPa, as determined by Brink, 1983 (Table 5.2) is therefore considered to be more representative of the tillite in the quarries studied.

5.2.2 Schmidt Hammer Rebound Test

The conventional unconfined compression tests and point load tests are not really representative when evaluating slight reductions of joint wall strength, which in turn results in the JCS becoming less than σ_c. In this case the thickness of material controlling the shear strength may be only a fraction of a millimetre for planar joints or a few millimetres for rougher joints.

The L-type Schmidt hammer (Plate 5.1) has been adapted for recording the rebound of a spring loaded plunger after its impact with a joint surface.

Plate 5.1: Schmidt Hammer



It is suitable for measuring JCS values between 20 MPa and 300 MPa (Barton and Choubey, 1977). The rebound number "R" was found to have a correlation to the unconfined compressive strength σ_c when multiplied by the dry density of the rock (Deere and Miller, 1966):

$$\log_{10}(\sigma_c) = 0.00088 \times \gamma \times R + 1.01 \qquad \text{Equation 5.3}$$

where σ_c = unconfined compressive strength (MPa)

γ = dry rock density (kN/m^3)

R = rebound number

Schmidt Hammer rebound tests were carried out on all joint surfaces measured. Although the International Society of Rock Mechanics (ISRM) recommends a total of 20 tests on each joint surface (Barton et.al., 1978) this was often not possible due to the small area of joint surface exposed. An average of between three to ten readings was taken. The ISRM also stipulates that the upper 50% of values be averaged to omit unexpected low readings due to the crushing of loose grains or block movement. Göktan and Ayday (1993) suggested that such a recording technique might not accurately reflect the strength variation caused by rock texture and micro-structure on rock surfaces and all values should therefore be included for any analyses. Due to the limited number of readings taken during this study, all values were included to obtain a mean value.

The Schmidt hammer rebound number reaches a maximum when the instrument is held vertically downward and a minimum when the hammer is held vertically upwards. During the study readings were taken at various angles, necessitating applying a correction factor to those values taken when the hammer was not in a vertical downward position (Table 5.3).

Table 5.3: Schmidt Hammer correction factors (Barton et.al., 1978)

Rebound	Downwards		Upwards		Horizontal
R	α = -90°	α = -34°	α = +90°	α = +45°	α = 0°
10	0	-0.8			-3.2
20	0	-0.9	-8.8	-6.9	-3.4
30	0	-0.8	-7.8	-6.2	-3.1
40	0	-0.7	-6.6	-5.3	-2.7
50	0	-0.6	-5.3	-4.3	-2.2
60	0	-0.4	-4.0	-3.3	-1.7

The correction factors were applied after an average Schmidt hammer value was obtained for each joint. All values for a particular joint set were than averaged to obtain a mean value for that particular joint set (Appendix B).

Equation 5.3 was applied to the mean corrected Schmidt Hammer values for each joint set using a dry density value of 26.66 kN/m^3 (see Chapter 4). The results summarised as per weathering grade are as follows:

Table 5.4: Average joint wall compressive strength values per weathering grade derived from the Schmidt Hammer.

Weathering Grade	**Mean JCS value (MPa)**	**Average dispersion of strength (MPa)**
Grade 1	117.04	± 70
Grade 2	121.51	± 45
Grade 3	42.01	± 30

The average dispersions (Barton et. al., 1978) of the above mean JCS values are very high. These values are therefore not considered accurate enough for comparison to other strength values such as those derived by Brink (1983).

5.3 Basic Friction Angle

The residual tilt test is a shear test under very low normal stress and is an effective way of measuring the basic friction angle (ϕ_b), which varies with surface texture and mineralogy of the joint surface. Two different tests were used to determine ϕ_b.

5.3.1. Stimpson's tilt test

Two pieces of core of equal length are placed adjacent to one another on a tilting table. A third piece of core, which was free to slide, was placed on top. The table is than inclined until the sliding of the upper piece of core occurs. The angle of inclination or tilt (α) is measured. The following formula is used to calculate the basic angle of friction (Stimpson, 1981):

$$\phi_a = \tan^{-1}[1.155 \times \tan(\alpha)] \qquad \text{Equation 5.4}$$

where α = angle of inclination.

Three tests, each comprising 10 sliding experiments, were carried out on core from the Inanda Expressway exploration programme (Table 5.5). The weathering grade of the samples used for this test was 2 (slightly weathered rock).

Table 5.5: Results of Stimpson's tilt test.

	Test 1	**Test 2**	**Test 3**
Tilt angle (α°)	34	35	36
	32	35	36
	32	35	36
	33	35	36.5
	31	35	35
	33	35	36
	33	31	35
	32	35	35.5
	32	34	35.5
	33	35	35
Average	32.5	34.5	35.6
Basic Friction angle	**36.3**	**38.4**	**39.5**

An average basic angle of friction (ϕ_b) of 38.1° was obtained.

5.3.2. Residual Tilt Test

The residual tilt, used by Barton and Choubey (1977), involves cutting samples of rock or core with a diamond saw. After cleaning the surfaces and air-drying of the samples, pairs of sawn surfaces are mated and tilted, using a similar apparatus as used in the Stimpson's tilt test. The tilt angle at which sliding occurred was measured. Rock samples from the Westville and Margate Quarry as well as core from the Inanda Express Way exploration programme was used. Ten tests were performed on each sample. The results are summarised in Table 5.6. Samples with a weathering grade of 2 have a higher average friction angle (35.9°) than unweathered grade 1 samples (33.4°). This is to be expected, as weathered core will most likely exhibit rougher surfaces compared to fresh rock. An overall average basic friction angle (ϕ_b) of 34.5° was obtained.

Table 5.6: Results of Residual Tilt Test

Sample No.	Alpha (α)									Average	Basic friction angle (ϕ_b)
Grade 1 samples											
WQ/1/1	32	33.5	32.5	33	33	32	32.5	32	32	31.5	32.4	36.2
	25	26	25	26	25	25.5	26	26.5	26	26.5	25.8	29.1
	31	32	32	31	32	31.5	33	31.5	31.5	31	31.7	35.4
	27	28	29	28	28	28	28	27	28	29	28.0	31.6
WQ/1/2	22.5	22	22	22	22	2.5	22	23	22	22	22.2	25.2
	39	39	39	28.5	39.5	38.5	38.5	39	38	38.5	37.8	41.8
MAR/3/1	30	33	30	31	30	30	31	30	31	30	30.7	34.4
											Average	**33.4**
Grade 2 samples											
BH3/4	36	36	35	35	35	36	35	35	36	35	35.4	39.4
	32	30	32	33	34	32.5	32	31.5	32	32	32.1	35.9
BH2/16	29	28.5	29	29	32	29	30	29	29	28.5	29.3	32.9
BH4/10	31	29	28	30	32	29.5	30	29	29	30	29.8	33.4
BH4/5	36	36	34	33.5	33	33.5	34	34	33.5	34	34.2	38.1
											Average	**35.9**
											Overall Average	**34.5**

It was found that Stimpson's method yielded higher values for the basic friction angle. For the purpose of determining the empirical shear strength of the tillite, it was decided to use the average basic friction angle values as derived by the Barton and Choubey method.

5.4 Back analysis calculation

Using the values (JCS = 65 MPa and $\phi_b = 33.4°$ for grade 1 rock and $\phi_b = 35.9°$ for grade 2 rock) obtained and discussed in the above sections, Barton & Choubey's empirical equations (Equation 5.1 & 5.2) were used to determine expected peak shear strength and JRC values. Back analysis was carried out on the 13 samples used for shear testing.

5.4.1 JRC

Knowing JCS, ϕ_b, the normal stress (σ_n) and peak shear strength (τ) (σ_n and τ values obtained from the shear testing - see Chapter 4), an attempt was made to back calculate JRC. These were then compared to JRC values physically estimated, using roughness profiles by Barton and Choubey, from the joint surfaces on the core samples before shear testing. These estimated JRC values are not considered to be completely accurate due to the small sample area of the core.

It is obvious from the results that there is no comparison between the estimated and calculated values (Table 5.7). In fact most calculated values are negative. This indicates that either the JCS or ϕ_b or τ values are incorrect. Varying ϕ_b or τ in the equation did not make a significant change in the back-calculated JCS values. The peak shear strength values obtained from the shear testing are therefore too low, perhaps due to premature failure of samples. The weathered nature of the joint surfaces and joint infilling, joint pairs not exactly in position or having undergone some displacement before shearing commenced could lead to premature failure.

5.4.2 Peak shear strength

Applying Equation 5.1 and using the same variables as above the values of peak shear strength were calculated. The physically measured JRC values of each individual joint surface were used in place of the calculated values (Table 5.7). All of the back-analysed peak shear strength values are much higher (in one case as much as five times) than those obtained from the actual shear box tests. Although sample preparation could affect peak shear strength during testing, the consistent low values obtained suggest that this is minimal. The low values can again be contributed to those reasons mentioned in the paragraph above. Another possible explanation could be the small joint surface area presented by a core sample.

Table 5.7 Summary of results from the back analysis calculations.

Sample No.	Normal Stress (MPa)	JRC Values		Peak shear strength (MPa)		Normalized Peak shear strength at a stress of 1Mpa
		Estimated	Back analysis	Shear Box Test	Back analysis	
3	1.03	5	-0.95	0.636	0.941	0.915
4	0.82	19	-2.04	0.464	2.191	2.456
16	0.78	7	1.67	0.634	0.908	1.134
15a	0.82	9	-4.35	0.429	1.088	1.290
15b	0.98	7	-3.87	0.466	1.114	1.134
15c	0.96	7	-5.17	0.477	1.093	1.134
15d	0.83	5	3.08	0.741	0.841	0.999
13	1.69	7	-8.02	0.696	1.812	1.134
14	1.01	9	-6.25	0.463	1.301	1.290
5a	0.98	9	4.02	0.844	1.159	1.180
10b	0.93	7	-3.70	0.517	1.063	1.134
7	0.82	7	-2.08	0.511	0.950	1.134
8	0.96	9	2.76	0.833	1.245	1.290

The peak shear strengths for the 10 different roughness profiles and for varying normal stresses and with JCS = 65 MPa and the basic friction angle = 34.5° are presented in Table 5.8.

Table 5.8: Peak shear strengths (MPa) for varying roughness profiles and normal stresses.

JRC	Normal Stress (MPa)				
value	0.5	1.0	2.0	5.0	10.0
0 – 2	0.372	0.735	1.454	3.581	7.084
2 – 4	0.432	0.837	1.622	3.884	7.519
4 – 6	0.501	0.951	1.805	4.206	7.973
6 – 8	0.581	1.080	2.006	4.549	8.448
8 – 10	0.676	1.227	2.230	4.918	8.946
10 – 12	0.793	1.399	2.481	5.316	9.470
12 – 14	0.940	1.605	2.768	5.748	10.024
14 – 16	1.134	1.857	3.101	6.221	10.609
16 – 18	1.407	2.176	3.492	6.742	11.231
18 – 20	1.823	2.598	3.964	7.320	11.894

As one would expect, the peak shear strength increases with increasing roughness and normal stress.

6. Rock Mass Classification of the Dwyka Tillite

Over many years it has become increasingly popular to make use of rock mass classification systems in the civil and mining engineering industry. The purpose is to:

- Identify the most important parameters influencing the behaviour of a rock mass;
- Divide the rock mass into several classes depending on its quality;
- Derive quantitative data for engineering design;
- Provide a common basis for communication between engineers and geologist;
- Recommend support guidelines for tunnels, mines and rock slopes.

One of the benefits of such a system is to improve the quality of a site investigation by collecting classification parameters and to enable better engineering judgement. It should however be noted that any classification system would never give the ultimate solution to any design problem. In addition most rock mass rating systems cater for tunnel design and support and not for open rock slopes.

The output of any classification system is only as good as the input data. A selection of parameters, each with varying significance, is required to fully describe a jointed rock mass. These can be summarised as follows:

- The strength of the rock material such as the uniaxial compressive strength;
- The rock quality designation (RQD), which is a measure of the drilling quality, first developed by Deere in 1967 (Hoek, 1998), can be obtained from measuring the length of core pieces. Palmstrøm (1985) suggested that if no core is available but discontinuity surveys are possible on surface exposures the RQD might be estimated from the number of discontinuities per unit volume (J_v). The suggested relationship is as follows:

  $$RQD = 115 - 3.3J_v \qquad \text{Equation 6.1}$$

  $$(RQD = 100 \text{ for } J_v < 4.5)$$

  Using this method was not always successful as the number of joints of each set is to be counted along perpendicular scan lines. This made it almost impossible to count near horizontal joints up a slope face. Where this method was attempted a J_v value of

< 4.5 was obtained which equated to a RQD value of 100. From experience this is not a true reflection of the rock quality. For this project the RQD was calculated in the same way as from drill core but using the survey line instead. The RQD can therefore be defined as:

$$\text{RQD} = \frac{\sum \text{Length of rock sections between joints} > 10 \text{ cm length}}{\text{Total length of the survey line}} \times 100\%$$

Equation 6.2

- The basic geological parameters collected during a discontinuity survey (see Chapter 4) were:
- Water conditions
- Stress field and
- Major faults and folds

There are four major systems available used for the quantification of rock mass parameters and which to a degree relates a final index that can be used for estimating the stability of rock slopes. These are the "Q" Index of Barton (1976), the Geomechanics Classification or Rock Mass Rating (RMR) system developed by Bieniawski (1976), the Mining Rock Mass Rating (MRMR) of Laubscher (1990) and the Slope Mass Rating (SMR) classification system by Romana (1997). Each of the above systems will only be explained briefly as the detailed description and explanation to the usage of each system falls outside the scope of this report. The results obtained from each of the various systems are summarised in Table 6.1. Classification Parameters and their ratings of each system are listed in Appendix F.

6.1 Rock Tunnelling Quality Index, Q

Barton et al (1974) of the Norwegian Geotechnical Institute have developed a Tunnelling Quality Index, Q, for the determination of rock mass characteristics and tunnel support requirements. The rock mass quality is a function of six parameters and can be defined as follows:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \qquad \text{Equation 6.3}$$

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
SRF	is the stress reduction factor

In Equation 6.3 the first quotient (RQD/ J_n) represents the structure of the rock mass and is a crude measure of block or particle size. The second quotient (J_r/J_a) represents the roughness and frictional characteristics of the joint wall or filling materials. The third quotient (J_w/SRF) measures the active stress. The rock mass quality can range from Q = 0.0001 to Q = 1000 on a logarithmic rock mass quality scale.

6.2 Geomechanics classification

The Rock Mass Rating (RMR) system or Geomechanics Classification was developed by Bieniawski during 1972 – 1973. The system was refined over the years and for the purpose of this study the 1989 version of the classification is used (Bieniawski (1989).

The following six parameters are used to classify a rock mass using the RMR system:

- Uniaxial compressive strength of the rock material
- Rock Quality Designation
- Spacing of discontinuities
- Condition of discontinuities

- Groundwater conditions
- Orientation of discontinuities

The Rock Mass Rating system is presented in Appendix F, giving the ratings for each of the six parameters listed above. The ratings are summed up to give a RMR value. Based on these values the rock mass can be classified into five classes. It should be noted that the importance rating given for joint spacing apply to rock masses having three sets of discontinuities. When less than three sets of discontinuities are present, the rating for joint spacing may be increased by 30%. (Bieniawski, 1989).

6.3 Laubscher Rock Mass Rating System

Laubscher's Rock Mass Rating (RMR) and Mining Rock Mass Rating (MRMR) classification system was developed to cater for diverse mining situations. The Geomechanics RMR had to be adjusted according to the mining environment so that the final MRMR could be used for mining design. The geological parameters that are required are (Laubscher, 1990):

- Intact rock strength (IRS)
- Rock Quality Designation (RQD)
- Joint/fracture spacing
- Fracture Frequency per metre (FF/m)
- Joint condition/water

The parameters and ratings to obtain RMR as well as the adjustments for the MRMR are shown in Appendix F. Two techniques can be used for assessing the spacing of fracture and joints. The more detailed technique uses the rock quality designation (RQD) and joint spacing (JS) separately, the maximum ratings being 15 and 25 respectively. The second technique measures all the discontinuities and records these as the fracture frequency per metre (FF/m) with a maximum rating of 40. The results of both techniques are very similar (see Figure 6.1 for comparison). Only the results of the first technique are listed in Table 6.1.

One of the advantages of this classification system over the Bieniawski RMR classification system is that although a maximum of a three-joint set is assumed, the system does provide adjusted ratings for a one or two joint set situations.

The RMR is used to calculate the Mining Rock Mass Rating (MRMR) (Table 6.1) using the adjustments ratings found in Tables E, F, G and H in Appendix F. These adjustments, which include the weathering of the rock mass, joint orientation and blasting effects, recognise the life of the excavation and the time-dependant nature of the rock mass. The adjustment ratings are empirical, having been based on numerous observations in the field.

6.4 Romano's Slope Mass Rating System

None of the above classification systems have really been designed for assessing slope instability risks, although Bieniawski's RMR does include a rating adjustment for discontinuity orientations in slopes (Bieniawski, 1976). This adjustment is however considered very limited as no real guidelines were published for the definition of the various classes.

The Slope Mass Rating (SMR) system developed by Romano is obtained from the Bieniawski's RMR by subtracting a factorial adjustment factor depending on the joints-slope relationship and adding a factor depending on the method of excavation (Romano, 1997):

$$SMR = RMR - (F_1 \cdot F_2 \cdot F_3) + F_4 \qquad \text{Equation 6.4}$$

where - F_1 depends on parallelism between joints and slope face strikes.

- F_2 refers to the joint dip angle in the planar mode of failure.
- F_3 reflects the relationship between slope face and joint dips.
- F_4 is the adjustment factor for the method of excavation.

Table 6.1: Results of the Rock Mass Classification Systems.

Site	Face	Barton			Bieniaskwi			Romano			Laubscher (Technique 1)			Laubscher (Technique 2)		
		Q	Class No.	Desc.	RMR	Class No.	Desc.	SMR	Class No.	Desc.	RMR	Class No.	Desc.	RMR	Class No.	Desc.
WQ	1	7.0	III	Fair	69	II	Good	73	II	Good	40	IV	Poor	42	III	Fair
	2/1	3.2	IV	Poor	64	II	Good	71	II	Good	57	III	Fair	51	III	Fair
	2/23	2.4	IV	Poor	58	III	Fair	59	III	Fair	49	III	Fair	52	III	Fair
	4	12.7	II	Good	78	II	Good	85	II	Good	52	III	Fair	47	III	Fair
RVQ	1	3.4	III	Fair	62	II	Good	75	II	Good	41	III	Fair	42	III	Fair
	2	8.1	III	Fair	52	III	Fair	60	III	Fair	41	III	Fair	45	III	Fair
	3	6.0	III	Fair	68	II	Good	71	II	Good	49	III	Fair	56	III	Fair
	4	8.4	III	Fair	67	II	Good	70	II	Good	52	III	Fair	58	III	Fair
	5	4.0	III	Fair	48	III	Fair	56	III	Fair	52	III	Fair	62	II	Good
ZQ	1	29.7	II	Good	79	II	Good	83	I	V. Good	53	III	Fair	51	III	Fair
	2	3.6	III	Fair	71	II	Good	86	I	V. Good	48	III	Fair	57	III	Fair
	3	5.4	III	Fair	52	III	Fair	56	III	Fair	45	III	Fair	52	III	Fair
	4	6.0	III	Fair	70	II	Good	71	II	Good	47	III	Fair	54	III	Fair
	5	11.2	II	Good	78	II	Good	82	I	V. Good	48	III	Fair	49	III	Fair
MQ	1	4.3	III	Fair	61	II	Good	71	II	Good	46	II	Fair	51	III	Fair
	2	3.6	III	Fair	54	III	Fair	67	II	Good	40	IV	Poor	46	III	Fair
	3	17.2	II	Good	43	IV	Poor	51	III	Fair	49	III	Fair	50	III	Fair
UQ	1	3.8	III	Fair	63	II	Good	67	II	Good	55	III	Fair	51	III	Fair
	5	6.7	III	Fair	78	II	Good	84	I	V. Good	59	III	Fair	61	II	Good

Explanations:

WQ	-	Westville Quarry
RVQ	-	Ridge View Quarry
ZQ	-	Zululand Quarry
MQ	-	Margate Quarry
UQ	-	Umkomaas Quarry

Descrption	Stability
V. Good	Completely stable; No failures.
Good	Stable; Some blocks.
Fair	Partially stable; Some joint or many wedges.
Poor	Unstable; Planar failure or big wedges.
V. Poor	Completely unstable; Big planar failure or soil-like.

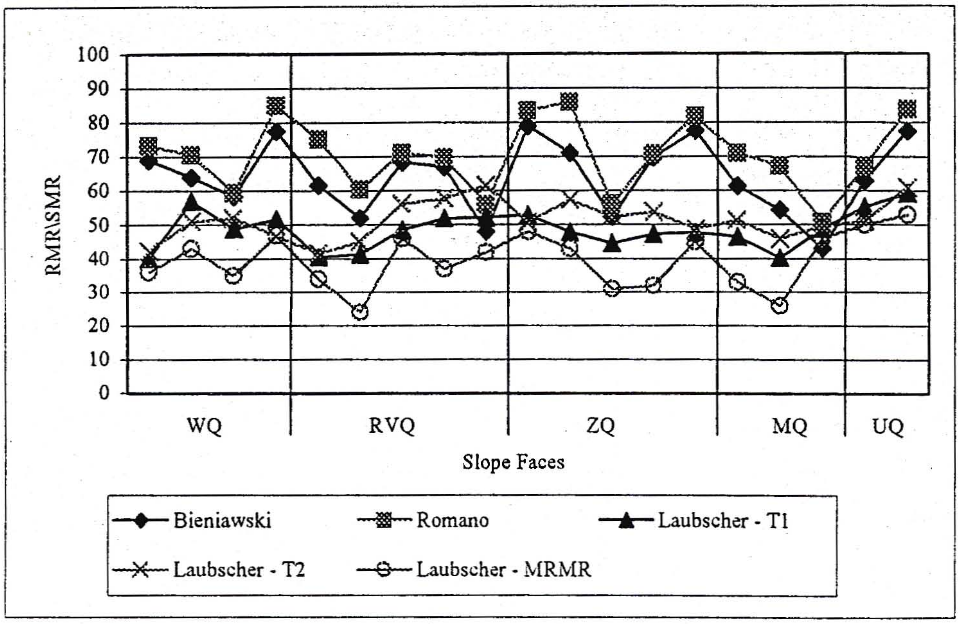
SMR is a very useful tool for the preliminary assessment of slope stability and gives some simple rules about instability mode and the required support measures. It cannot be regarded as a substitute for detailed analysis of each slope.

6.5 Discussion

From Table 6.1 the following conclusions can be drawn:

- Overall the rock mass quality of all slopes analysed fall within the 'Fair' to 'Good' range meaning that slopes are partially stable to stable.
- The Bieniawski (RMR) and Romano (SMR) Rock Mass Classification Systems generally describe the slopes to be more stable than compared to Barton Q value and Laubscher's RMR. Romano's SMR rating even suggests that some slopes (e.g. Zululand Quarry Face 1 and 2) are completely stable.
- The RMR's of Laubscher are generally lower than the ratings of Bieniawski and Romano (Figure 6.1). The RMR values for the first technique of Laubscher's (using RQD and JS) are generally lower when compared to the second technique (using FF/m).
- The RMR values of Bieniawski and Laubscher show a greater variation between the different slopes.
- Laubscher's MRMR values are, as expected, lower than the RMR values. Because of the adjustments to the RMR for weathering, joint orientation and blasting, the MRMR is considered to be a more accurate reflection of the true rock mass quality. Rock mass quality for the slopes studied varies between poor to fair.
- Romano's SMR values are consistently higher than the RMR values of Bieniawski.

Figure 6.1: Line graphs comparing the Rock Mass Rating of Bieniawski, Romano and Laubscher.



6.5.1 Correlation between Bieniawski's RMR and Barton's Q-index.

The relationship between the two most widely used classification indices, the Rock Mass Rating (RMR) of Bieniawski and the Rock Mass Quality (Q) of Barton has been proposed by many researchers. Bieniawski (1976) proposed the following relationship, which was found to be applicable to tunnels:

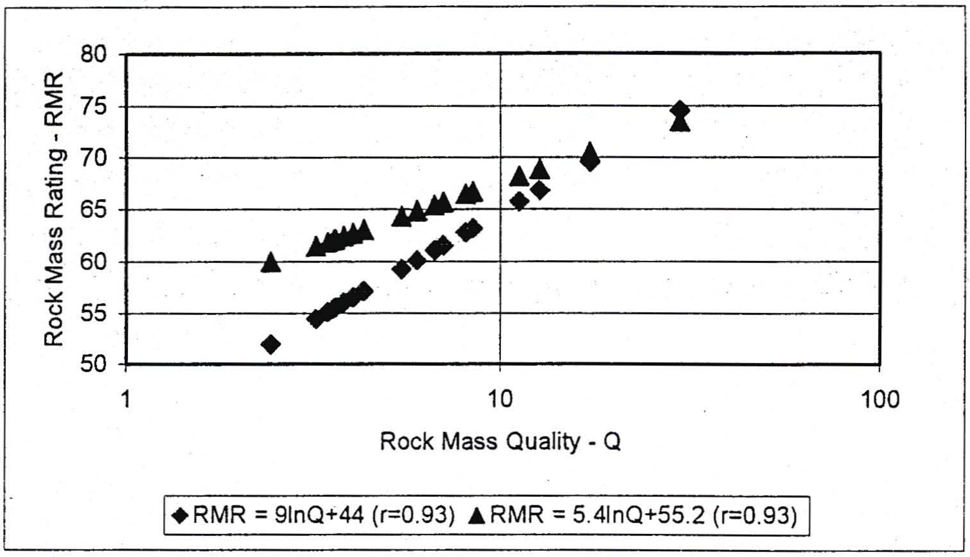
$$RMR = 9 \ln Q + 44 \qquad \text{Equation 6.5}$$

Moreno (Goel et al, 1996) proposed a different correlation between the RMR and Q as presented in Equation 6.6.

$$RMR = 5.4 \ln Q + 55.2 \qquad \text{Equation 6.6}$$

Using Equation 6.5 and 6.6, the results (values from Table 6.1) are plotted in Figure 6.2.

Figure 6.2: Various relationships between RMR and Q.



Good relationships exist between both Bieniawski equation (r = 0.93) and between the relationship proposed by Moreno (r = 0.93). It must, however, be kept in mind that these relationships are based on a very small data set and which does not cover the entire spectrum of possible Q (0.001 to 1000) and RMR values (0 to 100).

A new approach to correlate RMR and Q has been proposed by Goel et al, 1996. They argue that the two rock mass rating systems are not truly equivalent. The RMR does not consider the stress condition of the rock mass, while the Q-system does not consider joint orientation and intact rock strength as independent parameters. Goel et al (1996) defined two new rock indices:

- The rock mass number N, defined as the rock mass quality with SRF (stress reduction factor) as 1;
- The rock condition rating RCR, defined as RMR without rating for joint orientation and intact rock strength.

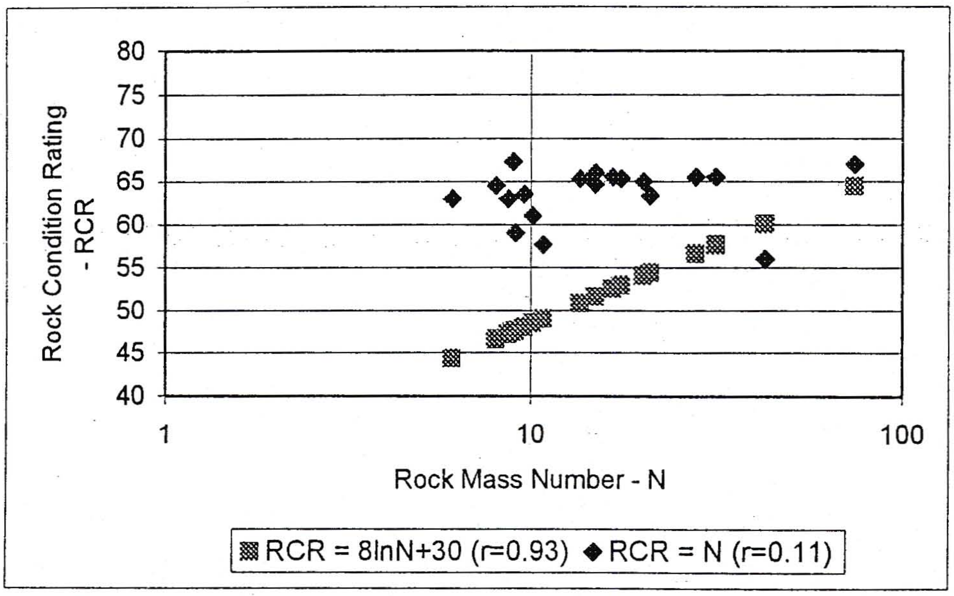
Goel et al (1996) used 63 cases to calculate values of RCR and N and to obtain a correlation:

$$RCR = 8 \ln N + 30 \qquad \text{Equation 6.7}$$

Using these two new definitions, the N and RCR values was calculated for each of the slopes analysed. The results are plotted on Figure 6.3. Once again there is almost no correlation between RCR and N (r = 0.11) but once Equation 6.7 is applied a very good correlation (r = 0.93) does exist.

The above results show that in case of this particular data set, the correlation between RMR and Q as proposed by Moreno (Equation 6.6) and the correlation between RCR and N, as proposed by Goel et. al. (Equation 6.7) work equally well. Both show a correlation coefficient of 0.93.

Figure 6.3: Relationship between N and RCR.



7. Slope Stability Analysis

The primary aim of this project was to establish why almost all of the rock slopes found in the tillite quarries investigated are so remarkably stable, considering that some of these quarries were last worked over thirty years ago. Localised slope failures of small rock masses along two or more joint faces have been occasionally noted, but these have not significantly affected the overall stability of the slopes.

7.1 Factors affecting slope stability

The stability of rock slopes in cut faces depends largely upon the presence and nature of planes of weaknesses or discontinuities within a rock mass, rather than the strength of rock itself (Piteau, 1970). The basic principles on which any slope stability study depends are:

- The system of jointing and other discontinuities
- Their relationship to possible failure surfaces or slopes
- The strength parameters of the joints which include the properties of both the joint planes (basic angle of friction, cohesive strength and density) and any infilling material
- Water pressure in the joints

Regional stresses are not considered to have any significant impact on the stability of slopes in the relatively small quarries investigated.

7.2 Factor of Safety of a slope

The stability of any slope depends upon the margin by which the forces, which tends to resist failure, exceed those that tend to cause failure. This concept is defined as the Factor of Safety (FOS) and serves as an index to compare the stability of slopes.

$$\text{FOS} = \frac{\text{Sum of the l argest forces which may be mobilized for resisting forces}}{\text{Sum of disturbing forces for the slope under consideration}}$$

Equation 7.1

When the slope is on the point of failure a condition of limited equilibrium exists in which the resisting and disturbing forces are equal. The factor of safety would than be equal to 1. A slope is considered stable when the value for the factor of safety is greater than unity. For the purpose of this report a FOS = 1.5 is used although a FOS of 1.3 is generally considered adequate for mine and quarry slopes (Hoek and Bray, 1977).

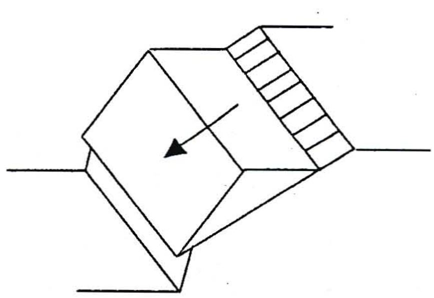
7.3 Type of slope failures

Slopes for which factor of safety can be calculated are (Hoek and Bray, 1977):

- *Plane failures*

A plane failure occurs when a discontinuity strikes approximately parallel to the slope face and dips into the excavation at an angle greater than the angle of friction, but less than slope faces.

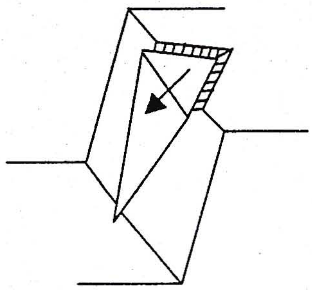
Figure 7.1: Planar failure in rock with highly ordered structure.



- *Wedge failure*

Two discontinuities strike obliquely across the slope face and their line of intersection daylights in the slope face. The wedge will slide on this line of intersection if the inclination of this line is greater than the basic angle of friction but less than the slope face.

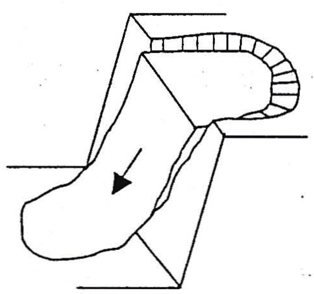
Figure 7.2: Wedge failure on two intersecting discontinuities.



- *Circular failure*

This occurs in waste rock and heavily fractured rock but failure will be defined by a circular failure surface.

Figure 7.3: Circular failure in waste rock or heavily fractured rock with no identifiable structural pattern.

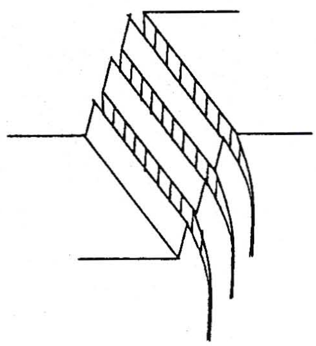


Slopes for which a factor of safety are not calculated:

- *Toppling failure*

Toppling failure in hard rock slopes occurs when blocks of rock rotate or topple down an incline if the slope angle is greater than the basic angle of friction and where deeply dipping discontinuities occur.

Figure 7.4: Toppling failure in rock with steeply dipping Discontinuities



- *Ravelling slopes*

This is scree material and generally consists of small individual pieces of rock that become dislodged from the rock mass and collect at the base of the slope. The main contributing factor causing ravelling is the weathering of the rock mass. It is therefore common on all slopes and will not be discussed in greater detail.

Potential wedge, plane and toppling failure have been identified in the quarries studied and will be discussed in greater detail below. Circular failure has not been identified on any of the slopes concerned.

7.4 Wedge failure

Using the stereonet plots (Appendix C) the Markland and Hocking's test (Hoek and Bray, 1977) was employed to determine the joints most likely to produce wedge failure (Table 7.1). A total of 21 potential wedges were identified from the various slopes (Table 7.1)

Plate 7.1: Example of Wedge failure at the Ridge View Quarry.

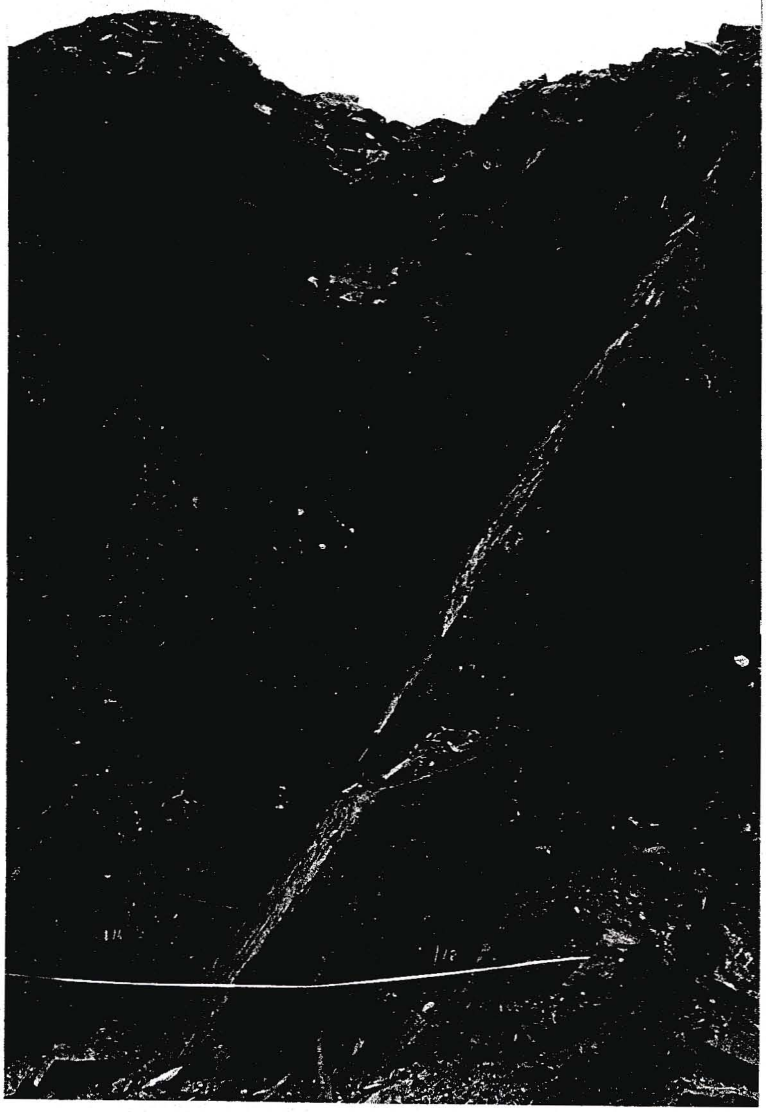
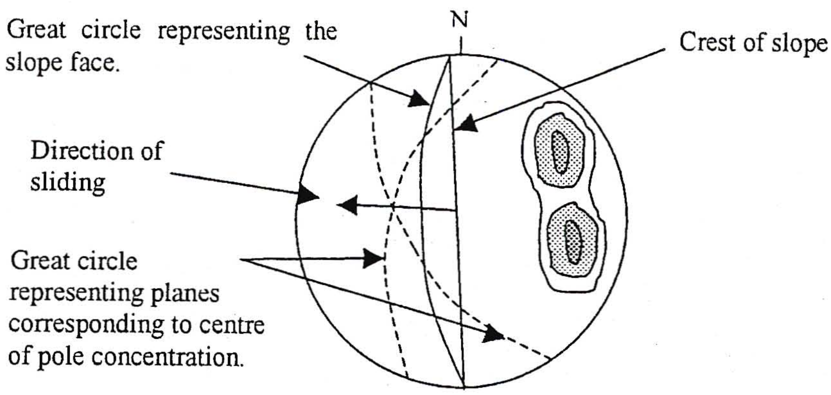


Plate 7.2: Possible wedge failure at the Ridge View Quarry – Face 2. Note the steep joint sets and the stepped nature of the joint surfaces.

Figure 7.5: Stereonet plot showing potential wedge failure



Two different programmes were used to analyse potential wedge failure and to determine their factors of safety. Both programs use the deterministic approach.

- Wedge Failure Analysis Module V2.0, developed by E.B. Kroeger, Dept. of Mining and Mineral Resources Engineering, Southern Illinois University. This program is a modification of the wedge failure analysis by Hoek and Bray (1977).
- Swedge V1.0, developed by Hoek et.al., Dept. of Civil Engineering, University of Toronto.

The following fixed parameters were used:

Density of rock	-	24.6 kN/m^3
Basic friction angle for both joint sets	-	34.5°

The rock density used is that for grade III weathered material. Varying the basic friction angle did not appear to have a major effect on the result of the wedge analysis.

A sensitivity analysis was carried out whereby parameters were varied to observe the effect of dry, partial dry and saturated slopes on the factor of safety. As no value for cohesion was obtained from the shear box tests, the wedge failure analysis were carried out at both 10 kPa and 100 kPa.

Density of water – saturated condition	-	9.81 kN/m^3
Density of water – partially dry condition	-	4.91 kN/m^3
Density of water – dry conditions	-	0.0 kN/m^3
Cohesion for both joint sets	-	10kPa/100kPa

Table 7.1 lists the joints, as determined by the Markland and Hocking's test, that are most likely to produce wedge failure. Table 7.2 lists the results of the analysis.

From Table 7.1 it can be seen that there are five instances where no wedges were formed. The results listed in Table 7.2 show that, with the exception of 4 cases, all slopes generally have a factor of safety of zero at both 10kPa and 100kPa cohesion. Of the four slopes with a positive FOS, only two are considered stable i.e. FOS > 1.5. At partially dry conditions the number of stable slopes has increased to five. At dry conditions and with a cohesion value of 100kPa all slopes are stable, whereas with cohesion at 10kPa four slopes become unstable. Overall the FOS values at C = 10kPa appear to be more realistic (maximum FOS = 11.3) than those recorded at C = 100kPa (maximum FOS = 110.2). The above results suggest that the slopes of the quarries are very stable at dry conditions but become quickly unstable in partially saturated conditions.

Table 7.1: List of joints to produce potential wedge failure

Quarry	Face	Slope Dip/Dipdir.	Height of slope (m)	Joint set	Dip/ Dipdir	Joint set	Dip/ Dipdir
Margate	1	88/163	10	JS2b	88/170	JS7	67/068
	1			JS2b	88/170	JS1b	86/278
	2	85/255	8	JS1a	77/260	JS2a	80/138
	2			JS1a	77/260	JS2b	81/169
	2			JS1a	77/260	JS9	69/035
Westville	1	88/184	32	JS1a	84/261	JS2b	89/075
	2LS2 3	70/275	25	JS1a	70/254	JS2b	82/335
	4	90/360	32	JS1a	80/269	JS2a	85/139
Ridge View	2	80/304	10	JS1a	73/265	JS8	88/358
	2			JS2a	80/315	JS4	78/196
	2			JS1a	73/265	JS4	78/196
	2			JS2a	80/315	JS8	88/358
	3	86/217	7	JS1a	89/063	JS8	88/003
	4	87/140	10	JS1a	84/261	JS2b	83/163
	4			JS1a	84/261	JS4	71/195
	5	85/303	12	JS1a	90/243	JS2a	80/306
Umkomaas	1	83/349	30	JS2a	85/326	JS6	57/109
	2	82/162	20	JS1a	61/259	JS4	53/195
	2			JS4	53/195	JS6	61/107
Zululand	3	90/053	12	JS4	67/184	JS5	59/135
	4	58/106	12	JS3	66/156	JS5	59/135

Table 7.2: Summary of wedge analysis results.

Quarry	Face	Wedge weight (tons)	Line of intersection Dip/Dipdir	Saturated at C = 100kPa		Partially dry at C = 100kPa		Dry at C = 100kPa		Saturated at C = 10kPa		Dry at C = 10kPa	
				FOS	Sliding Plane	FOS	Sliding Plane	FOS	Sliding Plane	FOS	Sliding Plane	FOS	Sliding Plane
Margate	1	9.7	66/085	0	CLOBP	1.7	COD2	29.2	COBD	0	CLOBP	3.3	COBD
	2	70.2	67/203	1.1	COD2	1.3	COD2	11.9	COBD	0.4	COD2	1.7	COBD
	2	60.5	74/225	0.6	COD2	0.9	COD2	11.5	COBD	0.1	COD2	1.4	COD2
	2	15.2	51/333	0	CLOBP	0	CLOBP	19.6	COBD	0	CLOBP	2.1	COBD
Westville	2LS23	0.8	70/267	0	CLOBP	0	CLOBP	110.2	COBD	0	CLOBP	11.3	COD1
Ridge View	2	7.2	73/275	0	CLOBP	0	CLOBP	20.5	COBD	0	CLOBP	2.3	COBD
	2	7.7	69/252	0	CLOBP	0	CLOBP	28.5	COBD	0	CLOBP.	3.3	COBD
	2	1.0	77/277	0	CLOBP	0	CLOBP	80.8	COBD	0	CLOBP	8.4	COBD
	4	0.2	80/208	0	CLOBP	0	CLOBP	86.5	COD2	0	CLOBP	8.7	COD2
	4	16.9	70/189	0	CLOBP	0	CLOBP	14.1	COBD	0	CLOBP	1.7	COBD
	5	100	78/333	0	CLOBP	0	CLOBP	12.3	COD2	0	CLOBP	1.4	COD2
Umkomaas	1	3624	40/052	4.0	COBD	4.5	COBD	5.1	COBD	1.0	COBD	2.1	COBD
	2	704	52/213	0	CLOBP	3.1	COD2	3.5	COD2	0	CLOBP	0.8	COD2
	2	4552	47/160	2.1	COBD	2.3	COBD	2.5	COBD	0.6	COBD	0.9	COBD
Zululand	3	1.4	59/139	0	CLOBP	0	CLOBP	31.9	COD2	0	CLOBP	3.6	COBD
	4	0.3	56/107	0	CLOBP	0	CLOBP	87.0	COBD	0	CLOBP	10.1	COBD

Note: CLOBP – contact lost on both planes; COBD – contact on both discontinuities; COD1 – contact on discontinuity 1

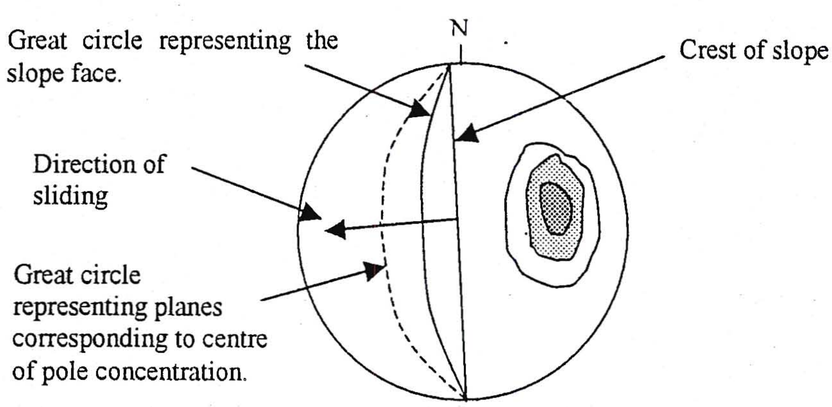
COD2 – contact on discontinuity 2

7.5 Plane failure

Plane failure in rock slopes is less common than wedge failure, as certain geometrical conditions must be satisfied in order for sliding to occur. These conditions are:

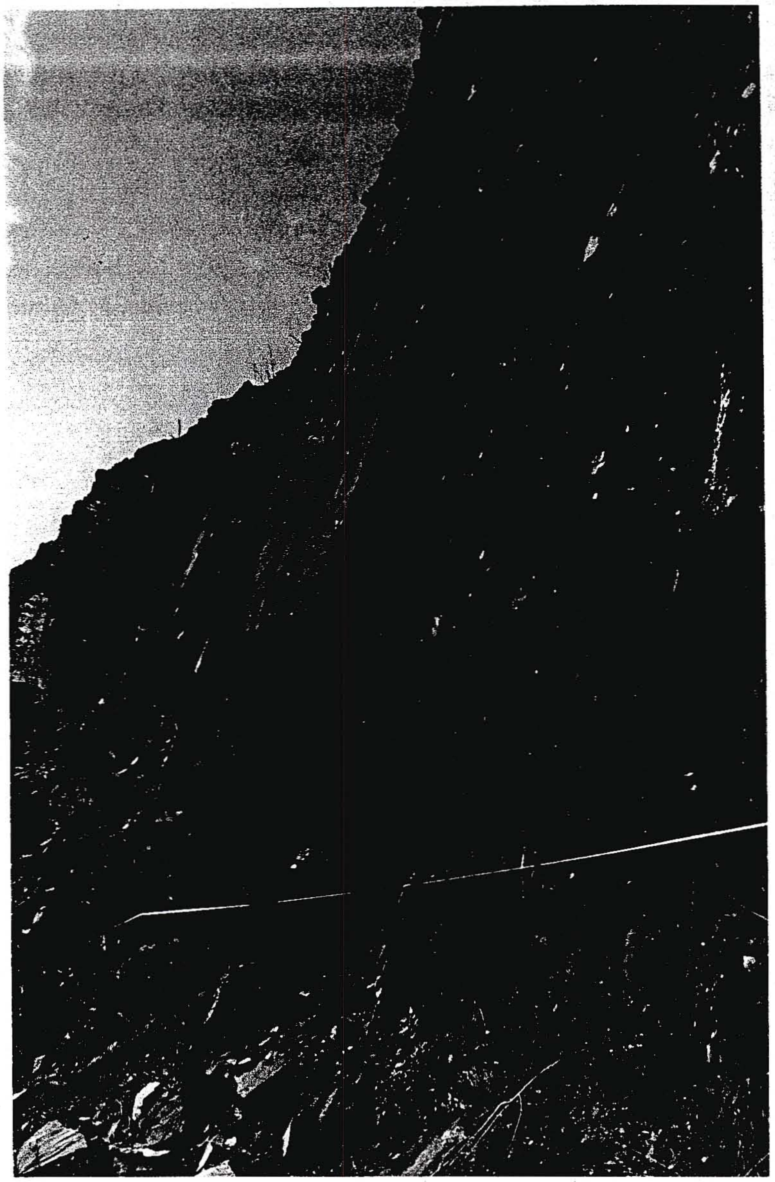
- The failure surface must strike parallel or nearly parallel (± 20°) to the slope surface.
- The dip of the failure plane must be less than the dip of the slope, i.e. $\psi_f > \psi_p$.
- The dip of the failure plane must be greater than the angle of friction of the plane, i.e. $\psi_p > \phi$.
- Release surfaces, which provide little resistance, must be present in the rock mass to define lateral boundaries.

Figure 7.6: Stereonet plot showing a potential plane failure



Using the above conditions only two slopes could be identified with potential plane failure, namely at the Margate Quarry, Face 2 (JS1a) and at the Ridge View Quarry, Face 5 (JS2a). No tension cracks were evident in the upper bench surfaces. These were either not present or were hidden by roads.

Plate 7.3: Example of possible plane failure at the Margate Quarry – Face 2.



The factor of safety for slopes with no tension cracks can be calculated using the following equations (Hoek and Bray, 1977):

$$F = \frac{cA + (W \cos \psi_p - U) \tan \phi}{W \sin \psi_p}$$ Equation 7.2 where

$$W = \frac{\gamma_r H^2}{2} (\cot \psi_p - \cot \psi_f) \qquad A = \frac{H}{\sin \psi_p} \qquad U = \frac{\gamma_w H_w^{\,2}}{2 \sin \psi_p}$$

c = cohesion of the failure plane

ψ_p = failure plane angle

γ_r = unit weight of rock

H = height of slope

ψ_f = overall slope angle

γ_w = unit weight of water

When the slope is completely drained, i.e. there is no water on the sliding plane, then U reduces to zero and equation 7.2 can be modified to:

$$F = \frac{cA + (W \cos\psi_p - U)\tan\phi}{W \sin\psi_p} \qquad \text{Equation 7.3}$$

Using the above equations the factors of safety for the two slopes was calculated for both saturated and dry conditions (Table 7.3). For these calculations the same parameters values as used for wedge failure analysis were employed.

Table 7.3: FOS for plane failure analysis using the Deterministic method.

Method	**Hoek and Bray**		**Prokon**
Slope	**Wet**	**Dry**	**Dry**
Margate Quarry – Face 2	5.11	7.62	7.62
Ridge View Quarry – Face 5	4.80	7.98	7.98

The factor of safety for plane failure was also calculated using the Geotechnical Analysis and Design programme developed by ARQ Assoc. cc and Prokon Software Consultants. Both the deterministic (Table 7.3) and probabilistic (Table 7.4) method were used. The probabilistic method allows for varying the significant variables over their maximum credible range in order to determine its influence on the factor of safety. The significant or random variables are those that do not have a single fixed value but may assume any number of values. Parameters such as the failure plane angle, rock density, angle of friction and cohesive strength are classed as random variables. The distribution of most random variables conform to a normal or Gaussian distribution, which is generally used for probabilistic studies in geotechnical engineering (Hoek, 1998).

Table 7.4: Probabilistic slope analysis for plane failure.

Margate Quarry – Face 2		**Factor of safety =**	**2.63**	
Fixed variables				
Slope height (m)	8			
Overall slope angle (deg)	85			
Random Variables				
Quantity	Mean	SDev	Min.	Max.
Failure plane angle (deg)	77	7	62	89
Friction angle (deg)	34.5	4.6	25.2	41.8
Cohesive strength (kPa)	100	5	95	100
Density of rock	24.6	1.9	22.5	27.5

Ridge View Quarry – Face 5		**Factor of safety =**	**2.93**	
Fixed variables				
Slope height (m)	12			
Overall slope angle (deg)	85			
Random Variables				
Quantity	Mean	SDev	Min.	Max.
Failure plane angle (deg)	80	5.6	71	85
Friction angle (deg)	34.5	4.6	25.2	41.8
Cohesive strength (kPa)	100	5	95	100
Density of rock	24.6	1.9	22.5	27.5

The calculated FOS is at a 95% probabilistic limit.

Very high factors of safety are obtained from the deterministic calculations regardless of whether the failure plane are wet or dry. Using the probabilistic method resulted in FOS values much lower than those obtained from deterministic approach but are still well above the cut-off value of 1.5.

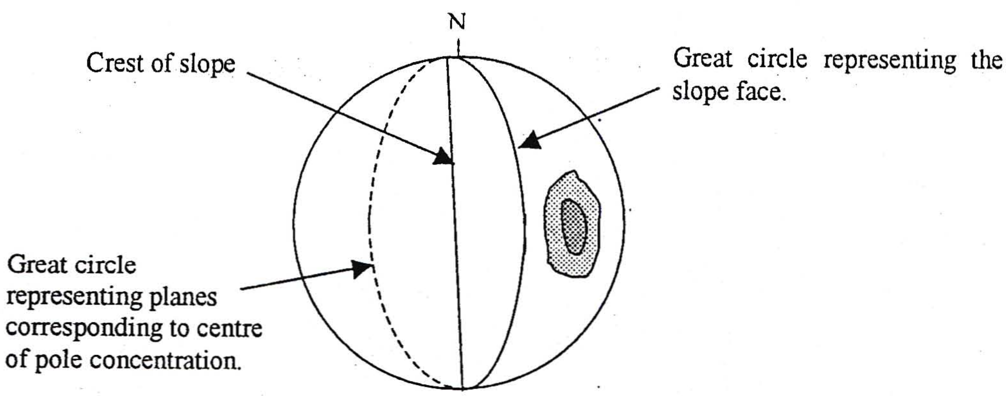
7.6 Toppling failure

Several different types of toppling failures have been described in the literature and these are briefly summarised below (Hoek and Bray, 1977):

- Flexural toppling – columns of rock, which become separated from the main rock mass by steeply dipping discontinuities, break in flexure as they bend forward. Tension cracks form at the top of the slope and the entire process is usually initiated by erosion of the toe of the slope.
- Block toppling – occurs when columns of hard rock are divided by widely spaced orthogonal joints. Columns forming the toe of the slope are pushed forward by the load of overturned columns behind. Sliding of the toe will allow further toppling to take place.
- Block-flexure toppling – is characterised by pseudo-continuous flexure along columns, which are divided by numerous cross-joints.

Toppling failure can be identified from stereonet plots (Figure 7.7).

Figure 7.7: Stereonet plot showing potential toppling failure.

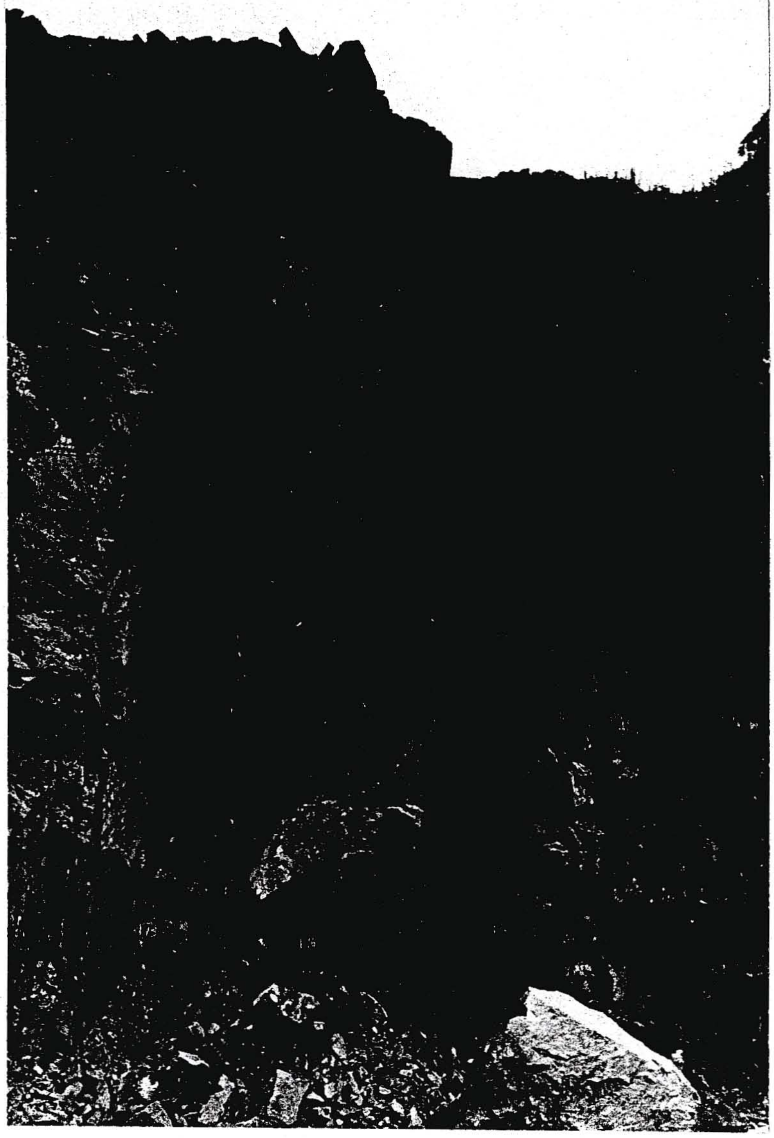


Using the above criteria a total of nine potential toppling failure cases could be identified in the quarries studied (Table 7.5). Variations within the dip of the high angle joints, which are typical of the Dwyka tillite in the area, can lead to toppling failure and in some instances tension cracks were noted close and parallel to the crest of the slopes. All of these cases could be classed as potential flexural toppling failure.

Table 7.5: Slopes showing potential toppling failure.

Quarry	Face	Joint set
Margate	2	JS7, JS9
	3	JS1a
Westville	2LS23	JS3
Ridge View	1	JS1a
	3	JS3
	4	JS2a
Zululand	1	JS2b
	2	JS5
	5	JS4

Plate 7.4: Potential toppling failure at the Rídge View Quarry – Face 3. Note the tension crack and near horizontal jointing.



8. Conclusion

Joint orientation data from the various sites have shown that two to three sets of high angle joints and one low angle joint set are common. The potential of wedge and planar failure is therefore very high. The steeply dipping discontinuities also promote the potential for flexural toppling failure and this was noted in several of the quarry faces.

Several different techniques were used to establish potential wedge, plane and toppling failure on the rock slopes in the Dwyka tillite quarries along the KwaZulu-Natal coastline. Factors of safety were calculated, using various equations and software programs, to assess the stability of each slope. The results of these techniques can be summarised as follows:

- At saturated conditions and with value of cohesion of 100 kPa, only four wedges, of the 16 potential wedge failures, showed a positive factor of safety. Only two of these are considered stable i.e. FOS > 1.5. The remaining 12 wedges show zero factor of safety with contact lost on both contacts.
- When saturated and with a value of cohesion of 10 kPa, all of the above four wedges have FOS values less than 1.5 i.e. all wedges are unstable.
- At partially dry conditions and with a value of cohesion of 100 kPa, four wedges have a FOS value of greater than 1.5. At completely dry conditions at C = 100 kPa all wedges are very stable. Reducing the value of cohesion to 10 kPa at dry condition then only 12 of the 16 wedges are stable (FOS > 1.5)
- Only two slopes were identified as having potential plane failure and both deterministic and probabilistic methods (wet and dry conditions) were used to calculate their factor of safety. In all cases the results returned values far above the cut-off value of 1.5 for the FOS suggesting no danger of possible and imminent plane failure.
- Nine possible cases of potential toppling failure were identified. The stability of these cases could not be assessed.

The following conclusions can be drawn from the above information:

- The majority of slopes, under saturated conditions, will be unstable. Slope stability improves as they become less saturated and in dry conditions, depending on the value of cohesion used, the slopes are stable.

- Slope stability is affected by the value of cohesion and slopes become increasingly more stable with higher cohesion.

In addition to the above techniques, an alternative but increasingly popular method of determining the quality of the rock mass was used to estimate the stability of the slopes concerned. Four different rock mass classification systems were used and compared for the quantification of the rock mass quality. The results can be summarised as follows:

- The rock mass quality of the various slopes can generally be described, by all four systems, as being partially stable to stable.
- The RMR of Bieniawski and the SMR of Romano returned higher overall ratings compared to Barton and Laubscher. Romano's slope mass rating system was the only system to classify any slopes (four) as being completely stable.
- The MRMR of Laubscher is considered to be the closest to the true rock mass quality as it takes into account the weathering of the rock mass, the joint orientation and the effect of blasting on the rock mass. The MRMR values are lower than any of the RMR values.

The above points allow one to conclude that wedge failures, at partially saturated conditions, are the main source of potential slope instability on the quarry rock faces. In addition the rock mass rating systems describe the rock quality of the tillite faces as only being partially stable to stable. However, in reality the slopes are, apart from small, localised failures, very stable. The answer to this contradiction lies in the nature of the discontinuities found within the rock slopes. Possible reasons for the high degree of slope stability can be explained as follows:

- The slopes never really become saturated enough to become unstable.
- The widely spaced jointing (Table 3.2) within the rock mass provides interlocking conditions contributing to a high rock mass cohesion.
- The low persistence of joints (Table 3.3) as compared to the actual slope height. This factor will limit the size and weight of potential wedges and therefore reduces the chances of failure.
- Termination indices (Table 3.4) indicate that a relatively high percentage (up to 90% in cases) of discontinuities terminate within the rock mass or against other discontinuities. This will also contribute to interlocking conditions and higher rock mass quality.

- Joint surfaces, particularly in the older quarries, are mostly covered by a rough, yellow-brown limonite staining similar in texture to sandpaper. This will help to hinder any movement along joint planes.
- Very little evidence was found of any type of joint infill or gauge material such as clay that could serve to weaken the rock mass and encourage movement along joint planes, especially during the rainy season. The most common infill found consists of coarse grained to gritty, sub-rounded to very angular tillite. These little pieces of rock can assist in stabilising any potential wedge failure.
- Joint roughness varies from rough planar to smooth undulating to smooth stepped. Slickensided joint surfaces were rarely found suggesting almost no movement along discontinuity planes prior to mining.
- Mapping of the quarries was carried out during the rainy season. Despite that, joints overall were remarkably dry with minor seepage only occurring in the sub-horizontal and horizontal joints sets (JS3).

9. Acknowledgements

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Appendix A

Quantitative descriptions of discontinuities

ROCK MASS DESCRIPTION DEFINITIONS

Joint Spacing

EX	extremely close	< 0.02 m
VC	very close	0.02 m - 0.06 m
C	close	0.06 m - 0.2 m
M	moderate	0.2 m - 0.6 m
W	wise	0.6 m - 2 m
VW	very wide	2 m - 6 m
EW	extremely wide	> 6 m

Joint Persistance

VL	very low	<1 m
L	low	1 - 3 m
M	medium	3 - 10 m
H	high	10 - 20 m
VH	very high	> 20 m

Joint Aperture

VT	very tight	< 0.1 mm	
T	tight	0.1 - 0.25 mm	"Closed" feature
PO	partly open	0.25 - 0.5 mm	
O	open	0.5 - 2.5 mm	
MW	moderately wide	2.5 - 10 mm	"Gapped" feature
W	wide	> 10 mm	
VW	very wide	1 - 10 cm	
EW	extremely wide	10 - 100 cm	"Open" feature
C	cavernous	> 1m	

Block shape

M	massive	few joints or very wide spacing
B	blocky	approx. equidimensional
T	tabular	one dim. considerably smaller than the other two
C	columnar	one dim. considerably larger than the other two
I	irregular	wide variations of block size and shape
CR	crushed	heavily jointed or "sugar cube"

Block size

		Joints/m3
VL	very large	< 1.0
L	large	1 - 3
M	medium	3 - 10
S	small	10 - 30
VS	very small	> 30

Type of joint infill

Q	Quartz
C	Clay
CC	Calcite
Ch	Chlirite
F	Fe/Mg staining
T	Tillite

Nature of joint infill

1. Clean
2. Surface staining
3. Non-cohesive
4. Inactive clay or clay matrix
5. Swelling clay or clay matrix
6. Cemented
7. Chlorite, talc, gypsum
8. Others - specify
9. Veining

Weathering grade of rock

1. Fresh
2. Slightly weathered
3. Moderately weathered
4. Highly weathered
5. Completely weathered
6. Residual soil

Water/Seepage

1. Dry
2. Damp
3. Wet, occasional drops
4. Continuous flow of water
5. Occasional filling washed out
6. Strong flow. Filling wahed out

Hardness of cohesive soil

		Approx. range of UCS (MPa)
S1	Very soft clay	<0.025
S2	Soft clay	0.025 - 0.05
S3	Firm clay	0.05 - 0.10
S4	Stiff clay	0.10 - 0.25
S5	Very stiff clay	0.25 - 0.50
S6	Hard clay	> 0.50

Hardness of Rock

R 0	Extremely weak rock	0.25 - 1.0
R 1	Very weak rock	1.0 - 5.0
R 2	Weak rock	5.0 - 25
R 3	Medium strong rock	25 - 50
R 4	Strong rock	50 - 100
R 5	Very strong rock	100 - 250
R 6	Extremely strong rock	> 250

Type of termination

Tr	Discontinuity terminates in rock
Tx	Discontinuity terminates outside exposure
Td	Discontinuity terminates against another discontinuity.

Appendix B

Summary of the Rock Mass Classification Data

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry: Westville	Face:	1		Date	02/02/1997
Blasting: No	Photo:	Yes			
Length of survey line (m):		24.6	Height of slope (m):		32
Total number of joints:		98	Direction of slope (dip/dipdir):		88/184
Average FF/m:		3.98	Overall slope angle (deg):		88

Rock mass information

Weathering grade of rock mass:	2	RQD:	95
Hardness of rock mass:	R 5	Block size index (Ib):	0.47
Intact strength of rock (MPa) :	100 - 250	Block shape:	Irregular

Discontinuity information

		Set1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS1a	JS2b	JS3			
Dip direction (deg):	Mean	261	334	075			
	Min.	323/08	327/148	27			
	Max.	289/061	340/161	117			
Dip (deg):	Mean	84	89	08			
	Min.	73/98	79/85	03			
	Max.	90	90	14			
No. of joints per line:		52	9	5			
Joint Spacing (m) :		0.33	0.29	0.78			
Joint Persistence (m):		7.2	4.0	3.3			
Joint Aperture (m):		T	PO	T			
Joint Roughness:		8	8	8			
JRC value:		2	1	2			
Joint Filling:	Type:	F/T	F/C	C			
	Nature:	1/2/3	1/4	4			
Joint wall rock weathering:		2	1	2			
Seepage:		1	1	1			
Termination Index (%):	Tr	19	33	20			
	Tx	63	22	10			
	Td	18	45	70			
Corr. Schmidt Hammer value:		40	45	39			

Structure (Bedding, Faults etc)

General Remarks

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.D52

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	Westville	Face:	2LS1	Date	09/02/1997
Blasting:	No	Photo:	No		

Length of survey line (m):	12.0	Height of slope (m):	30
Total number of joints:	29	Direction of slope (dip/dipdir):	68/315
Average FF/m:	2.42	Overall slope angle (deg):	68

Rock mass information

Weathering grade of rock mass:	2	RQD:	96
Hardness of rock mass:	R 6	Block size index (Ib):	0.45
Intact strength of rock (MPa) :	>250	Block shape:	Irregular

Discontinuity information

		Set1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS1a	JS2b	JS3	JS4		
Dip direction (deg):	Mean	249	333	332	199		
	Min.	238	323/145	279	194		
	Max.	263	342/161	360	201		
Dip (deg):	Mean	77	85	10	80		
	Min.	62	75/87	3	79		
	Max.	90	90	11	86		
No. of joints per line:		15	8	2	2		
Joint Spacing (m) :		0.60	0.60	0.15	0.25		
Joint Persistence (m):		6.3	1.9	0.9	2.4		
Joint Aperture (m):		O	T	PO	PO		
Joint Roughness:		7	2	8	8		
JRC value:		3	3	3	3		
Joint Filling:	Type:	CC	CC				
	Nature:	1/2	2	1	1		
Joint wall rock weathering:		1	1	2	1		
Seepage:		1	1	2	1		
Termination Index (%):	Tr	10	31	0	25		
	Tx	67	69	25	75		
	Td	23	0	75	0		
Corr. Schmidt Hammer value:		44	50	--	46		

Structure (Bedding, Faults etc)

General Remarks

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.D22

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	Westville	Face:	2LS23	Date	11/02/1997
Blasting:	No	Photo:	Yes		

Length of survey line (m):	14.4	Height of slope (m):	25
Total number of joints:	41	Direction of slope (dip/dipdir):	70/274
Average FF/m:	2.85	Overall slope angle (deg):	70

Rock mass information

Weathering grade of rock mass:	2	RQD:	97
Hardness of rock mass:	R 6	Block size index (Ib):	0.45
Intact strength of rock (MPa) :	>250	Block shape:	Blocky to columnar

Discontinuity information

		Set1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS1a	JS2b	JS2b	JS3		
Dip direction (deg):	Mean	254	335	168	044		
	Min.	230	325	164	014		
	Max.	270	345	172	064		
Dip (deg):	Mean	70	82	78	13		
	Min.	58	73	73	02		
	Max.	88	90	81	20		
No. of joints per line:		13	6	5	5		
Joint Spacing (m) :		0.32	0.58	0.62	0.45		
Joint Persistence (m):		3.9	3.4	2.7	7.4		
Joint Aperture (m):		T	T	PO	PO		
Joint Roughness:		7	8	7	5		
JRC value:		3	2	4	5		
Joint Filling:	Type:	CC	CC/F	CC/CH	CC		
	Nature:	2	2	2	1/2		
Joint wall rock weathering:		2	2	2	2		
Seepage:		1	1	1	1		
Termination Index (%):	Tr	23	17	40	0		
	Tx	77	42	50	25		
	Td	0	41	10	75		
Corr. Schmidt Hammer value:		46	49	45	48		

Structure (Bedding, Faults etc)

Bedding plane at 18/052 (See photo in Chapter 5).

General Remarks

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.D22

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry: Westville | Face: 4 | Date 12/02/1997

Blasting: No | Photo: Yes

Length of survey line (m):	15.0	Height of slope (m):	32
Total number of joints:	54	Direction of slope (dip/dipdir):	90/360
Average FF/m:	3.60	Overall slope angle (deg):	90

Rock mass information

Weathering grade of rock mass:	2	RQD:	95
Hardness of rock mass:	R 5	Block size index (Ib):	--
Intact strength of rock (MPa) :	100 - 250	Block shape:	Irregular

Discontinuity information

		Set1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS1a	JS2a				
Dip direction (deg):	Mean	269	139				
	Min.	254	134				
	Max.	282	146				
Dip (deg):	Mean	80	85				
	Min.	69	81				
	Max.	90	89				
No. of joints per line:		33	8				
Joint Spacing (m) :		0.27	0.37				
Joint Persistence (m):		7.1	3.6				
Joint Aperture (m):		PO	T				
Joint Roughness:		7	7				
JRC value:		2	3				
Joint Filling:	Type:	Q	T				
	Nature:	9	1/3				
Joint wall rock weathering:		2	2				
Seepage:		1	1				
Termination Index (%):	Tr	14	25				
	Tx	70	63				
	Td	16	12				
Corr. Schmidt Hammer value:		48	47				

Structure (Bedding, Faults etc)

General Remarks

A number of low angle joints were recorded but the orientations varied a gret deal. No average dip/dipdirection could be obtained. No block size index was calculated.

Quartz veins in cases measured up to 2 cm in thickness.

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.

-- denotes insufficient data to calculate.D22

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	MQ	Face:	1	Date	30/03/1997
Blasting:	No	Photo:	Yes		

Length of survey line (m):	31.1	Height of slope (m):	10
Total number of joints:	83	Direction of slope (dip/dipdir):	88/162
Average FF/m:	2.67	Overall slope angle (deg):	88

Rock mass information

Weathering grade of rock mass:	2	RQD:	97
Hardness of rock mass:	R 5	Block size index (Ib):	0.21
Intact strength of rock (MPa):	100 - 250	Block shape:	Irregular

Discontinuity information

		Set1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS1b	JS2b	JS7			
Dip direction (deg):	Mean	278	170	68			
	Min.	262/82	343/163	61			
	Max.	294/114	357/176	75			
Dip (deg):	Mean	86	88	67			
	Min.	74/80	85/82	62			
	Max.	90	90	73			
No. of joints per line:		42	10	6			
Joint Spacing (m):		0.25	0.19	0.19			
Joint Persistence (m):		6.2	9.4	1.5			
Joint Aperture (m):		O	MW	O			
Joint Roughness:		2	2	5			
JRC value:		3	3	4			
Joint Filling:	Type:	CC	CC	CC			
	Nature:	2	2	2			
Joint wall rock weathering:		2	2	2			
Seepage:		1/2	1	1			
Termination Index (%):	Tr	5	0	0			
	Tx	67	100	25			
	Td	28	0	75			
Corr. Schmidt Hammer value:		41	45	41			

Structure (Bedding, Faults etc)

General Remarks

Joint infill consists predominately of thin calcite staining. In the upper weathered zone (top 3 m) the tillite appears to moderately weathered and joint surfaces are typically orange-brown in colour.

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	MQ	Face:	2	Date	31/03/1997
Blasting:	Yes	Photo:	Yes		

Length of survey line (m):	27.2	Height of slope (m):	8
Total number of joints:	99	Direction of slope (dip/dipdir):	85/255
Average FF/m:	3.65	Overall slope angle (deg):	85

Rock mass information

Weathering grade of rock mass:	1	RQD:	93
Hardness of rock mass:	R 6	Block size index (Ib):	0.13
Intact strength of rock (MPa) :	>250	Block shape:	Crushed

Discontinuity information

		Set 1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS1a	JS2a	JS2b	JS3	JS7	JS7
Dip direction (deg):	Mean	260	138	169	356	65	35
	Min.	251	129	157/337	306	55	25
	Max.	269	145	182/002	61	76	44
Dip (deg):	Mean	77	80	81	4	56	69
	Min.	62	73	69/82	1	47	63
	Max.	89	90	90	11	66	77
No. of joints per line:		10	10	23	5	8	8
Joint Spacing (m) :		0.11	0.11	0.16	0.04	0.31	0.16
Joint Persistence (m):		3.0	6.1	5.1	3.1	1.8	1.6
Joint Aperture (m):		O	O	O	M	L	O
Joint Roughness:		2	2	7	5	5	7
JRC value:		3	2	2	4	3	2
Joint Filling:	Type:	CC	CC	CC	CC	CC	CC
	Nature:	1/2	2	2	2	2	2
Joint wall rock weathering:		1	1/2	2	1	1/2	2
Seepage:		1	1	1	1	1	1
Termination Index (%):	Tr	5	0	7	10	19	25
	Tx	65	85	67	0	0	19
	Td	30	15	26	90	81	56
Corr. Schmidt Hammer value:		43	52	49	44	45	36

Structure (Bedding, Faults etc)

General Remarks

Calcite development along joint surfaces on this face is not as well developed as seen on the other faces in this quarry.

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	MQ	Face:	3	Date	01/04/1997
Blasting:	Yes	Photo:	Yes		

Length of survey line (m):	21.5	Height of slope (m):	10
Total number of joints:	50	Direction of slope (dip/dipdir):	79/090
Average FF/m:	2.33	Overall slope angle (deg):	79

Rock mass information

Weathering grade of rock mass:	2	RQD:	98
Hardness of rock mass:	R 6	Block size index (Ib):	0.11
Intact strength of rock (MPa) :	>250	Block shape:	Irregular

Discontinuity information

		Set1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS1a	JS2a				
Dip direction (deg).	Mean	62	124				
	Min.	229/50	114				
	Max.	258/78	133				
Dip (deg):	Mean	86	78				
	Min.	80/65	70				
	Max.	90	86				
No. of joints per line:		30	10				
Joint Spacing (m) :		0.12	0.21				
Joint Persistence (m):		2.0	5.2				
Joint Aperture (m):		VT	MW				
Joint Roughness:		7	5				
JRC value:		3	4				
Joint Filling:	Type:	CC	CC				
	Nature:	2	2				
Joint wall rock weathering:		2	2				
Seepage:		1/2	1/2				
Termination Index (%):	Tr	13	5				
	Tx	75	80				
	Td	12	15				
Corr. Schmidt Hammer value:		41	41				

Structure (Bedding, Faults etc)

General Remarks

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	Ridge View	Face:	I	Date	19/02/1999
Blasting:	Yes	Photo:	Yes		

Length of survey line (m):	11.3	Height of slope (m):	20
Total number of joints:	42	Direction of slope (dip/dipdir):	80/086
Average FF/m:	3.72	Overall slope angle (deg):	80

Rock mass information

Weathering grade of rock mass:	2	RQD:	93
Hardness of rock mass:	R 3	Block size index (Ib):	0.54
Intact strength of rock (MPa) :	25 - 50	Block shape:	Columnar

Discontinuity information

		Set1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS1a	JS2a	JS3			
Dip direction (deg):	Mean	087	146	030			
	Min.	083	132	018			
	Max.	091	160	0.46			
Dip (deg):	Mean	83	76	26			
	Min.	84	66	16			
	Max.	88	86	32			
No. of joints per line:		5	18	6			
Joint Spacing (m) :		1.00	0.40	0.21			
Joint Persistence (m):		8.8	4.8	3.1			
Joint Aperture (m):		T	O	PO			
Joint Roughness:		2	7	5			
JRC value:		3	2	3			
Joint Filling:	Type:	F	F	F			
	Nature:	2	2	2			
Joint wall rock weathering:		3	3	3			
Seepage:		I	I	I			
Termination Index (%):	Tr	0	3	8			
	Tx	90	53	42			
	Td	10	44	50			
Corr. Schmidt Hammer value:		30	28	20			

Structure (bedding planes, faults etc)

General Remarks

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes for insufficient data to calculate.

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	Ridge View	Face:	2	Date	23/02/1997
Blasting:	Yes	Photo:	Yes		

Length of survey line (m):	20.8	Height of slope (m):	10
Total number of joints:	38	Direction of slope (dip/dipdir):	80/303
Average FF/m:	1.83	Overall slope angle (deg):	80

Rock mass information

Weathering grade of rock mass:	2	RQD:	97
Hardness of rock mass:	R 3	Block size index (Ib):	1.01
Intact strength of rock (MPa) :	25 - 50	Block shape:	Irregular

Discontinuity information

		Set1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS1a	JS2a	JS3	JS4	JS8	
Dip direction (deg):	Mean	265	315	078	196	358	
	Min.	247	310	003	190	347	
	Max.	277	325	128	201	008	
Dip (deg):	Mean	73	80	10	78	88	
	Min.	63	68	0	72	81	
	Max.	83	90	19	85	90	
No. of joints per line:		10	5	8	4	6	
Joint Spacing (m) :		0.53	--	0.36	2.15	1.38	
Joint Persistence (m):		3.3	0.7	10.1	4.1	3.8	
Joint Aperture (m):		VW	T	MW	O	M	
Joint Roughness:		7	7	4	5	4	
JRC value:		3	2	5	4	9	
Joint Filling:	Type:	F	F	C	F	F	
	Nature:	2	2	5	2	2	
Joint wall rock weathering:		2	2	2	2	2	
Seepage:		1	1	2	2	2	
Termination Index (%):	Tr	0	10	38	25	0	
	Tx	56	30	19	25	30	
	Td	44	60	43	15	70	
Corr. Schmidt Hammer value:		37	28	15	39	27	

Structure (Bedding, Faults etc)

Bedding plane at dip/dipdirection of 12/071. Tillite below the bedding plane is olive green to blue in colour compared to the brown tillite above. Infilling consists of soft, weathered clay/tillite material.

General Remarks

Shallow angle joints or bedding planes have thick infilling consisting of clay and chlorite.

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.D22

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	Ridge View	Face:	3	Date	24/02/1997
Blasting:	Yes	Photo:	Yes		

Length of survey line (m):	19.8	Height of slope (m):	7
Total number of joints:	37	Direction of slope (dip/dipdir):	86/217
Average FF/m:	1.87	Overall slope angle (deg):	85

Rock mass information

Weathering grade of rock mass:	I	RQD:	98
Hardness of rock mass:	R 6	Block size index (Ib):	0.28
Intact strength of rock (MPa) :	>250	Block shape:	Irregular

Discontinuity information

		Set I	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS1a	JS2a	JS3	JS8		
Dip direction (deg):	Mean	063	318	073	003		
	Min.	230/052	308/131	007	355/175		
	Max.	256/072	326/145	121	008/188		
Dip (deg):	Mean	89	84	10	88		
	Min.	82/87	75/87	01	84/87		
	Max.	90	90	20	90		
No. of joints per line:		12	8	7	5		
Joint Spacing (m) :		0.33	0.27	0.24	0.43		
Joint Persistence (m):		2.8	5.7	5.0	3.7		
Joint Aperture (m):		T	PO	O	T		
Joint Roughness:		5	7	5	4		
JRC value:		3	2	3	7		
Joint Filling:	Type:	F	F/CC	F	F/C		
	Nature:	2	2/9	2	1/2		
Joint wall rock weathering:		2	2	2	2		
Seepage:		1	2	2	2		
Termination Index (%):	Tr	29	13	36	10		
	Tx	63	87	14	80		
	Td	8	0	50	10		
Corr. Schmidt Hammer value:		43	49	48	44		

Structure (Bedding, Faults etc)

General Remarks

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.D22

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	Ridge View	Face:	4	Date	24/02/1997
Blasting:	Yes	Photo:	Yes		

Length of survey line (m):	29.9	Height of slope (m):	10
Total number of joints:	86	Direction of slope (dip/dipdir):	87/140
Average FF/m:	2.88	Overall slope angle (deg):	87

Rock mass information

Weathering grade of rock mass:	I	RQD:	95
Hardness of rock mass:	R 5	Block size index (Ib):	0.23
Intact strength of rock (MPa) :	100 - 250	Block shape:	Irregular

Discontinuity information

		Set I	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS1a	JS2a	JS2b	JS3	JS4	
Dip direction (deg):	Mean	261	312	163	064	195	
	Min.	61/240	303	156	000	185	
	Max.	106/284	322	168	120	201	
Dip (deg):	Mean	84	71	83	09	71	
	Min.	86/78	66	78	00	62	
	Max.	90	78	87	16	79	
No. of joints per line:		34	6	6	10	5	
Joint Spacing (m) :		0.33	0.31	0.13	0.23	1.10	
Joint Persistence (m):		4.6	2.1	7.8	3.5	6.6	
Joint Aperture (m):		PO	T	T	PO	PO	
Joint Roughness:		6	6	6	5	5	
JRC value:		3	2	2	5	5	
Joint Filling:	Type:	F/T	F/T	F	F/T	F	
	Nature:	2/3	2/3	2	2/3	2	
Joint wall rock weathering:		2	2	2	2	2	
Seepage:		I	I	I	I	I	
Termination Index (%):	Tr	10	8	17	15	0	
	Tx	66	92	25	10	80	
	Td	24	0	58	75	20	
Corr. Schmidt Hammer value:		46	43	36	45	43	

Structure (Bedding, Faults etc)

General Remarks

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.D22

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	Ridge View	Face:	5	Date	27/02/1997
Blasting:	Yes	Photo:	Yes		

Length of survey line (m):	14.5	Height of slope (m):	12
Total number of joints:	23	Direction of slope (dip/dipdir):	85/303
Average FF/m:	1.59	Overall slope angle (deg):	85

Rock mass information

Weathering grade of rock mass:	2	RQD:	99
Hardness of rock mass:	R 5	Block size index (Ib):	0.26
Intact strength of rock (MPa) :	100 - 250	Block shape:	Columnar

Discontinuity information

		Set 1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS1a	JS2a	JS3			
Dip direction (deg):	Mean	243	306	067			
	Min.	231/050	297	033			
	Max.	254/071	317	092			
Dip (deg):	Mean	90	80	17			
	Min.	79/79	71	12			
	Max.	90	85	24			
No. of joints per line:		9	4	3			
Joint Spacing (m) :		0.29	0.31	0.18			
Joint Persistence (m):		6.6	5.1	5.6			
Joint Aperture (m):		T	T	PO			
Joint Roughness:		5	7	5			
JRC value:		3	3	3			
Joint Filling:	Type:	F/T	F	F			
	Nature:	2/3	2	2			
Joint wall rock weathering:		2	2	2			
Seepage:		1	1	1			
Termination Index (%):	Tr	0	25	0			
	Tx	72	75	0			
	Td	38	0	100			
Corr. Schmidt Hammer value:		49	47	22			

Structure (Bedding, Faults etc)

General Remarks

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.D22

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	UQ	Face:	1	Date	03/03/1997
Blasting:	No	Photo:	Yes		

Length of survey line (m):	12.2	Height of slope (m):	30
Total number of joints:	20	Direction of slope (dip/dipdir):	83/349
Average FF/m:	1.64	Overall slope angle (deg):	83

Rock mass information

Weathering grade of rock mass:	I	RQD:	98
Hardness of rock mass:	R 5	Block size index (Ib):	0.07
Intact strength of rock (MPa) :	100 - 250	Block shape:	Irregular

Discontinuity information

		Set1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS2a	JS3	JS6			
Dip direction (deg):	Mean	326	343	109			
	Min.	319	0	97			
	Max.	332	360	121			
Dip (deg):	Mean	85	5	57			
	Min.	79	0	47			
	Max.	90	8	71			
No. of joints per line:		3	2	7			
Joint Spacing (m) :				0.20			
Joint Persistence (m):		0.7	18.5	3.6			
Joint Aperture (m):		PO	T	MW			
Joint Roughness:		7	5	7			
JRC value:		3	3	3			
Joint Filling:	Type:	F	F	T			
	Nature:	2	2	3/4			
Joint wall rock weathering:		2	2	2			
Seepage:		1	2	2			
Termination Index (%):	Tr	17	0	14			
	Tx	33	75	36			
	Td	50	25	50			
Corr. Schmidt Hammer value:		49	45	41			

Structure (Bedding, Faults etc)

General Remarks

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.D22

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	UQ	Face:	2	Date	03/03/1997
Blasting:	No	Photo:	Yes		

Length of survey line (m):	Spot Readings	Height of slope (m):	30
Total number of joints:	26	Direction of slope (dip/dipdir):	30/163 & 82/162
Average FF/m:		Overall slope angle (deg):	30 & 82

Rock mass information

Weathering grade of rock mass:	2	RQD:	
Hardness of rock mass:	R 5	Block size index (Ib):	0.73
Intact strength of rock (MPa) :	100 - 250	Block shape:	Irregular

Discontinuity information

		Set1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS1a	JS3	JS4	JS6		
Dip direction (deg):	Mean	259	89	195	107		
	Min.	248	61	181	69		
	Max.	269	113	208	116		
Dip (deg):	Mean	61	15	53	61		
	Min.	52	10	45	46		
	Max.	70	23	61	76		
No. of joints per line:		5	2	6	7		
Joint Spacing (m) :		0.15	1.05	1.00	0.17		
Joint Persistence (m):		16.6		14.3	2.3		
Joint Aperture (m):		T	PO	O	VT		
Joint Roughness:		7	4	6	5		
JRC value:		2	5	2	5		
Joint Filling:	Type:	F	F/T	F	F		
	Nature:	2	2/3	2	2		
Joint wall rock weathering:		2	2	2	2		
Seepage:		1	2	1	1		
Termination Index (%):	Tr	0	--	0	36		
	Tx	70	--	83	43		
	Td	30	--	17	21		
Corr. Schmidt Hammer value:		39	--	50	50		

Structure (Bedding, Faults etc)

General Remarks

No line survey was carried out. Only spot readings of joint orientations were taken along this face. The initial slope of this face was 30 degrees but changed to 82 degrees further back.

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.D22

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	UQ	Face:	4	Date	03/03/1997
Blasting:	No	Photo:	Yes		

Length of survey line (m):	Spot readings	Height of slope (m):	20
Total number of joints:	14	Direction of slope (dip/dipdir):	78/320
Average FF/m:		Overall slope angle (deg):	78

Rock mass information

Weathering grade of rock mass:	2	RQD:	--
Hardness of rock mass:	R 5	Block size index (Ib):	--
Intact strength of rock (MPa) :	100 - 250	Block shape:	Irregular

Discontinuity information

		Set1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS3	JS6				
Dip direction (deg):	Mean	74	98				
	Min.	52	89				
	Max.	95	107				
Dip (deg):	Mean	19	65				
	Min.	7	58				
	Max.	31	76				
No. of joints per line:		5	3				
Joint Spacing (m) :		--	0.55				
Joint Persistence (m):		5.4	2.4				
Joint Aperture (m):		PO	VW				
Joint Roughness:		6	4				
JRC value:		5	8				
Joint Filling:	Type:	T	T				
	Nature:	3	3				
Joint wall rock weathering:		2	2				
Seepage:		1	1				
Termination Index (%):	Tr	20	--				
	Tx	50	83				
	Td	30	17				
Corr. Schmidt Hammer value:		44	41				

Structure (Bedding, Faults etc)

General Remarks

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	UQ	Face:	5	Date	04/03/1997
Blasting:	No	Photo:	Yes		

Length of survey line (m):	17.5	Height of slope (m):	18
Total number of joints:	16	Direction of slope (dip/dipdir):	89/192
Average FF/m:	0.91	Overall slope angle (deg):	89

Rock mass information

Weathering grade of rock mass:	2	RQD:	100
Hardness of rock mass:	R 5	Block size index (Ib):	--
Intact strength of rock (MPa) :	100 - 250	Block shape:	Irregular

Discontinuity information

		Set I	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS1a	JS3				
Dip direction (deg):	Mean	266	91				
	Min.	249	71				
	Max.	283	111				
Dip (deg):	Mean	58	24				
	Min.	51	19				
	Max.	66	32				
No. of joints per line:		7	4				
Joint Spacing (m) :		0.31	--				
Joint Persistence (m):		5.4	6.3				
Joint Aperture (m):		T	O				
Joint Roughness:		6	6				
JRC value:		3	3				
Joint Filling:	Type:	F	F/T				
	Nature:	2	3				
Joint wall rock weathering:		2	2				
Seepage:		1	2				
Termination Index (%):	Tr	29	--				
	Tx	50	50				
	Td	21	50				
Corr. Schmidt Hammer value:		49	49				

Structure (Bedding, Faults etc)

General Remarks

Joint surfaces of the JS3a joint set are slightly weathered due to water seepage. Joint surfaces have a brown discolouration.

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	ZQ	Face:	1	Date	16/03/1997
Blasting:	No	Photo:	No		

Length of survey line (m):	22.2	Height of slope (m):	17
Total number of joints:	40	Direction of slope (dip/dipdir):	85/178
Average FF/m:	1.80	Overall slope angle (deg):	85

Rock mass information

Weathering grade of rock mass:	2	RQD:	99
Hardness of rock mass:	R 5	Block size index (Ib):	0.47
Intact strength of rock (MPa) :	100 - 250	Block shape:	Blocky

Discontinuity information

		Set1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS2b	JS3				
Dip direction (deg):	Mean	340	81				
	Min.	327	68				
	Max.	350	105				
Dip (deg):	Mean	77	28				
	Min.	68	22				
	Max.	88	38				
No. of joints per line:		20	6				
Joint Spacing (m) :		1.10	0.31				
Joint Persistence (m):		10.7	5.7				
Joint Aperture (m):		O	T				
Joint Roughness:		7	4				
JRC value:		3	6				
Joint Filling:	Type:	CC	F				
	Nature:	2	2				
Joint wall rock weathering:		2	2				
Seepage:		1	1				
Termination Index (%):	Tr	5	42				
	Tx	75	33				
	Td	20	25				
Corr. Schmidt Hammer value:		46	40				

Structure (Bedding, Faults etc)

General Remarks

JS2b is characterised by minor calcite staining as joint infill.

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	ZQ	Face:	2	Date	16/03/1997
Blasting:	No	Photo:	Yes		

Length of survey line (m):	38.6	Height of slope (m):	10
Total number of joints:	66	Direction of slope (dip/dipdir):	88/316
Average FF/m:	1.71	Overall slope angle (deg):	88

Rock mass information

Weathering grade of rock mass:	2	RQD:	98
Hardness of rock mass:	R 5	Block size index (Ib):	0.26
Intact strength of rock (MPa) :	100 - 250	Block shape:	Irregular

Discontinuity information

		Set1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS3	JS4	JS5			
Dip direction (deg):	Mean	16	209	133			
	Min.	294	201	120			
	Max.	68	219	143			
Dip (deg):	Mean	6	74	59			
	Min.	4	66	46			
	Max.	10	85	70			
No. of joints per line:		4	13	12			
Joint Spacing (m) :		0.32	0.13	0.32			
Joint Persistence (m):		3.4	0.8	2.6			
Joint Aperture (m):		T	VT	VT			
Joint Roughness:		4	7	2			
JRC value:		4	2	3			
Joint Filling:	Type:	F	F	F			
	Nature:	2	2	2			
Joint wall rock weathering:		2	2	2			
Seepage:		1	1	1			
Termination Index (%):	Tr	8	31	--			
	Tx	88	12	25			
	Td	4	56	75			

Corr. Schmidt Hammer value:

Structure (Bedding, Faults etc)

General Remarks

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	ZQ	Face:	3	Date	17/03/1997
Blasting:	Yes	Photo:	Yes		

Length of survey line (m):	24.8	Height of slope (m):	12
Total number of joints:	42	Direction of slope (dip/dipdir):	90/053
Average FF/m:	1.69	Overall slope angle (deg):	90

Rock mass information

Weathering grade of rock mass:	2	RQD:	98
Hardness of rock mass:	R 5	Block size index (Ib):	0.24
Intact strength of rock (MPa) :	100 - 250	Block shape:	Irregular

Discontinuity information

		Set1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS2a	JS4	JS5			
Dip direction (deg):	Mean	145	184	135			
	Min.	140	175	140			
	Max.	150	195	150			
Dip (deg):	Mean	90	67	59			
	Min.	85	56	47			
	Max.	90	77	72			
No. of joints per line:		5	12	13			
Joint Spacing (m) :		0.16	0.33	0.23			
Joint Persistence (m):		2.8	2.5	3.9			
Joint Aperture (m):		PO	VT	PO			
Joint Roughness:		7	7	5			
JRC value:		2	2	3			
Joint Filling:	Type:	F/CC	F/CC	F/CC			
	Nature:	2	2	2			
Joint wall rock weathering:		2	2	2			
Seepage:		1	1	1			
Termination Index (%):	Tr	30	38	23			
	Tx	30	38	27			
	Td	40	24	50			
Corr. Schmidt Hammer value:		58	52	54			

Structure (Bedding, Faults etc)

General Remarks

Mostly, fresh, blue, very hard tillite but zone of reddish-brown tillite does occurr.

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	ZQ	Face:	4	Date	17/03/1997
Blasting:	No	Photo:	Yes		

Length of survey line (m):	23.2	Height of slope (m):	12
Total number of joints:	38	Direction of slope (dip/dipdir):	58/106
Average FF/m:	1.64	Overall slope angle (deg):	58

Rock mass information

Weathering grade of rock mass:	2	RQD:	98
Hardness of rock mass:	R 6	Block size index (Ib):	0.35
Intact strength of rock (MPa) :	>250	Block shape:	Irregular

Discontinuity information

		Set1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS2a	JS2b	JS5			
Dip direction (deg):	Mean	156	180	131			
	Min.	151	167	121			
	Max.	163	191	144			
Dip (deg):	Mean	66	69	59			
	Min.	60	59	50			
	Max.	72	80	69			
No. of joints per line:		5	11	7			
Joint Spacing (m) :		0.44	0.18	0.44			
Joint Persistence (m):		4.5	1.4	6.1			
Joint Aperture (m):		O	VT	O			
Joint Roughness:		4	7	7			
JRC value:		4	3	3			
Joint Filling:	Type:	CC	F/CC	F/CC			
	Nature:	2	2	2			
Joint wall rock weathering:		2	2	2			
Seepage:		1	1	1			
Termination Index (%):	Tr	--	41	7			
	Tx	80	36	64			
	Td	20	23	29			
Corr. Schmidt Hammer value:		54	51	51			

Structure (Bedding, Faults etc)

General Remarks

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.

ROCK MASS DESCRIPTION DATA SHEET

General Information

Quarry:	ZQ	Face:	5	Date	18/03/1997
Blasting:	Yes	Photo:	Yes		

Length of survey line (m):	12.6	Height of slope (m):	10
Total number of joints:	33	Direction of slope (dip/dipdir):	80/044
Average FF/m:	2.62	Overall slope angle (deg):	80

Rock mass information

Weathering grade of rock mass:	I	RQD:	98
Hardness of rock mass:	R 6	Block size index (Ib):	0.08
Intact strength of rock (MPa) :	>250	Block shape:	Irregular

Discontinuity information

		Set1	Set 2	Set 3	Set 4	Set 5	Set 6
Project Reference:		JS2a	JS4				
Dip direction (deg):	Mean	309	197				
	Min.	299	185				
	Max.	318	208				
Dip (deg):	Mean	84	72				
	Min.	75	68				
	Max.	90	79				
No. of joints per line:		13	7				
Joint Spacing (m) :		0.11	0.14				
Joint Persistence (m):		4.6	1.2				
Joint Aperture (m):		O	O				
Joint Roughness:		7	7				
JRC value:		4	5				
Joint Filling:	Type:	CC/Q	CC				
	Nature:	2/9	2				
Joint wall rock weathering:		I	I				
Seepage:		I	I				
Termination Index (%):	Tr	23	0				
	Tx	58	71				
	Td	19	29				
Corr. Schmidt Hammer value:		42	48				

Structure (Bedding, Faults etc)

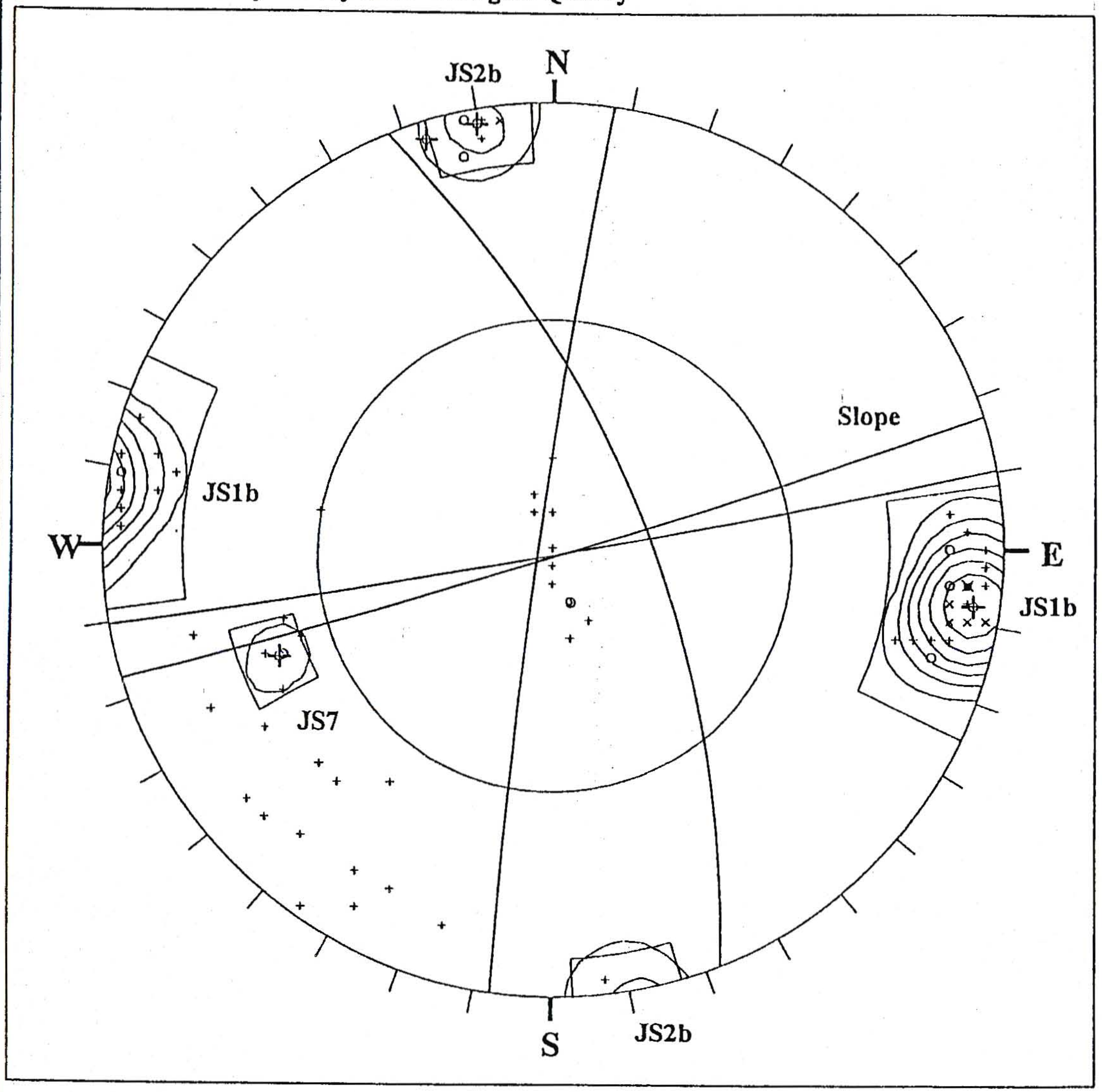
General Remarks

Note: For parameter definitions see Appendix A for the quantitative descriptions of discontinuities.
-- denotes insufficient data to calculate.

Appendix C

Stereonet Plots

Discontinuity Survey Data - Margate Quarry



Face 1

EQUAL ANGLE
LOWER HEMISPHERE

SCATTER LEGEND

NUM. OF POLES

1 pole
2 poles
3 poles
4 poles

CONTOUR LEGEND

FISHER POLE CONCENTRATIONS
% of total per 1.0 % area

Minimum Contour	=	5
Contour Interval	=	5
Max.Concentration	=	34.9

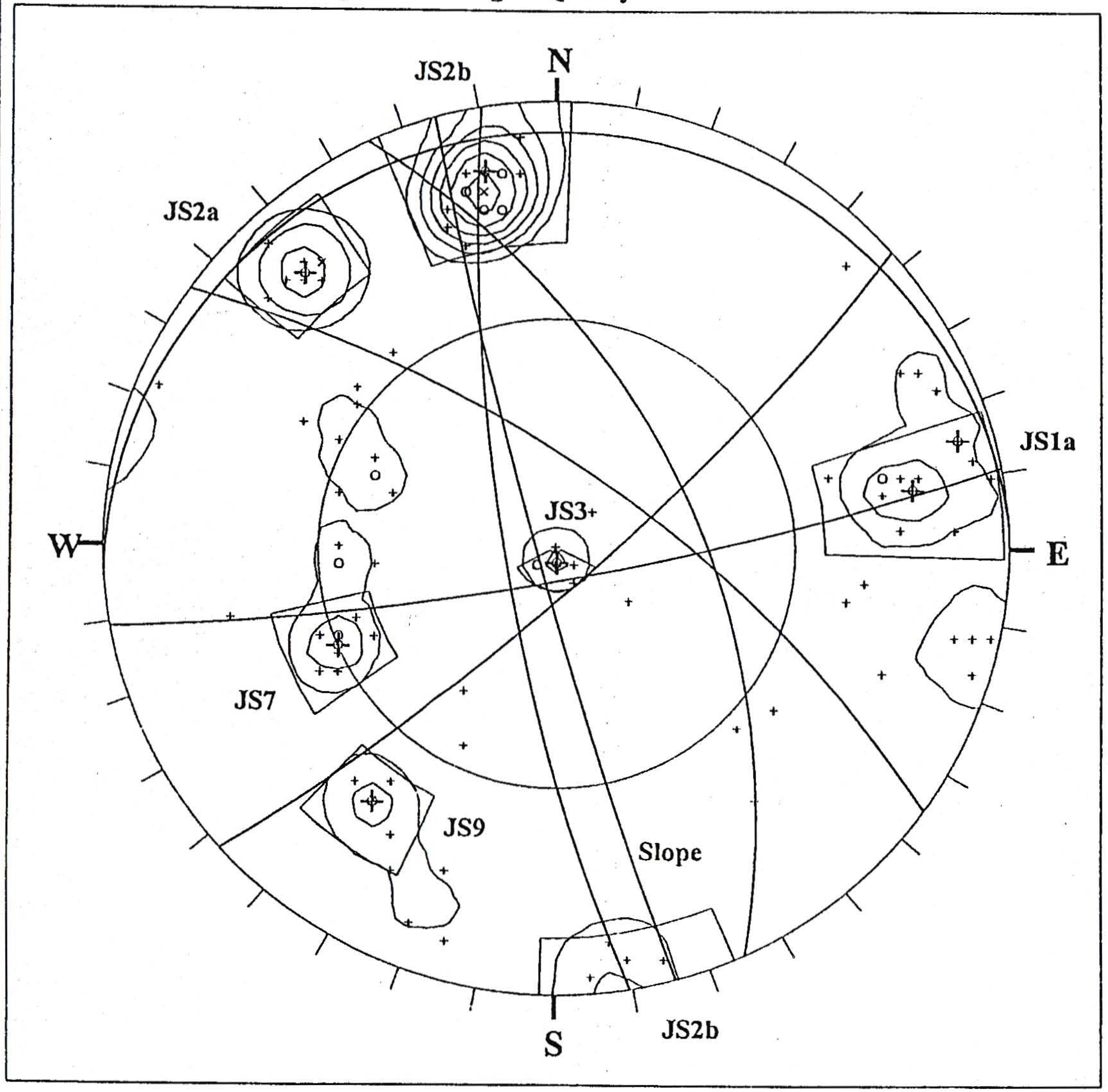
MAJOR PLANES

ORIENTATIONS

#	DIP/DIR.
Slope	88/163
JS1b	86/278
JS2b	88/170
JS7	67/068

84 Poles Plotted
84 Data Entries

Discontinuity Survey Data - Margate Quarry



EQUAL ANGLE
LOWER HEMISPHERE

SCATTER LEGEND

NUM. OF POLES

1 pole
2 poles
3 poles

CONTOUR LEGEND

FISHER POLE CONCENTRATIONS
% of total per
1.0 % area

Minimum Contour = 2.5
Contour Interval = 2.5
Max.Concentration = 16

MAJOR PLANES

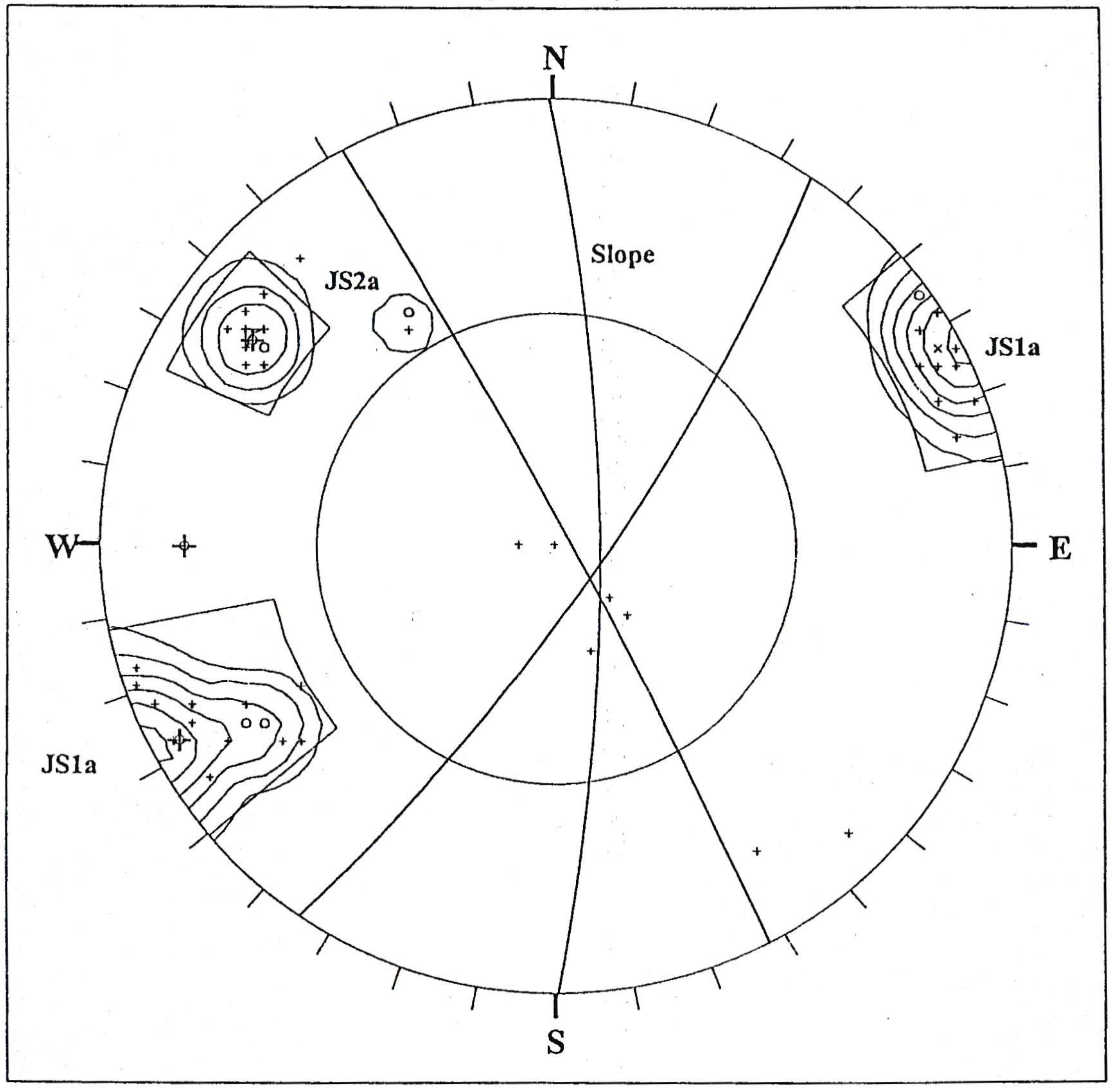
ORIENTATIONS

#	DIP/DIR.
Slope	85/255
JS1a	77/260
JS2b	81/169
JS2a	80/138
JS7	56/065
JS9	69/035
JS3	04/356

Face 2

100 Poles Plotted
100 Data Entries

Discontinuity Survey Data - Margate Quarry



Face 3

EQUAL ANGLE
LOWER HEMISPHERE

SCATTER LEGEND

NUM. OF POLES

1 pole
2 poles
3 poles

CONTOUR LEGEND

FISHER POLE CONCENTRATIONS
% of total per 1.0 % area

Minimum Contour	=	4.5
Contour Interval	=	4.5
Max.Concentration	=	28.3

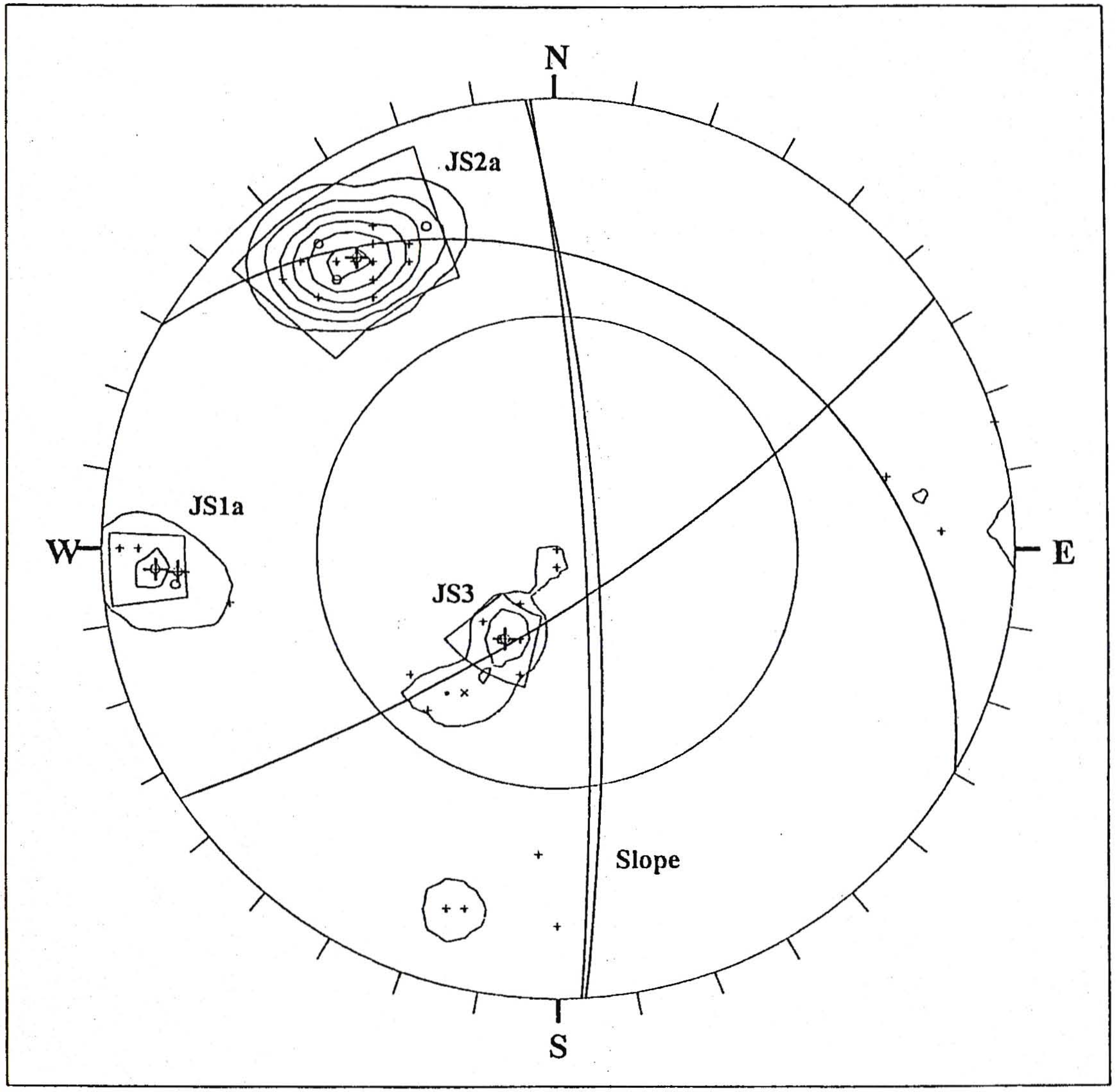
MAJOR PLANES

ORIENTATIONS
 # DIP/DIR.

Slope	79/090
JS1a	86/062
JS2a	78/124

51 Poles Plotted
51 Data Entries

Discontinuity Survey Data - Ridge View Quarry



Face 1

EQUAL ANGLE
LOWER HEMISPHERE

SCATTER LEGEND

NUM. OF POLES

1 pole
2 poles
3 poles

CONTOUR LEGEND

FISHER POLE CONCENTRATIONS
% of total per
1.0 % area

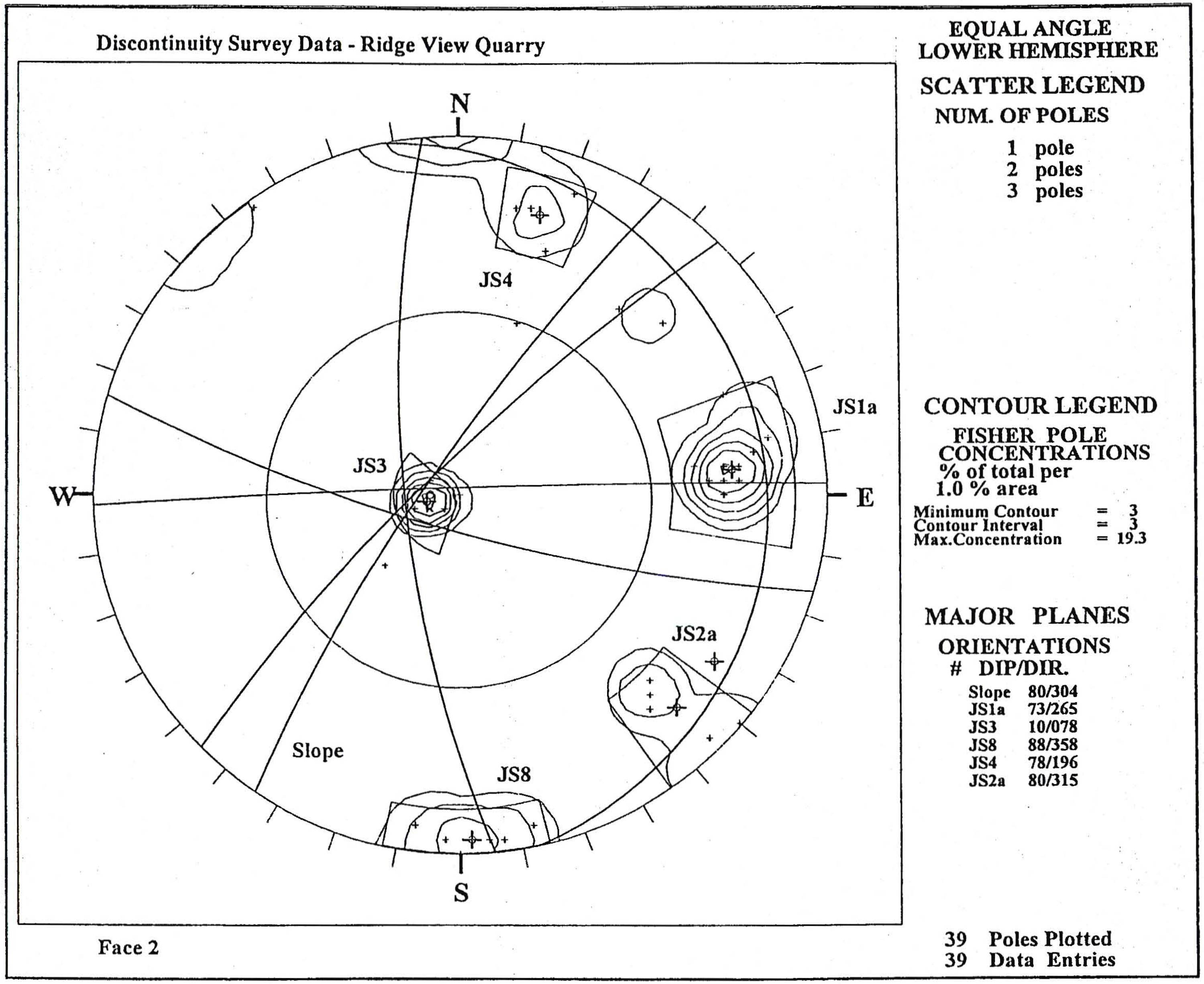
Minimum Contour	=	4
Contour Interval	=	4
Max.Concentration	=	26

MAJOR PLANES

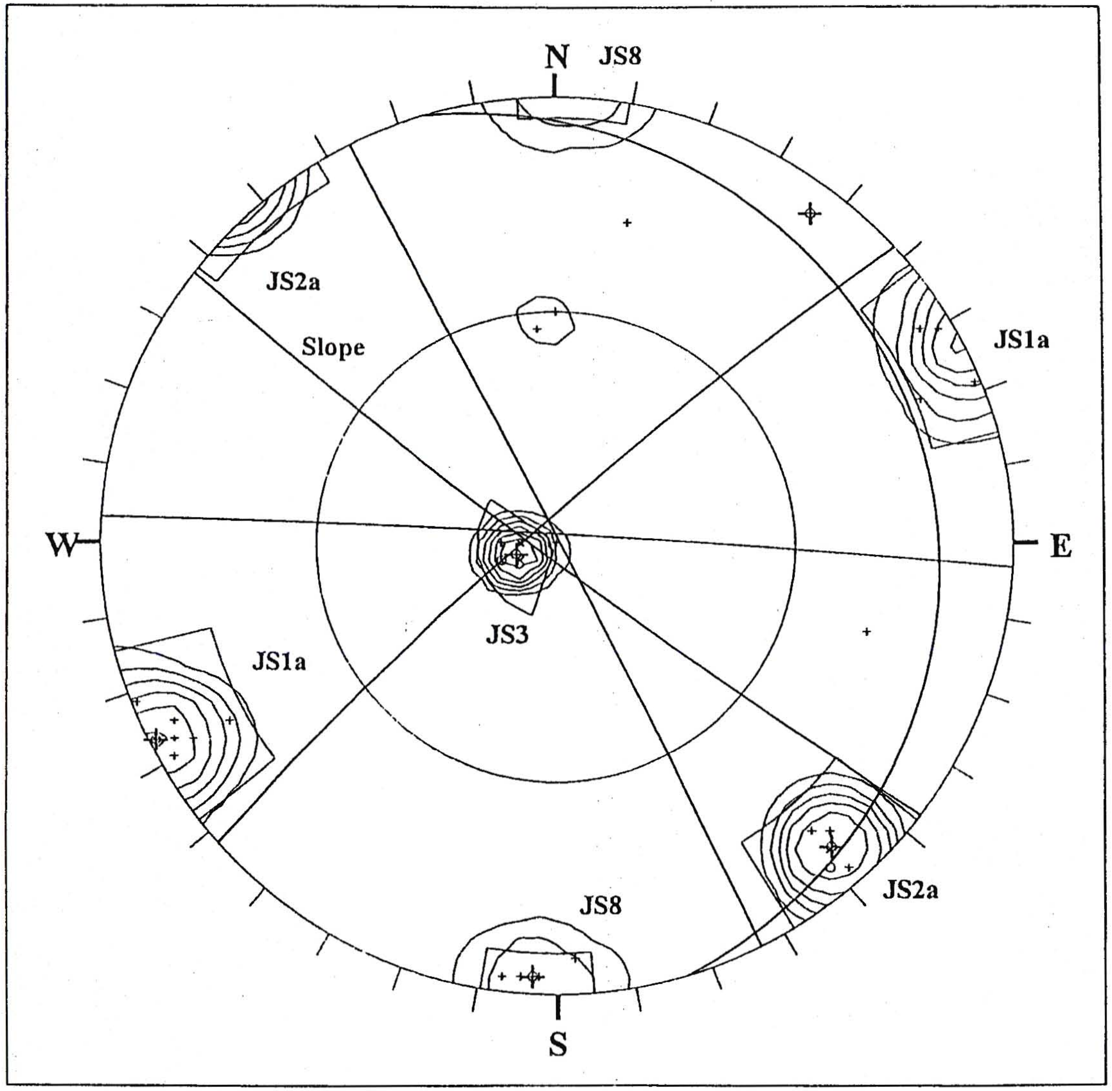
ORIENTATIONS

#	DIP/DIR.
Slope	80/086
JS1a	83/087
JS2a	76/146
JS3	26/030

43 Poles Plotted
43 Data Entries


Discontinuity Survey Data - Ridge View Quarry
N
E
S
W
JS4
JS1a
JS3
JS2a
Slope
JS8
Face 2
EQUAL ANGLE
LOWER HEMISPHERE
SCATTER LEGEND
NUM. OF POLES
1 pole
2 poles
3 poles
CONTOUR LEGEND
FISHER POLE
CONCENTRATIONS
% of total per
1.0 % area
Minimum Contour = 3
Contour Interval = 3
Max.Concentration = 19.3
MAJOR PLANES
ORIENTATIONS
DIP/DIR.
Slope 80/304
JS1a 73/265
JS3 10/078
JS8 88/358
JS4 78/196
JS2a 80/315
39 Poles Plotted
39 Data Entries

Discontinuity Survey Data - Ridge View Quarry



Face 3

EQUAL ANGLE
LOWER HEMISPHERE

SCATTER LEGEND

NUM. OF POLES

1 pole
2 poles
3 poles

CONTOUR LEGEND

FISHER POLE CONCENTRATIONS
% of total per
1.0 % area

Minimum Contour = 3.5
Contour Interval = 3.5
Max.Concentration = 23.5

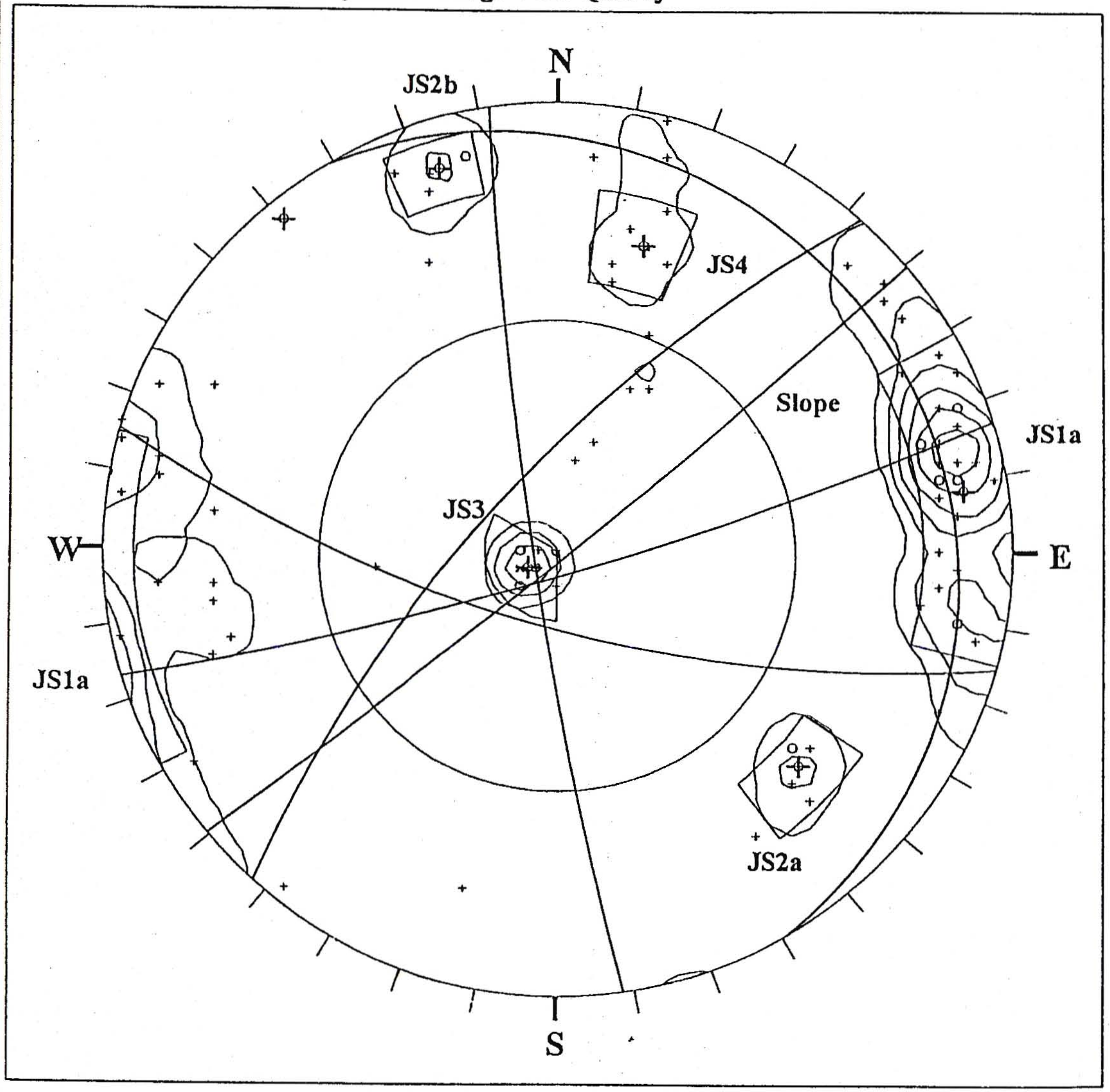
MAJOR PLANES

ORIENTATIONS

#	DIP/DIR.
Slope	86/217
JS1a	89/063
JS2a	84/318
JS3	10/073
JS8	88/003

38 Poles Plotted
38 Data Entries

Discontinuity Survey Data - Ridge View Quarry



EQUAL ANGLE
LOWER HEMISPHERE

SCATTER LEGEND

NUM. OF POLES

1 pole
2 poles
3 poles

CONTOUR LEGEND

FISHER POLE
CONCENTRATIONS
% of total per
1.0 % area

Minimum Contour = 2.5
Contour Interval = 2.5
Max.Concentration = 17

MAJOR PLANES

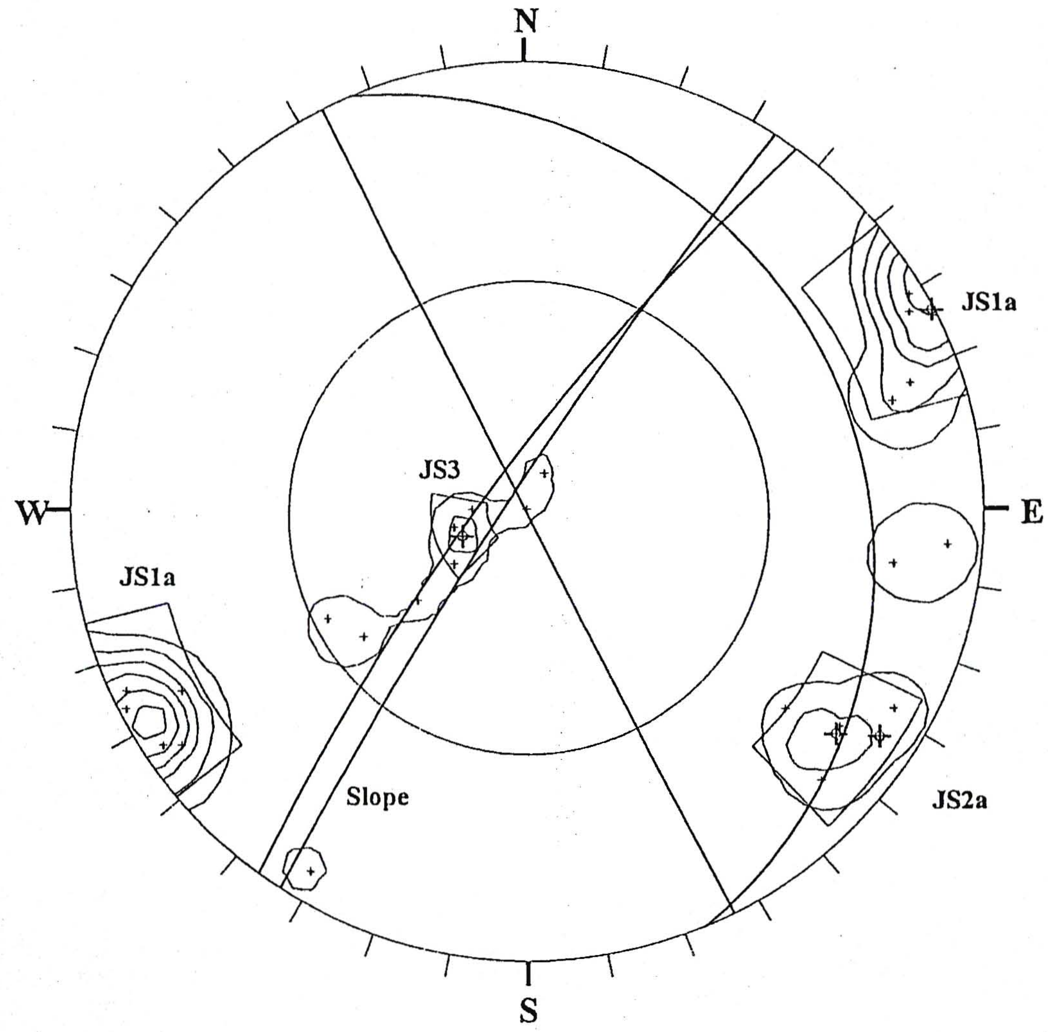
ORIENTATIONS

#	DIP/DIR.
Slope	87/140
JS1a	84/261
JS2a	71/312
JS3	08/059
JS2b	83/163
JS4	71/195

Face 4

87 Poles Plotted
87 Data Entries

Discontinuity Survey Data - Ridge View Quarry



Face 5

EQUAL ANGLE
LOWER HEMISPHERE

SCATTER LEGEND
NUM. OF POLES
1 pole

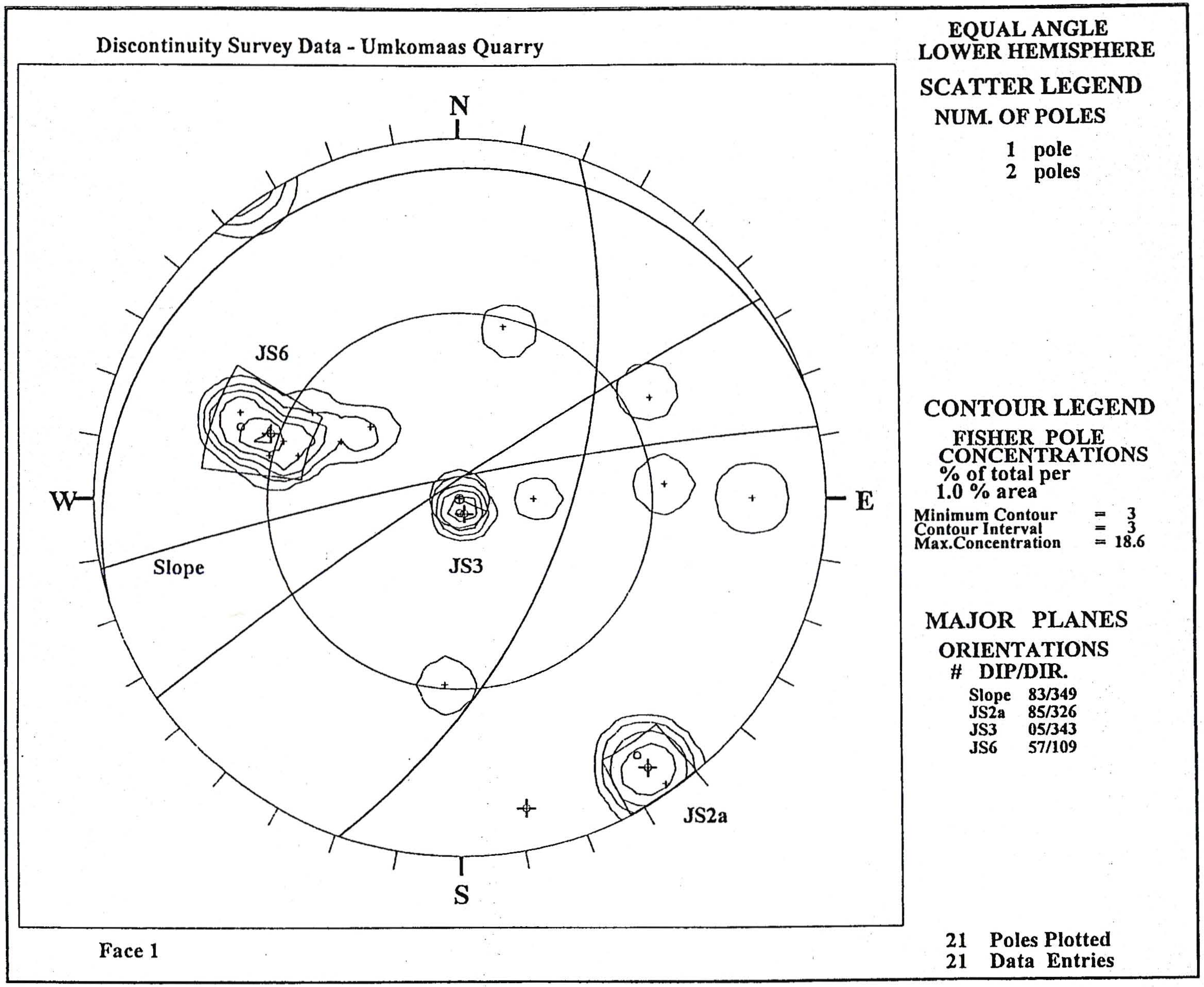
CONTOUR LEGEND
FISHER POLE CONCENTRATIONS
% of total per 1.0 % area

Minimum Contour	=	4
Contour Interval	=	4
Max.Concentration	=	25.9

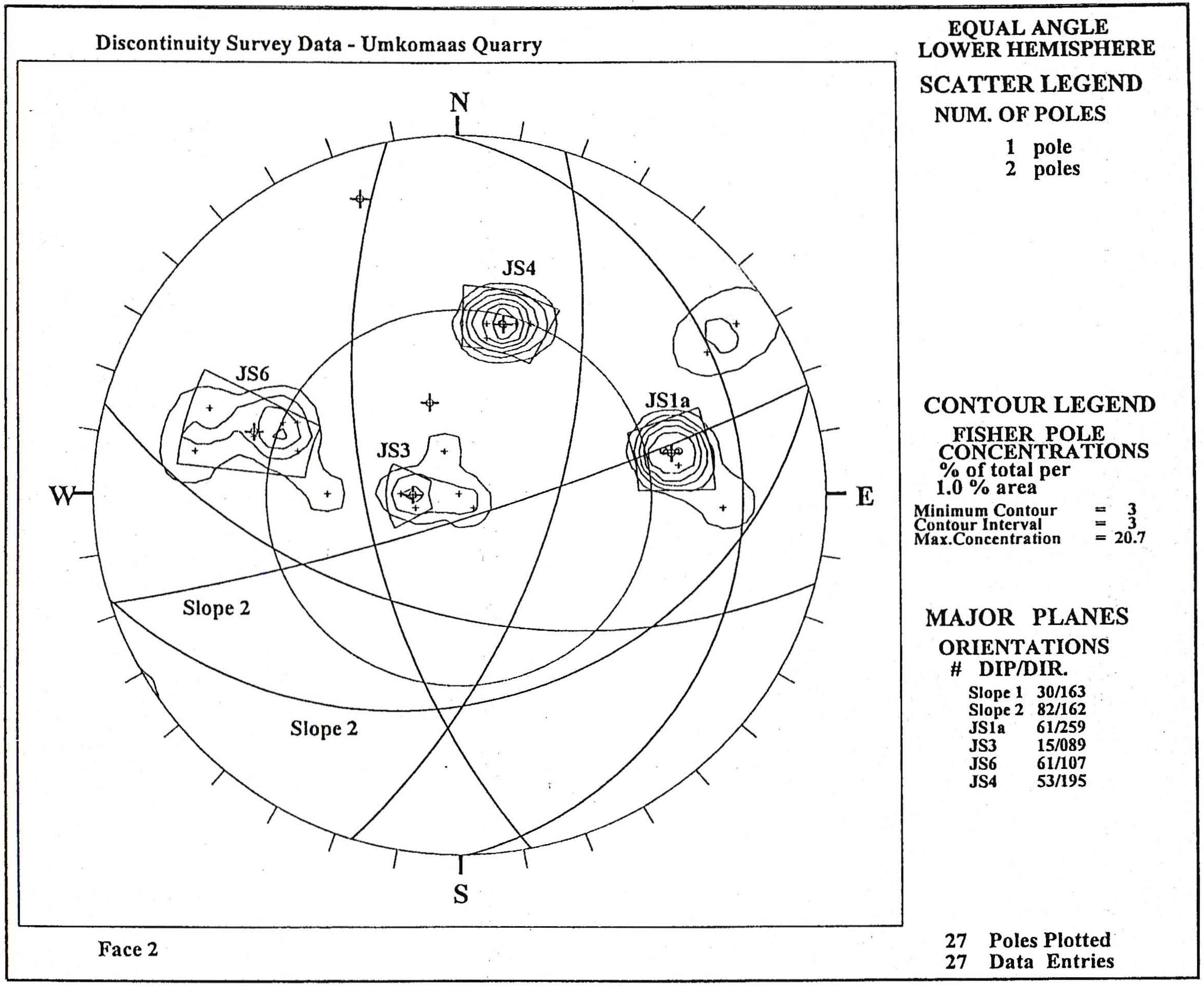
MAJOR PLANES
ORIENTATIONS

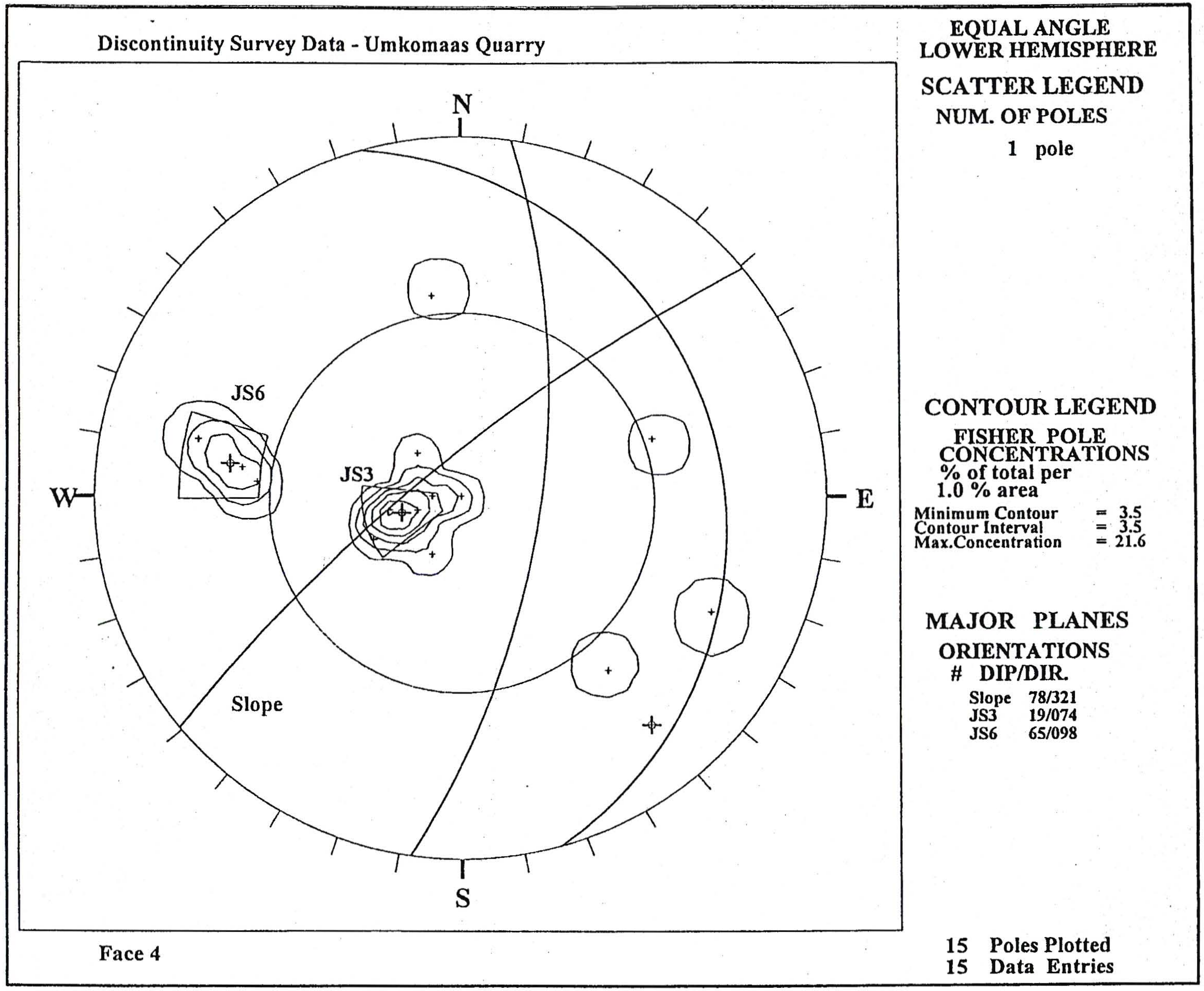
#	DIP/DIR.
Slope	85/303
JS1a	90/243
JS2a	80/306
JS3	17/067

24 Poles Plotted
24 Data Entries

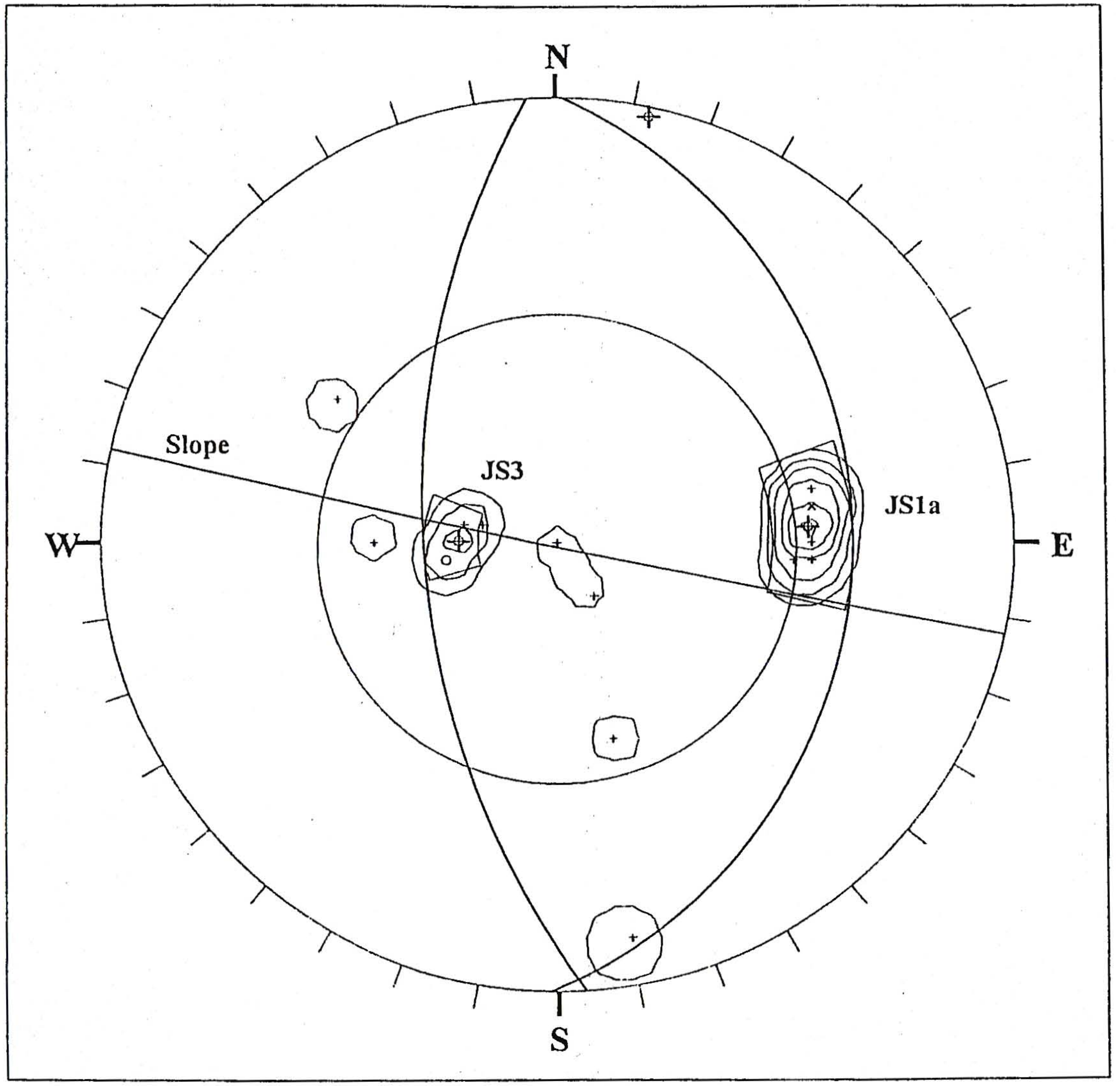


Discontinuity Survey Data - Umkomaas Quarry
N
W
E
S
JS6
Slope
JS3
JS2a
Face 1
EQUAL ANGLE
LOWER HEMISPHERE
SCATTER LEGEND
NUM. OF POLES
1 pole
2 poles
CONTOUR LEGEND
FISHER POLE
CONCENTRATIONS
% of total per
1.0 % area
Minimum Contour = 3
Contour Interval = 3
Max.Concentration = 18.6
MAJOR PLANES
ORIENTATIONS
DIP/DIR.
Slope 83/349
JS2a 85/326
JS3 05/343
JS6 57/109
21 Poles Plotted
21 Data Entries


Discontinuity Survey Data - Umkomaas Quarry
N
W
E
S
JS4
JS6
JS1a
JS3
Slope 2
Slope 2
Face 2
EQUAL ANGLE
LOWER HEMISPHERE
SCATTER LEGEND
NUM. OF POLES
1 pole
2 poles
CONTOUR LEGEND
FISHER POLE
CONCENTRATIONS
% of total per
1.0 % area
Minimum Contour = 3
Contour Interval = 3
Max.Concentration = 20.7
MAJOR PLANES
ORIENTATIONS
DIP/DIR.
Slope 1 30/163
Slope 2 82/162
JS1a 61/259
JS3 15/089
JS6 61/107
JS4 53/195
27 Poles Plotted
27 Data Entries


Discontinuity Survey Data - Umkomaas Quarry
N
E
S
W
JS6
JS3
Slope
EQUAL ANGLE
LOWER HEMISPHERE
SCATTER LEGEND
NUM. OF POLES
1 pole
CONTOUR LEGEND
FISHER POLE
CONCENTRATIONS
% of total per
1.0 % area
Minimum Contour = 3.5
Contour Interval = 3.5
Max.Concentration = 21.6
MAJOR PLANES
ORIENTATIONS
DIP/DIR.
Slope 78/321
JS3 19/074
JS6 65/098
Face 4
15 Poles Plotted
15 Data Entries

Discontinuity Survey Data - Umkomaas Quarry



Face 5

EQUAL ANGLE
LOWER HEMISPHERE

SCATTER LEGEND

NUM. OF POLES

1 pole
2 poles
3 poles

CONTOUR LEGEND

FISHER POLE CONCENTRATIONS
% of total per 1.0 % area

Minimum Contour	=	5
Contour Interval	=	5
Max.Concentration	=	31.6

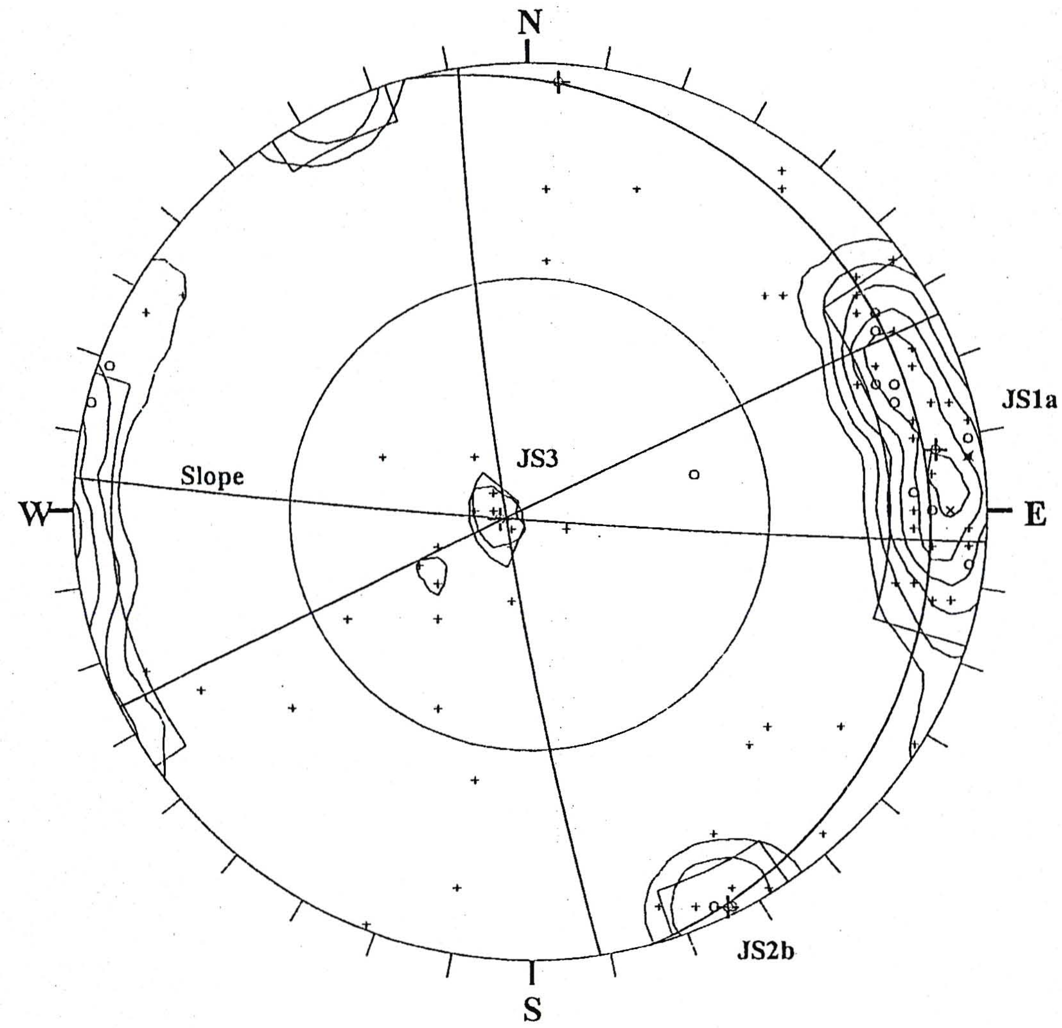
MAJOR PLANES

ORIENTATIONS

#	DIP/DIR.
Slope	89/192
JS1a	58/266
JS3	24/091

17 Poles Plotted
17 Data Entries

Discontinuity Survey Data - Westville Quarry



Face 1

EQUAL ANGLE
LOWER HEMISPHERE

SCATTER LEGEND

NUM. OF POLES

1 pole
2 poles
3 poles
4 poles

CONTOUR LEGEND

FISHER POLE CONCENTRATIONS
% of total per
1.0 % area

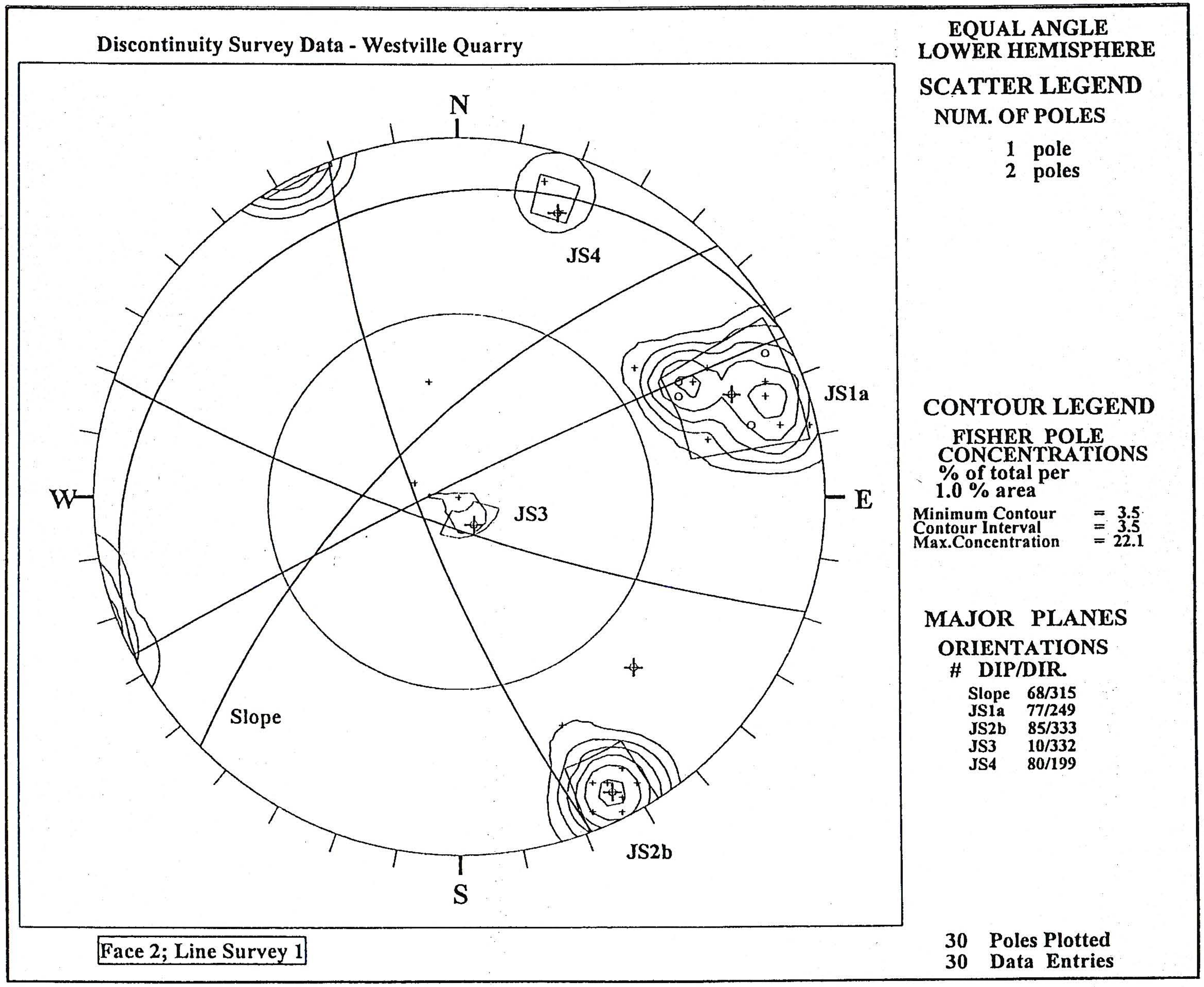
Minimum Contour	=	2.5
Contour Interval	=	2.5
Max.Concentration	=	16.4

MAJOR PLANES

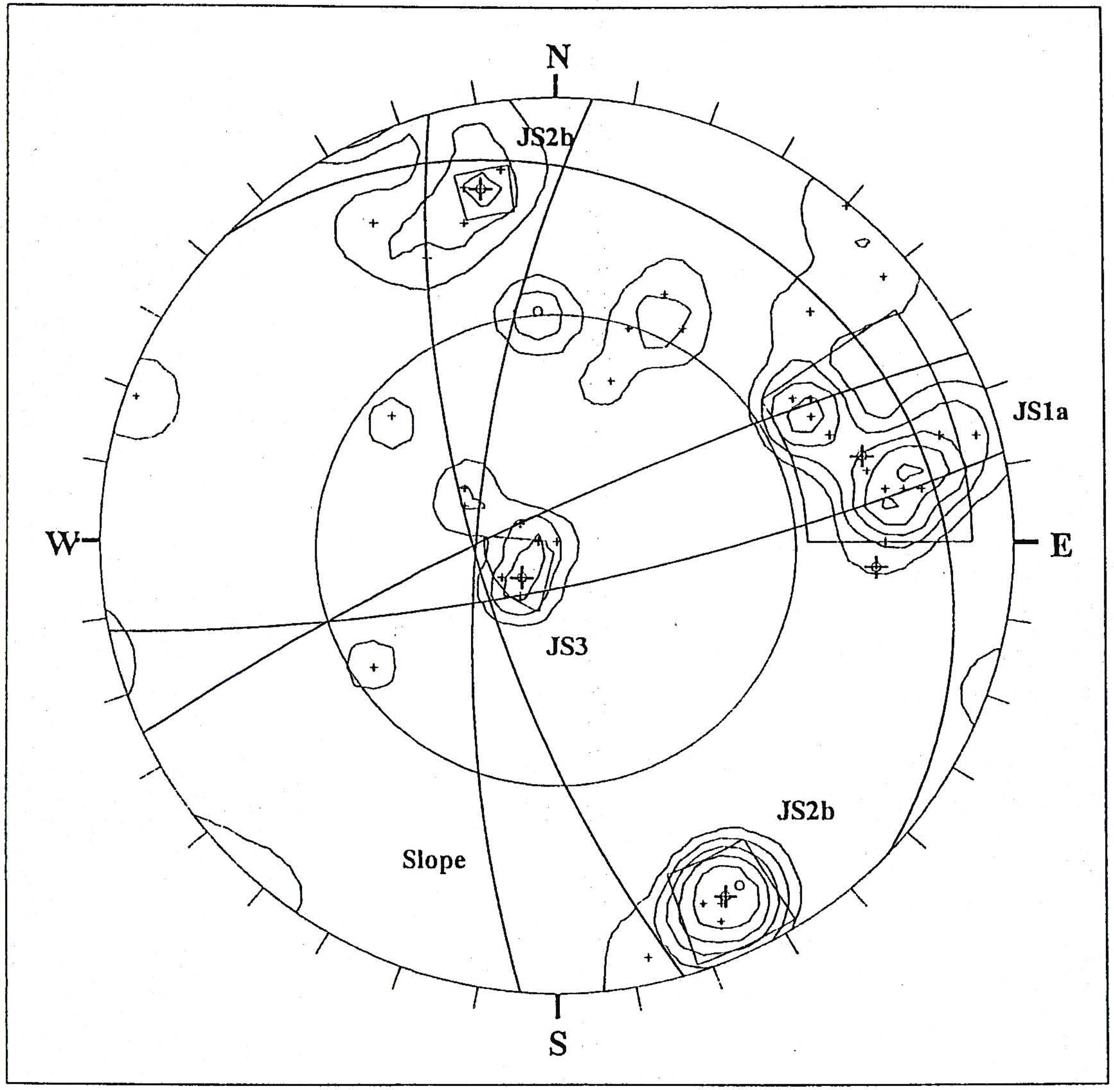
ORIENTATIONS

#	DIP/DIR.
Slope	88/184
JS1a	84/261
JS2b	89/334
JS3	08/075

99 Poles Plotted
99 Data Entries


Discontinuity Survey Data - Westville Quarry
N
S
E
W
JS4
JS1a
JS3
JS2b
Slope
EQUAL ANGLE
LOWER HEMISPHERE
SCATTER LEGEND
NUM. OF POLES
1 pole
2 poles
CONTOUR LEGEND
FISHER POLE CONCENTRATIONS
% of total per
1.0 % area
Minimum Contour = 3.5
Contour Interval = 3.5
Max.Concentration = 22.1
MAJOR PLANES
ORIENTATIONS
DIP/DIR.
Slope 68/315
JS1a 77/249
JS2b 85/333
JS3 10/332
JS4 80/199
Face 2; Line Survey 1
30 Poles Plotted
30 Data Entries

Discontinuity Survey Data - Westville Quarry



Face 2; Line Survey 2 & 3

EQUAL ANGLE
LOWER HEMISPHERE

SCATTER LEGEND

NUM. OF POLES

1 pole
2 poles

CONTOUR LEGEND

FISHER POLE CONCENTRATIONS
% of total per
1.0 % area

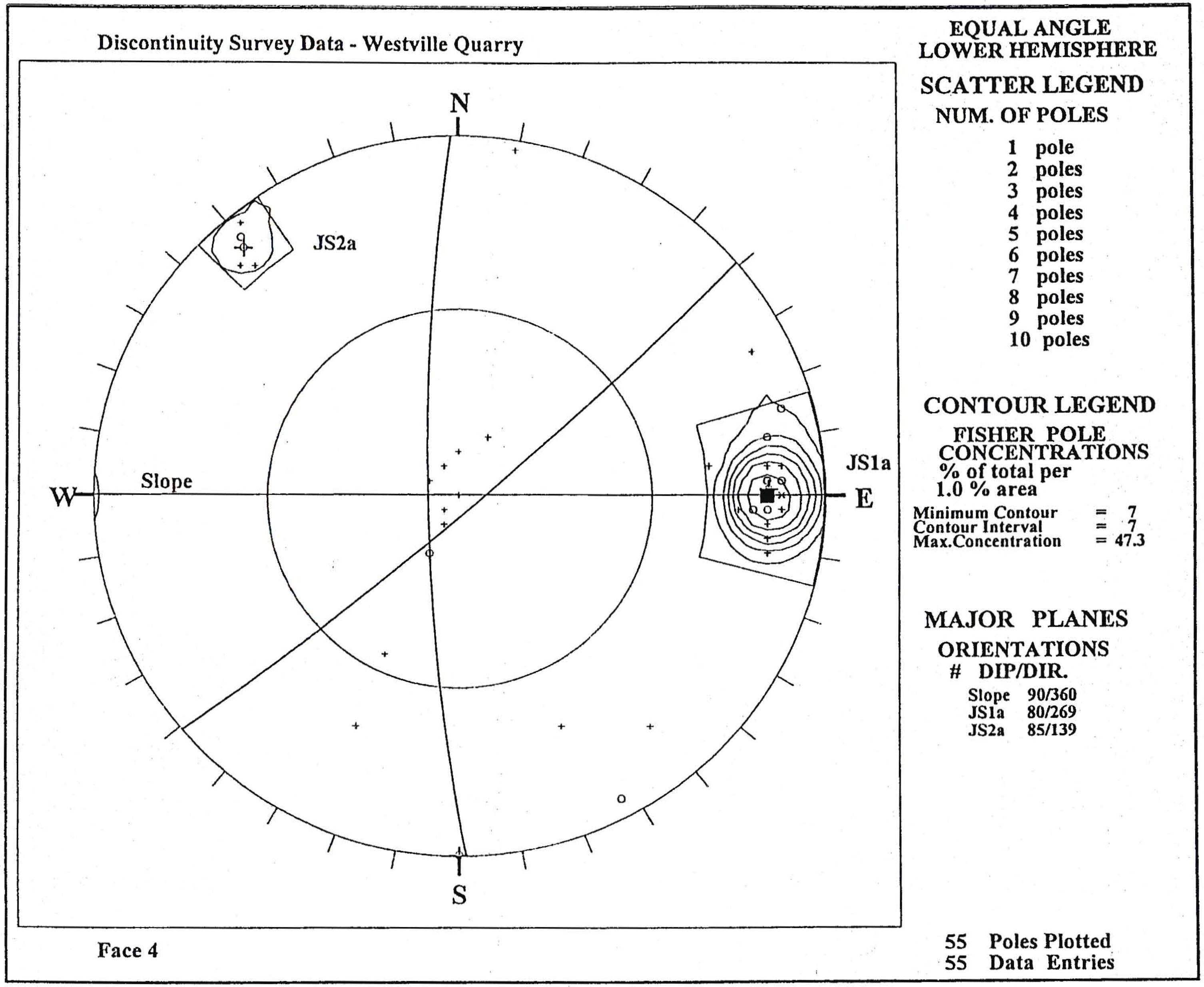
Minimum Contour	=	2
Contour Interval	=	2
Max.Concentration	=	11.6

MAJOR PLANES

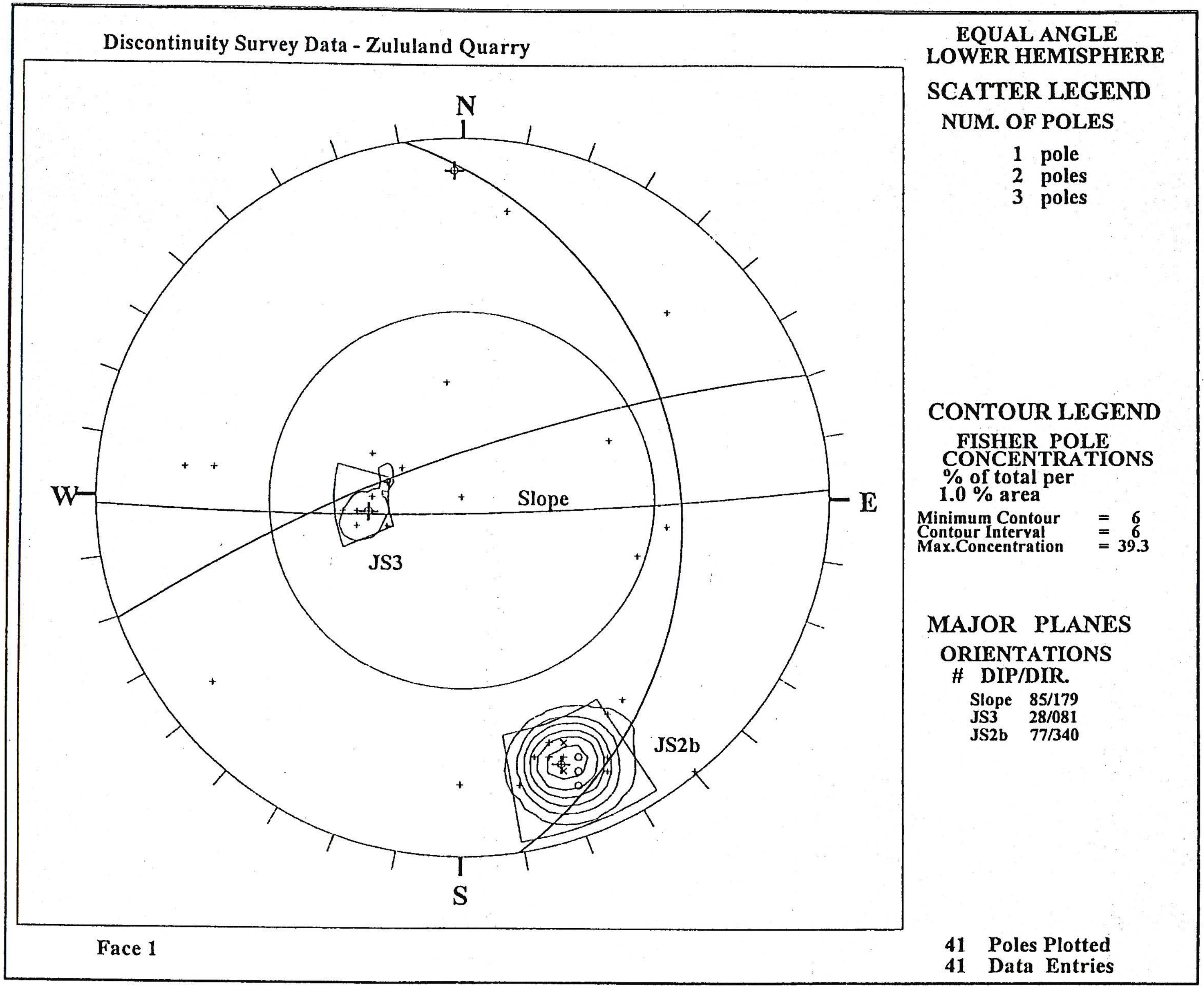
ORIENTATIONS

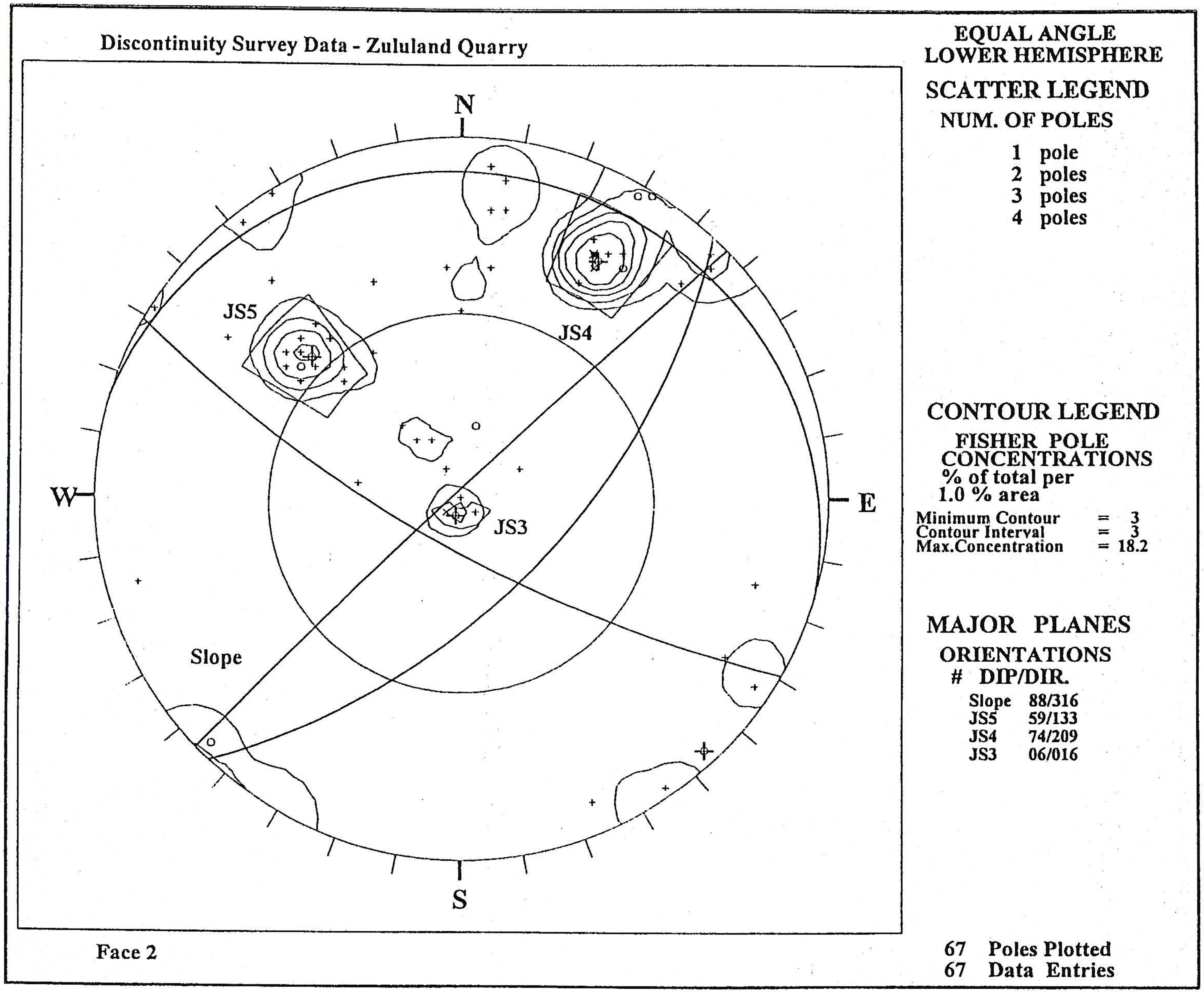
#	DIP/DIR.
Slope	70/275
JS1a	70/254
JS2b	82/335
JS3	13/044
JS2b	78/168

42 Poles Plotted
42 Data Entries



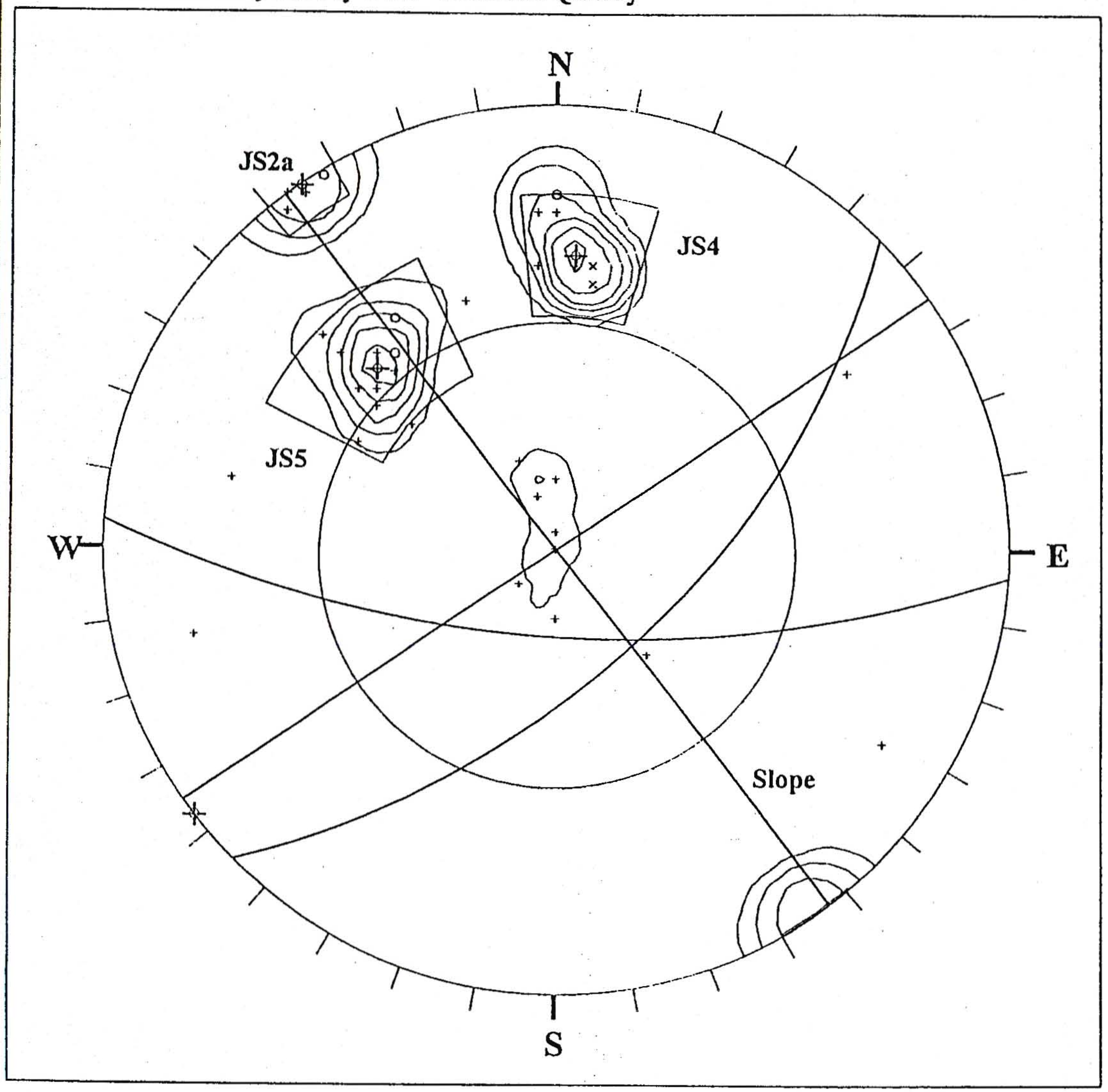
Discontinuity Survey Data - Westville Quarry
N
S
W
E
Slope
JS1a
JS2a
EQUAL ANGLE
LOWER HEMISPHERE
SCATTER LEGEND
NUM. OF POLES
1 pole
2 poles
3 poles
4 poles
5 poles
6 poles
7 poles
8 poles
9 poles
10 poles
CONTOUR LEGEND
FISHER POLE CONCENTRATIONS
% of total per
1.0 % area
Minimum Contour = 7
Contour Interval = 7
Max.Concentration = 47.3
MAJOR PLANES
ORIENTATIONS
DIP/DIR.
Slope 90/360
JS1a 80/269
JS2a 85/139
Face 4
55 Poles Plotted
55 Data Entries


Discontinuity Survey Data - Zululand Quarry
N
W
E
S
Slope
JS3
JS2b
Face 1
EQUAL ANGLE
LOWER HEMISPHERE
SCATTER LEGEND
NUM. OF POLES
1 pole
2 poles
3 poles
CONTOUR LEGEND
FISHER POLE
CONCENTRATIONS
% of total per
1.0 % area
Minimum Contour = 6
Contour Interval = 6
Max.Concentration = 39.3
MAJOR PLANES
ORIENTATIONS
DIP/DIR.
Slope 85/179
JS3 28/081
JS2b 77/340
41 Poles Plotted
41 Data Entries



Discontinuity Survey Data - Zululand Quarry
N
E
S
W
JS5
JS4
JS3
Slope
Face 2
EQUAL ANGLE
LOWER HEMISPHERE
SCATTER LEGEND
NUM. OF POLES
1 pole
2 poles
3 poles
4 poles
CONTOUR LEGEND
FISHER POLE CONCENTRATIONS
% of total per
1.0 % area
Minimum Contour = 3
Contour Interval = 3
Max.Concentration = 18.2
MAJOR PLANES
ORIENTATIONS
DIP/DIR.
Slope 88/316
JS5 59/133
JS4 74/209
JS3 06/016
67 Poles Plotted
67 Data Entries

Discontinuity Survey Data - Zululand Quarry



Face 3

EQUAL ANGLE
LOWER HEMISPHERE

SCATTER LEGEND

NUM. OF POLES

1 pole
2 poles
3 poles

CONTOUR LEGEND

FISHER POLE CONCENTRATIONS
% of total per 1.0 % area

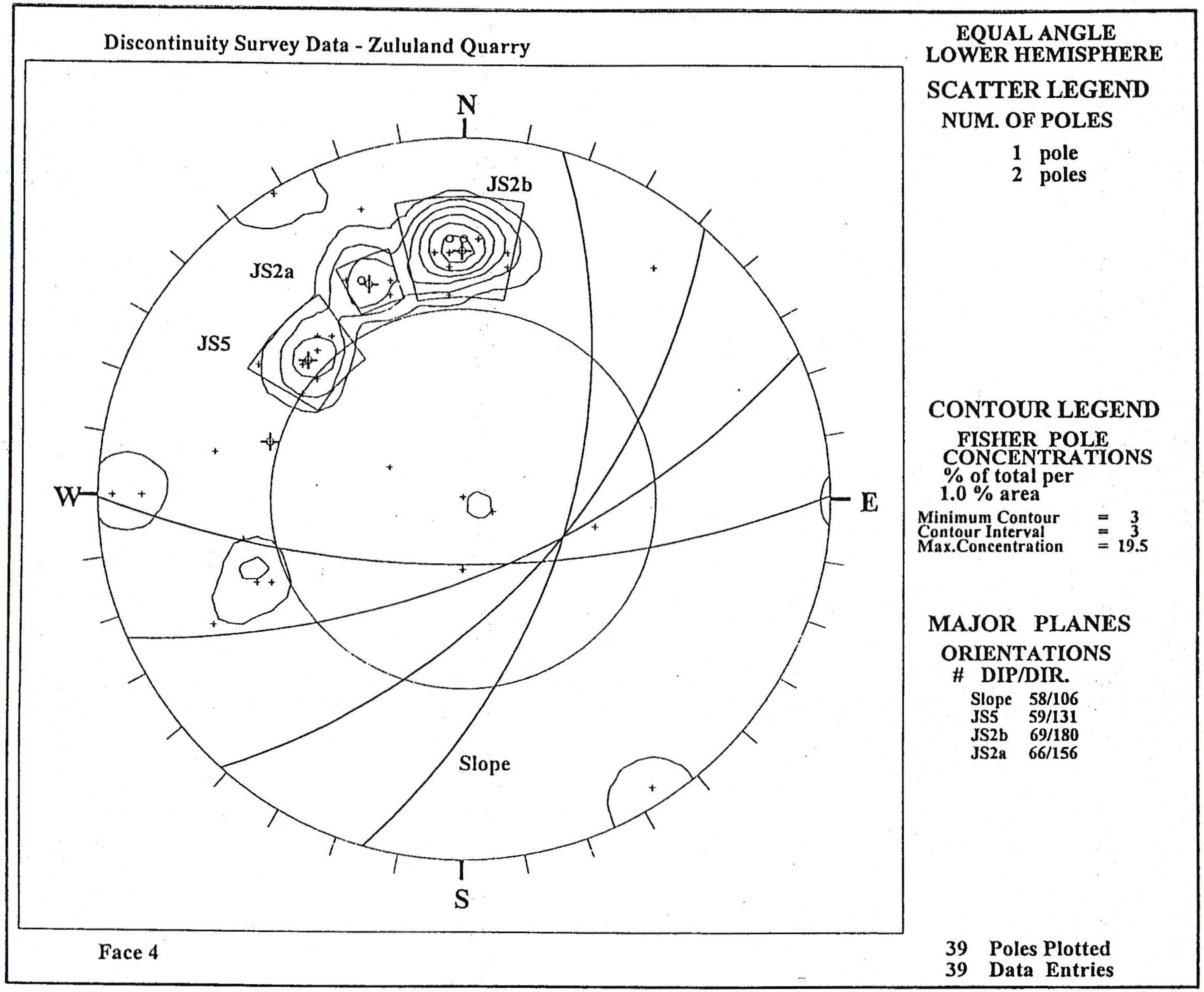
Minimum Contour	=	3
Contour Interval	=	3
Max.Concentration	=	19.5

MAJOR PLANES

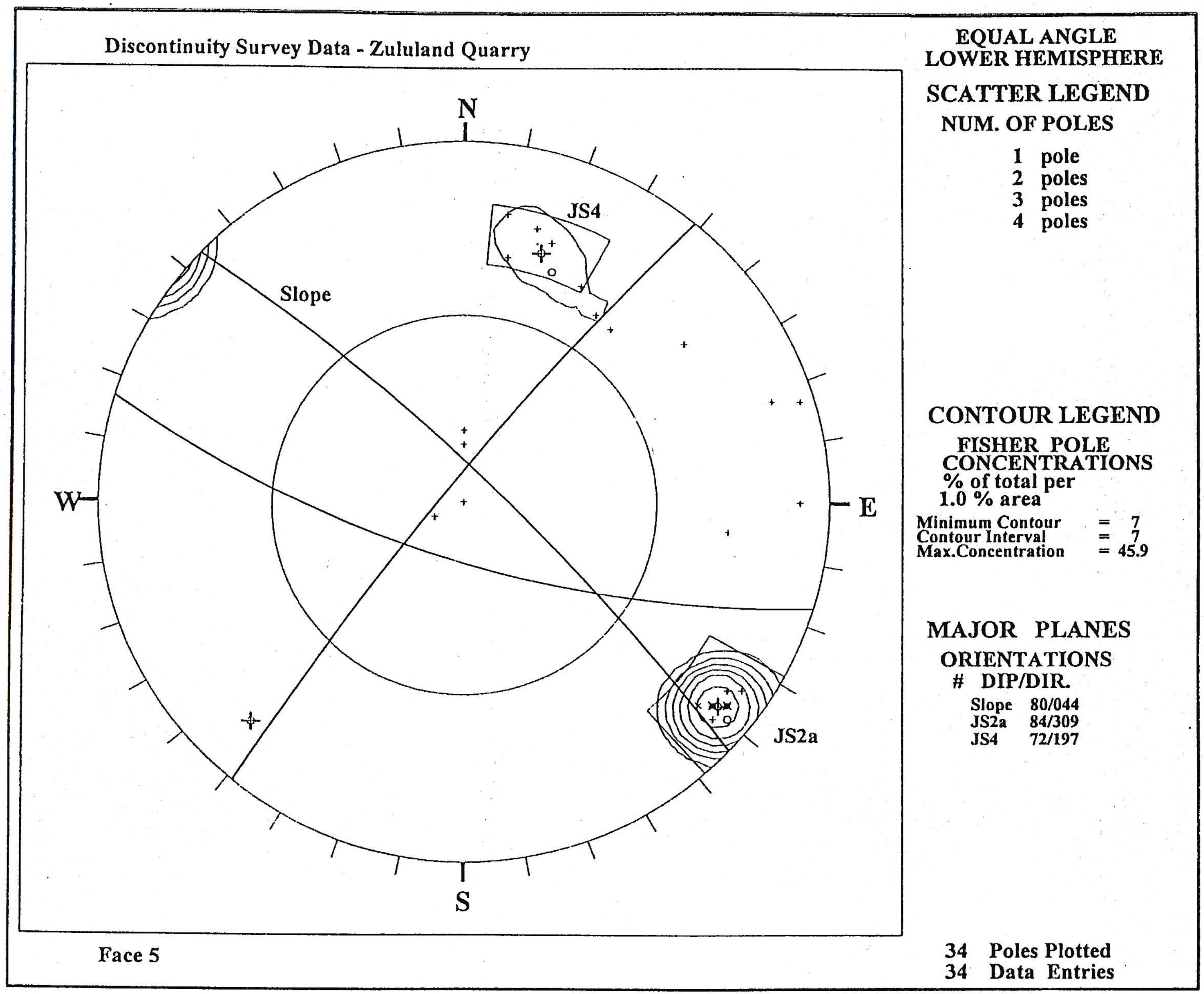
ORIENTATIONS

#	DIP/DIR.
Slope	90/053
JS5	59/135
JS2a	90/145
JS4	67/184

43 Poles Plotted
43 Data Entries



Discontinuity Survey Data - Zululand Quarry
N
E
S
W
JS2b
JS2a
JS5
Slope
EQUAL ANGLE
LOWER HEMISPHERE
SCATTER LEGEND
NUM. OF POLES
1 pole
2 poles
CONTOUR LEGEND
FISHER POLE
CONCENTRATIONS
% of total per
1.0 % area
Minimum Contour = 3
Contour Interval = 3
Max.Concentration = 19.5
MAJOR PLANES
ORIENTATIONS
DIP/DIR.
Slope 58/106
JS5 59/131
JS2b 69/180
JS2a 66/156
Face 4
39 Poles Plotted
39 Data Entries


Discontinuity Survey Data - Zululand Quarry
N
S
E
W
JS4
Slope
JS2a
EQUAL ANGLE
LOWER HEMISPHERE
SCATTER LEGEND
NUM. OF POLES
1 pole
2 poles
3 poles
4 poles
CONTOUR LEGEND
FISHER POLE
CONCENTRATIONS
% of total per
1.0 % area
Minimum Contour = 7
Contour Interval = 7
Max.Concentration = 45.9
MAJOR PLANES
ORIENTATIONS
DIP/DIR.
Slope 80/044
JS2a 84/309
JS4 72/197
Face 5
34 Poles Plotted
34 Data Entries

Appendix D

Shear Test – Description of joint surfaces

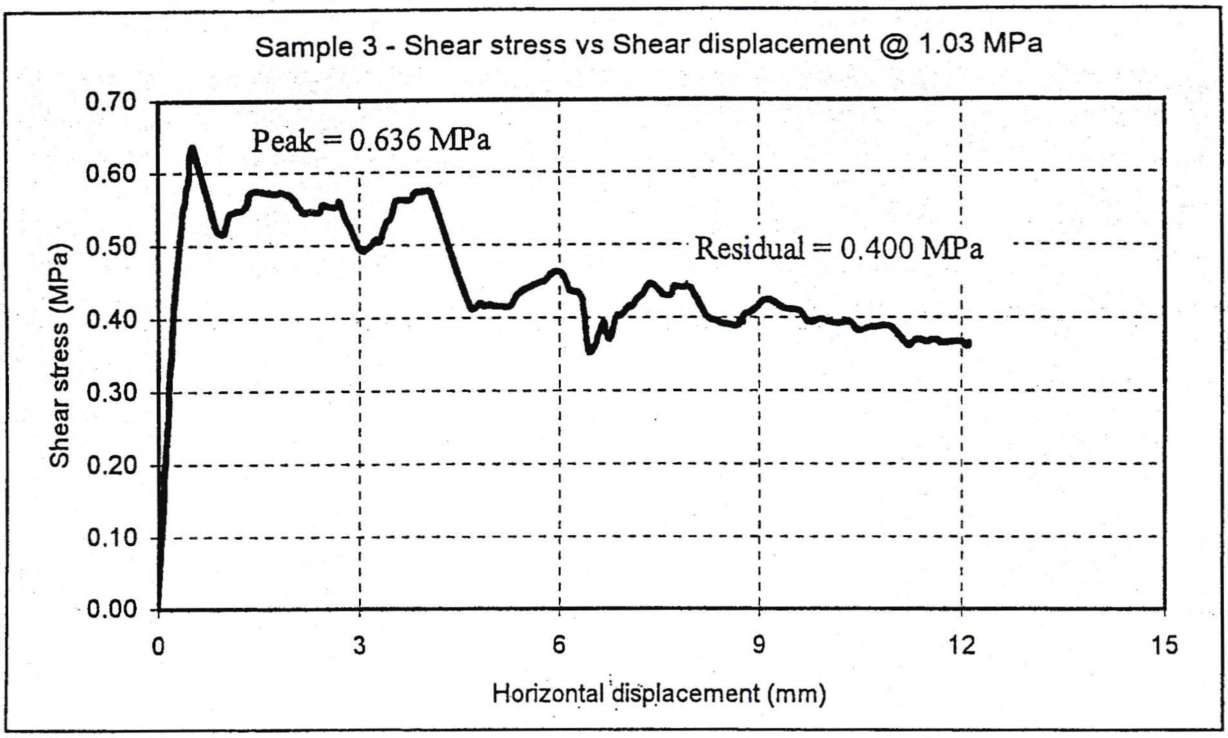
Shearbox Samples

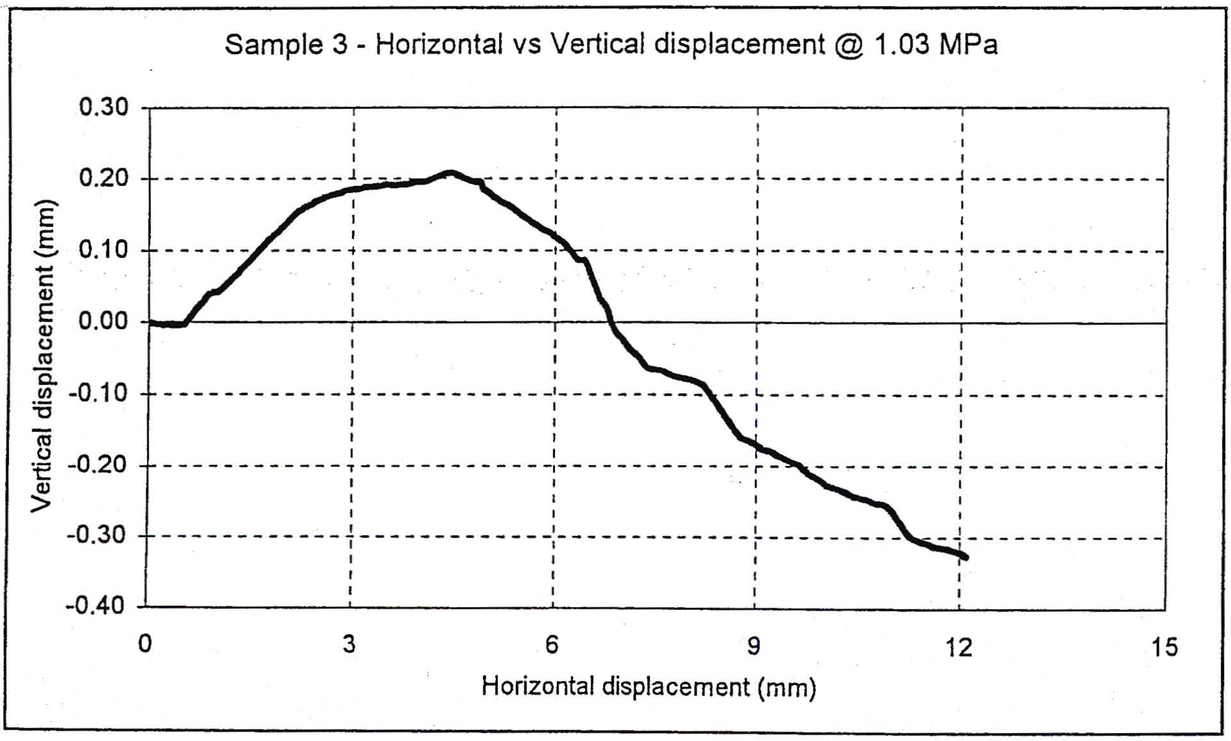
Inanda Expressway
Diameter of core (m) 0.00546

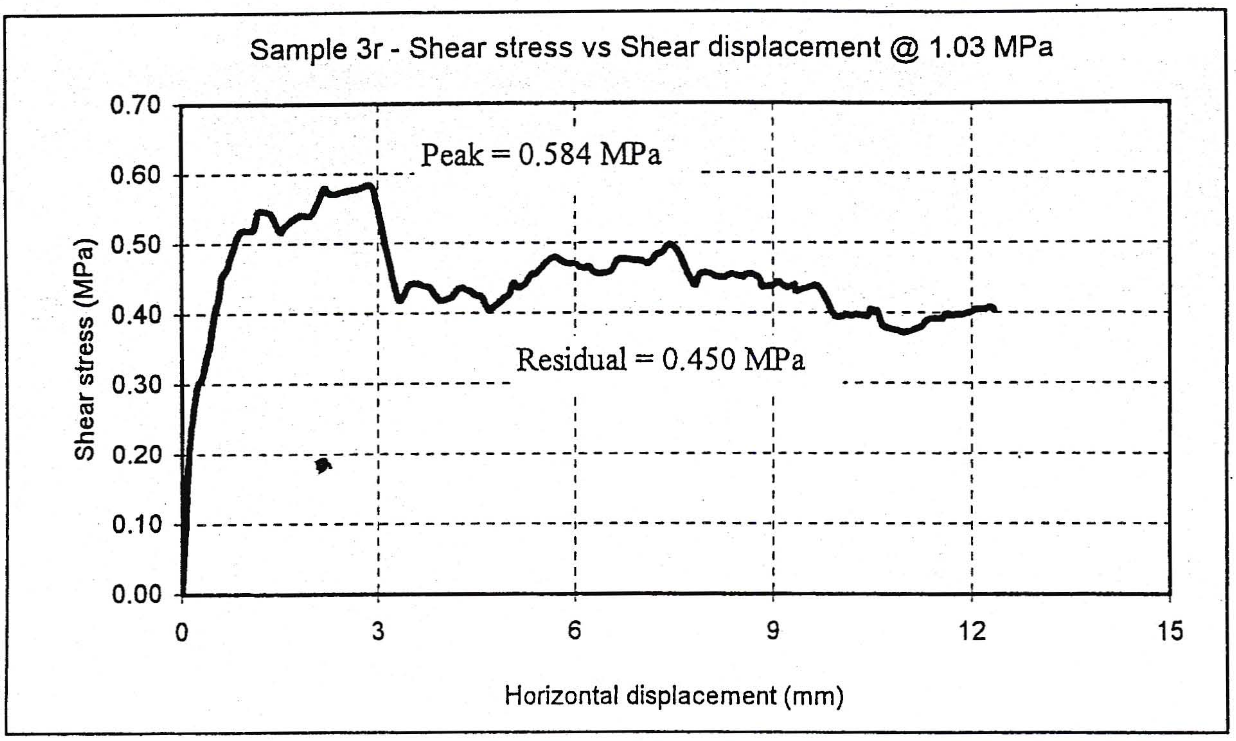
Sample No.	Borehole	Depth (m)	Weathering grade	Hardness	Aperture	Roughness	JRC value	Filling		Dip
								Type	Nature	
3	BH1	14.5	1	R5	T	SU	5	F	2	87
4	BH1	17.1	1	R6	T	RU	19		2/3	41
16	BH2	8.5	2	R5/R4	T	RP	7			40
15a	BH2	7.49	2	R5	T	RU	9	F	2	41
15b	BH2	7.67	2	R5	T	SU	7	F	2	30
15c	BH2	7.72	2	R5		SU	7	F	2	23
15d	BH2	7.78	2	R5		RP	5	F	2	32
13	BH2	7.25	2	R5		RP	7	F	2	73
14	BH2	7.38	2	R5		RU	9	F	2	12
5a	BH4	6.91	1	R6	T	SU	9	F	2	16
10b	BH4	16.97	2	R5	PO	SU	7	F	2	36
7	BH4	4.86	2	R5	T	RP	7	F	2	82
8	BH4	5.41	2	R5	PO	RU	9	F	2	16

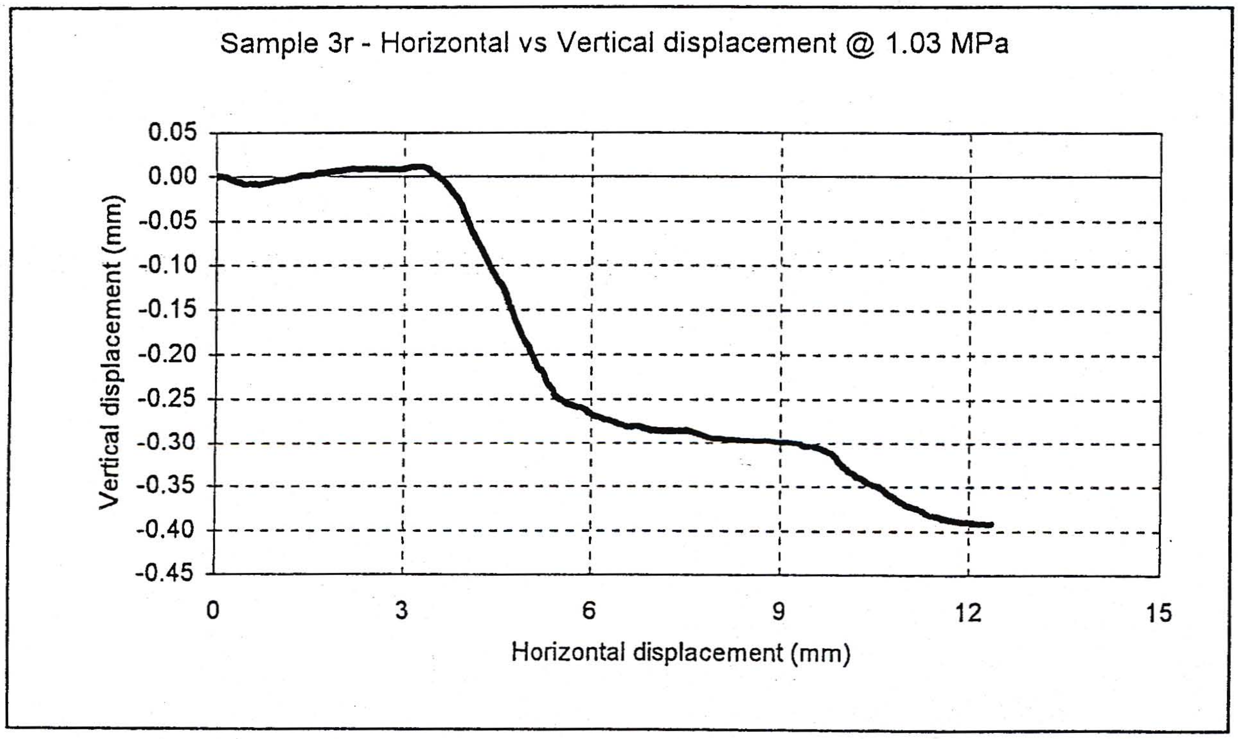
Appendix E

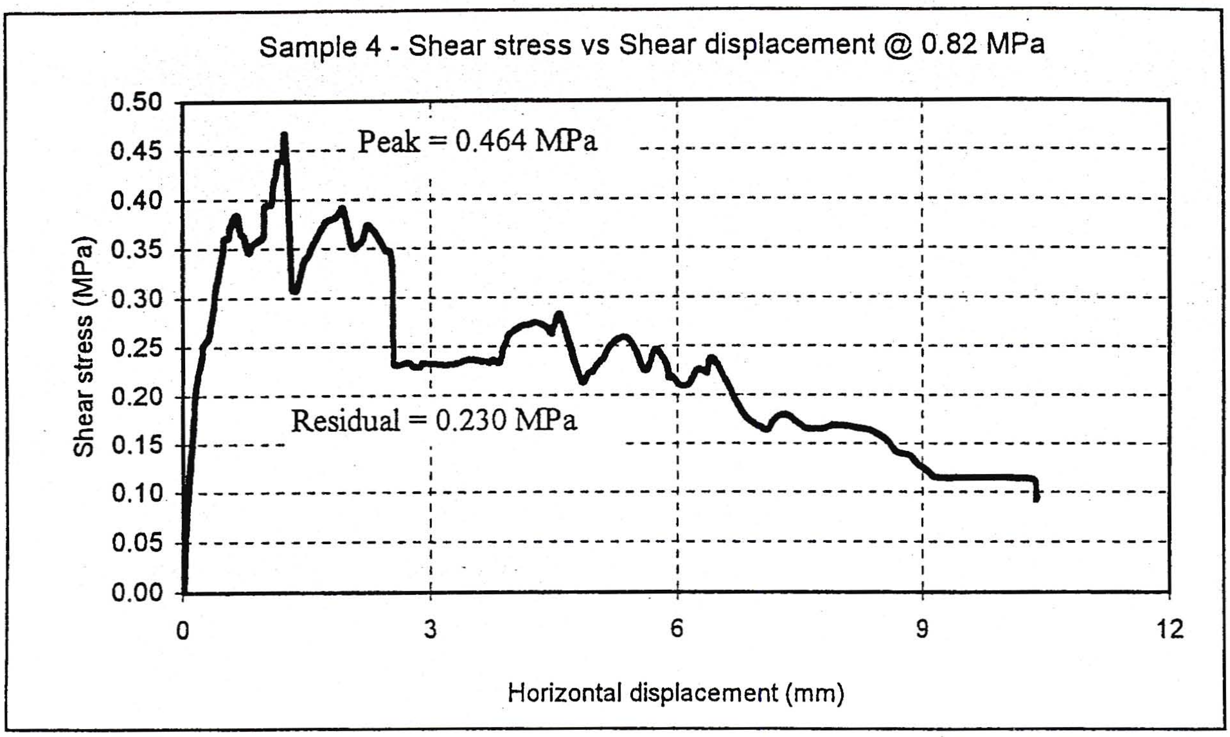
Plots of shear test results

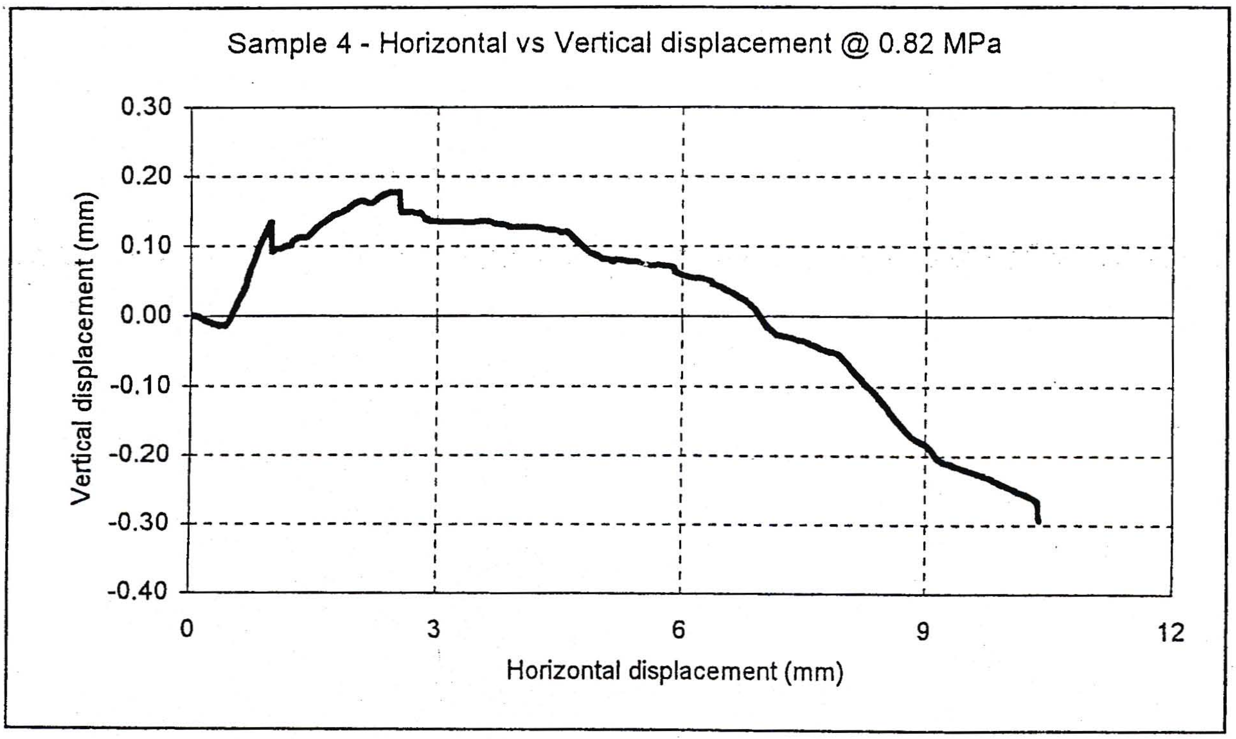

Sample 3 - Shear stress vs Shear displacement @ 1.03 MPa
Peak = 0.636 MPa
Residual = 0.400 MPa
Shear stress (MPa)
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
3
6
9
12
15
Horizontal displacement (mm)

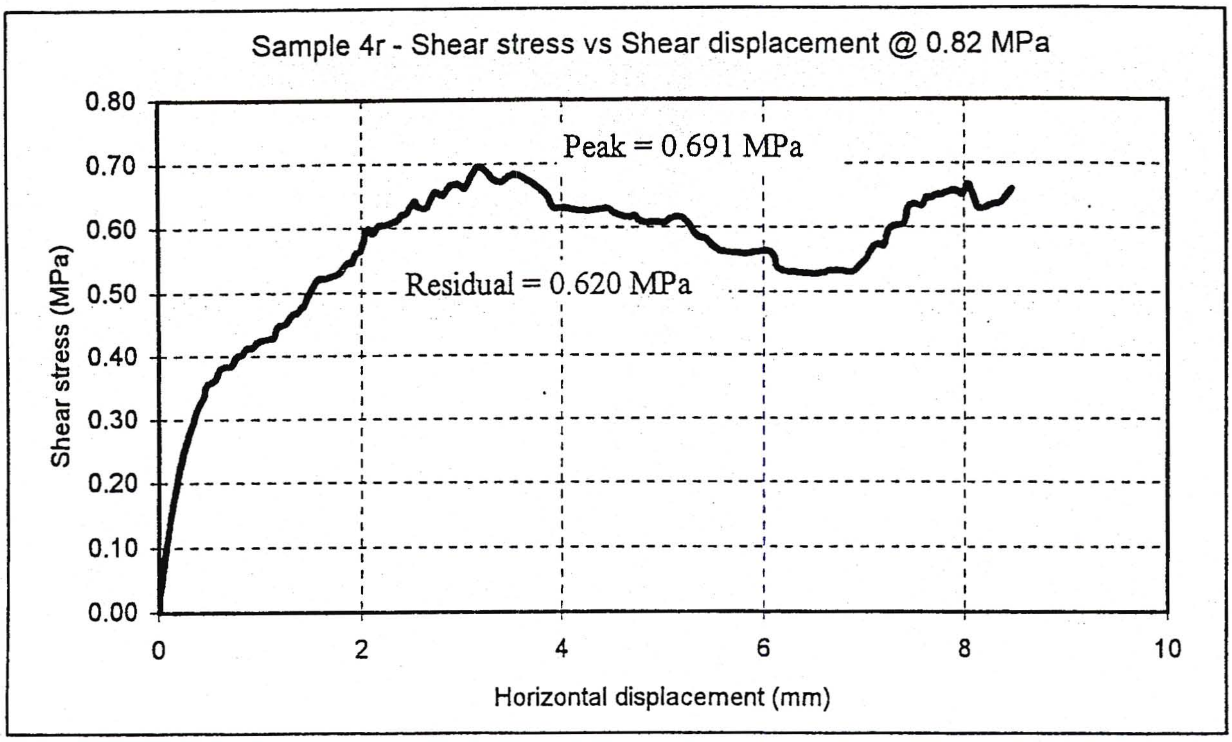

Sample 3 - Horizontal vs Vertical displacement @ 1.03 MPa
Vertical displacement (mm)
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
0
3
6
9
12
15
Horizontal displacement (mm)

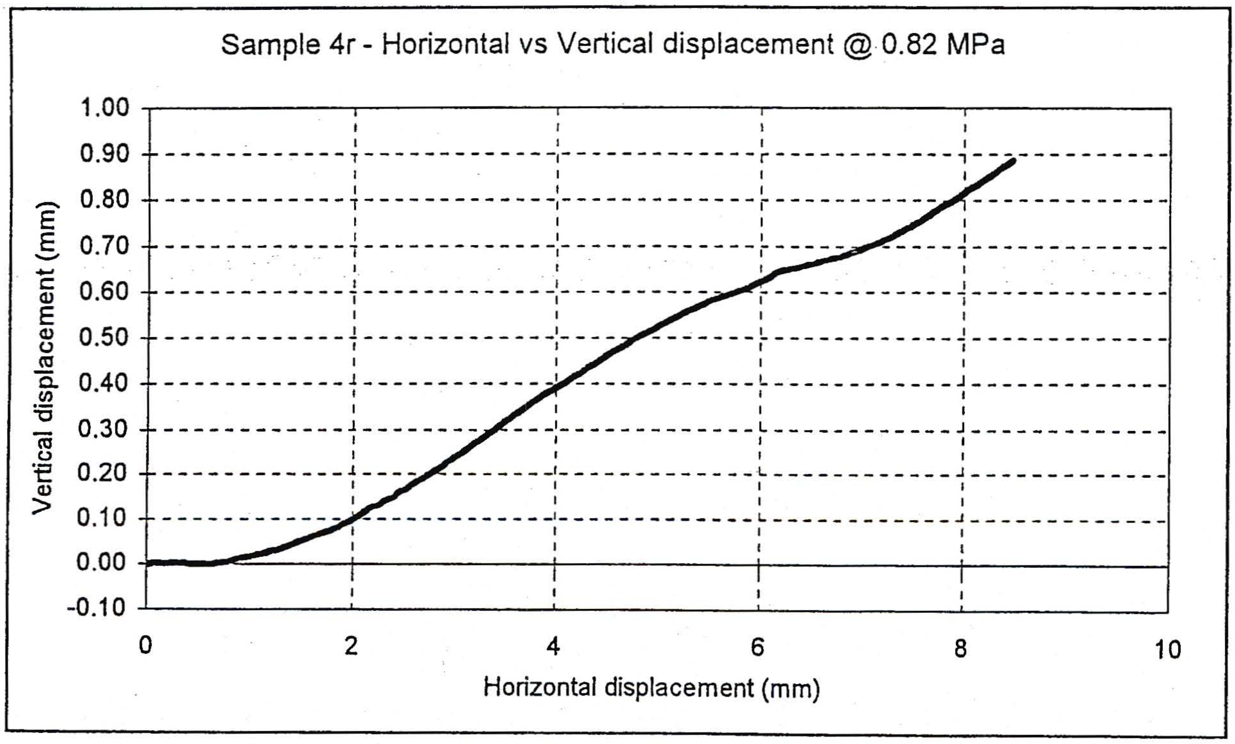

Sample 3r - Shear stress vs Shear displacement @ 1.03 MPa
Peak = 0.584 MPa
Residual = 0.450 MPa
Shear stress (MPa)
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
3
6
9
12
15
Horizontal displacement (mm)

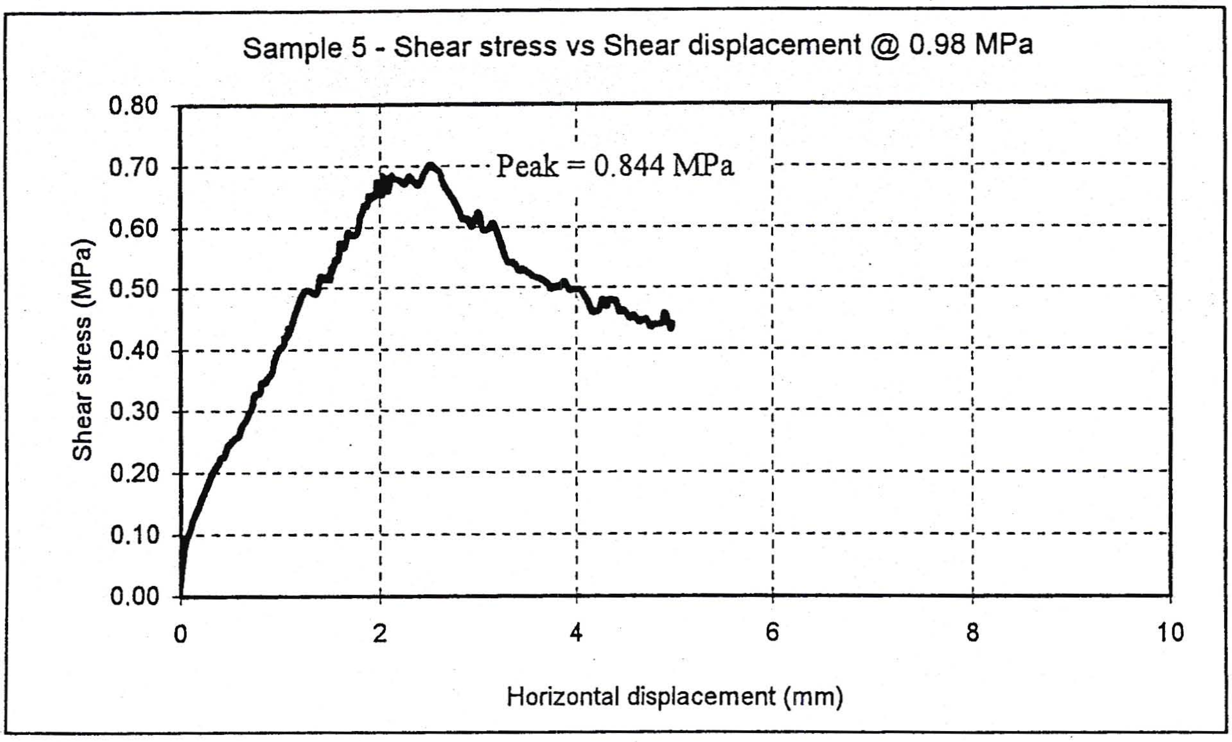

Sample 3r - Horizontal vs Vertical displacement @ 1.03 MPa
Vertical displacement (mm)
0.05
0.00
-0.05
-0.10
-0.15
-0.20
-0.25
-0.30
-0.35
-0.40
-0.45
0
3
6
9
12
15
Horizontal displacement (mm)


Sample 4 - Shear stress vs Shear displacement @ 0.82 MPa
Peak = 0.464 MPa
Residual = 0.230 MPa
Shear stress (MPa)
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
3
6
9
12
Horizontal displacement (mm)

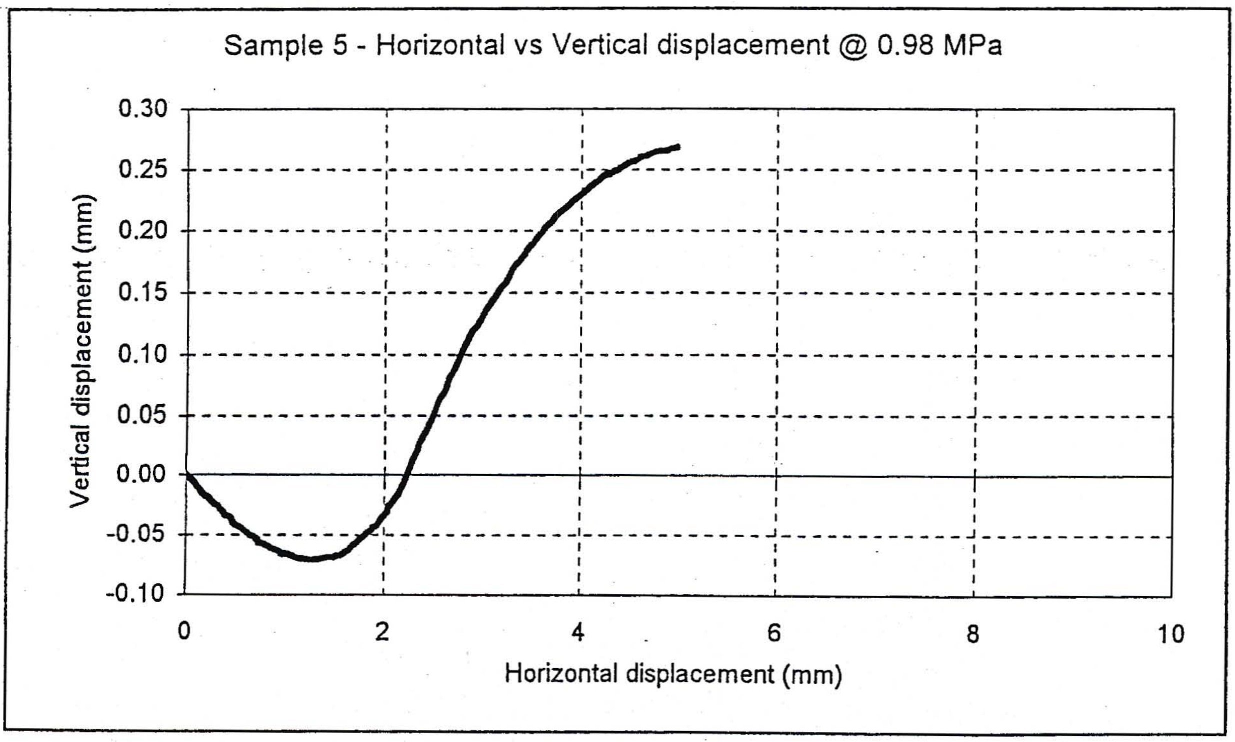

Sample 4 - Horizontal vs Vertical displacement @ 0.82 MPa
Vertical displacement (mm)
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
0
3
6
9
12
Horizontal displacement (mm)


Sample 4r - Shear stress vs Shear displacement @ 0.82 MPa
Peak = 0.691 MPa
Residual = 0.620 MPa
Shear stress (MPa)
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
2
4
6
8
10
Horizontal displacement (mm)

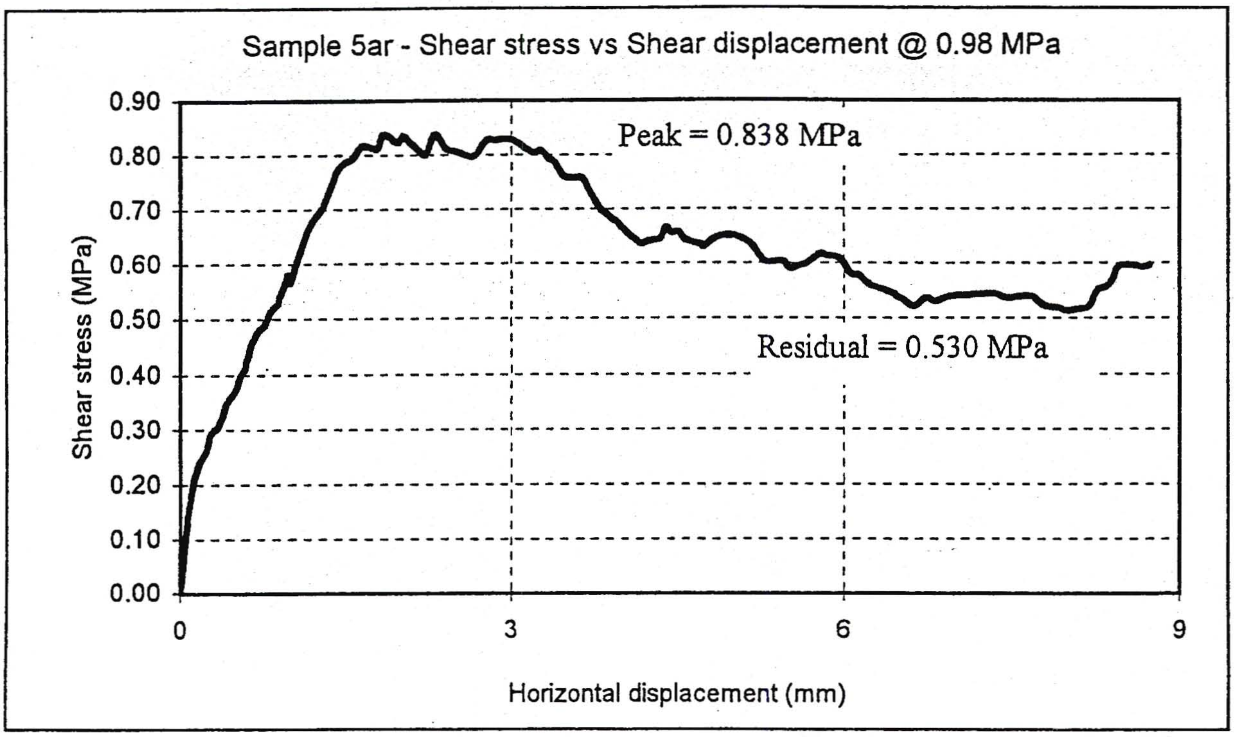

Sample 4r - Horizontal vs Vertical displacement @ 0.82 MPa
Vertical displacement (mm)
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
0
2
4
6
8
10
Horizontal displacement (mm)



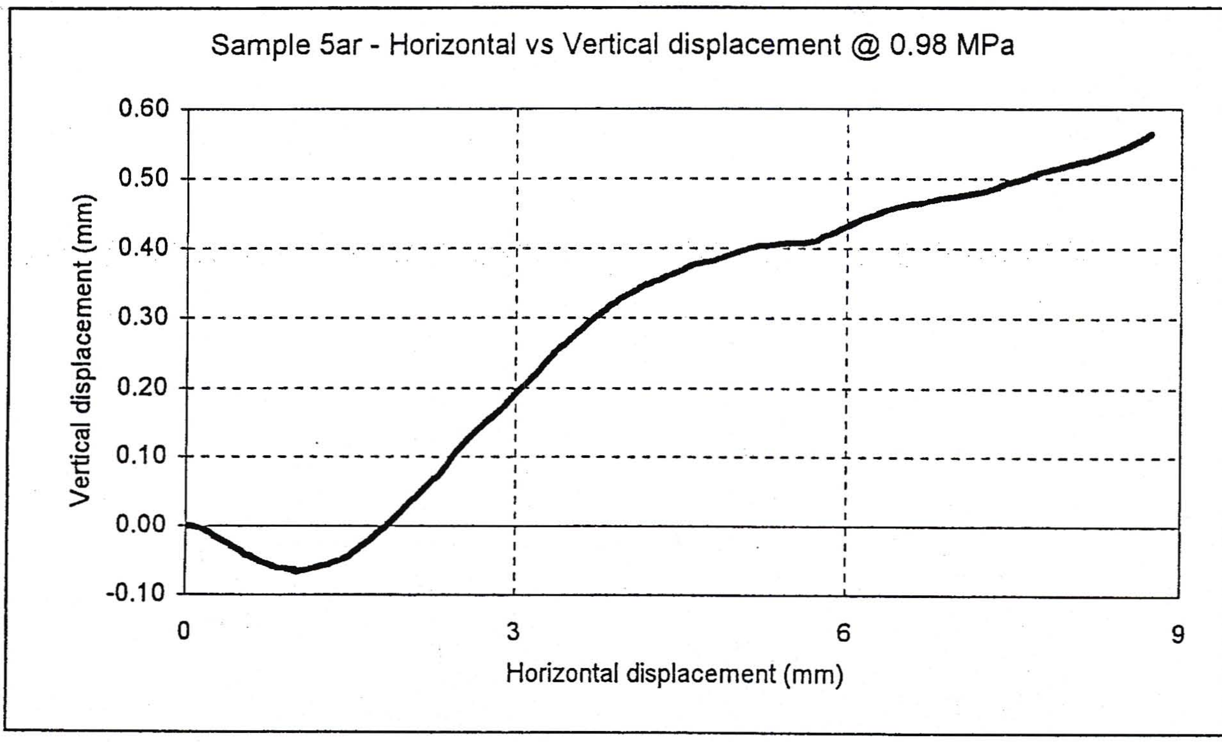
Sample 5 - Shear stress vs Shear displacement @ 0.98 MPa
Peak = 0.844 MPa
Shear stress (MPa)
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
2
4
6
8
10
Horizontal displacement (mm)



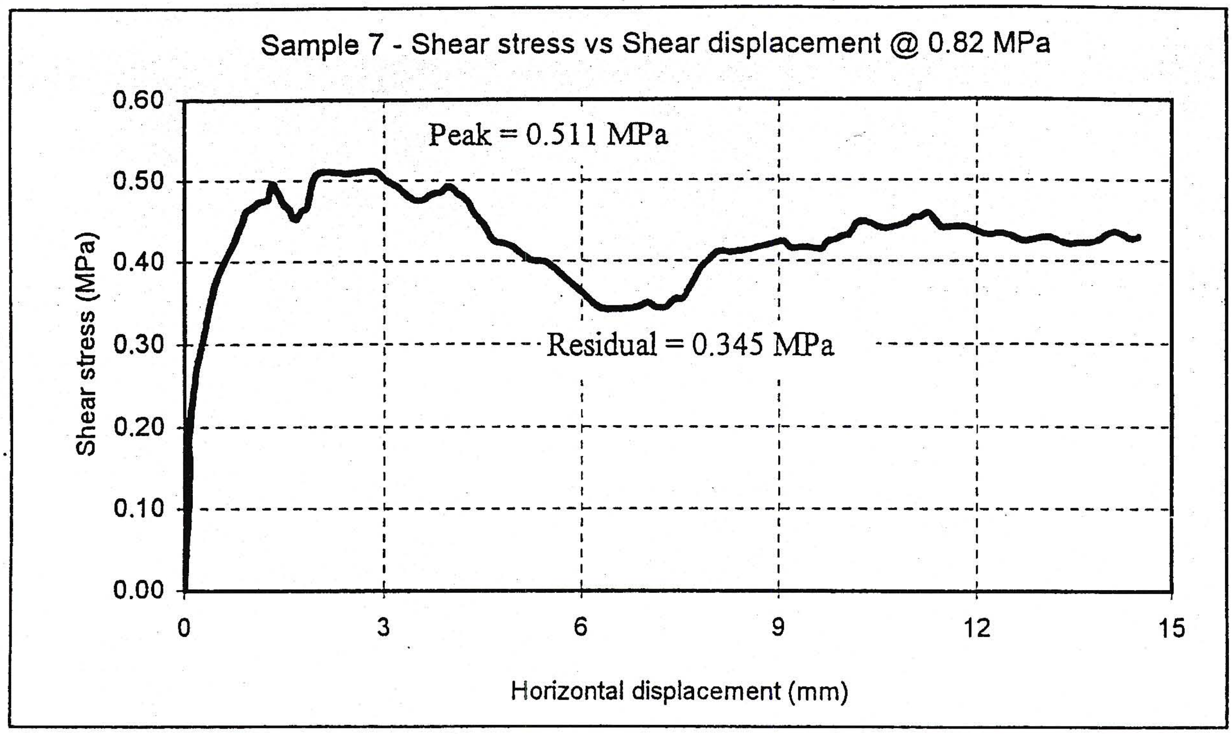
Sample 5 - Horizontal vs Vertical displacement @ 0.98 MPa
Vertical displacement (mm)
0.30
0.25
0.20
0.15
0.10
0.05
0.00
-0.05
-0.10
0
2
4
6
8
10
Horizontal displacement (mm)

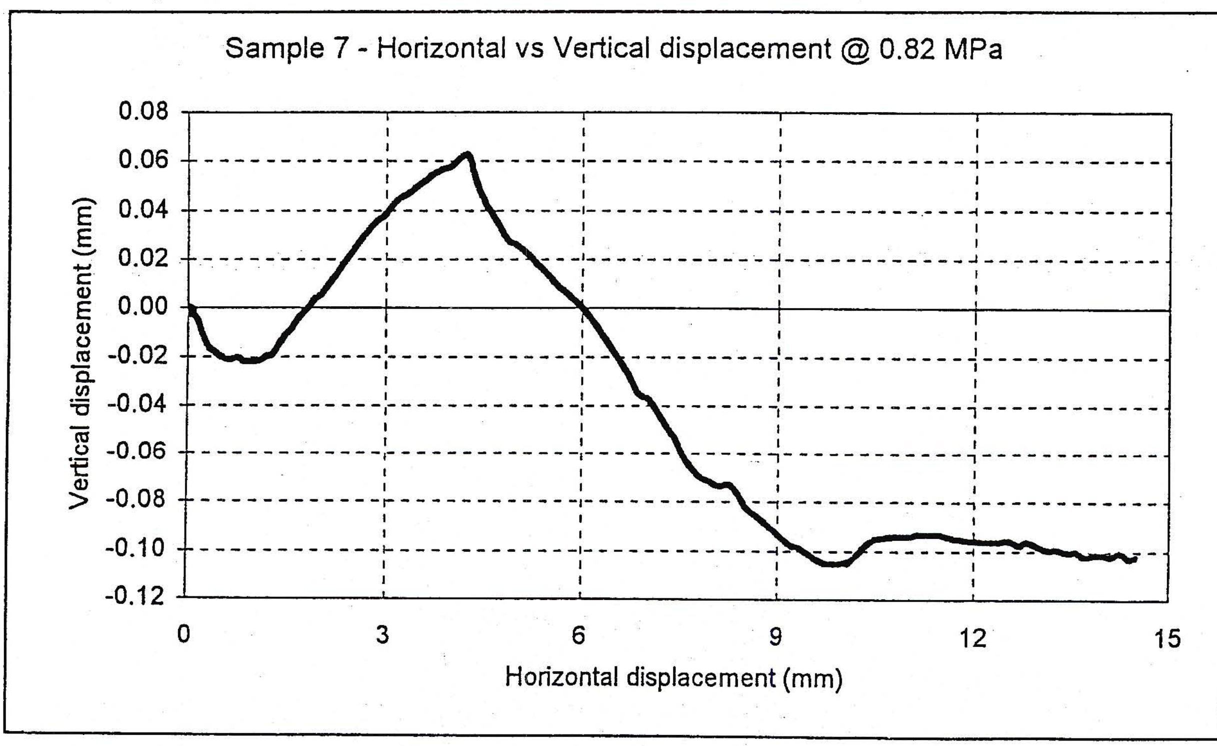


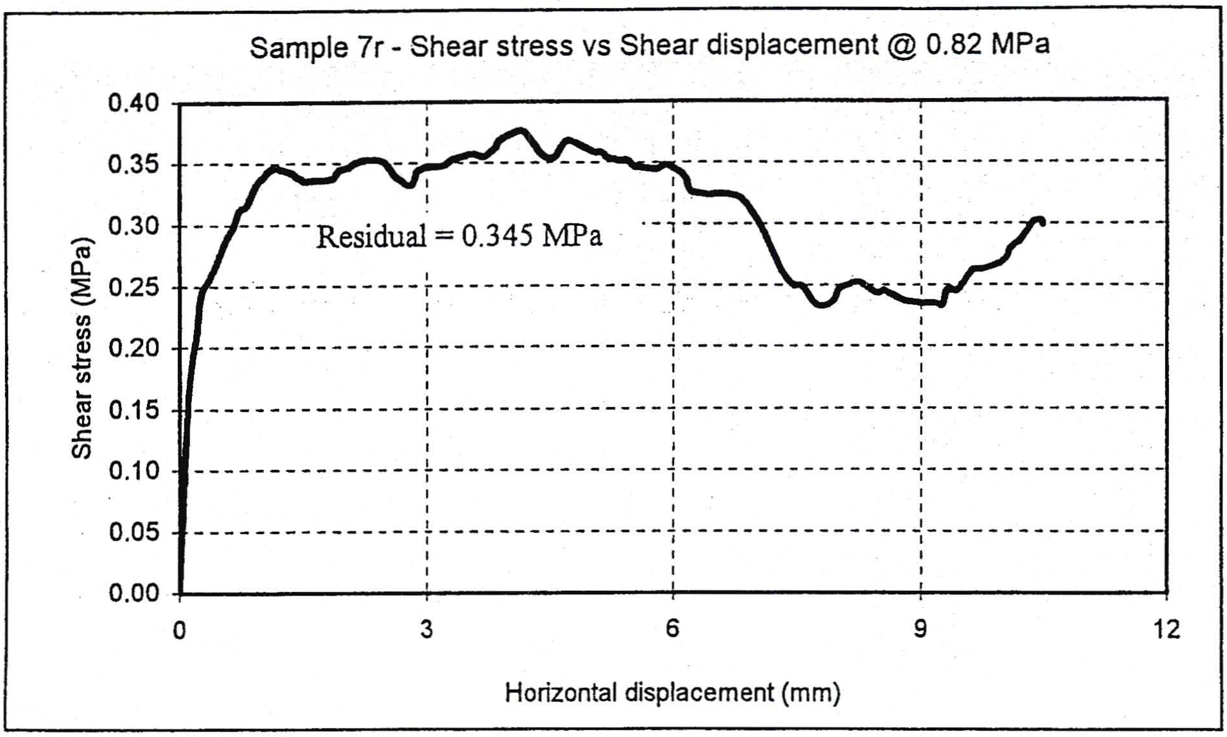
Sample 5ar - Shear stress vs Shear displacement @ 0.98 MPa
Peak = 0.838 MPa
Residual = 0.530 MPa
Shear stress (MPa)
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
3
6
9
Horizontal displacement (mm)

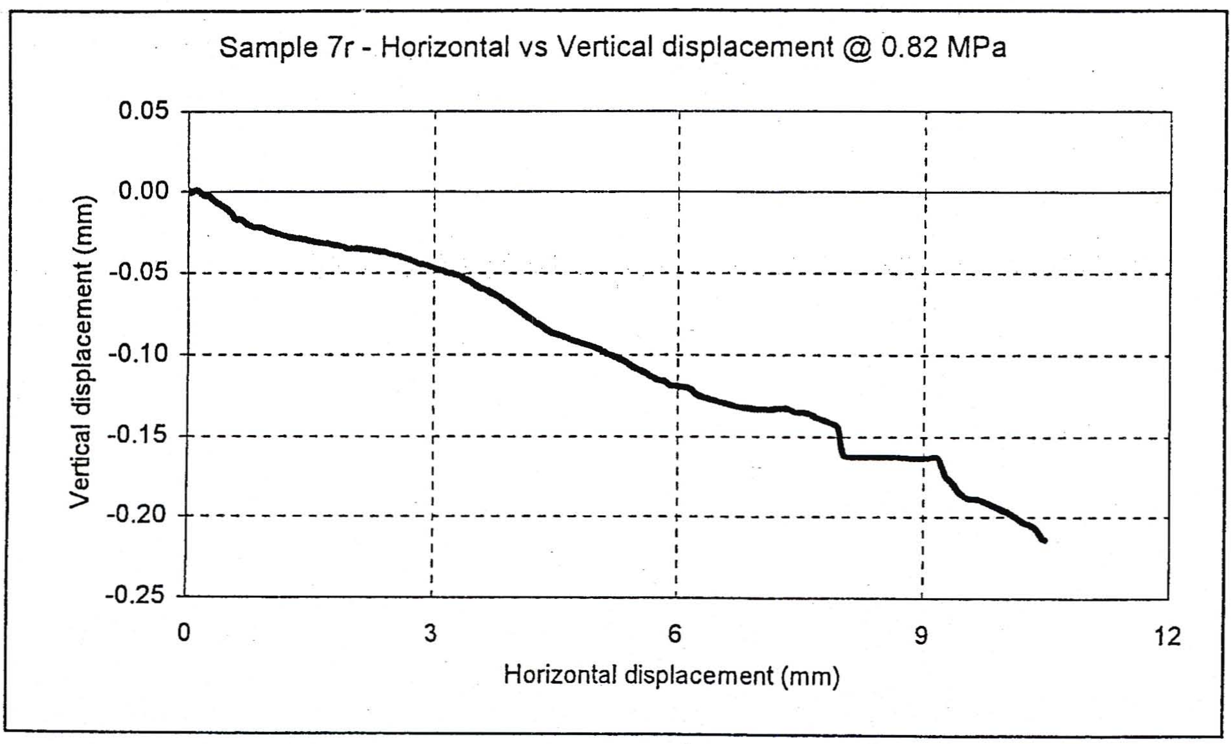


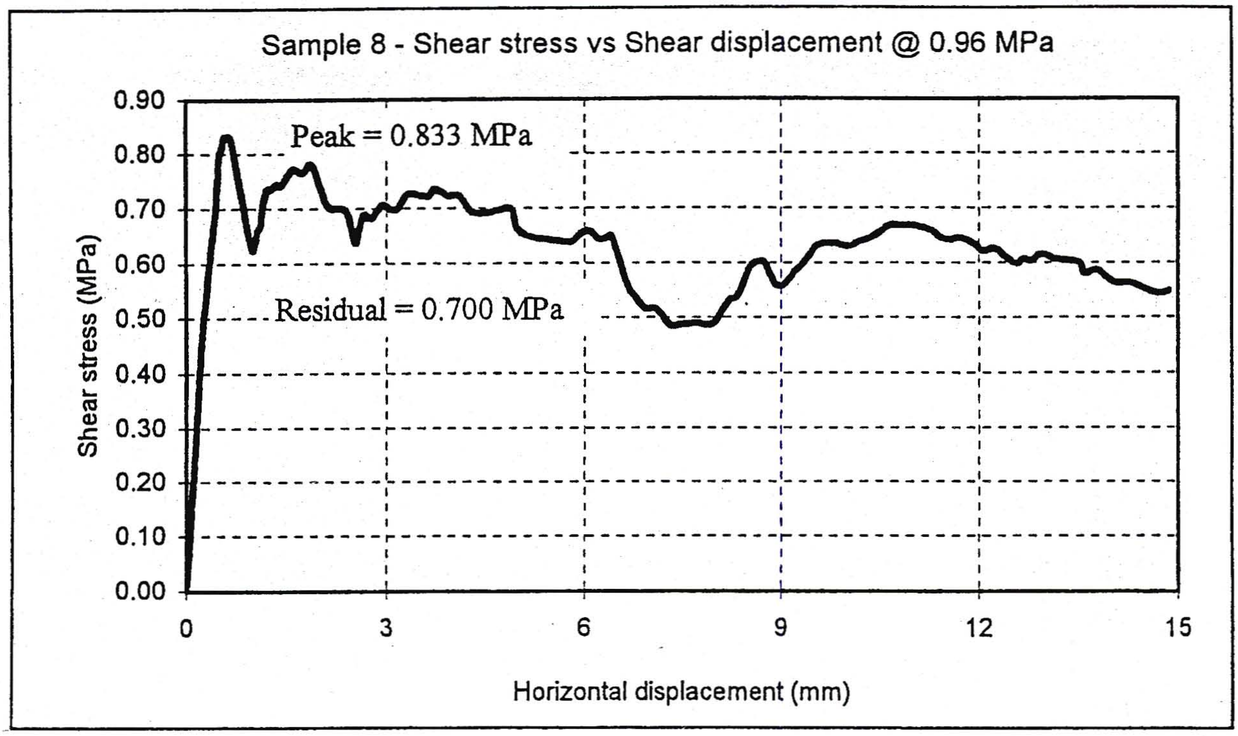
Sample 5ar - Horizontal vs Vertical displacement @ 0.98 MPa
Vertical displacement (mm)
0.60
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
0
3
6
9
Horizontal displacement (mm)


Sample 7 - Shear stress vs Shear displacement @ 0.82 MPa
Peak = 0.511 MPa
Residual = 0.345 MPa
Shear stress (MPa)
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
3
6
9
12
15
Horizontal displacement (mm)

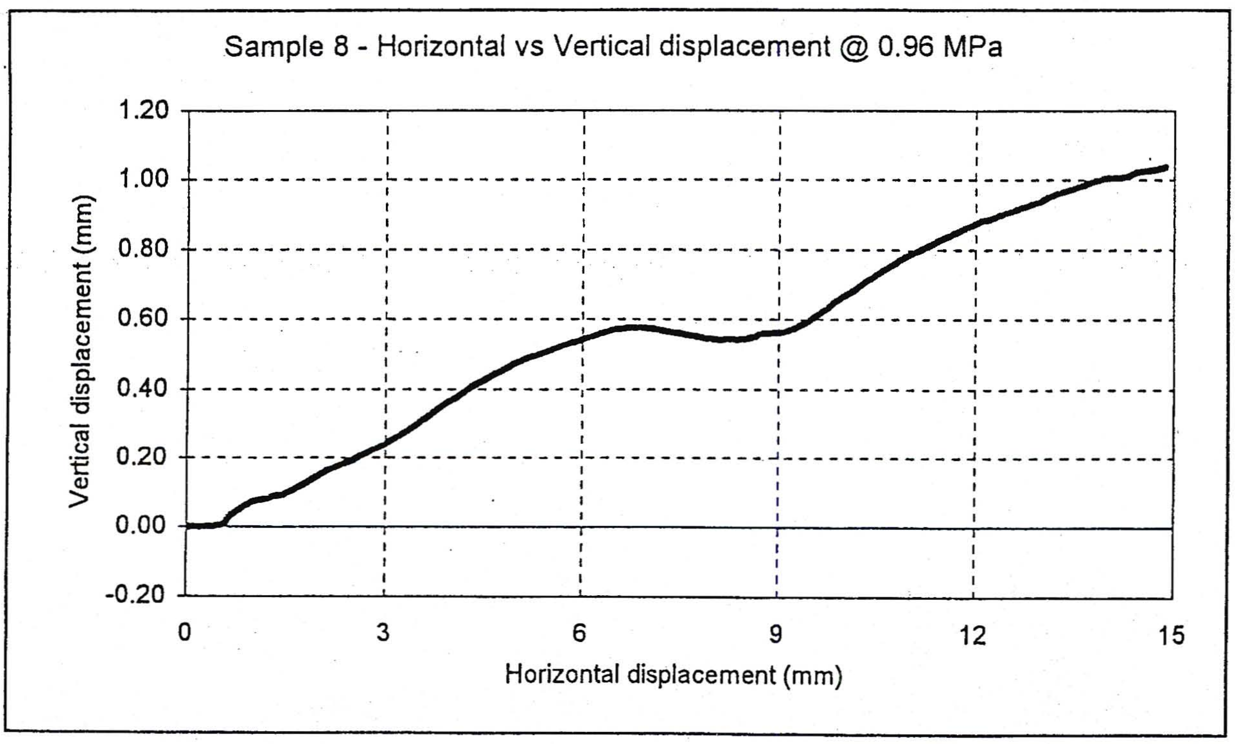

Sample 7 - Horizontal vs Vertical displacement @ 0.82 MPa
Vertical displacement (mm)
0.08
0.06
0.04
0.02
0.00
-0.02
-0.04
-0.06
-0.08
-0.10
-0.12
0
3
6
9
12
15
Horizontal displacement (mm)


Sample 7r - Shear stress vs Shear displacement @ 0.82 MPa
Residual = 0.345 MPa
Shear stress (MPa)
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
3
6
9
12
Horizontal displacement (mm)

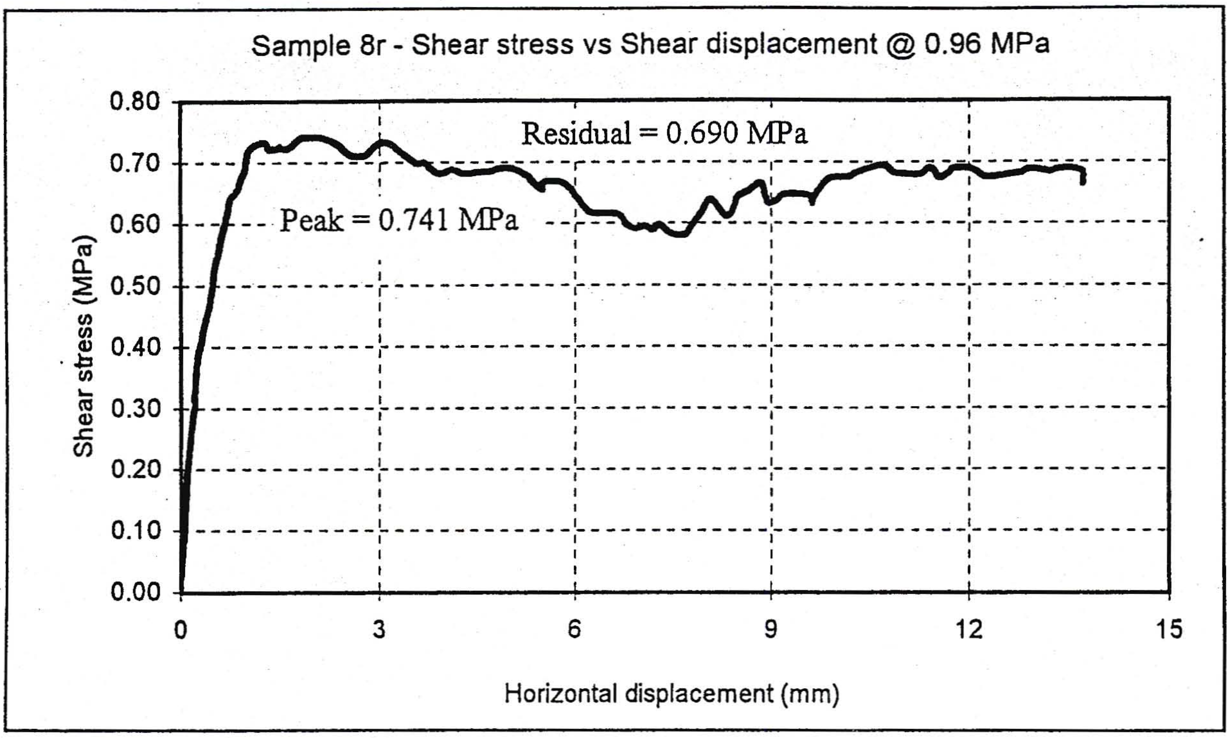

Sample 7r - Horizontal vs Vertical displacement @ 0.82 MPa
Vertical displacement (mm)
0.05
0.00
-0.05
-0.10
-0.15
-0.20
-0.25
0
3
6
9
12
Horizontal displacement (mm)

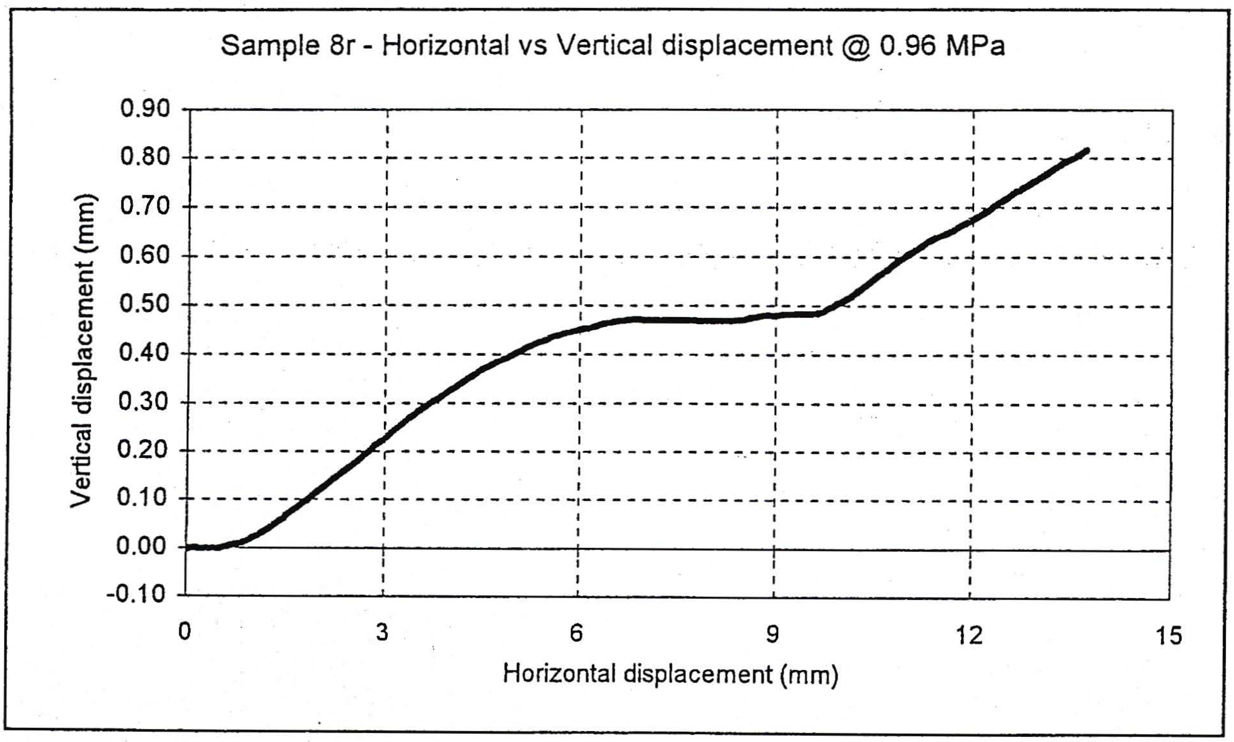


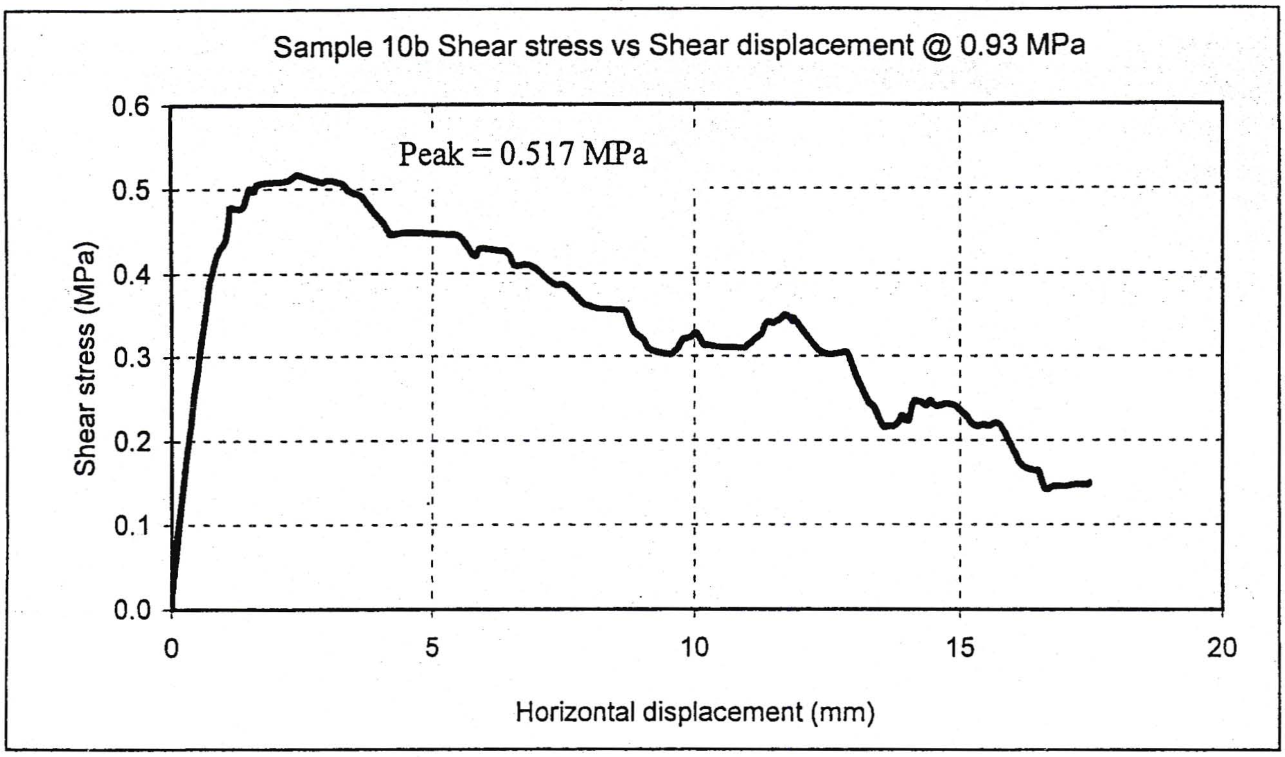
Sample 8 - Shear stress vs Shear displacement @ 0.96 MPa
Peak = 0.833 MPa
Residual = 0.700 MPa
Shear stress (MPa)
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
3
6
9
12
15
Horizontal displacement (mm)

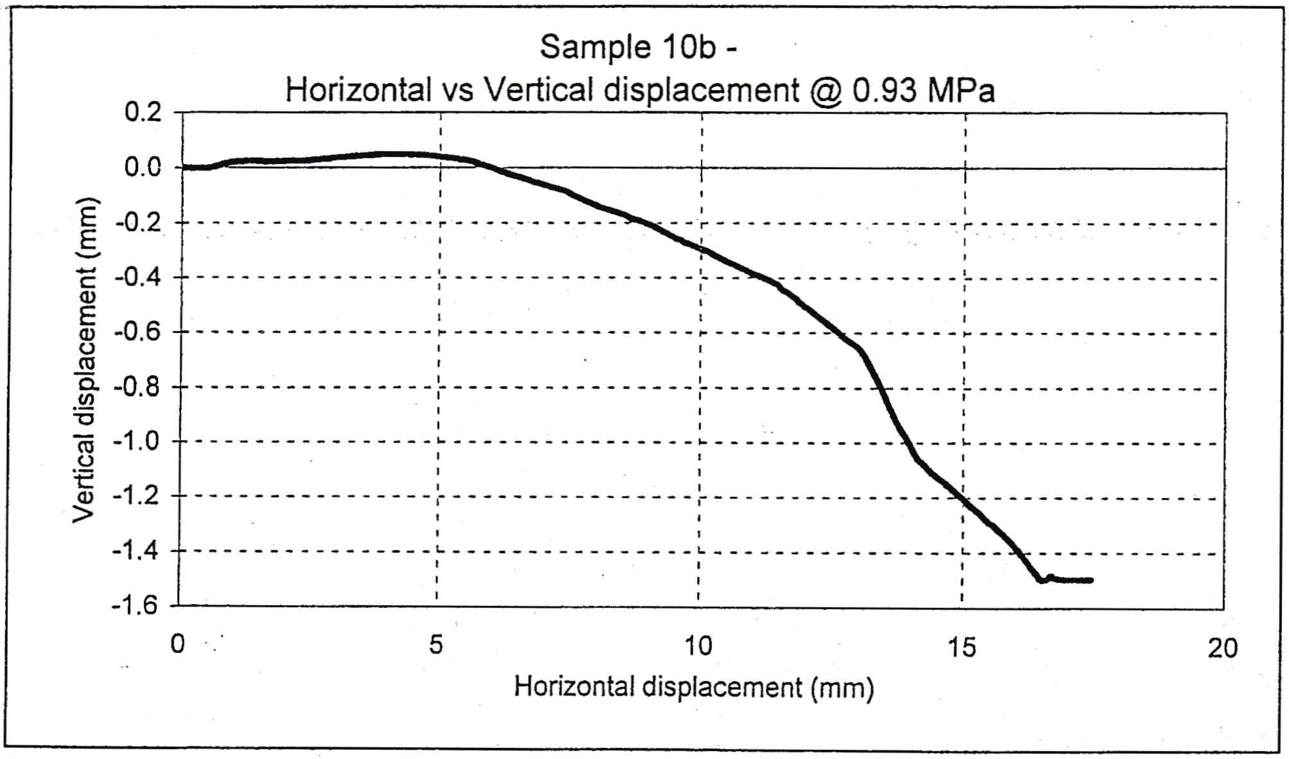


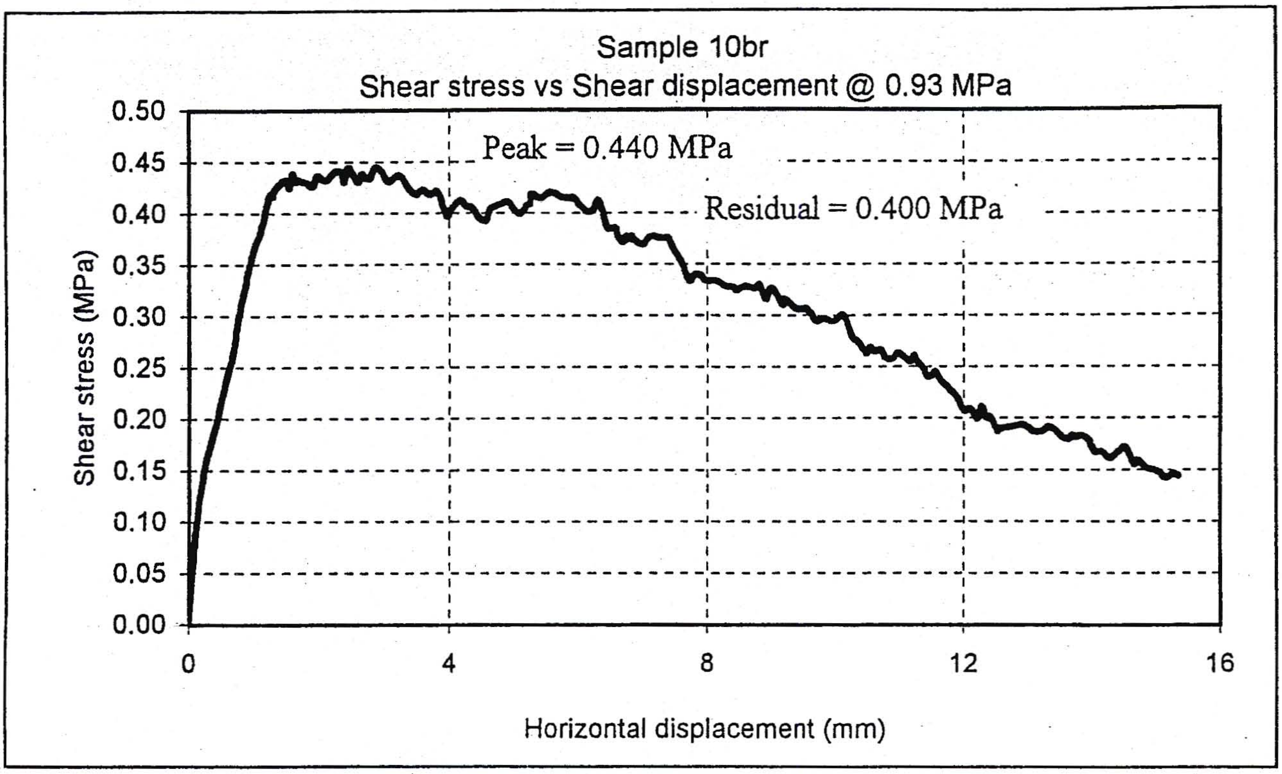
Sample 8 - Horizontal vs Vertical displacement @ 0.96 MPa
Vertical displacement (mm)
1.20
1.00
0.80
0.60
0.40
0.20
0.00
-0.20
0
3
6
9
12
15
Horizontal displacement (mm)

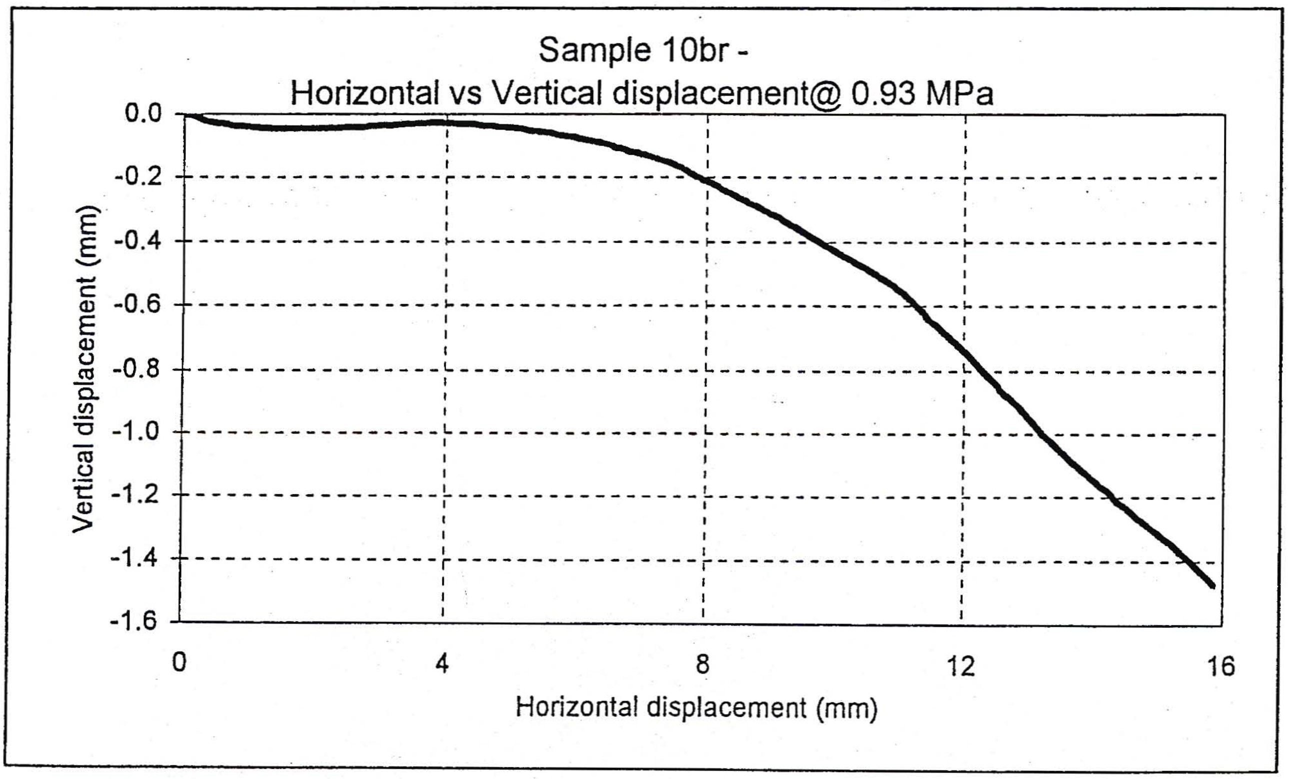

Sample 8r - Shear stress vs Shear displacement @ 0.96 MPa
Residual = 0.690 MPa
Peak = 0.741 MPa
Shear stress (MPa)
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
3
6
9
12
15
Horizontal displacement (mm)

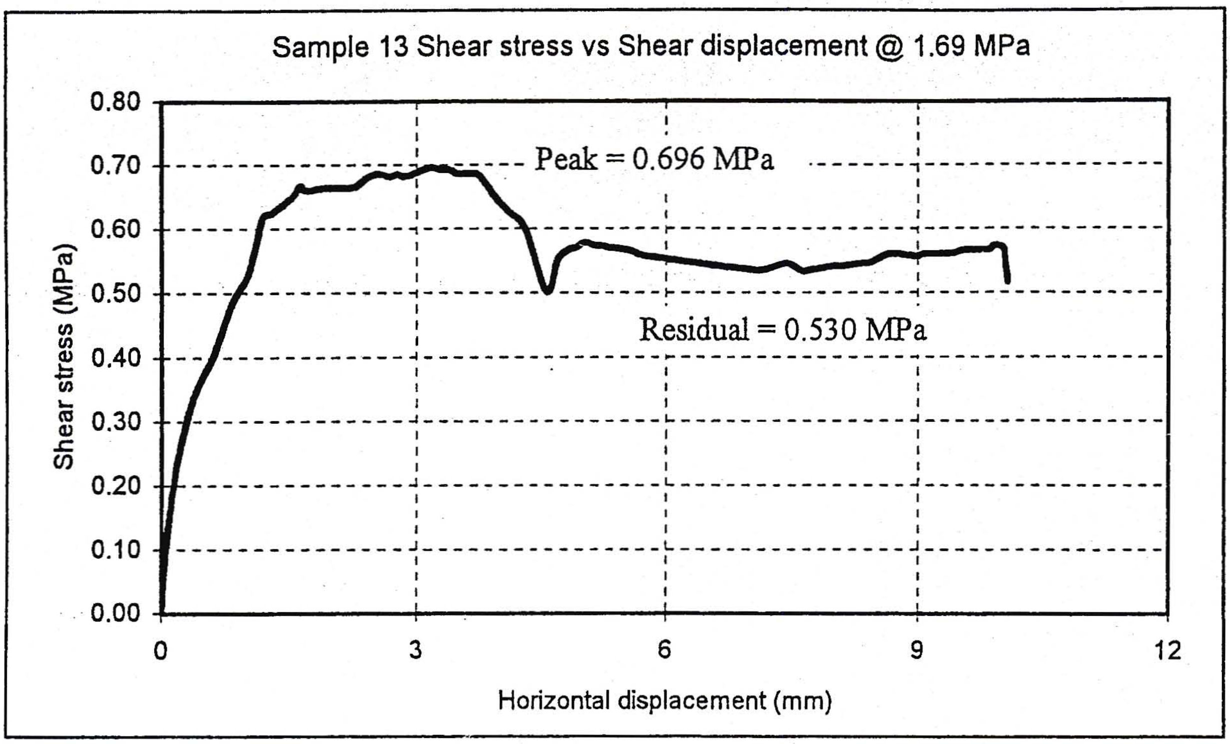

Sample 8r - Horizontal vs Vertical displacement @ 0.96 MPa
Vertical displacement (mm)
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
0
3
6
9
12
15
Horizontal displacement (mm)

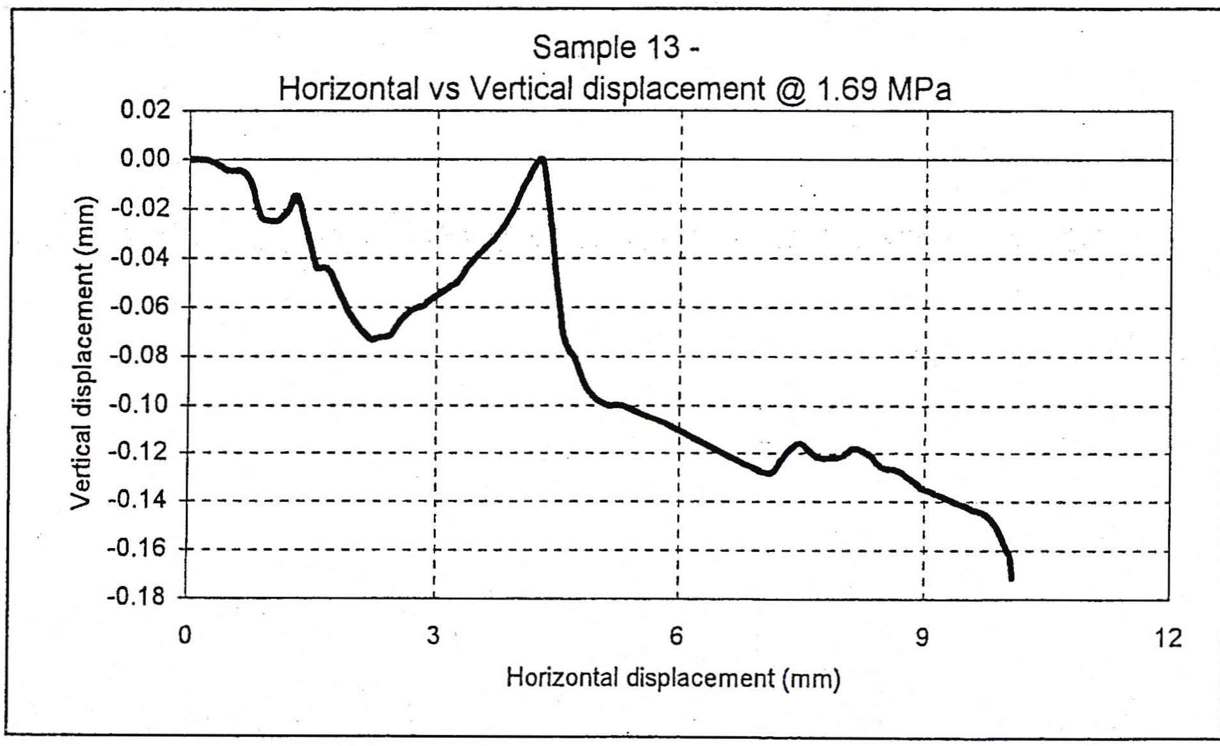

Sample 10b Shear stress vs Shear displacement @ 0.93 MPa
Peak = 0.517 MPa
Shear stress (MPa)
0.6
0.5
0.4
0.3
0.2
0.1
0.0
0
5
10
15
20
Horizontal displacement (mm)

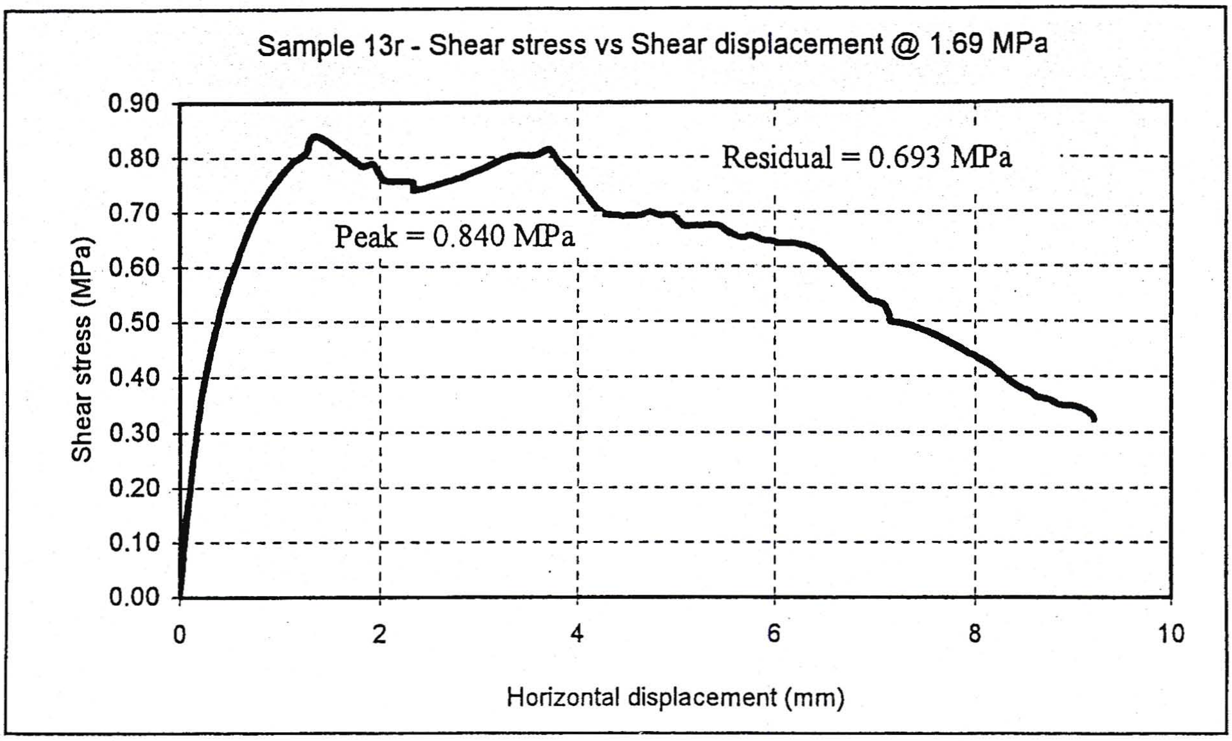

Sample 10b -
Horizontal vs Vertical displacement @ 0.93 MPa
Vertical displacement (mm)
0.2
0.0
-0.2
-0.4
-0.6
-0.8
-1.0
-1.2
-1.4
-1.6
0
5
10
15
20
Horizontal displacement (mm)

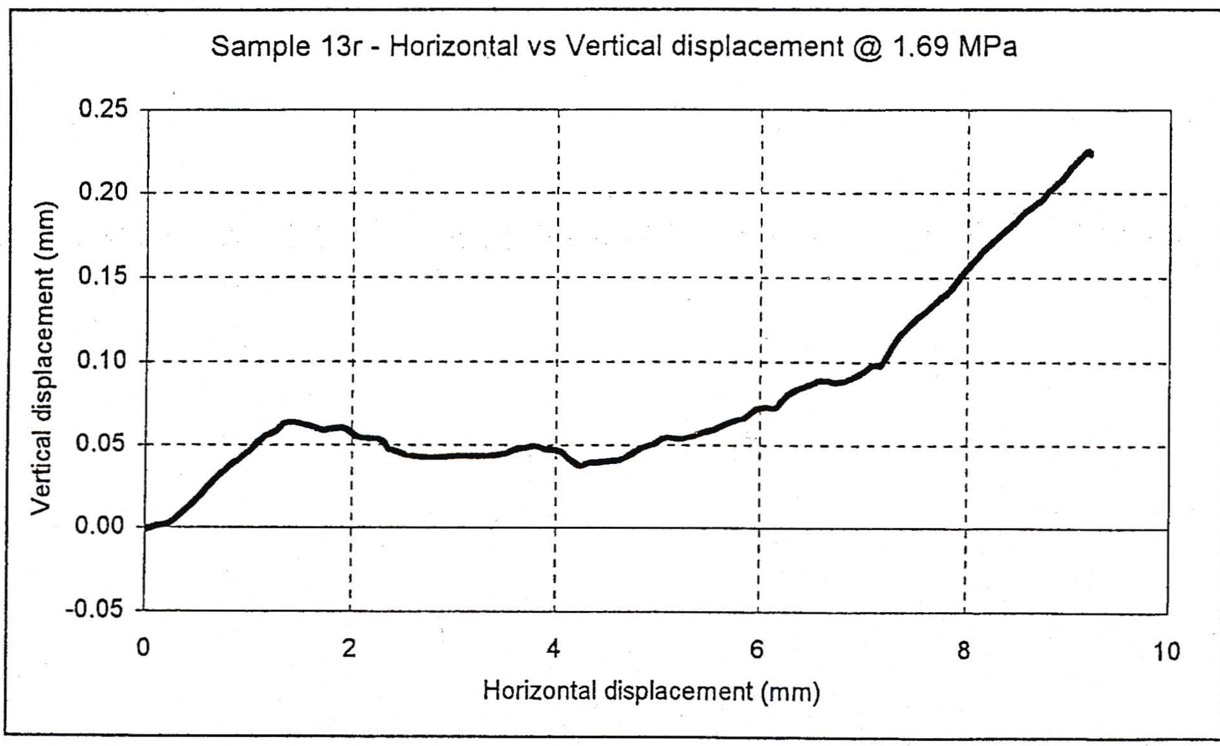

Sample 10br
Shear stress vs Shear displacement @ 0.93 MPa
Peak = 0.440 MPa
Residual = 0.400 MPa
Shear stress (MPa)
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
4
8
12
16
Horizontal displacement (mm)

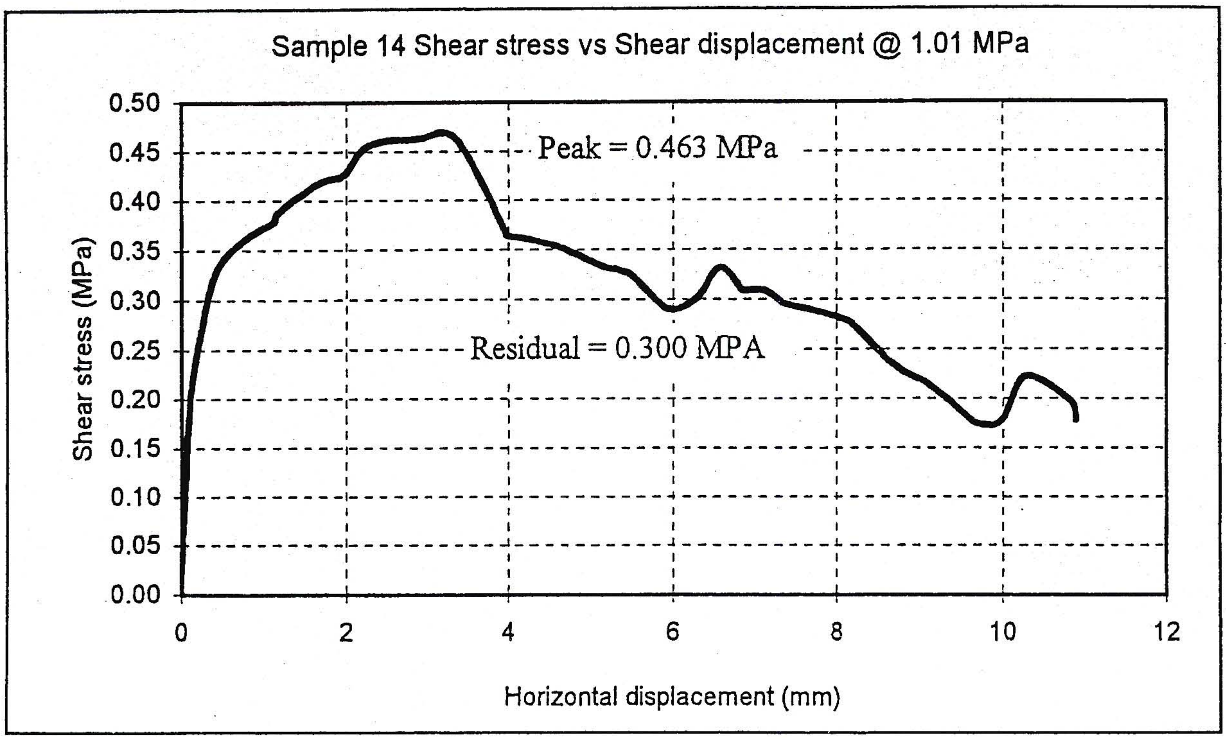

Sample 10br -
Horizontal vs Vertical displacement@ 0.93 MPa
Vertical displacement (mm)
0.0
-0.2
-0.4
-0.6
-0.8
-1.0
-1.2
-1.4
-1.6
0
4
8
12
16
Horizontal displacement (mm)

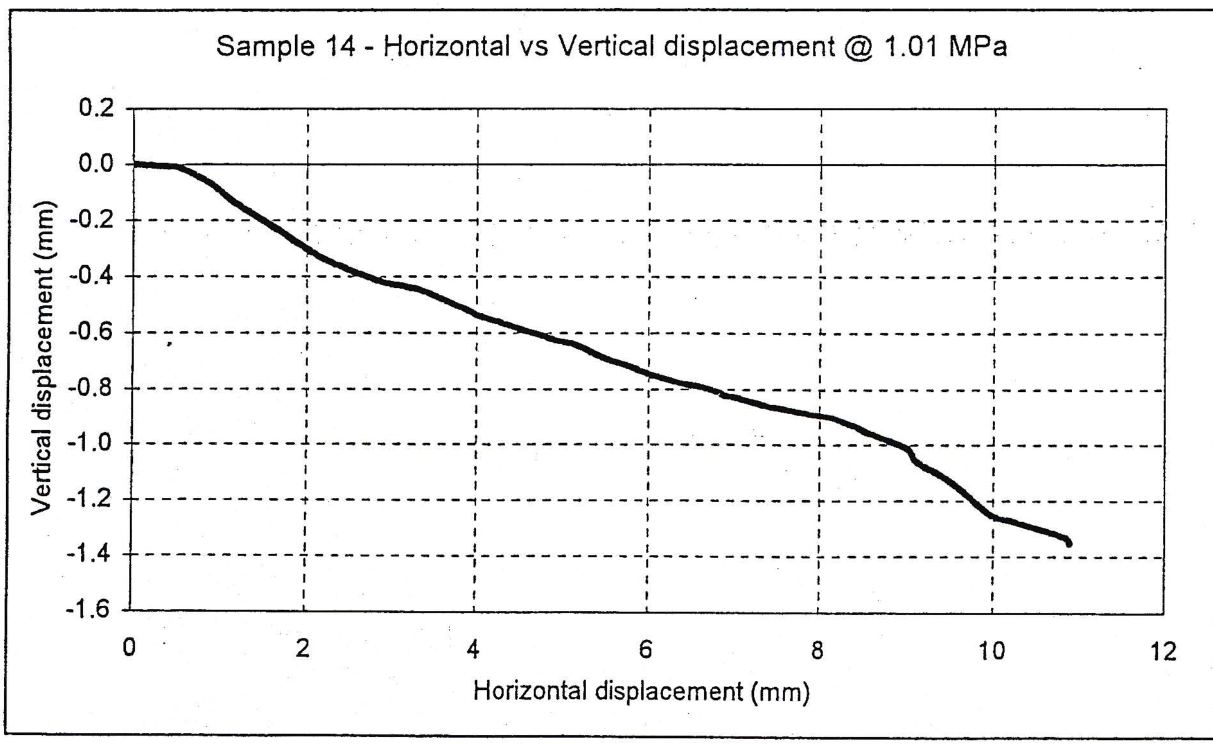

Sample 13 Shear stress vs Shear displacement @ 1.69 MPa
Peak = 0.696 MPa
Residual = 0.530 MPa
Shear stress (MPa)
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
3
6
9
12
Horizontal displacement (mm)

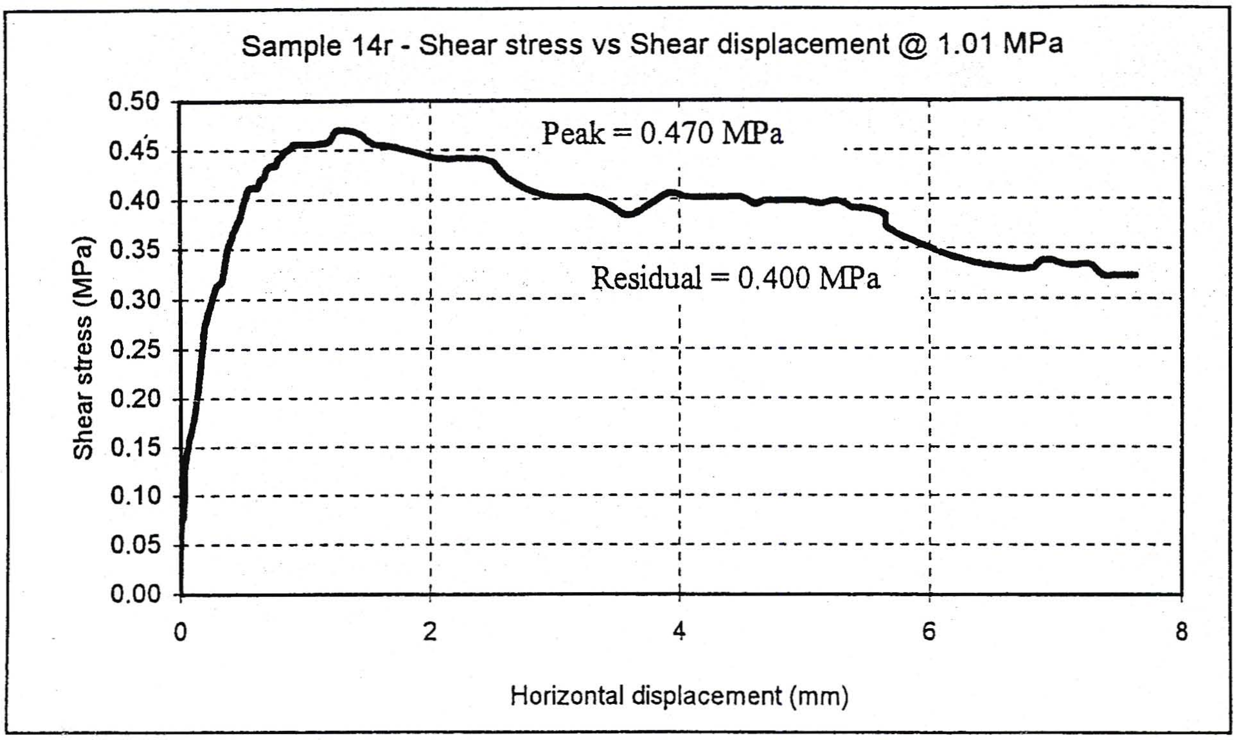

Sample 13 -
Horizontal vs Vertical displacement @ 1.69 MPa
Vertical displacement (mm)
0.02
0.00
-0.02
-0.04
-0.06
-0.08
-0.10
-0.12
-0.14
-0.16
-0.18
0
3
6
9
12
Horizontal displacement (mm)

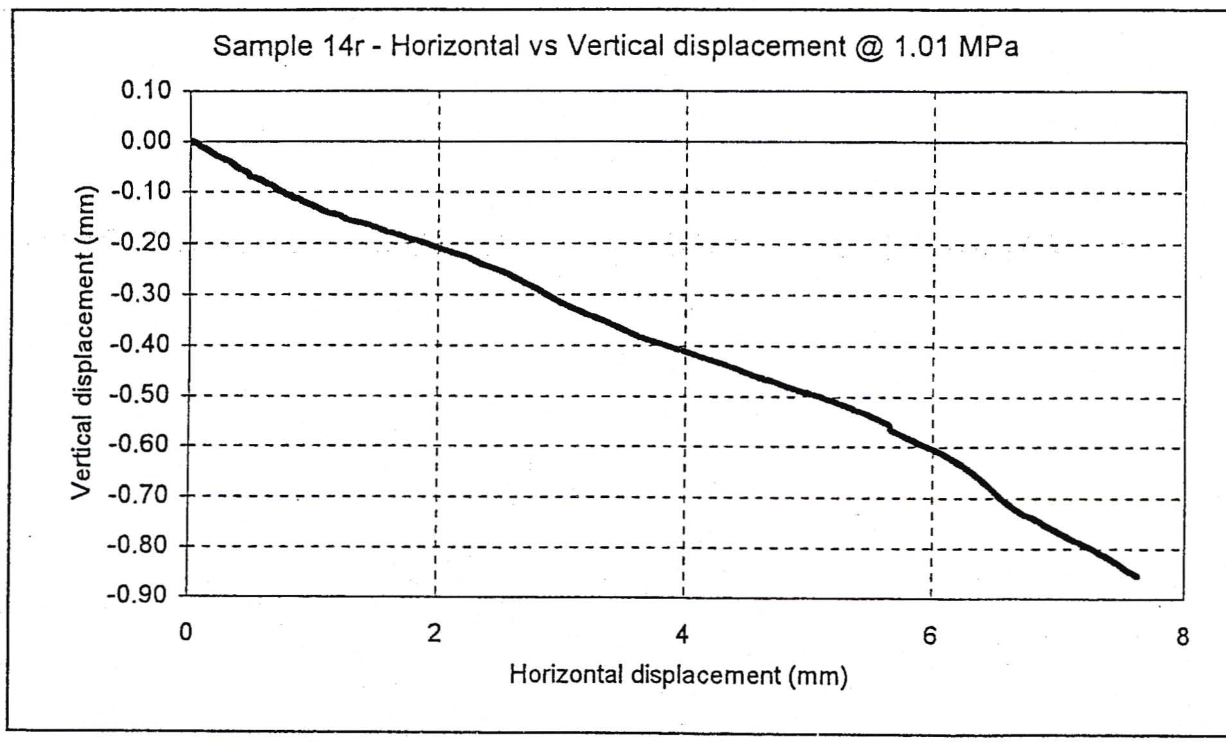

Sample 13r - Shear stress vs Shear displacement @ 1.69 MPa
Residual = 0.693 MPa
Peak = 0.840 MPa
Shear stress (MPa)
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
2
4
6
8
10
Horizontal displacement (mm)

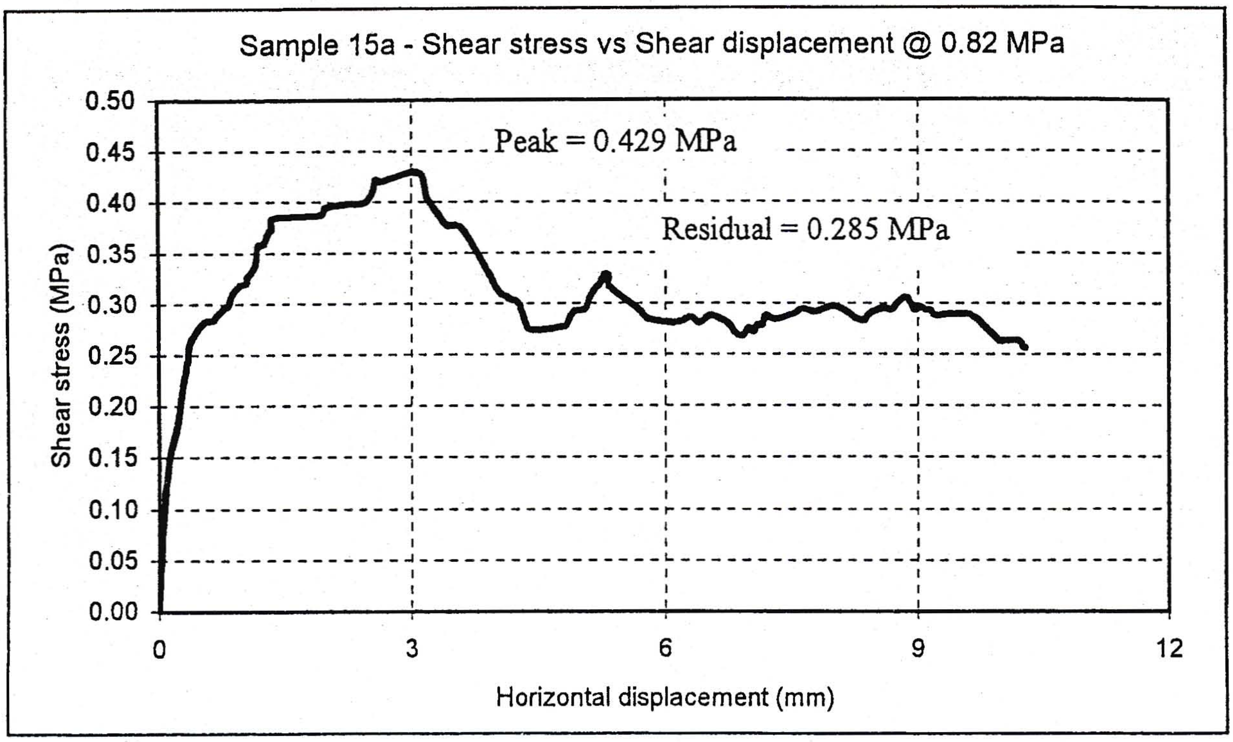

Sample 13r - Horizontal vs Vertical displacement @ 1.69 MPa
Vertical displacement (mm)
0.25
0.20
0.15
0.10
0.05
0.00
-0.05
0
2
4
6
8
10
Horizontal displacement (mm)

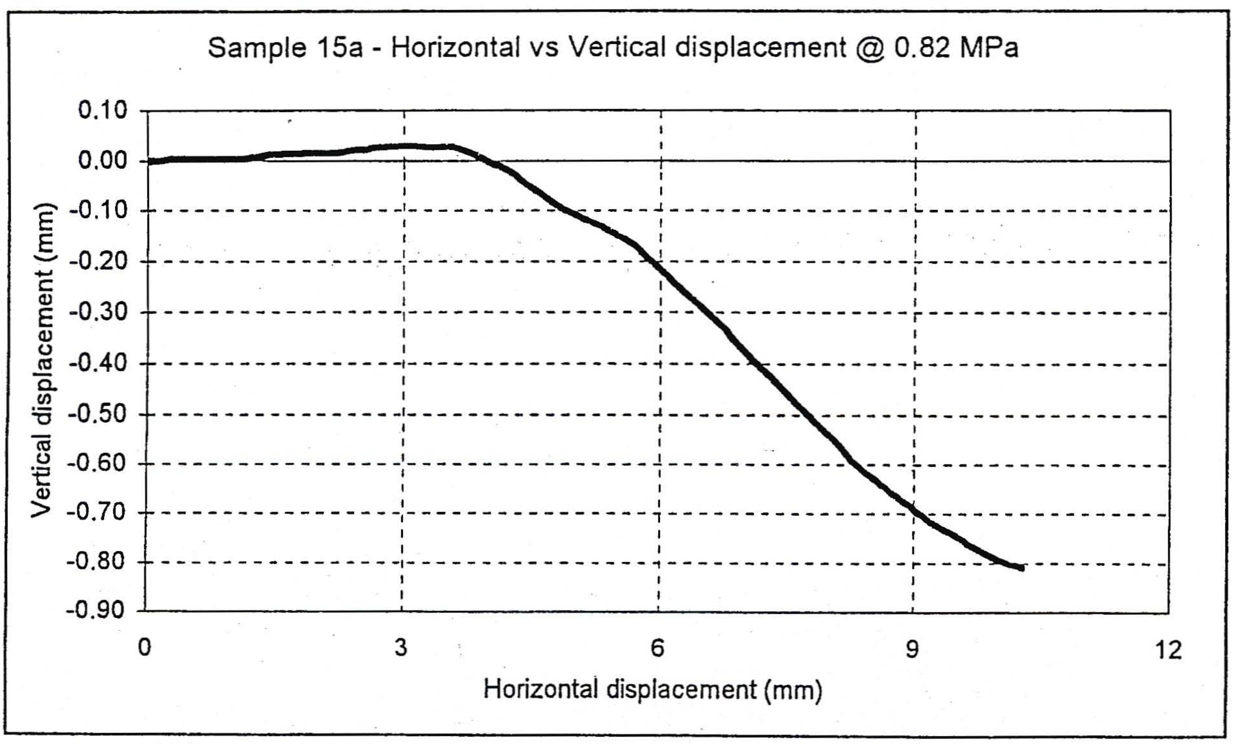

Sample 14 Shear stress vs Shear displacement @ 1.01 MPa
Peak = 0.463 MPa
Residual = 0.300 MPA
Shear stress (MPa)
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
2
4
6
8
10
12
Horizontal displacement (mm)

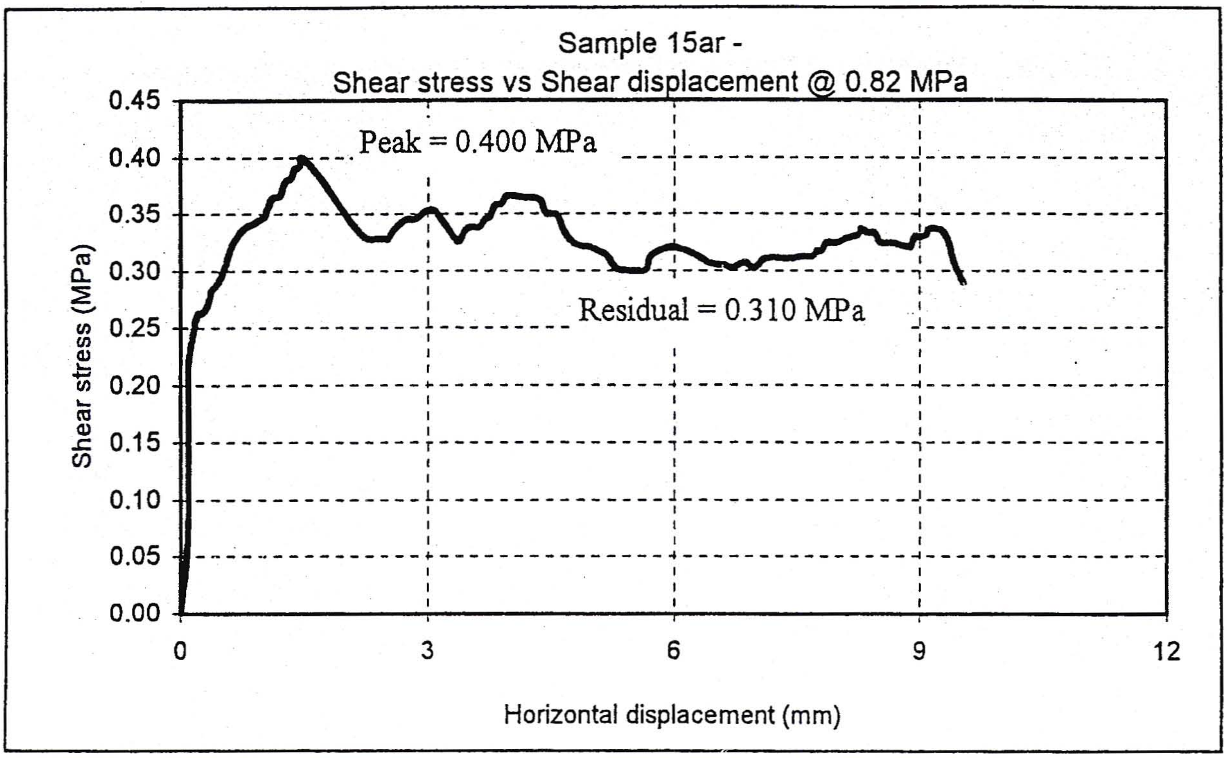

Sample 14 - Horizontal vs Vertical displacement @ 1.01 MPa
Vertical displacement (mm)
0.2
0.0
-0.2
-0.4
-0.6
-0.8
-1.0
-1.2
-1.4
-1.6
0
2
4
6
8
10
12
Horizontal displacement (mm)

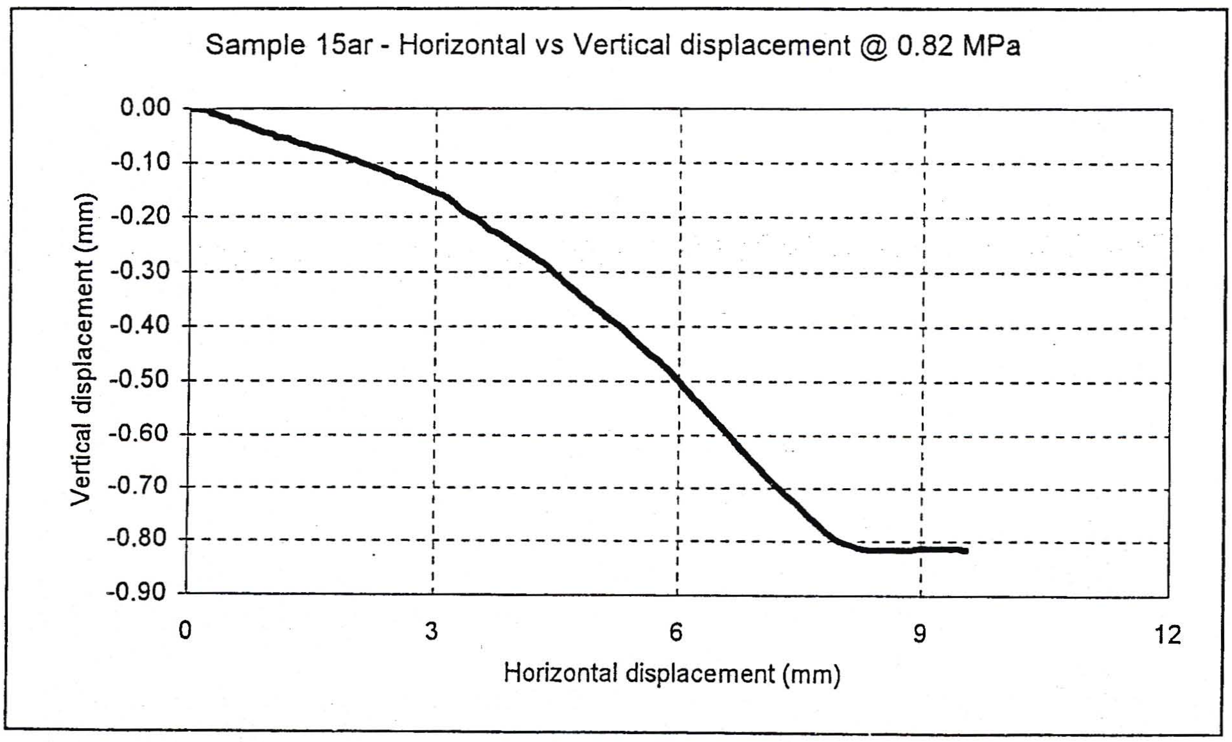

Sample 14r - Shear stress vs Shear displacement @ 1.01 MPa
Peak = 0.470 MPa
Residual = 0.400 MPa
Shear stress (MPa)
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
2
4
6
8
Horizontal displacement (mm)

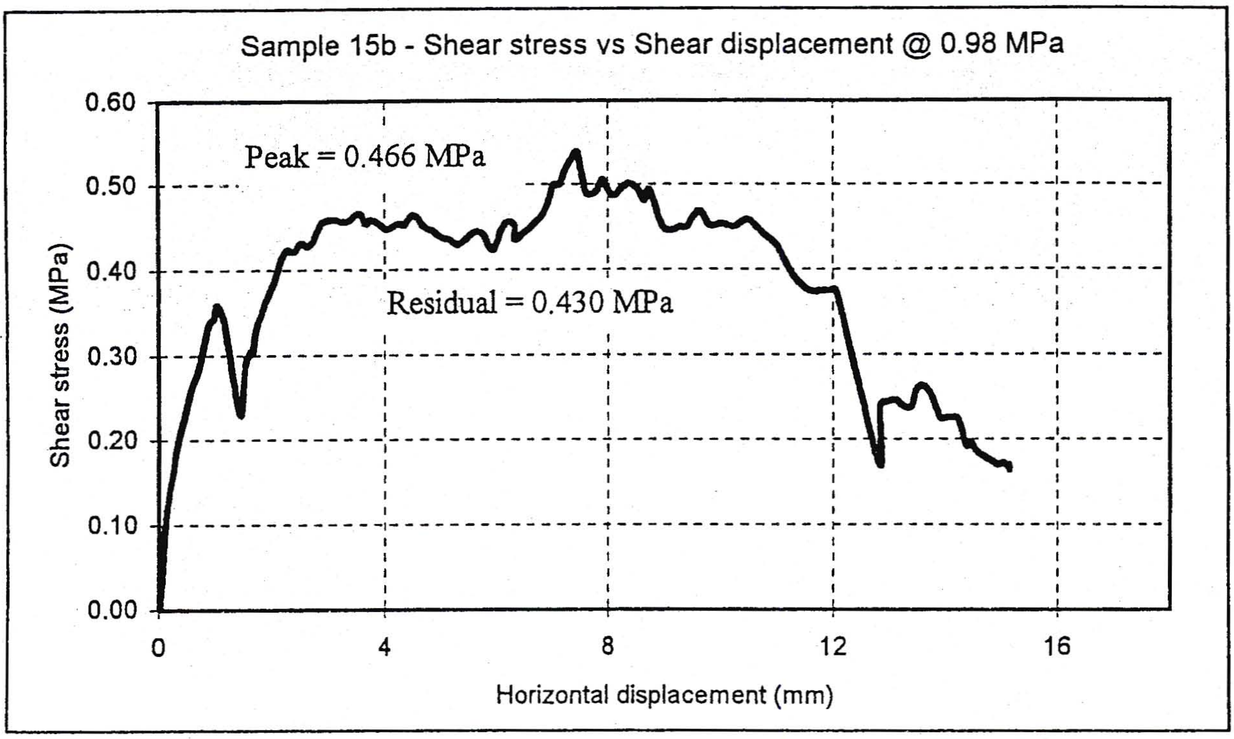

Sample 14r - Horizontal vs Vertical displacement @ 1.01 MPa
Vertical displacement (mm)
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
-0.60
-0.70
-0.80
-0.90
0
2
4
6
8
Horizontal displacement (mm)


Sample 15a - Shear stress vs Shear displacement @ 0.82 MPa
Peak = 0.429 MPa
Residual = 0.285 MPa
Shear stress (MPa)
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
3
6
9
12
Horizontal displacement (mm)

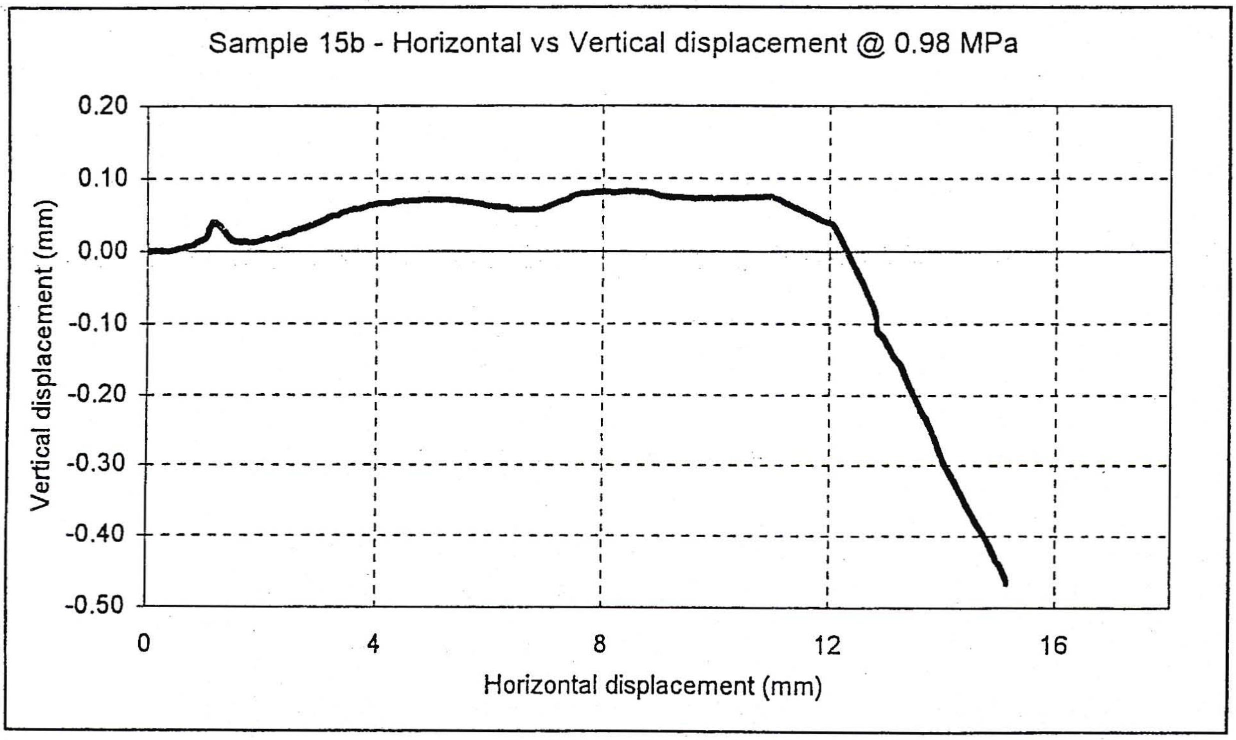

Sample 15a - Horizontal vs Vertical displacement @ 0.82 MPa
Vertical displacement (mm)
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
-0.60
-0.70
-0.80
-0.90
0
3
6
9
12
Horizontal displacement (mm)


Sample 15ar -
Shear stress vs Shear displacement @ 0.82 MPa
Peak = 0.400 MPa
Residual = 0.310 MPa
Shear stress (MPa)
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
3
6
9
12
Horizontal displacement (mm)

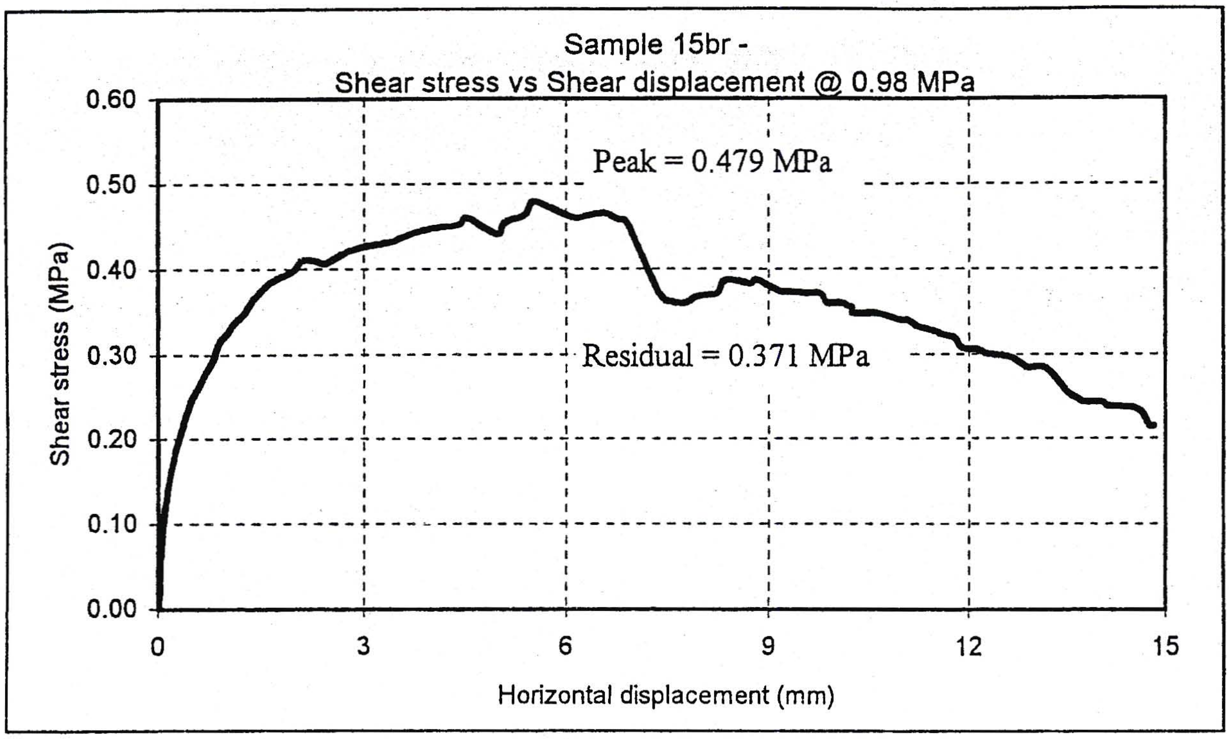

Sample 15ar - Horizontal vs Vertical displacement @ 0.82 MPa
Vertical displacement (mm)
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
-0.60
-0.70
-0.80
-0.90
0
3
6
9
12
Horizontal displacement (mm)

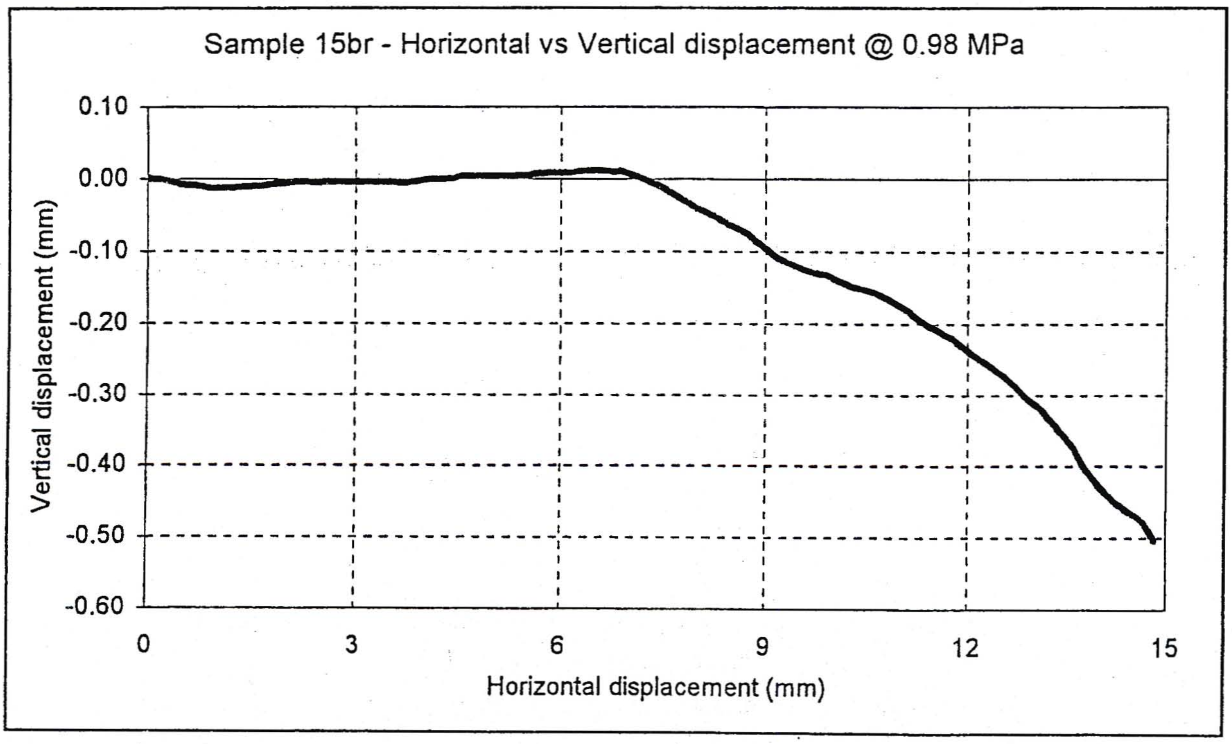


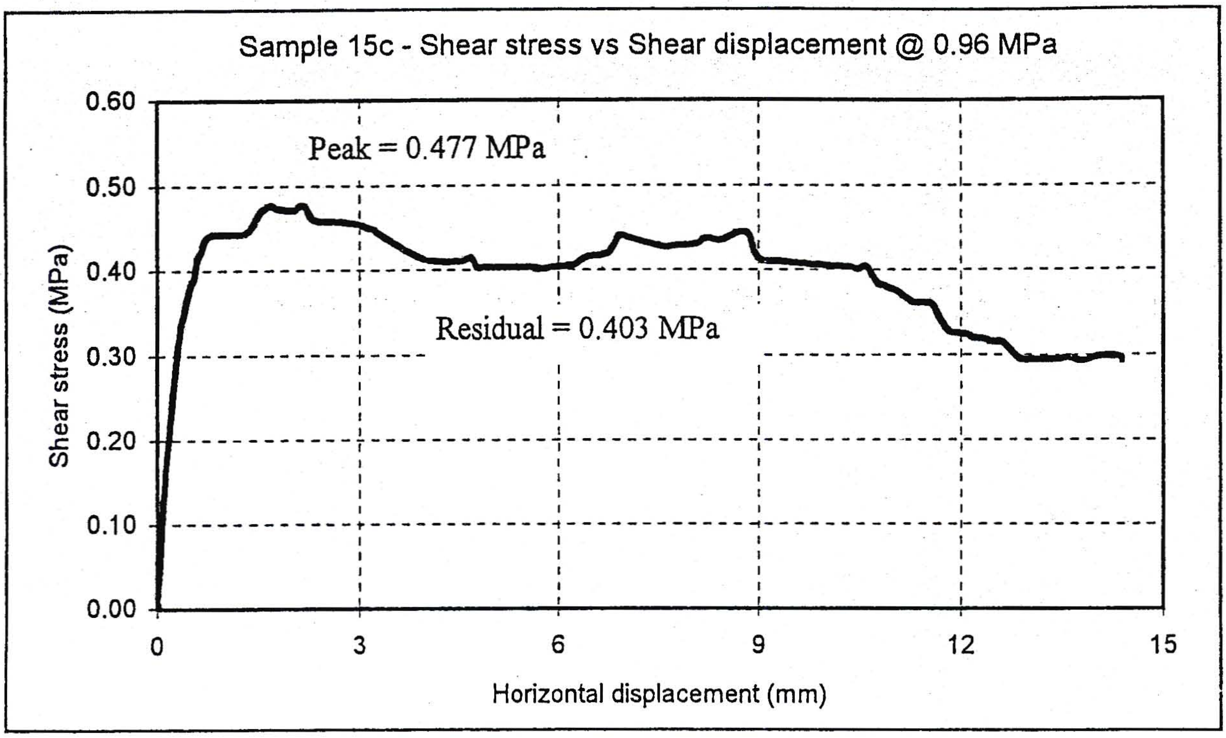
Sample 15b - Shear stress vs Shear displacement @ 0.98 MPa
Peak = 0.466 MPa
Residual = 0.430 MPa
Shear stress (MPa)
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
4
8
12
16
Horizontal displacement (mm)

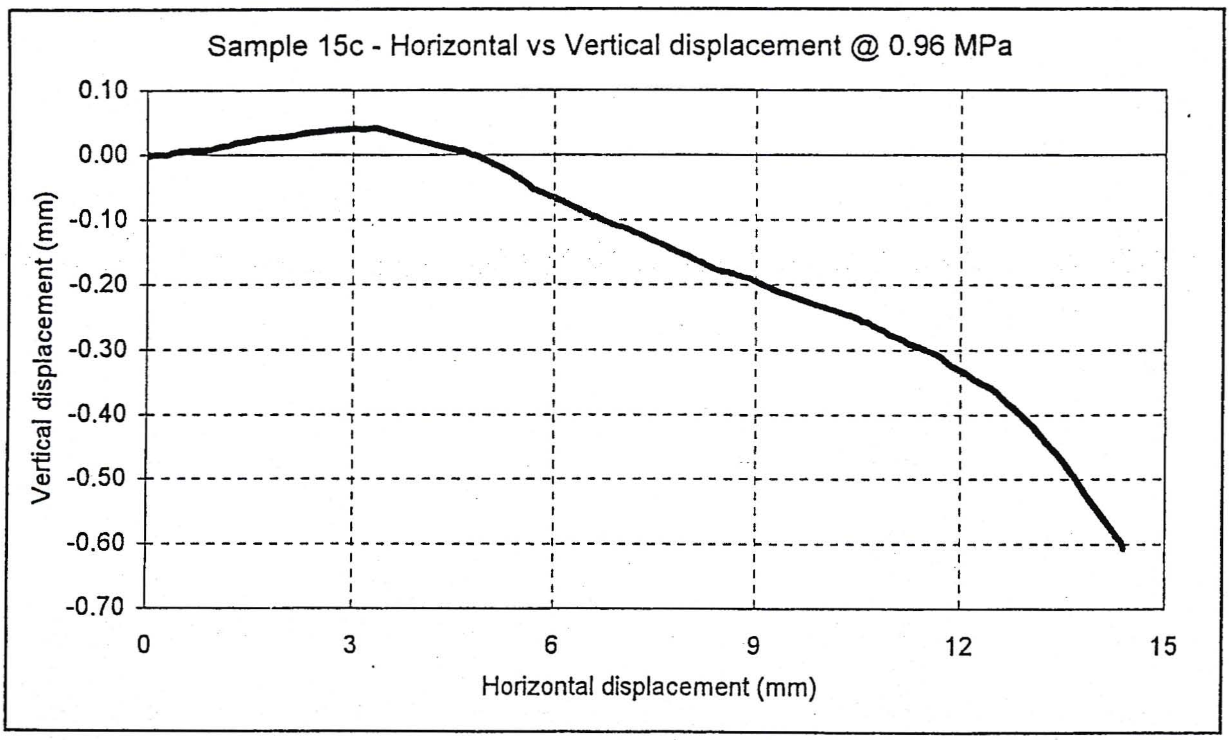


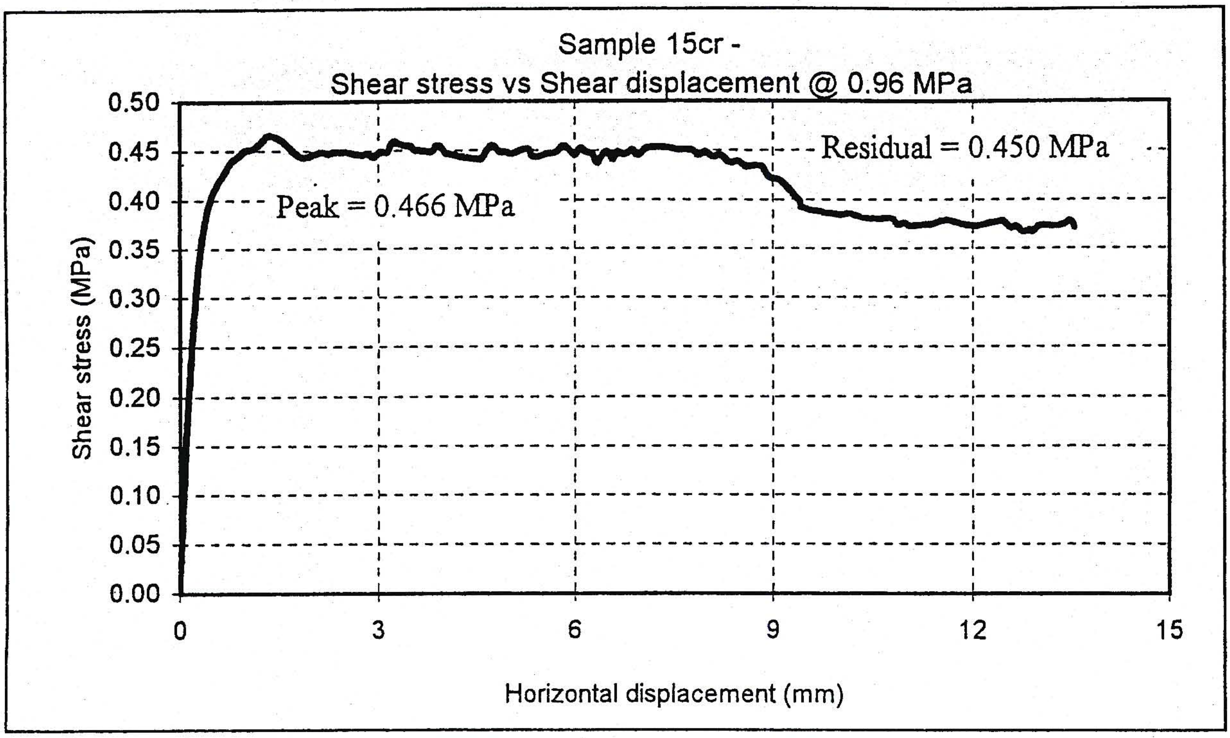
Sample 15b - Horizontal vs Vertical displacement @ 0.98 MPa
Vertical displacement (mm)
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
0
4
8
12
16
Horizontal displacement (mm)


Sample 15br -
Shear stress vs Shear displacement @ 0.98 MPa
Peak = 0.479 MPa
Residual = 0.371 MPa
Shear stress (MPa)
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
3
6
9
12
15
Horizontal displacement (mm)

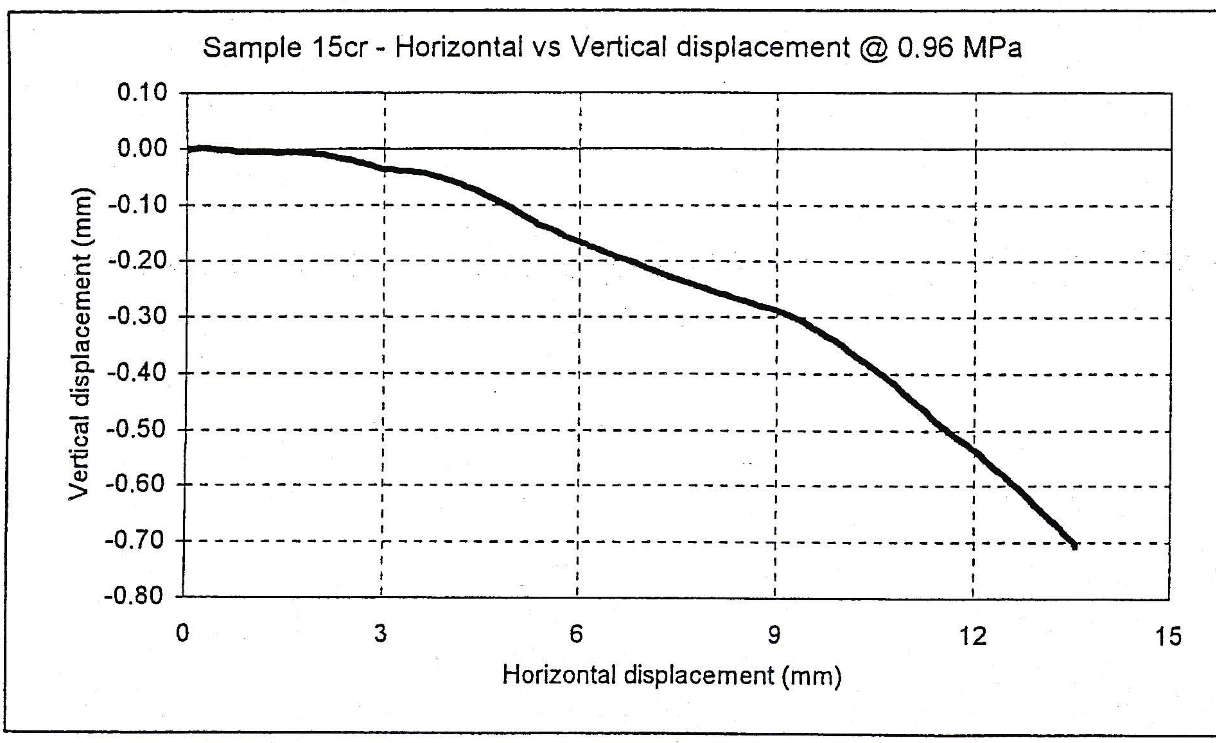

Sample 15br - Horizontal vs Vertical displacement @ 0.98 MPa
Vertical displacement (mm)
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
-0.60
0
3
6
9
12
15
Horizontal displacement (mm)


Sample 15c - Shear stress vs Shear displacement @ 0.96 MPa
Peak = 0.477 MPa
Residual = 0.403 MPa
Shear stress (MPa)
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
3
6
9
12
15
Horizontal displacement (mm)

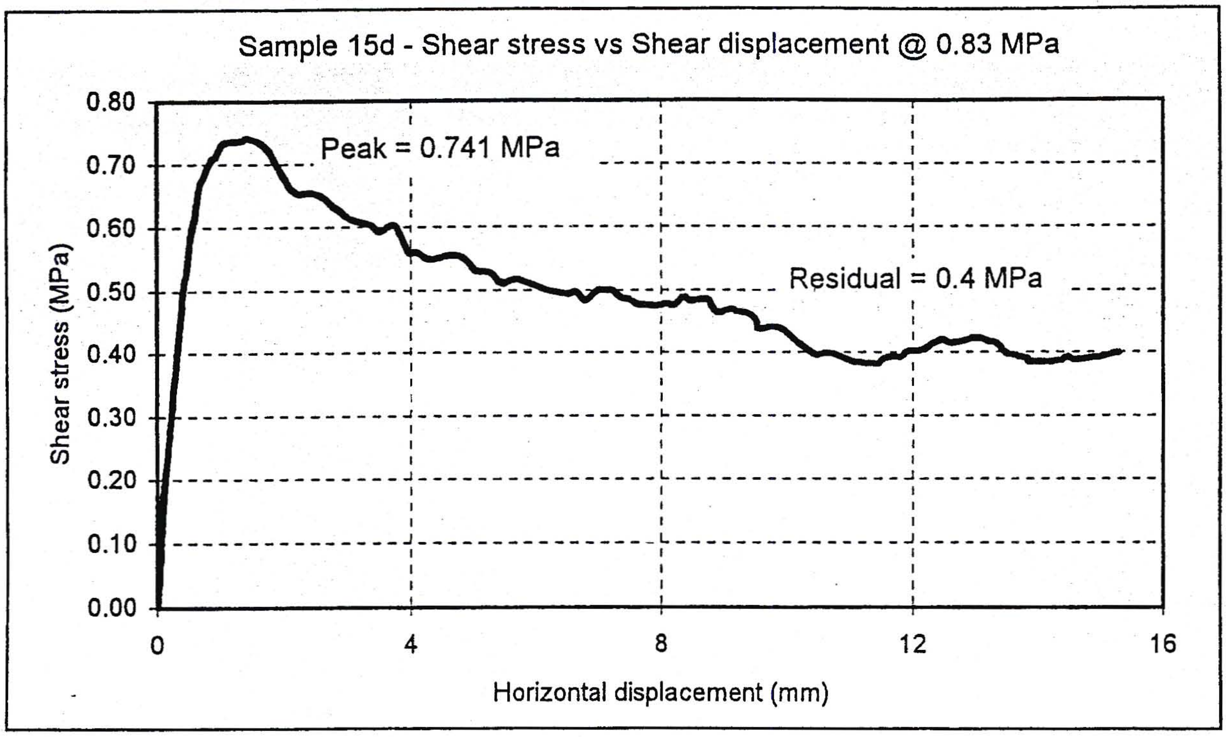

Sample 15c - Horizontal vs Vertical displacement @ 0.96 MPa
Vertical displacement (mm)
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
-0.60
-0.70
0
3
6
9
12
15
Horizontal displacement (mm)



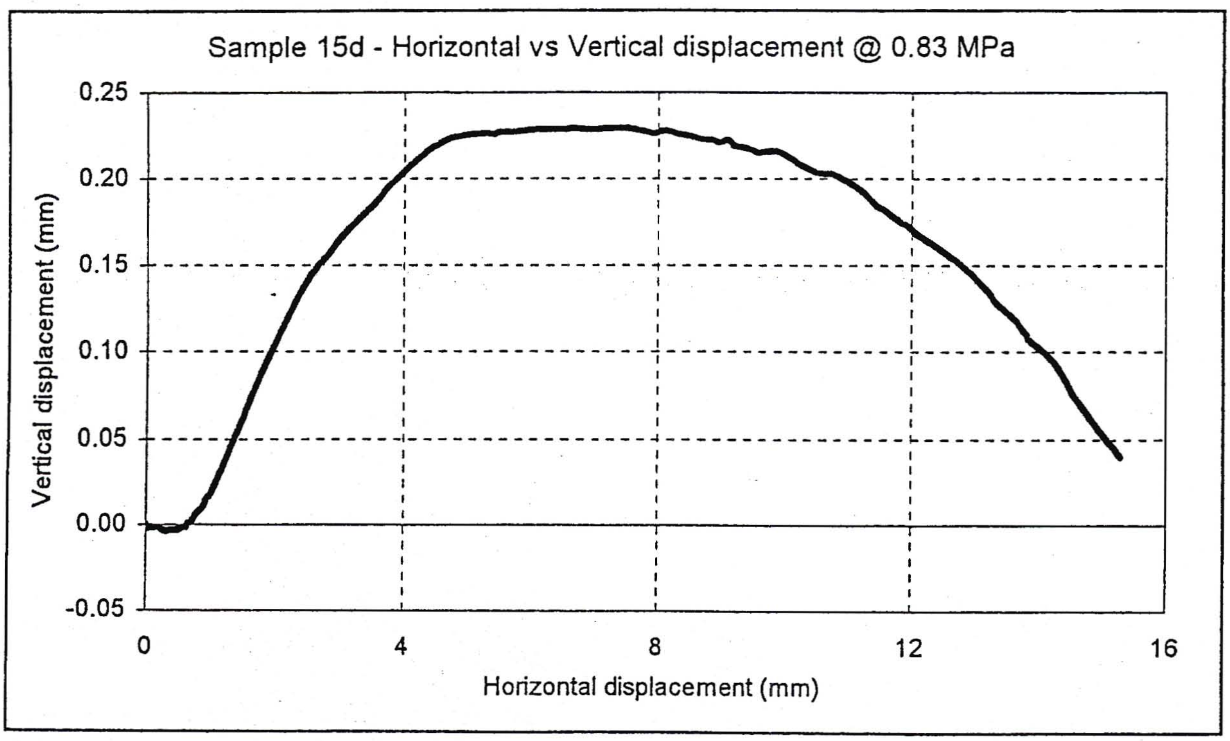
Sample 15cr -
Shear stress vs Shear displacement @ 0.96 MPa
Residual = 0.450 MPa
Peak = 0.466 MPa
Shear stress (MPa)
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
3
6
9
12
15
Horizontal displacement (mm)



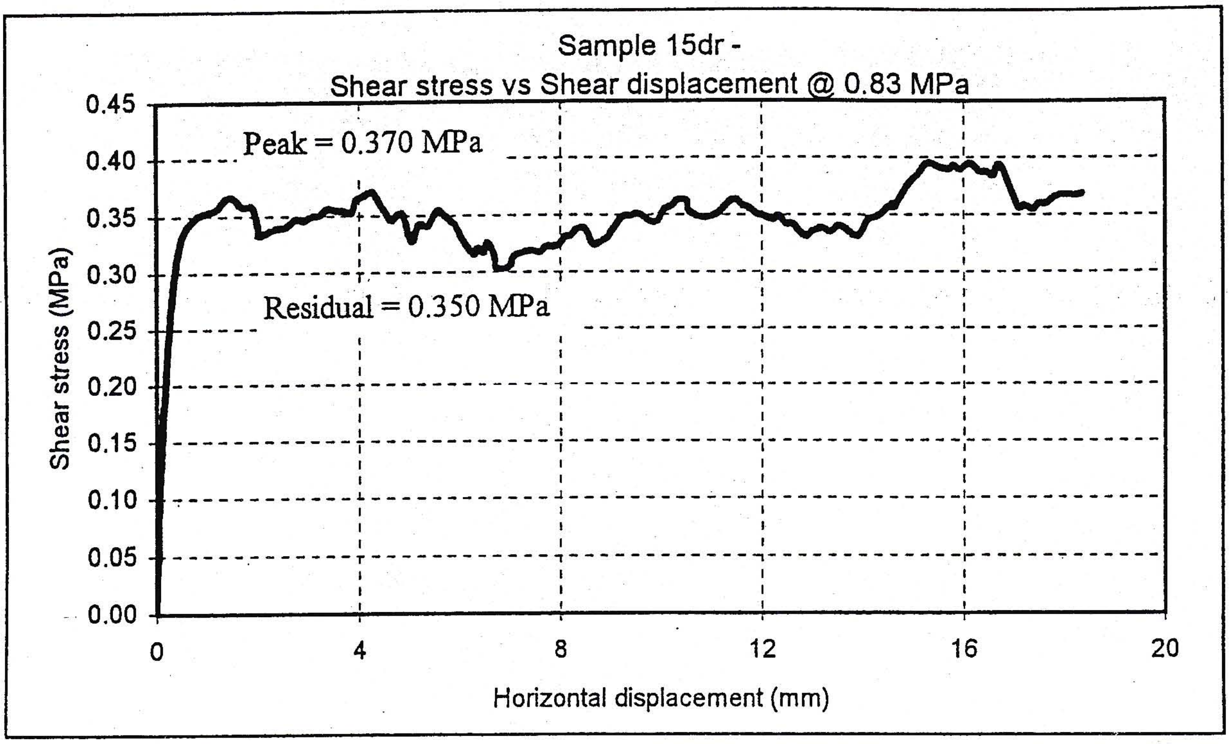
Sample 15cr - Horizontal vs Vertical displacement @ 0.96 MPa
Vertical displacement (mm)
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
-0.60
-0.70
-0.80
0
3
6
9
12
15
Horizontal displacement (mm)

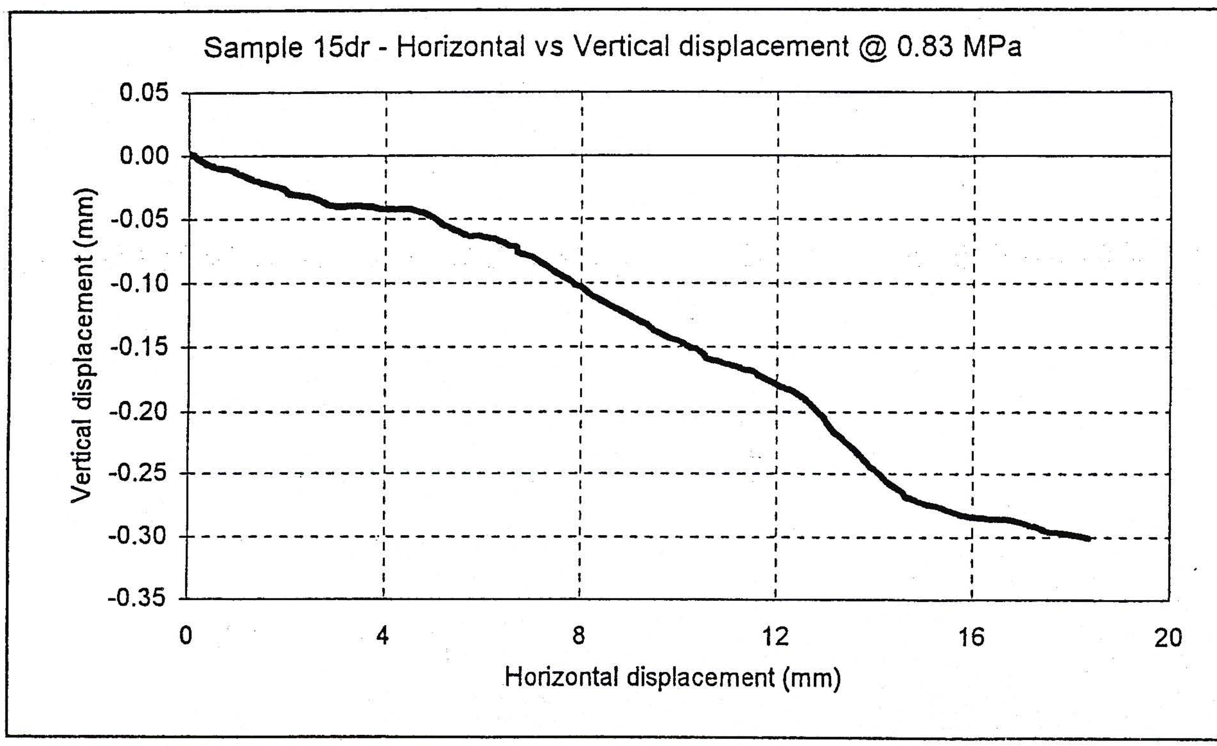


Sample 15d - Shear stress vs Shear displacement @ 0.83 MPa
Peak = 0.741 MPa
Residual = 0.4 MPa
Shear stress (MPa)
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
4
8
12
16
Horizontal displacement (mm)

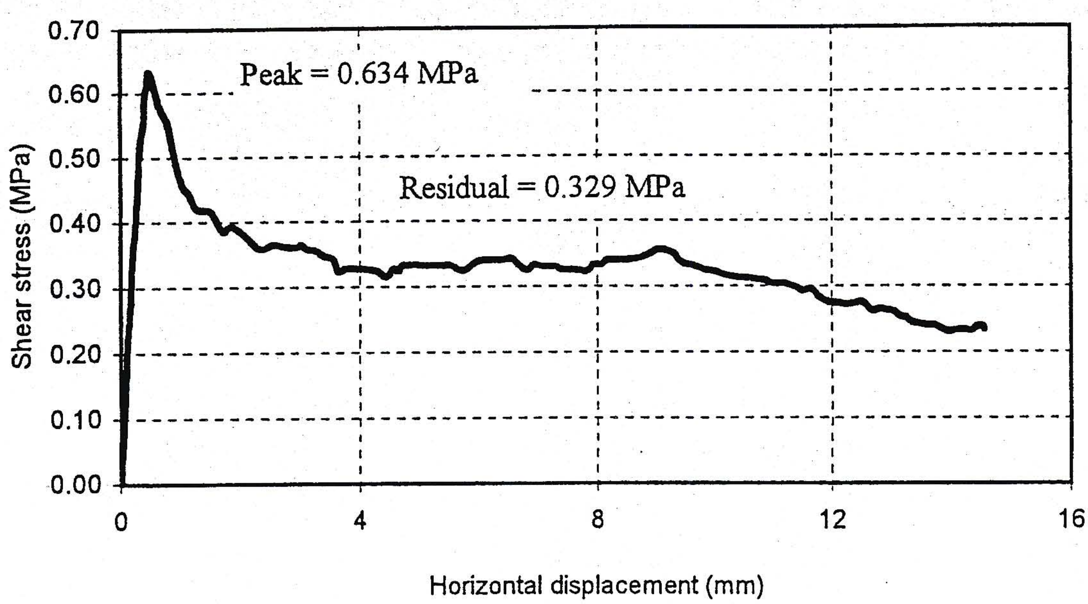


Sample 15d - Horizontal vs Vertical displacement @ 0.83 MPa
Vertical displacement (mm)
0.25
0.20
0.15
0.10
0.05
0.00
-0.05
0
4
8
12
16
Horizontal displacement (mm)

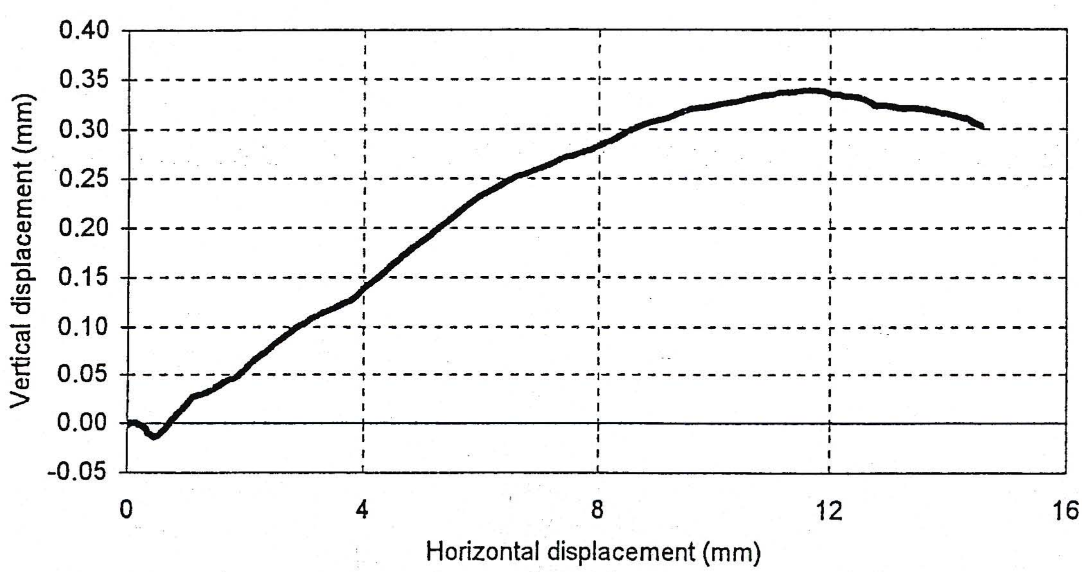

Sample 15dr -
Shear stress vs Shear displacement @ 0.83 MPa
Peak = 0.370 MPa
Residual = 0.350 MPa
Shear stress (MPa)
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
4
8
12
16
20
Horizontal displacement (mm)

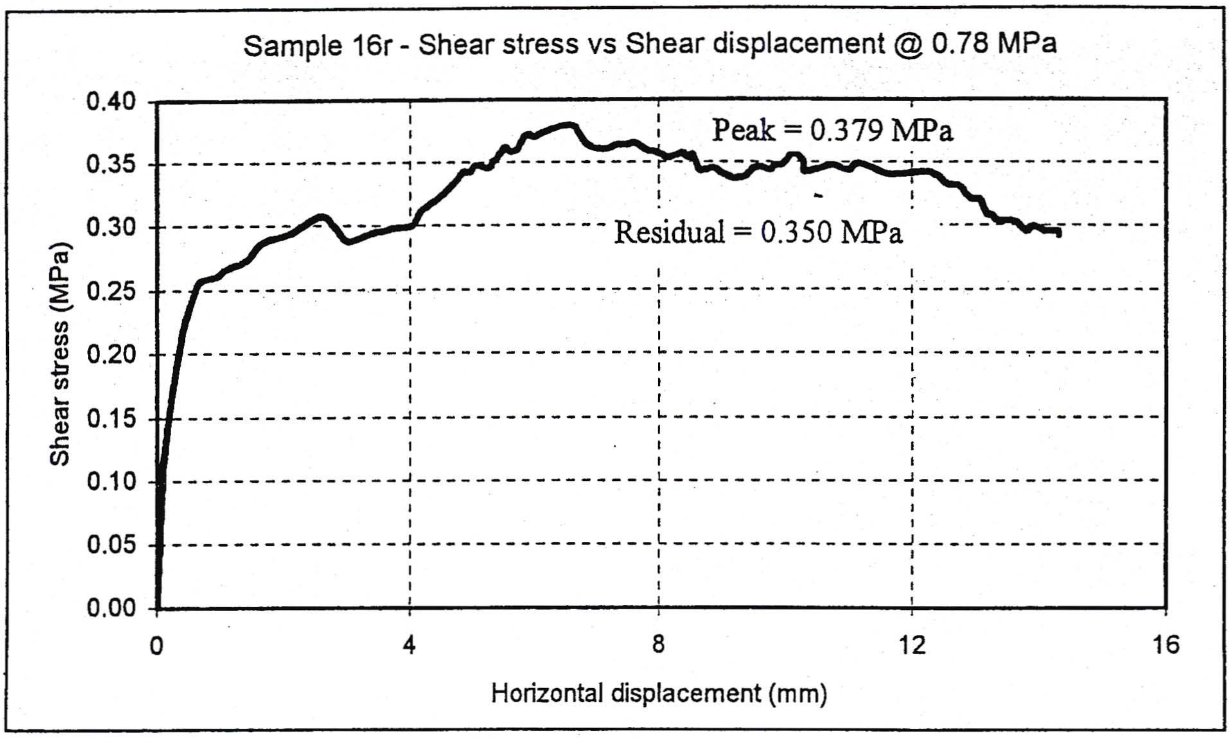

Sample 15dr - Horizontal vs Vertical displacement @ 0.83 MPa
Vertical displacement (mm)
0.05
0.00
-0.05
-0.10
-0.15
-0.20
-0.25
-0.30
-0.35
0
4
8
12
16
20
Horizontal displacement (mm)

Sample 16 - Shear stress vs Shear displacement @ 0.78 MPa

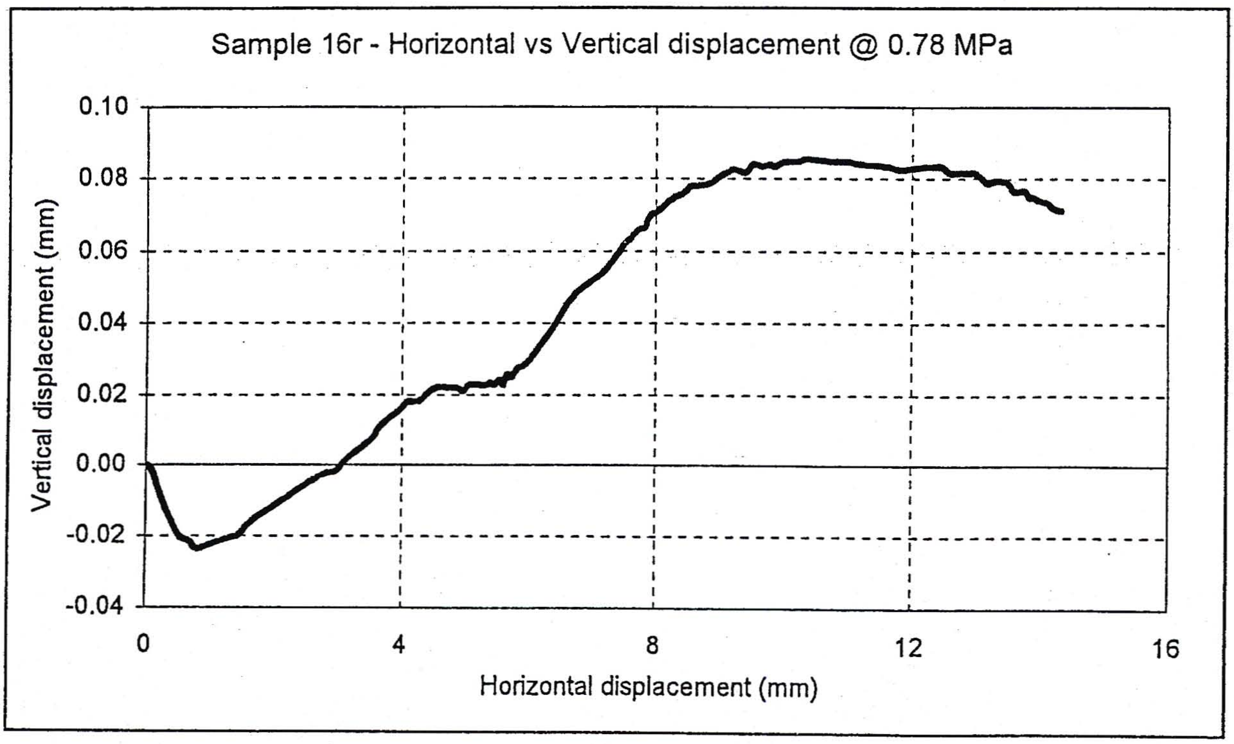


Sample 16 - Horizontal vs Vertical displacement @ 0.78 MPa





Sample 16r - Shear stress vs Shear displacement @ 0.78 MPa
Peak = 0.379 MPa
Residual = 0.350 MPa
Shear stress (MPa)
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
4
8
12
16
Horizontal displacement (mm)



Sample 16r - Horizontal vs Vertical displacement @ 0.78 MPa
Vertical displacement (mm)
0.10
0.08
0.06
0.04
0.02
0.00
-0.02
-0.04
0
4
8
12
16
Horizontal displacement (mm)

Appendix F

Rock Mass Classification Parameters and their Ratings

Geomechanics Classification of Rock masses: Rock Mass Rating (After Bieniawski, 1979)

Table A: Classification Parameters and their ratings

	Parameter		Ranges of Values						
1	Strength of intact rock material	Point-load strength index (MPa)	>10	4 - 10	2 - 4	1 - 2	For this range, uniaxial compressive strength test is preferred.		
		Uniaxial compressive strength (MPa)	>250	100 - 250	50 - 100	25 - 50	5 - 25	1 - 5	<1
	Rating		15	12	7	4	2	1	0
2	Drill Core Quality, RQD (%)		90 - 100	75 - 90	50 - 75	25 - 50	<25		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		>2 m	0.6 - 2 m	200 - 600 mm	60 - 200 mm	<60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation <1 mm Slightly weathered walls	Slightly rough surfaces Separation <1 mm Highly weathered walls	Slickensided surfaces or Gouge <5 mm thick or Separation 1 - 5 m Continuous	Soft gouge >5 mm thick or Separation >5 mm Continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (L/min)	None	<10	10 - 20	25 - 125	>125		
		Ratio of Joint water pressure/Major principal stress	0	<0.1	0.1 - 0.2	0.2 - 0.5	>0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

Table B: Rating adjustment for discontinuity orientations

Strike and Dip orientations of discontinuities		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels and mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

Table C: Rock Mass Classes determined from total ratings

Rating	100 - 81	80 - 61	60 - 41	40 - 21	<20
Class No.	I	III	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

aubscher's Mining Rock Mass Classification System (1990)

able A: Parameters and ratings

IRS (MPa)	rating % - possible rating of 20
>185	20
165 - 185	18
145 - 164	16
125 - 144	14
105 - 124	12
85 - 105	10
65 - 84	8
45 - 64	6
35 - 44	5
25 - 34	4
12 - 24	3
5 - 11	2
1 - 4	1

RQD	rating % - possible rating of 15
97 - 100	15
84 - 96	14
71 - 83	12
56 - 70	10
44 - 55	8
31 - 43	6
17 - 30	4
4 - 16	2
0 - 3	0

Fracture Frequency, FF/m - possible rating of 40			
Average per metre	Rating Set 1	Set 2	Set 3
0.10	40	40	40
0.15	40	40	40
0.20	40	40	38
0.25	40	38	36
0.30	38	36	34
0.50	36	34	31
0.80	34	31	28
1.00	31	28	26
1.50	29	26	24
2.00	26	24	21
3.00	24	21	18
5.00	21	18	15
7.00	18	15	12
10.00	15	12	10
15.00	12	10	7
20.00	10	7	5
30.00	7	5	2
40.00	5	2	0
Allow for core recovery			

Assement of joint condition - possible rating of 40					
Parameter	Description	Dry	Moist	Mod. Pressure 25 - 125 l/m	High pressure >125 l/m
A Large scale point expression	Multi wavy directional	100	100	95	90
	Uni	95	90	85	80
	Curved	85	80	75	70
	Slight undulation	80	75	70	65
	Straight	75	70	65	60
B Small scale joint expression	Rough stepped/irregular	95	90	85	80
	Smooth stepped	90	85	80	75
	Slickensided stepped	85	80	75	70
	Rough undulating	80	75	70	65
	Smooth undulating	75	70	65	60
	Slickensided undulating	70	65	60	55
	Rough planar	65	60	55	50
	Smooth planar	60	55	50	45
	Polished	55	50	45	40
C Joint wall alteration weaker than wall rock and only if it is weaker than the filling.		75	70	65	60
D Joint filling	Non-softening and sheared material - Coarse	90	85	80	75
	Non-softening and sheared material - Medium	85	80	75	70
	Non-softening and sheared material - Fine	80	75	70	65
	Soft sheared material - Coarse	70	65	60	55
	Soft sheared material - Medium	60	55	50	45
	Soft sheared material - Fine	50	45	40	45
	Gouge thickness < amplitude of irregularities	45	40	35	30
	Gouge thickness > amplitude of irregularities	30	20	15	10

ssessment of joint spacing - possible rating of 25
ote: x = spacing * 100

Joint set R = 25 * ((26.4 * log X) + 45)/100
Joint set R = 25 * ((25.9 * log X min) + 38)/100 * ((30 * log Xmax) + 28)/100
Joint set R = 25 * ((25.9 * log X min) + 30)/100 * ((29.6 * log Xint) + 20)/100 * ((33.3 * log Xmax) + 10)100

able B: Meaning of ratings

lass	1	2	3	4	5
ating	100 - 81	80 - 61	60 - 41	40 - 21	20 - 0
escription	Very Good	Good	Fair	Poor	Very poor

Table C: Factors to give average fracture frequency

Sampling procedure	Factor
One set of three sets on a line, or one set only	1.0
Two sets of three sets on a line or two sets only	1.5
All of the sets on a line or borehole core	2.0
Two sets on a line and one on another	2.4
Three sets on three lines at right angles	3.0

Table D: Factors by which joint frequencies are multiplied.

Continuous features %	Factor
100	1
90	0.9
80	0.8
70	0.7
60	0.6
50	0.5

Table E: Adjustments for weathering.

Degree of weathering	Potential weathering andadjustments, %				
	0.5 yrs	1 yrs	2 yrs	3 yrs	4+ yrs
Fresh	100	100	100	100	100
Slight	88	90	92	94	96
Moderate	82	84	86	88	90
High	70	72	74	76	78
Complete	54	56	58	60	62
Residual soil	30	32	34	36	38

Table F: Percentage adjustment for joint orientation.

No. of joints defining the block	No. of faces inclined away from the vertical				
	70%	75%	80%	85%	90%
3	3		2		
4	4	3		2	
5	5	4	3	2	1
6	6	5	4	3	2.1

Table G: Percentage adjustment for the plunge of the intersection of joints on the base of the blocks

Average rating	Plunge degree	Adjustment %	Plunge degree	Adjustment %	Plunge degree	Adjustment %
0 -5	10 - 30	85	30 - 40	75	> 40	70
5 - 10	10 - 20	90	20 - 40	80	> 40	70
10 - 15	20 - 30	90	30 - 50	80	>50	75
15 - 20	30 - 40	90	40 - 60	85	> 60	80
20 - 30	30 - 50	90	> 50	85		
30 - 40	40 - 60	90	> 50	90		

Table H: Blasting effects

Technique	Adjustment, %
Boring	100
Smooth-wall blasting	97
Good convential blasting	94
Poor blasting	80

Geomechanical Classification for Slopes : Slope Mass Rating (Romana, 1997)

Table A: Bieniawski (1979) Ratings for RMR

Table B: Adjustment ratings for joints.

Case		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
P	| Aj - As |	>30 deg.	30 - 20 deg.	20 - 10 deg.	10 - 5 deg.	< 5 deg.
T	| (Aj - As) - 180 deg |					
P/T	F1	0.15	0.40	0.70	0.85	1.00
P	| Bj |	< 20 deg.	20 - 30 deg.	30 - 35 deg.	35 - 45 deg.	> 45 deg.
P	F2	0.15	0.40	0.70	0.85	1.00
T	F2	1	1	1	1	1
P	Bj - Bs	> 10 deg.	10 - 0 deg.	0 deg.	0 to -10 deg.	< -10 deg.
T	Bj + Bs	<110 deg.	110 - 120 deg.	> 120 deg.	--	--
P/T	F3	0	-6	-25	-50	-60

P - Plane failure, T - Toppling failure, Aj - Joint dip direction, As - slope dip direction, Bj - Joint dip, Bs - Slope dip

Table C: Adjustment ratings for Methods of Excavation Slopes

Method	Natural Slope	Presplitting	Smooth blasting	Blasting or mechanical	Deficient blasting
F4	+15	+10	+10	0	-8

Table D: Tentative Description of SMR Classes

Class No.	V	IV	III	II	I
SMR	0 - 20	21 - 40	41 - 60	61 - 80	81 - 100
Description	Very Poor	Poor	Fair	Good	Very Good

Appendix F

Rock Mass Classification Parameters and their Ratings

Geomechanics Classification of Rock masses: Rock Mass Rating (After Bieniawski, 1979)

Table A: Classification Parameters and their ratings

	Parameter		Ranges of Values						
1	Strength of intact rock material	Point-load strength index (MPa)	>10	4 - 10	2 - 4	1 - 2	For this range, uniaxial compressive strength test is preferred.		
		Uniaxial compressive strength (MPa)	>250	100 - 250	50 - 100	25 - 50	5 - 25	1 - 5	<1
	Rating		15	12	7	4	2	1	0
2	Drill Core Quality, RQD (%)		90 - 100	75 - 90	50 - 75	25 - 50	<25		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		>2 m	0.6 - 2 m	200 - 600 mm	60 - 200 mm	<60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation <1 mm Slightly weathered walls	Slightly rough surfaces Separation <1 mm Highly weathered walls	Slickensided surfaces or Gouge <5 mm thick or Separation 1 - 5 m Continuous	Soft gouge >5 mm thick or Separation >5 mm Continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (L/min)	None	<10	10 - 20	25 - 125	>125		
		Ratio of Joint water pressure/Major principal stress	0	<0.1	0.1 - 0.2	0.2 - 0.5	>0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

Table B: Rating adjustment for discontinuity orientations

Strike and Dip orientations of discontinuities		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels and mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

Table C: Rock Mass Classes determined from total ratings

Rating	100 - 81	80 - 61	60 - 41	40 - 21	<20
Class No.	I	III	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

Geomechanical Classification for Slopes : Slope Mass Rating (Romana, 1997)

Table A: Bieniawski (1979) Ratings for RMR

Table B: Adjustment ratings for joints.

	Case	Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
P	| Aj - As |	>30 deg.	30 - 20 deg.	20 - 10 deg.	10 - 5 deg.	< 5 deg.
T	| (Aj - As) - 180 deg |					
P/T	F1	0.15	0.40	0.70	0.85	1.00
P	| Bj |	< 20 deg.	20 - 30 deg.	30 - 35 deg.	35 - 45 deg.	> 45 deg.
P	F2	0.15	0.40	0.70	0.85	1.00
T	F2	1	1	1	1	1
P	Bj - Bs	> 10 deg.	10 - 0 deg.	0 deg.	0 to -10 deg.	< -10 deg.
T	Bj + Bs	<110 deg.	110 - 120 deg.	> 120 deg.	--	--
P/T	F3	0	-6	-25	-50	-60

P - Plane failure, T - Toppling failure, Aj - Joint dip direction, As - slope dip direction, Bj - Joint dip, Bs - Slope dip

Table C: Adjustment ratings for Methods of Excavation Slopes

Method	Natural Slope	Presplitting	Smooth blasting	Blasting or mechanical	Deficient blasting
F4	+15	+10	+10	0	-8

Table D: Tentative Description of SMR Classes

Class No.	V	IV	III	II	I
SMR	0 - 20	21 - 40	41 - 60	61 - 80	81 - 100
Description	Very Poor	Poor	Fair	Good	Very Good

Laubscher's Mining Rock Mass Classification System (1990)

Table A: Parameters and ratings

IRS (MPa) rating % - possible rating of 20	
>185	20
165 - 185	18
145 - 164	16
125 - 144	14
105 - 124	12
85 - 105	10
65 - 84	8
45 - 64	6
35 - 44	5
25 - 34	4
12 - 24	3
5 - 11	2
1 - 4	1

RQD rating % - possible rating of 15	
97 - 100	15
84 - 96	14
71 - 83	12
56 - 70	10
44 - 55	8
31 - 43	6
17 - 30	4
4 - 16	2
0 - 3	0

Fracture Frequency, FF/m - possible rating of 40			
Average per metre	Set 1	Rating Set 2	Set 3
0.10	40	40	40
0.15	40	40	40
0.20	40	40	38
0.25	40	38	36
0.30	38	36	34
0.50	36	34	31
0.80	34	31	28
1.00	31	28	26
1.50	29	26	24
2.00	26	24	21
3.00	24	21	18
5.00	21	18	15
7.00	18	15	12
10.00	15	12	10
15.00	12	10	7
20.00	10	7	5
30.00	7	5	2
40.00	5	2	0
Allow for core recovery			

Assement of joint condition - possible rating of 40						
Parameter	Description		Dry	Moist	Mod. Pressure 25 - 125 l/m	High pressure >125 l/m
A Large scale point expression	Multi wavy directional		100	100	95	90
	Uni		95	90	85	80
	Curved		85	80	75	70
	Slight undulation		80	75	70	65
	Straight		75	70	65	60
B Small scale joint expression	Rough stepped/irregular		95	90	85	80
	Smooth stepped		90	85	80	75
	Slickensided stepped		85	80	75	70
	Rough undulating		80	75	70	65
	Smooth undulating		75	70	65	60
	Slickensided undulating		70	65	60	55
	Rough planar		65	60	55	50
	Smooth planar		60	55	50	45
	Polished		55	50	45	40
C Joint wall alteration weaker than wall rock and only if it is weaker than the filling.			75	70	65	60
D Joint filling	Non-softening and sheared material	Coarse	90	85	80	75
		Medium	85	80	75	70
		Fine	80	75	70	65
	Soft sheared material	Coarse	70	65	60	55
		Medium	60	55	50	45
		Fine	50	45	40	45
	Gouge thickness < amplitude of irregularities		45	40	35	30
	Gouge thickness > amplitude of irregularities		30	20	15	10

Assessment of joint spacing - possible rating of 25
Note: x = spacing * 100

1 Joint set R = 25 * ((26.4 * log X) + 45)/100
2 Joint set R = 25 * ((25.9 * log X min) + 38)/100 * ((30 * log Xmax) + 28)/100
3 Joint set R = 25 * ((25.9 * log X min) + 30)/100 * ((29.6 * log Xint) + 20)/100 * ((33.3 * log Xmax) + 10)100

Table B: Meaning of ratings

Class	1	2	3	4	5
Rating	100 - 81	80 - 61	60 - 41	40 - 21	20 - 0
Description	Very Good	Good	Fair	Poor	Very poor

Table C: Factors to give average fracture frequency

Sampling procedure	Factor
One set of three sets on a line, or one set only	1.0
Two sets of three sets on a line or two sets only	1.5
All of the sets on a line or borehole core	2.0
Two sets on a line and one on another	2.4
Three sets on three lines at right angles	3.0

Table D: Factors by which joint frequencies are multiplied.

Continuous features %	Factor
100	1
90	0.9
80	0.8
70	0.7
60	0.6
50	0.5

Table E: Adjustments for weathering.

Degree of weathering	Potential weathering and adjustments, %				
	0.5 yrs	1 yrs	2 yrs	3 yrs	4+ yrs
Fresh	100	100	100	100	100
Slight	88	90	92	94	96
Moderate	82	84	86	88	90
High	70	72	74	76	78
Complete	54	56	58	60	62
Residual soil	30	32	34	36	38

Table F: Percentage adjustment for joint orientation.

No. of joints defining the block	No. of faces inclined away from the vertical				
	70%	75%	80%	85%	90%
3	3		2		
4	4	3		2	
5	5	4	3	2	1
6	6	5	4	3	2.1

Table G: Percentage adjustment for the plunge of the intersection of joints on the base of the blocks

Average rating	Plunge degree	Adjustment %	Plunge degree	Adjustment %	Plunge degree	Adjustment %
0 -5	10 - 30	85	30 - 40	75	> 40	70
5 - 10	10 - 20	90	20 - 40	80	> 40	70
10 - 15	20 - 30	90	30 - 50	80	>50	75
15 - 20	30 - 40	90	40 - 60	85	> 60	80
20 - 30	30 - 50	90	> 50	85		
30 - 40	40 - 60	90	> 50	90		

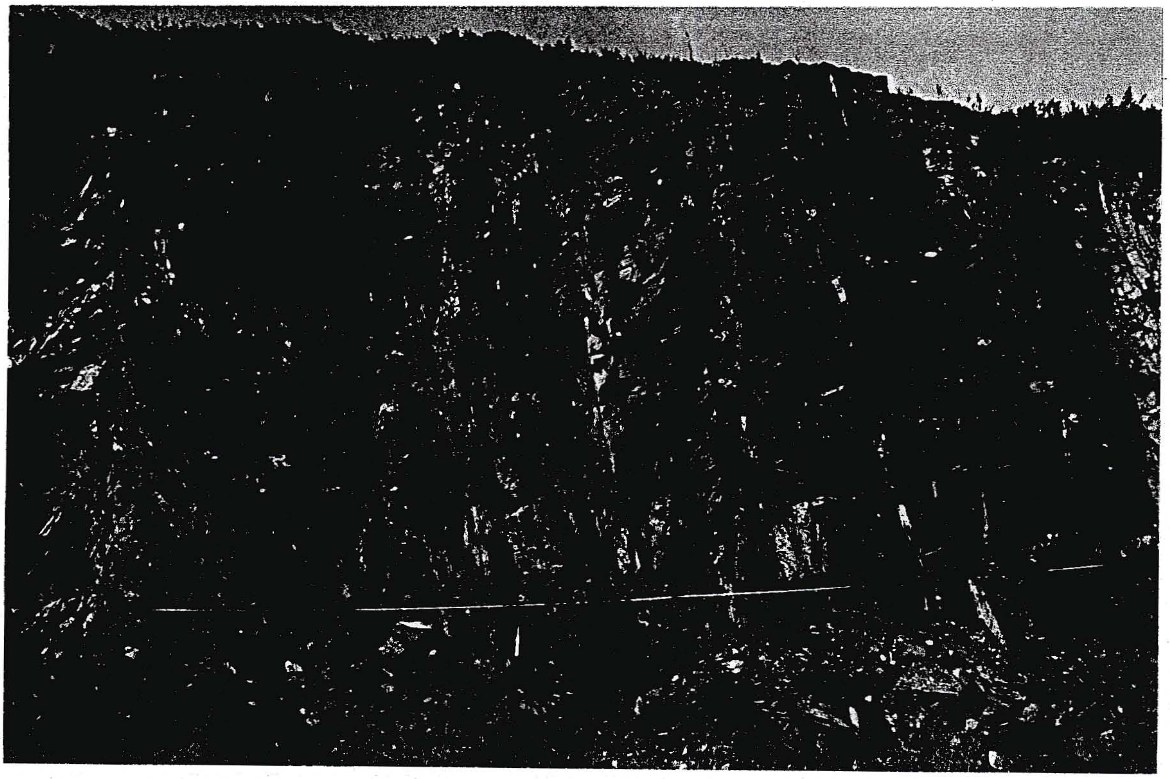
Table H: Blasting effects

Technique	Adjustment, %
Boring	100
Smooth-wall blasting	97
Good convential blasting	94
Poor blasting	80

Appendix G

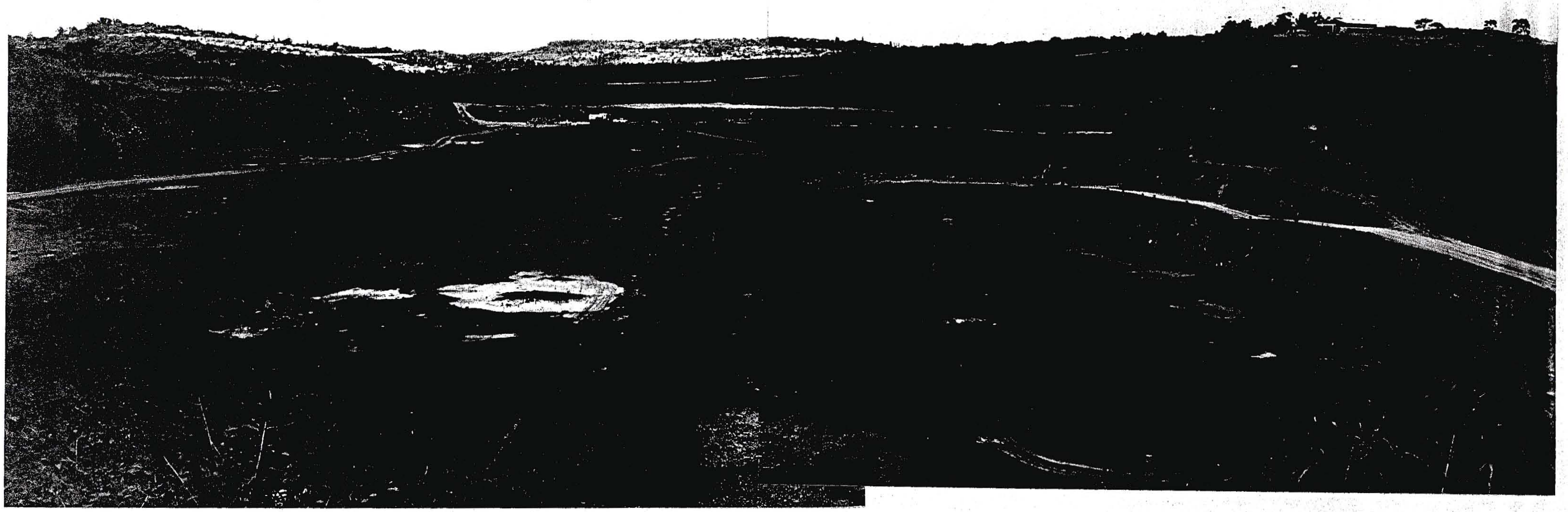
Photographic Summary

Margate Quarry – Face 1



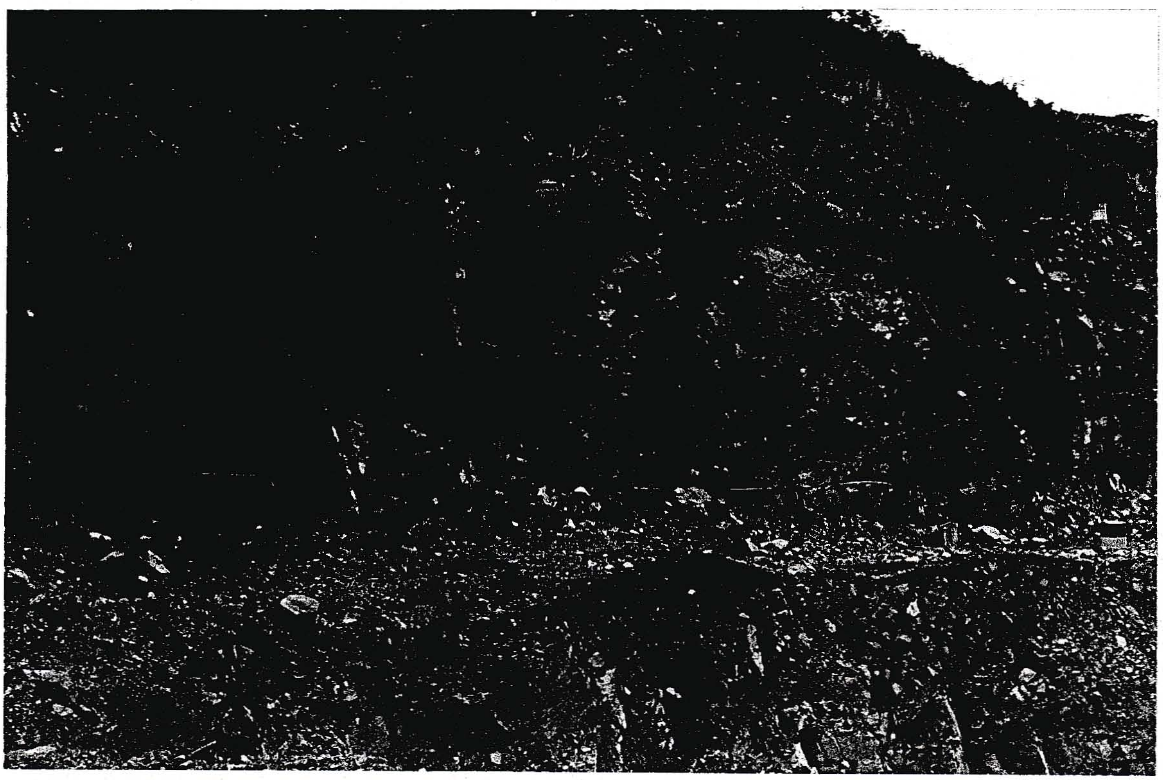
Margate Quarry – Face 2

Margate Quarry – Face 3

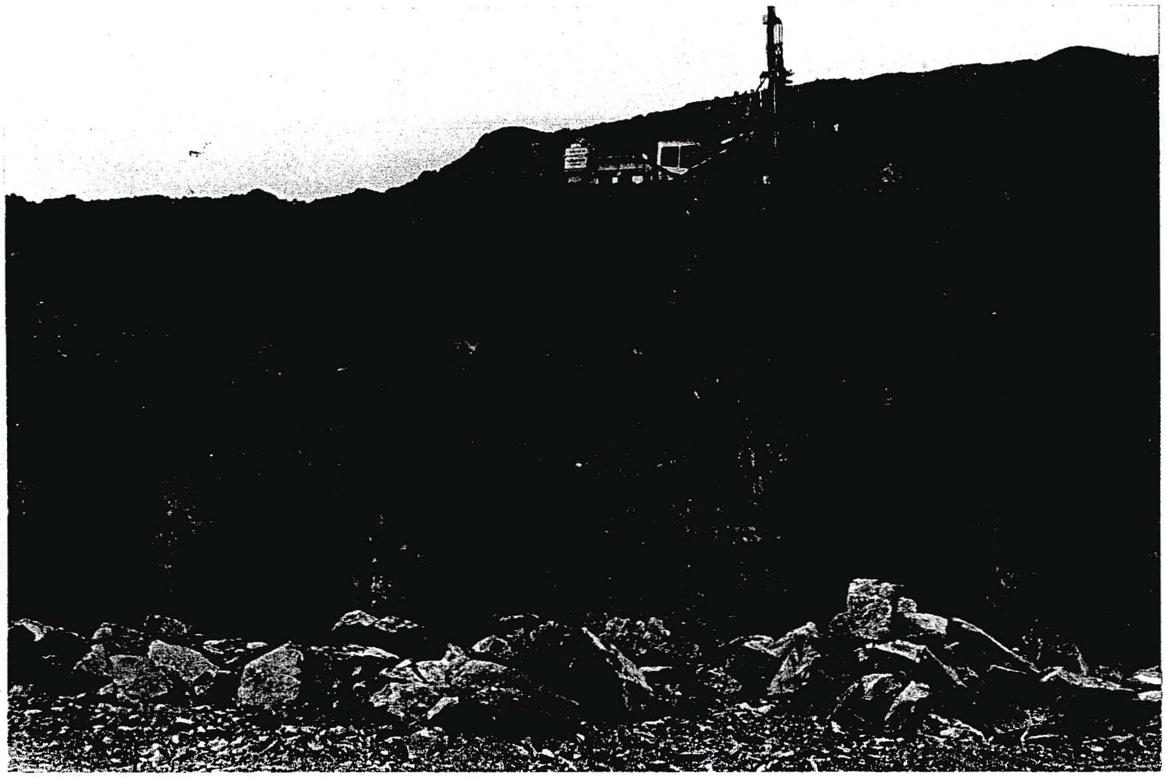


General overview of the Ridge View Quarry

Ridge View Quarry – Face 1



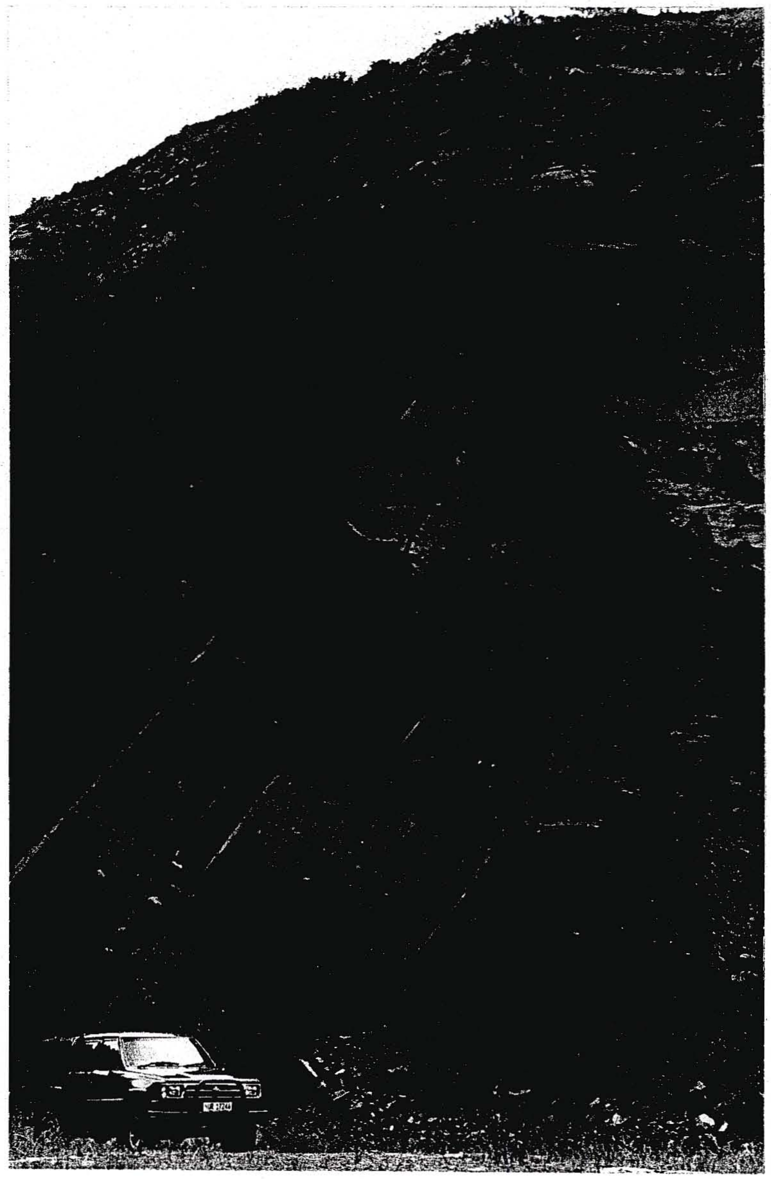
Ridge View Quarry – Face 2



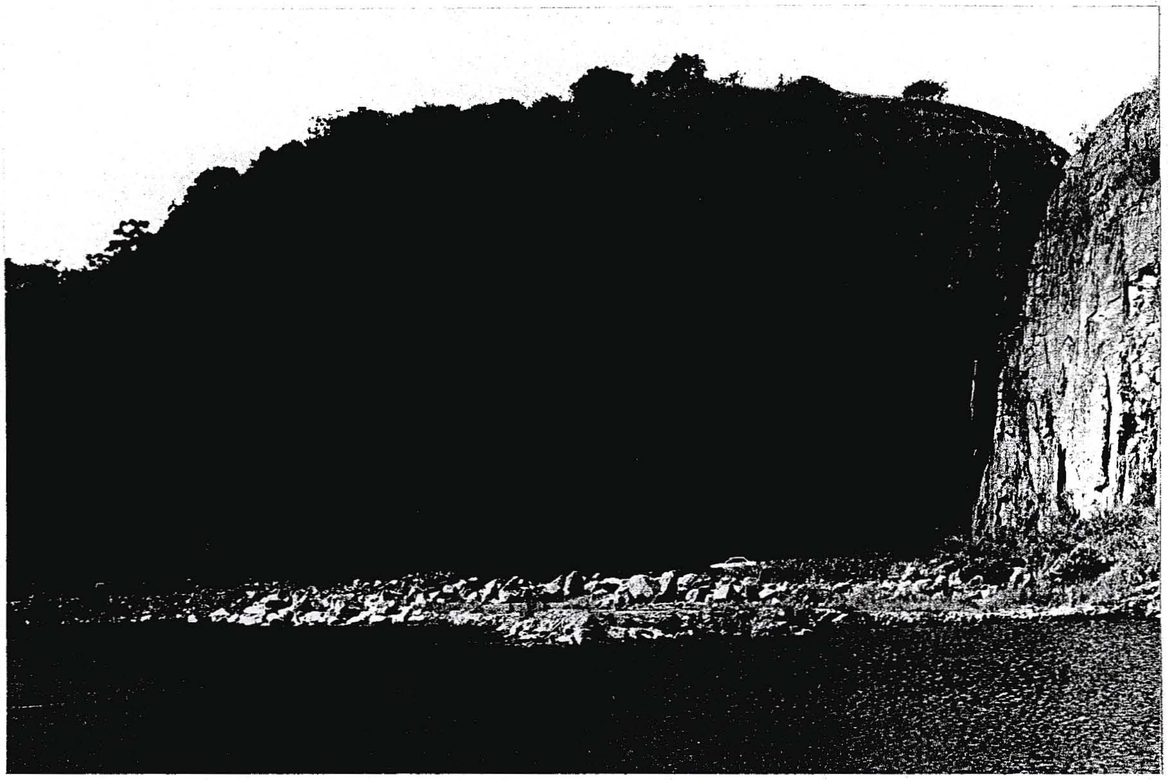
Ridge View Quarry – Face 3

Ridge View Quarry – Face 4

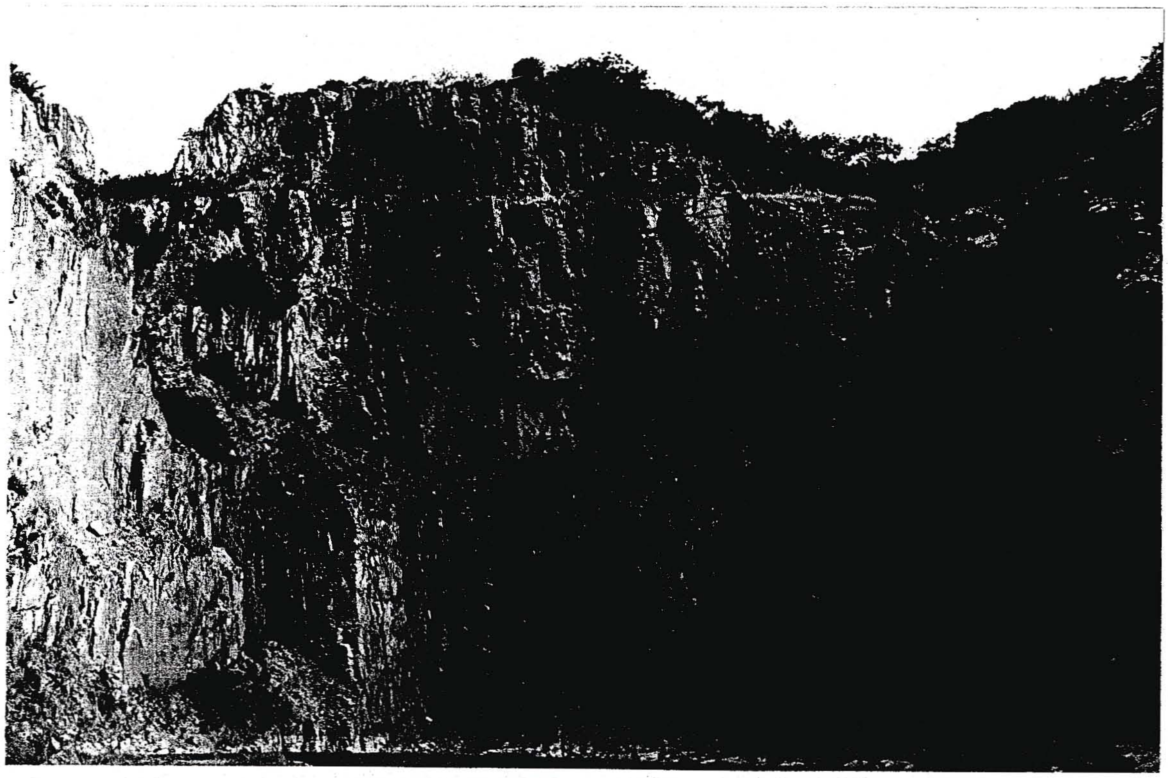
General view of the Umkomaas Quarry (facing south).



Umkomaas Quarry – Face 2.

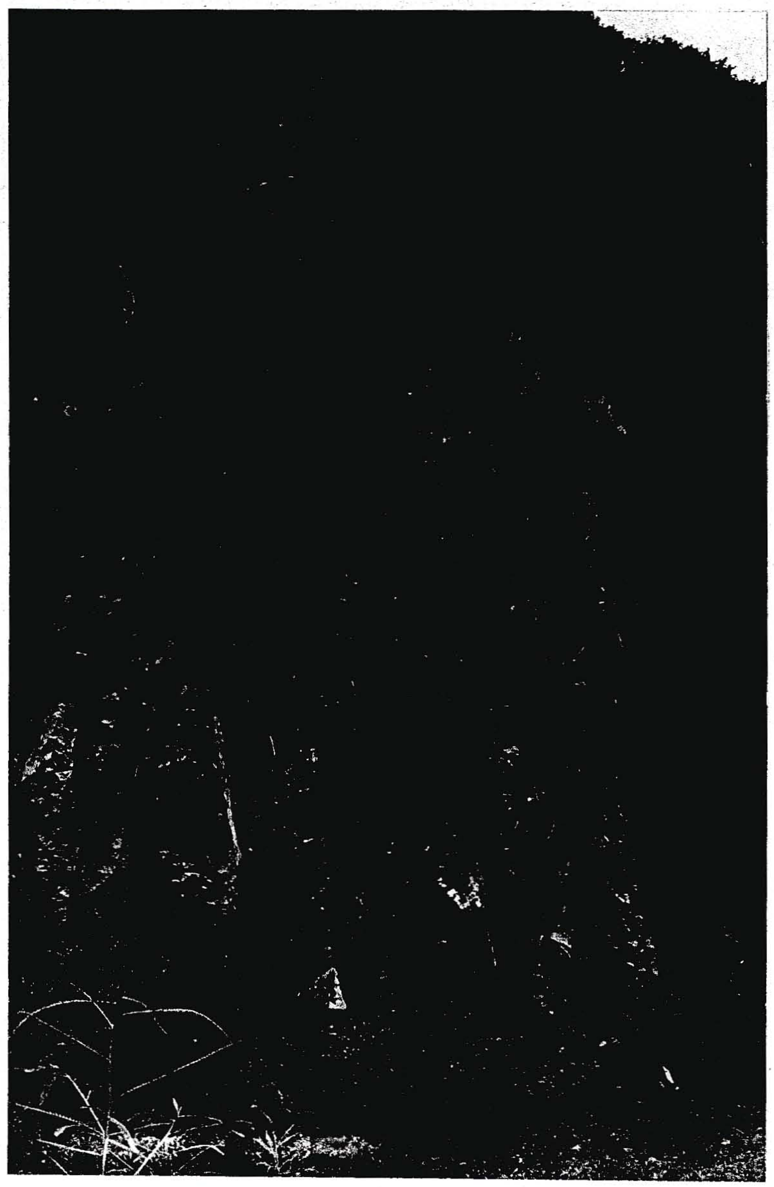


Westville Quarry – Face 1

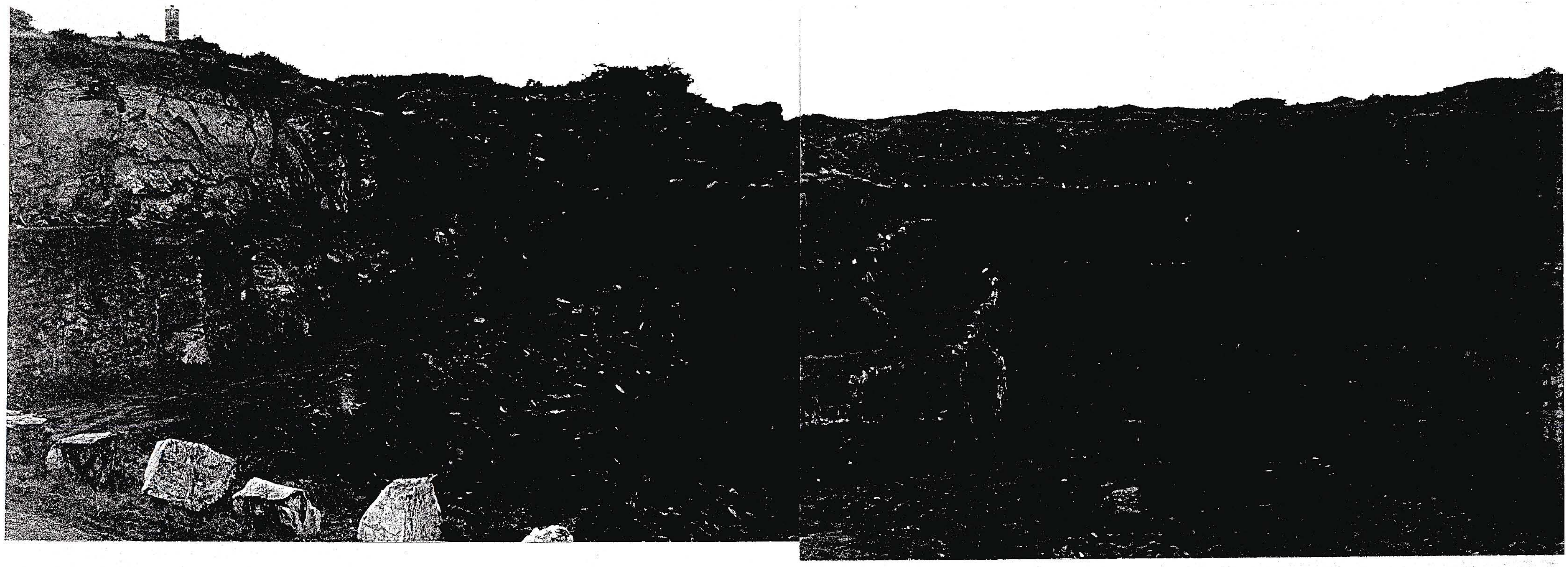


Westville Quarry – Face 4

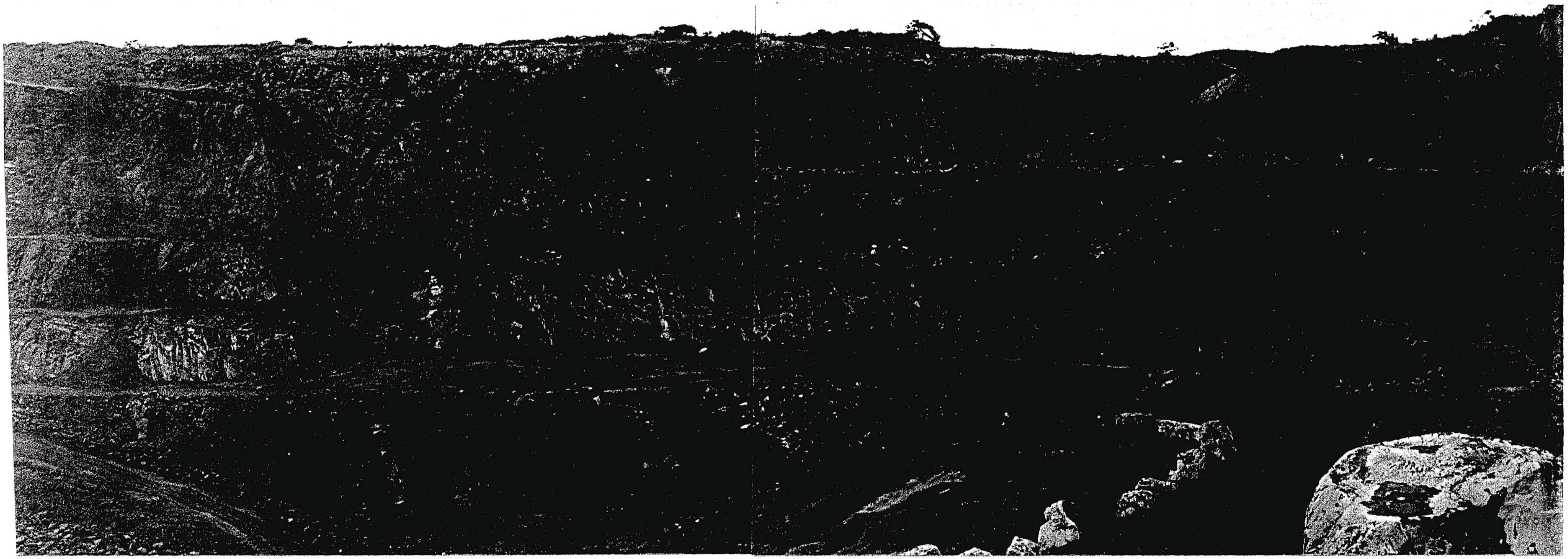
Westville Quarry – Face 2



Westville Quarry – Face 1. Note the blocky nature of tillite due to the jointing.



General overview of the Zululand Quarry (north-western side).



General overview of the Zululand Quarry (north-eastern side).