SAFETY IN MINES RESEARCH ADVISORY COMMITTEE

SIMRAC

Final Project Report

Title: A REASSESSMENT OF COAL PILLAR DESIGN PROCEDURES

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EXECUTIVE SUMMARY

The SIMRAC Project COLO21A entitled "A Reassessment of Coal Pillar Design Procedures" sets out to achieve a coal pillar design procedure that takes cognisance of different geological and structural factors as well as the influence of the surrounding strata.

Three enabling outputs were defined at the commencement of the project. These were, firstly, the determination of the individual seam strength of coal seams in the different coalfields, secondly, a study of pillar foundation failure, and thirdly, techniques and guidelines to ensure safe and stable bord and pillar workings.

The extensive laboratory testing program, on 12 coal blocks from 11 collieries, provided valuable information on the laboratory strength of South African coals. The statistical re-analysis showed that the strength of the eight blocks from the No 2 Seam, Witbank Coalfield occurred in a fairly tight strength range; and that laboratory coal strengths from individual seams or mines could deviate to a significant although relatively small extent from the overall average.

While the laboratory results cannot be directly applied to the field a methodology for the estimation of relative strength between coal seams has been established. This could be of significance when mining a greenfield region.

Results from an investigation into the back analysis of collapsed and intact pillar cases in Australian collieries were summarized. These results are considered highly significant as the resulting design formula is extremely similar to that obtained from South African coalfields. In the study it was concluded that the strength of a pillar, once the pillar width to height ratio

exceeds about 2,0, is increasingly determined by geometry and coal strata contact friction, and less by intrinsic coal strength.

Pillar rating of over 300 panels was conducted. The classification system indicated that there was a variation in effective strength within collieries and across the coalfields. The relative ranking showed that the seams, ordered from strongest to weakest were:

Top-Bottom	Klip River
Bottom	Klip River
Alfred	Vryheid
C	Eastern Transvaal
Dundas	Utrecht
No 7	Soutpansberg
No 4	Highveld
Gus	Utrecht
No 2	Highveld
No 1	Witbank
No 2	Witbank
No 5	Highveld
No 5	Witbank
No 2A	Vereeniging
No 2B	Vereeniging
Main	Zululand
No 3	Vereeniging
Alfred	Utrecht

The relative strength should be treated with caution.

An improvement to the classification system would be the incorporation of the extent of structural discontinuities within the coal pillar.

Three in situ field trials were conducted. Unfortunately the results from the field trials were not as productive as anticipated. Remote and continuous monitoring of field instrumentation is essential for the further understanding of pillar behaviour and systems have been developed to meet this requirement.

The influence of the surrounding strata on pillar strength was recognized and examined. Stable workings require all elements in the system, the roof, pillar and floor or foundation, to be in a stable condition.

Salamon's design formula works well where the coal pillar is the weakest element of the system. However, of the 81 known collapsed pillar cases reported on up to 1988, only 44 were included in previous analysis. Put slightly differently only 54 per cent of known pillar collapses have been adequately taken into account in current pillar design practices!

Collection of information regarding collapsed pillar cases was undertaken in the last six months of the project. Since 1988 at least 12 pillar collapses have occurred in South African Collieries. This important study is not completed and requires further effort in the collection and analysis of vital information.

The project initiated collection of data pertaining to floor heave, but colliery closures curtailed this aspect of the investigation. A collapse in 1995 due to foundation failure provided the opportunity for investigation of the influence of the floor strata on stability.

Given the potential seriousness of foundation failure on pillar stability the collection of index tests on suspect strata, for example at the No. 5 Seam Witbank Coalfield, should be initiated. While this mode of failure may only occur some time after mining the failure has the potential to result in the loss of access ways as experienced by the trapping of miners at Emaswati Colliery.

Two sites were monitored for floor behaviour during the project. Foundation stability is dependent on the floor strata properties and thickness in relation to the pillar width. To obtain these parameters an investigation of shear strength, frictional and other properties are indicated. Additional research is required before design criteria can be applied.

Techniques for ensuring safe and stable bord and pillar workings is to apply the Salamon design formulae, apply the classification system to confirm the performance of pillars formed in a colliery, incorporate an evaluation of the effects of discontinuities; should the performance or discontinuities influence the pillar adjust design. Conduct index and laboratory tests for geoduribility. Should the floor material be found to be susceptible to foundation failure the design of foundation stability becomes a paramount requirement.

Valuable information with regard to pillar behaviour and laboratory strength was recorded during the project duration. The loss of staff during the course of the project was disruptive and highlights the need for continuity of the research effort if further significant progress it to be made.

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1.0 PROJECT STATEMENT

The SIMRAC Project COLO2IA entitled "A Reassessment of Coal Pillar Design Procedures" sets out to achieve a coal pillar design procedure that takes cognisance of different geological and structural factors as well as the influence of the surrounding strata.

The project was initiated in 1993 and this report summarises the work to date. Coal pillar design is complex. Major research projects have been conducted world wide for decades and still continue in the larger coal producing countries. The complexity exists due to the variability in geological factors; and to the often conflicting requirements in the design, that is to maximize extraction while at the same time ensuring stability, often for the long term.

"Long term" adds a further complexity to the equation, as rock strength is known to be influenced by environmental factors, as well as by the loading due to the overlying strata over time. The extent of these influences is not completely known.

Three enabling outputs were defined at the commencement of the project. These were, firstly the determination of the individual seam strength of coal seams in the different coalfields, secondly a study of pillar foundation failure, and thirdly techniques and guidelines to ensure safe and stable bord and pillar workings.

The project investigated several factors influencing coal pillar strength. These studies included:-

- A case study of long term pillar determination
- Laboratory strength tests of coal blocks, and a comparison with Salamon's and Australian field data
- Numerical modelling
- Pillar classification rating

- Collection of case histories of pillar collapse
- In-situ field trials
- Pillar foundation failure

Extensive laboratory tests were conducted on 12 coal blocks from 11 collieries. While the question of extrapolation to full size pillars remains open, the tests did show up some relative differences between seams. A statistical analysis of the laboratory results was encouraging by showing the tight range within blocks from the No. 2 Seam, Witbank Colliery, where eight of the coal blocks were extracted.

A comparison between Australian and South African collapsed pillar cases was conducted by the University of New South Wales, Australia. Their conclusions are of significance and are summarised in this project report.

Numerical Simulation is an important tool in Rock Mechanics. Several models were examined for applicability to coal mining and, where suitable, used in the calculation of convergence of the roof and floor and pillar stress. Modelling was undertaken both for back calculation of the pillar compression and load as well as examining the in situ experimental field trials undertaken.

The characterisation of pillars by a pillar classification system was undertaken. Over 300 panels were visited and the pillars rated according to the amount of pillar scaling, the influence of discontinuities, weakness in the pillar and age of the workings.

Collection of information regarding collapsed pillar cases was undertaken in the last six months of the project. Since 1988 at least 12 pillar collapses have occurred in South African Collieries. This important study is not completed and requires further effort in the collection and analysis of vital information.

Three in situ field trials were conducted, two in pillar extraction panels and a third in a pillar splitting panel. Unfortunately the results from the field trials were not as productive as anticipated. For maximum information instrumentation must be installed well before the extraction operation and, in the case of surface boreholes prior to mining. Production sequencing changes can result in the instrumentation already installed not being in the most suitable position. Also damage of instruments due to mining or goafing can curtail the readings. The results from the field trial were disappointing due to both sequence alteration and instrumentation damage. However, valuable lessons have been learnt and these will be incorporated into future monitoring campaigns.

The influence of the surrounding strata on pillar strength was recognized and examined. Stable workings require all elements in the system to be in a stable condition. Conversely, failure of one element can result in a system failure. The elements for pillar stability are the roof, pillar and floor or foundation.

Salamon's design method is based on empirical data. In the collection of that data, pillar collapses due to foundation failure were excluded, as were cases where weak roof may have induced failure. Similarly, Madden's reassessment of coal pillar design conducted in the mid to late 1980's also excluded these types of collapsed cases. The result is that the Salamon design formula works well where the coal pillar is the weakest element of the system. However, of the 81 known collapsed pillar cases reported on up to 1988, only 44 were included in previous analysis. Put slightly differently only 54 per cent of known pillar collapses have been adequately taken into account in current pillar design practices!

The project initiated collection of data pertaining to floor heave, but colliery closures curtailed this aspect of the investigation. A collapse in 1995 due to foundation failure provided the opportunity for investigation of the influence of the floor strata on stability.

Valuable information with regard to pillar behaviour and laboratory strength was recorded during the project duration. The loss of staff during the course of the project was disruptive and highlights the need for continuity of the research effort if further significant progress it to be made.

2.0 LONG TERM STABILITY OF COAL PILLARS

Long term pillar stability affects the safety of the mine personnel in several ways. Firstly, pillar deterioration occurring in main developments could potentially lead to the isolation of crews. While larger pillars designed to higher safety factors in main developments are industry standards this effect should be investigated. Of greater potential risk is the intended extraction of previously formed pillars. Extensive reserves exist wherein the nominal safety factor was designed to the accepted norm of 1,8. The effect of the passage of long periods of time on pillar strength is required to be known as the extraction of seemingly suitable pillars standing for several decades will be increasingly contemplated in the future.

In October 1995 pillars in the No. 2 Seam, Witbank Coalfield, were inspected at Tavistock Colliery. In one panel the pillars were mined in 1958 prior to the introduction of Salamon's safety factor design formula. The pillars were formed to a calculated safety factor of 1.21. In a report dated July 1988, some 30 years after mining, Dr Carmbly stated that "after very many years the conditions are still excellent." At this stage, 1988, the pillars showed little sign of deterioration. Observations in 1995 however, revealed considerable scaling of the pillars in conjunction with roof falls. Obviously some trigger occurred between 1988 and 1995 to account for the deterioration in panel condition. Mining of an adjacent area did occur in 1991 with a substantial 24m wide barrier between the panels. To examine the possible influence of mining on the older pillars, numerical simulations using BEPIL, MAP3D and MINLAY were conducted. All models showed very small (0,1-0,2 MPa) increases in load on some of the pillars next to the barrier. It was concluded that the mining probably was not the reason for the deterioration.

The panel in question together with surrounding panels were visited and the pillars classified according to the existing rating system developed and reported by Madden (1985).

Studies by Wagner (1974) and Ozan (1992) showed that the failure of an unconfined pillar sidewall is independent of the width to height ratio, whereas the ultimate strength and post-peak behaviour of a pillar are strongly determined by its w/h ratio.

Madden (1991) carried out visual observations of bord and pillar workings. He concluded that meaningful results can be obtained with visual observations over a mining depth range of 50 to 120 m. The panels at Tavistock Colliery, at a depth of about 85 m, were considered suitable for visual classification of pillar classification.

The pillar classification process is based on detailed visual observations of bord and pillar conditions. At Tavistock Colliery, three pillars from each panel except W3N panel were rated using the rating system. The pillar conditions, mining dimensions, fracturing, scaling, roof and floor conditions, support performance, pillar sidewalls and corners as well as the effects of structural discontinuities on the pillar stability were investigated.

The results are shown in Table 2.1 together with photos of the pillars in Figures 2.1-2.10. Figure 2.11 shows a plot of the skin stress versus pillar rating, while Figure 2.12 shows the panel layout. It is suggested that other panels of both low and acceptable safety factors be observed at Tavistock Colliery to ascertain if the observed deterioration is an anomaly or a more frequent and widespread occurrence. The change in pillar condition over a relatively short time span of seven years, after some 30 years of showing little deterioration, requires further study as the impact of time on pillar stability is unknown. The limited pillar rating system at Tavistock Colliery could be expanded and assist in the determination of the effects of time.

Table 2.1. (A) The Visual Observations of the Pillars at Tavistock Colliery

SECTION	AGE OF PILLAR (YEAR)	PILLAR SIDES	PILLAR CORNERS	STONE DUSTING	TIME SINCE DUSTING	WEAKNESS IN PILLAR	COAL WEATHERING	DISCONTINUITY EFFECT ON PILLAR	TIME FACTOR	PILLAR PUNCHING	RATING	SIDE WALL STRESS (MPa)
W1N(A)	37	25	50	0	0	10	20	100	1.48	100	423 poor	6.88
W1N(B)	37	75	100	0	0	20	20	100	1.48	100	586 good	5.45
W1N (C)	37	75	75	0	0	0	20	100	1.48	100	514 fair	6.59
W2N(A)	24	75	50	0	1	10	0	100	1.45	100	486 fair	5.9
W2N(B)	24	100	75	0	1	15	20	100	1.45	100	565 good	5.73
W2N(C)	24	75	75	0	1	10	20	100	1.45	100	522 fair-good	6.38
W3N	21	100	100	5	1	20	20	100	1.44	100	611 very-good	5.54

(B) The Dimensions of the Pillars and the Panels

SECTION	PILLAR WIDTH (m)	BORD WIDTH (m)	MINING HEIGHT (m)	MINING DEPTH (m)
W1N(A)	6.5	7.2	3.5	88
W1N(B)	9.8	6.4	3.7	88
W1N (C)	6.9	6.8	3.4	88
W2N(A)	7.4	6.3	3.5	88
W2N(B)	8.5	6.5	3.5	88
W2N(C)	7.0	6.7	3.6	88
W3N	9.3	6.7	3.5	84

Figure 2.1. W1N (A) Rating 423 Poor. Roadway Profile

Figure 2.2. W1N (A) Rating 423 Poor. Showing Pillar Scaling Corners

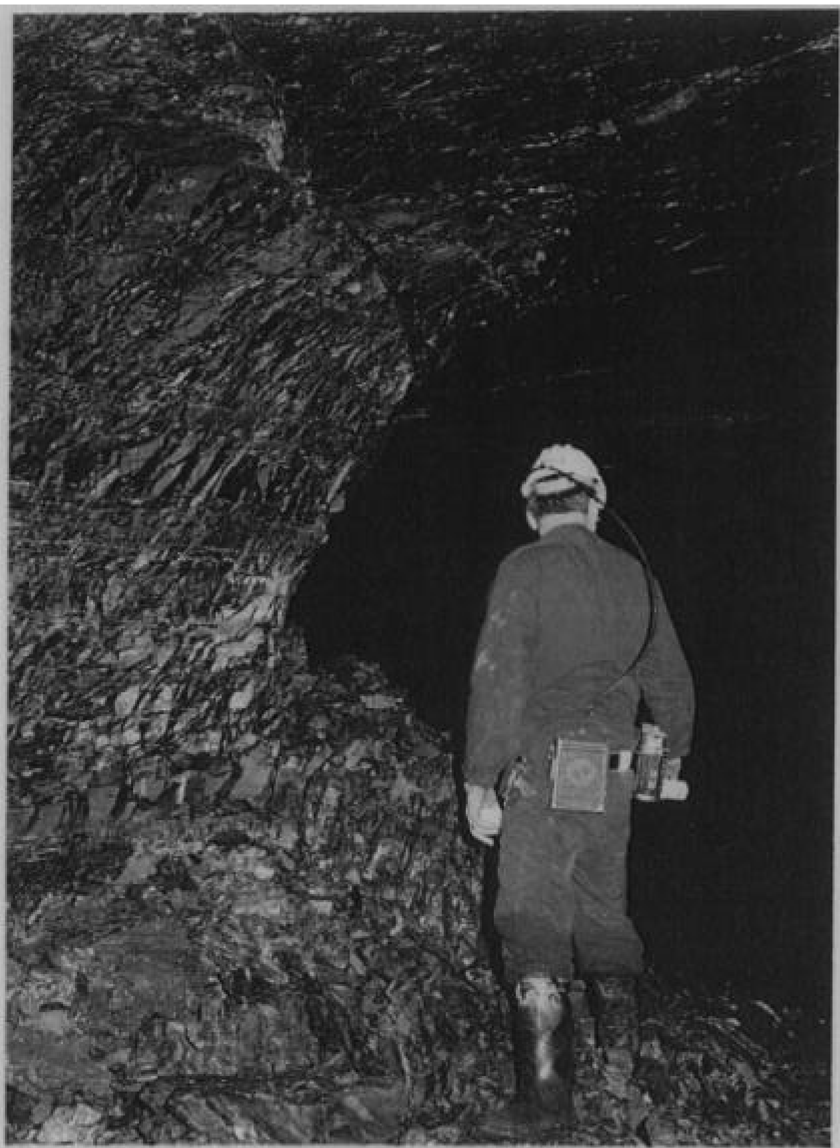


Figure 2.3. W1N (A) Rating 423 Poor. Scaling of Side and Corners

Figure 2.4. W1N (A) Rating 423 Poor. Pillar Scaling

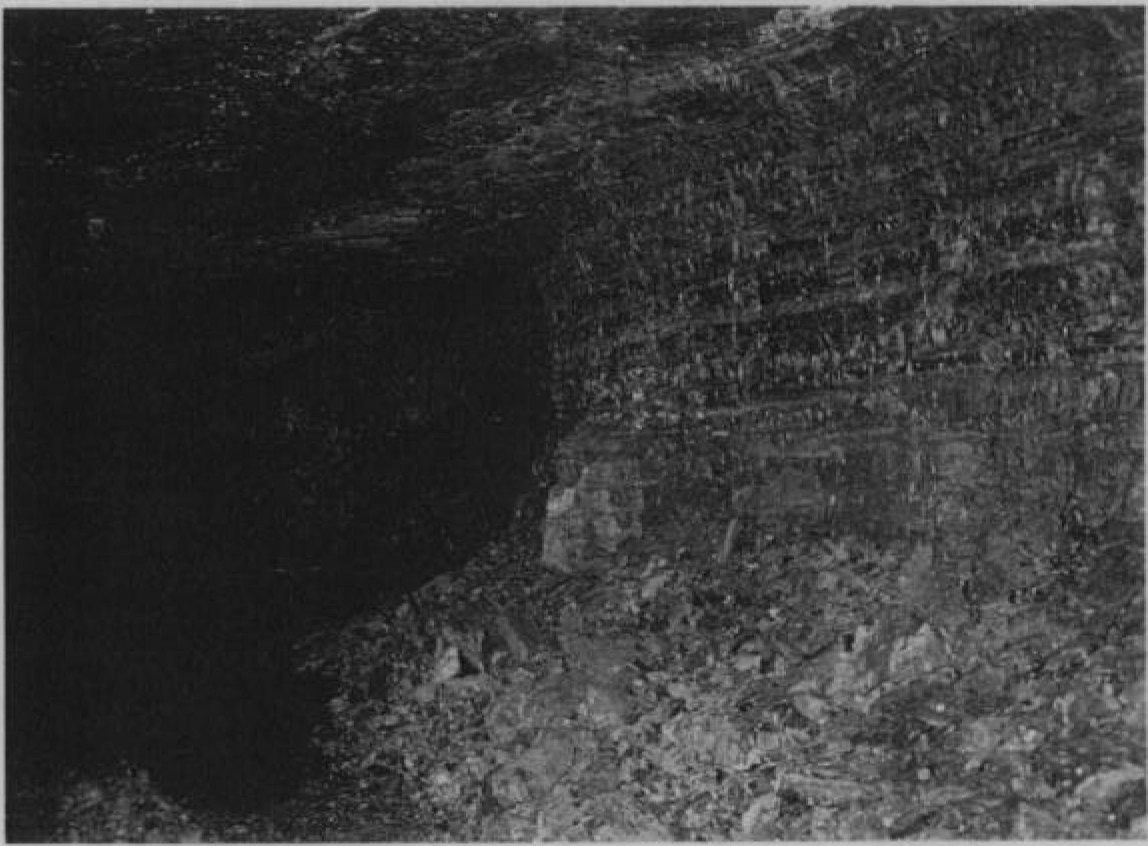


Figure 2.5. W1N (B) Rating 586 Good. Scaling of Side and Corner

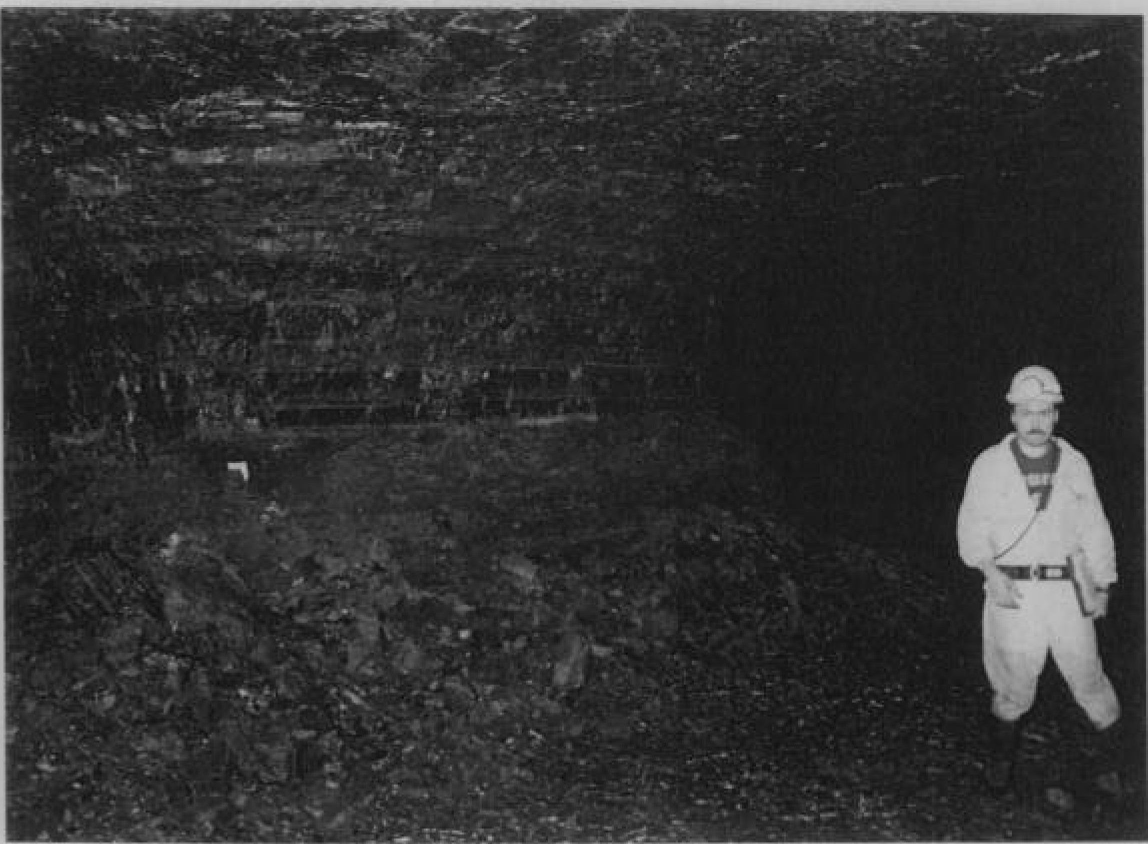


Figure 2. 6. W1N (C) Rating 549 Fair. Pillar Scaling

Figure 2.7. W2N (A) Rating 486 Fair. Pillar Scaling

Figure 2.8. W2N (A) Rating 486 Fair. Scaling of Side and Corner

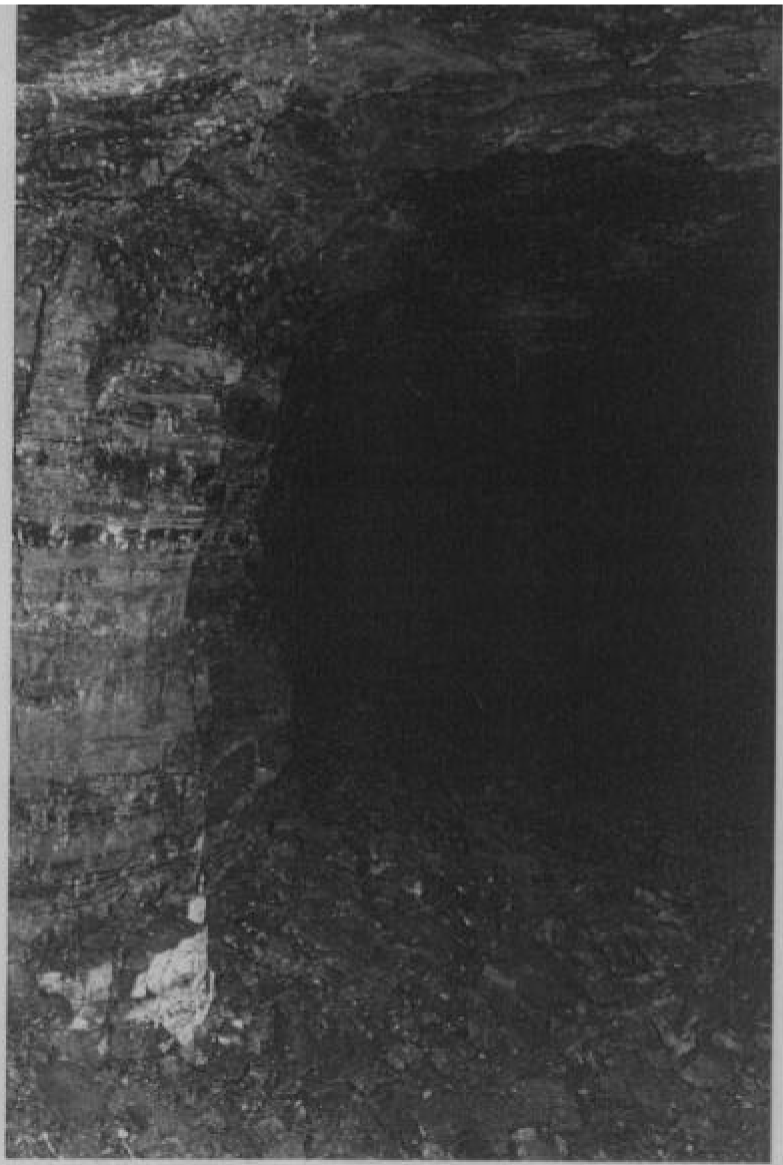


Figure 2.9. W2N (B) Rating 565 Good. Pillar Side Scaling

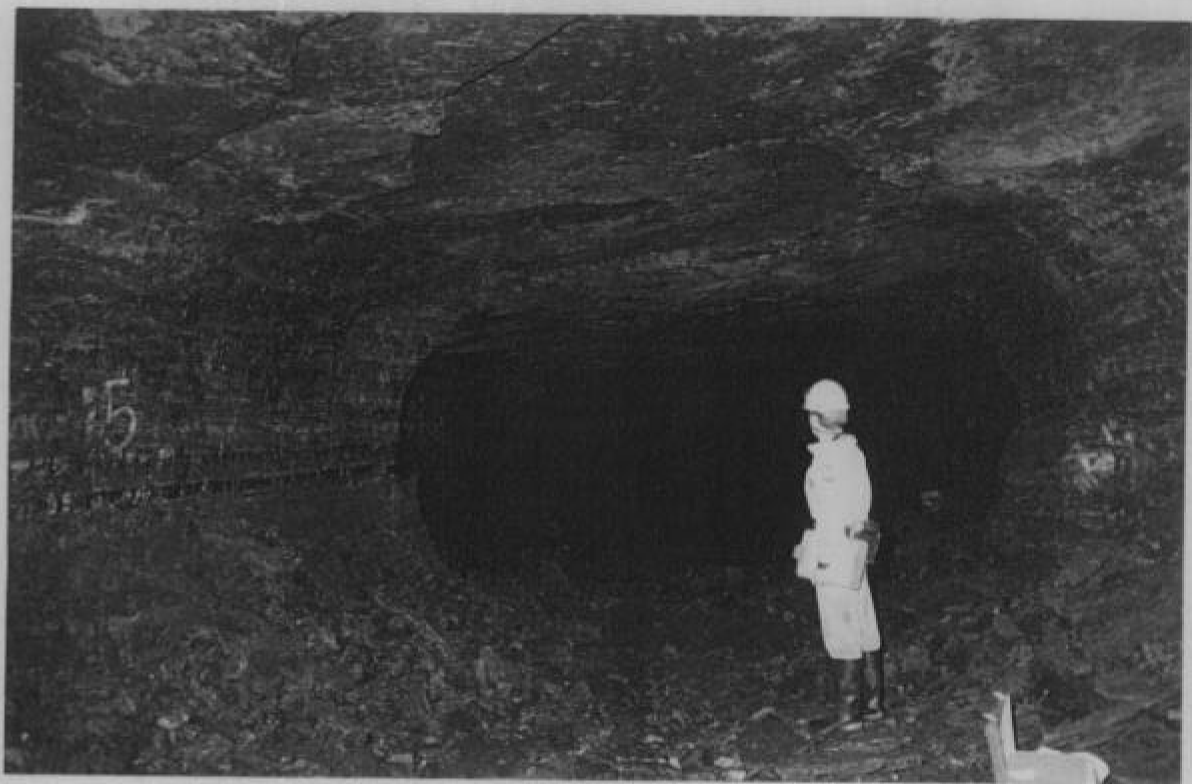


Figure 2.10. W2N (C) Rating 522 Fair - Good. Roadway Profile

Figure 2.12 Panel Layout of Tavistock Colliery No 2 Seam.

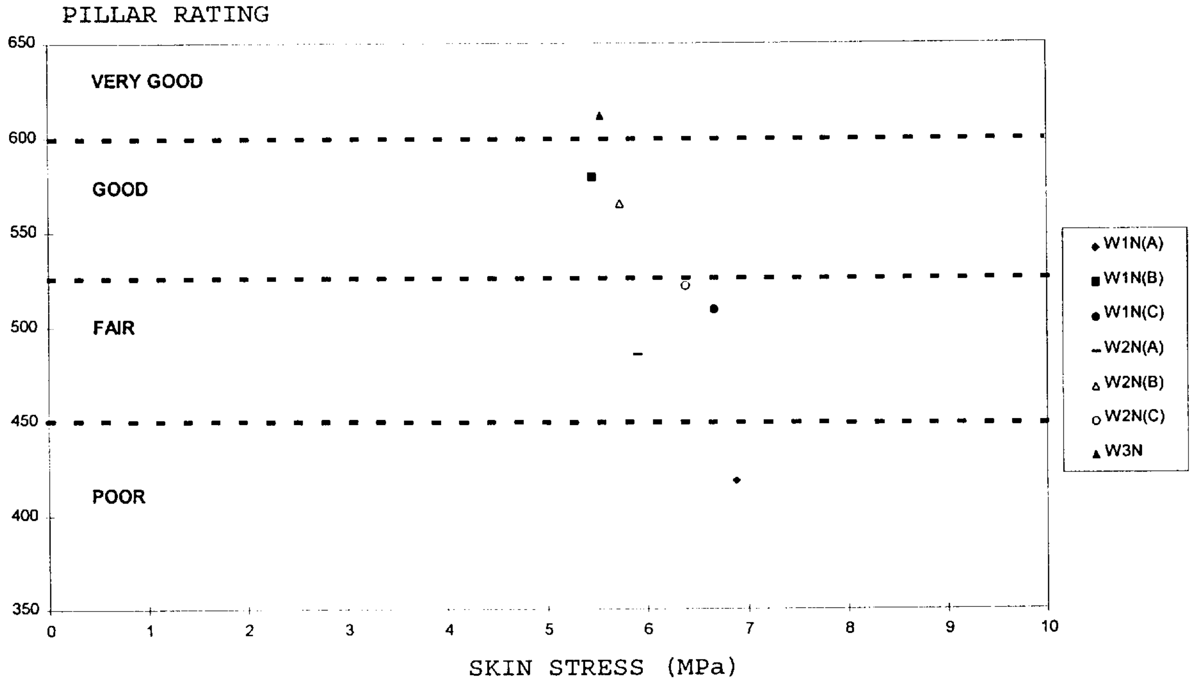


Figure 2.11. Pillar Rating versus Skin Stress, Tavistock Colliery

The question of time has been raised by van der Merwe (1993) in his paper evaluating the strength of coal pillars in the Vaal Basin. An approach worth consideration may be to re-examine other panels in which previous knowledge of pillar conditions were known. For example, several panels were assessed in 1985 for the extent of fracturing during an investigation into the effects of mining method on coal pillars. These experiments were conducted over 10 years ago. Re-evaluating these conditions may assist with the problem of assessing pillar deterioration over time.

3.0 SEAM AND PILLAR STRENGTH FORMULAE

3.1 Reanalysis Of Laboratory Test Results

Extensive laboratory testing of coal samples was conducted during the project. Some 930 tests from 5 seams in 11 collieries and 9 coalfields were compiled in the end of year report 1994 by Canbulat and Akermann and the mid-year report 1995 by Madden and Canbulat. The test sample ranged in diameter from 0,025 - 0,3 m and in the width to height ratio over the range 1,0 - 8,0.

Initially the methodology aimed at determining site specific strength formulae which incorporated the testing of samples prepared from a block of coal selected from the seam within a colliery. The questions of repeatability and how representative a 1,5 m^3 coal block is of the full seam and indeed the mine are yet to be answered. Site specific strength formulae based on laboratory testing has been extensively debated in the past. One of the main problems with this approach is the danger of dilution of the substantial data base of experience. The approach was shifted to examining the potential of laboratory testing to distinguish between seam strengths and relate the results to the field design.

To achieve this, the laboratory test results were re-analysed statistically. The re-analysis initially combined all the test results and obtained a relationship between predicted strength and for sample width and height. In this manner a coefficient of strength for each block was obtained. The mean strength and variance for the samples tested from each block were then compared and the relative variation in strength between each block was obtained.

The laboratory test results were then analysed fixing the constants for width and height to those obtained by Salamon from the statistical analysis of collapsed and intact pillar cases. A comparison was made between the two methods on individual strength of the coal blocks.

Comments on potential bias of the laboratory testing process and how the results can be of practical use were examined as were the relationship between the laboratory test results and those obtained by Salamon from the analysis of full sized pillars.

To gain additional insight into the results from the laboratory tests obtained earlier in the project the data was reanalysed by statistical methods. Some of the detailed results of this re-analysis are given in Appendix A, but the key points arising were as follows:-

1 - A common form of relationship with width and width to height ratio across blocks provides an adequate fit to the data with a coefficient varying according to the strength of the coal. The form of relationship is

$$\text{Strength} = \delta \text{ x width}^{0,139} / \text{height}^{0,449} \qquad \text{(MPa)} \qquad (3.1)$$

width and height of the laboratory samples are in metres.

The coefficient delta (δ) is as follows for the various collieries (the overall average value being 15,83 MPa).

Table 3.1 Delta (δ) Values For The Coal Blocks With "Optimized" α and β

Colliery	Seam	δ
Arnot	No 2	15.5
Bank	No 2	16.2
Blinkpan	No 2	14.7
Delmas block 2	No 2	16.6
Delmas block 1	No 2	18.8
Goedehoop	No 2	16.1
Greenside	No 2	15.4
Khutala	No 2	17.0
Kriel	No 4	14.3
Secunda	4C Lower	14.3
Sigma	No 2A	13.9
Zululand Anthracite	Main	17.2
Overall Average		15,83

2 - The strength of the different blocks exhibits consistent variation. The analysis of variance for between-blocks variability is presented overleaf. The variance between blocks is a highly significant effect.

Table 3.2. Variance between Blocks

Source	Sum of squares	degrees of freedom	Mean square
Between blocks	7.41	11	0.6736
Residual	48.99	917	0.0534
TOTAL	56.40	928	0.0608

3 - The strength of the different seams exhibits consistent variation, with No 2 Seam, Witbank Coalfield being stronger than the other blocks tested. The analysis of variance for between-seams variability is presented below. The variance between seams is a highly significant effect.

Table 3.3. Variance between Seams

Source	Sum of squares	degrees of freedom	Mean square
Between seams	4.11	4	1.0275
Residual	52.29	924	0.0566
TOTAL	56.40	928	0.0608

4 - The analysis of variance for between-blocks variability for all blocks drawn from the No 2 Seam, Witbank Coalfield is presented overleaf. This effect is also highly significant, although the variability is lower than for all blocks. This implies that blocks drawn from within the No 2 Seam, Witbank Coalfield are somewhat more similar in strength than blocks drawn at random.

Table 3.4. Variance between No 2 Seam Blocks

Source	Sum of squares	degrees of freedom	Mean square
Between blocks	3.30	7	0.4710
Residual	27.53	616	0.0447
TOTAL	30.83	623	0.0495

5 - The analysis of variance for between-blocks variability for all blocks drawn from seams other than the No 2 Seam, Witbank Coalfield is presented below. This effect is also highly significant, and the variability between blocks is similar to that for all blocks including the No 2 Seam.

Table 3.5. Variance between non No 2 Seam Blocks

Source	Sum of squares	degrees of freedom	Mean square
Between blocks	2.31	3	0.7697
Residual	21.46	301	0.0713
TOTAL	23.77	304	0.0782

6 - The improvement which can be achieved by fitting a separate form of model to each block, i.e. different exponents for width and width/height ratio, is relatively small. The reduction in the residual mean square, or error variance, is from 0,0528 for the common form of model to 0,0473 to the separate form of model. It is recommended that the common form of model is preferable as it is more robust, simpler to apply and there is limited evidence for specifying a different form of model for each block. Considering that the common model is also based on far more observations, the predictions which can be made using it are subject to a lower error variance. For example, the prediction for the average log(strength) of a block 1,0 metre in width with a width to height ratio of 1,0 at Arnot Colliery is subject to a standard deviation of 0,084

(strength ±17,6%) using a model for Arnot only, whereas it is subject to a standard deviation of 0,036 (strength ±7,4%) based on the common relationship.

7 - Another way of looking at the significance of fitting separate models to each block is by examining the variation between the regression coefficients obtained. The tables below present an analysis of variance on the coefficients for width and width to height ratio, which show that there is significant variation between these coefficients.

Table 3.6. Regression Coefficients

Source	Sum of squares	degrees of freedom	Mean square
Between blocks	0.066	11	0.0060
Residual	1.146	917	0.0012
TOTAL	1.212	928	0.0013

Source	Sum of squares	degrees of freedom	Mean square
Between blocks	0.060	11	0.0055
Residual	1.155	917	0.0013
TOTAL	1.215	928	0.0013

8 - In order to establish the relative strength of a new coal block to ±10% at a 95% confidence level, 23 samples would have to be tested. To reduce the uncertainty range by a factor of 2, the number of samples would have to be multiplied by 4. In the current set of tests, the 95% range for coal strength is generally of the order of ±5 or 6%.

9 - Some anomalous readings of strength were noted in the dataset for results from the blocks obtained from Sigma and Greenside Collieries. These do not affect the overall results of analysis to a great extent, although it would be worthwhile to investigate the outlying readings.

10 - The range for predicting the strength of a 1 m cube block at the 95% level is ±8% for most mines. Individual cubes of this dimension would be expected to exhibit a variation in strength of ±60%. However, these results should be treated with caution as this involves extrapolation well outside the range of the data, and different physical behaviours may come into play.

The series of samples from 12 coal blocks were tested for strength as a basis for establishing a relationship between geometrical properties (width or diameter and length or height), coal type and the strength. For each block, a balanced experiment was conducted with respect to the width (diameter) and width/height ratio of the samples. These variables were therefore used as the basis for defining the geometry of the sample.

The relationship previously fitted to accommodate geometrical variations as well as differing coal properties between the different blocks was given in equation (3.1).

The goodness of fit of this relationship was determined by considering the error variance after fitting the model. This was 0,0535 on the natural logarithm of sample strength.

On the suggestion of Professor Salamon, it was decided to fit a model for strength based on fixed parameters α and β. The values to be used were 0,46 and 0,66 as these values had resulted from the statistical analysis of coal pillar failures conducted by Salamon and Munro. The model fitted in this way was as follows.

Strength = δ^1 x width ^{0,46} / height ^{0,66} (MPa) (3.2)

δ^1 values for the various collieries are shown in Table 3.7.

Table 3.7. δ^1 Values For The Coal Blocks With "Salamon" α and β

Colliery	Seam	δ^1
Arnot	No 2	15.8
Bank	No 2	17.6
Blinkpan	No 2	15.3
Delmas block 2	No 2	17.0
Delmas block 1	No 2	24.0
Goedehoop	No 2	17.3
Greenside	No 2	15.4
Khutala	No 2	17.8
Kriel	No 4	15.6
Secunda	4C Lower	14.9
Sigma	No 2A	17.8
Zululand Anthracite	Main	19.9

The goodness of fit of the model expressed in terms of the error variance in the natural logarithm of sample strength has deteriorated to some extent to a value of 0,0814.

Figures 3.1 and 3.2 provide a graphic representation of the difference in error variance between the model fitted with "optimized" α and β and the model using Salamon's α and β. The increase in scatter around the 45 line is apparent. Figure 3.2 also provides evidence of conditional bias in the residuals provided by the "Salamon" model, with underprediction at low strengths and overprediction at high strengths.

More detailed examination of the residuals explains why this is the case, Figure 3.3 shows how the "optimized" model adequately explains the variability with respect to the sample width. By contrast, Figure 3.4 shows how the "Salamon" model leaves a distinct trend of unexplained variability underpredicting strength at low widths and overpredicting strength at large widths.

A similar conclusion is drawn from Figures 3.5 and 3.6 with respect to the sample width to height ratio. The "optimized" model residuals are unbiased over the range of the data, whereas the "Salamon" model underpredicts strength at low values of the ratio and overpredicts at high value.

The average conditional biases exhibited by the "Salamon" model with respect to width and width to height ratio are summarized in Table 3.8. A positive number represents a percentage overprediction of strength by the model.

Table 3.8. Strength Prediction by Salamon Field Model to Laboratory Results

		Width/height ratio		
		1	2	5
Width (m)	0.1	-14.2	-1.7	+12.8
	0.2	-7.6	+4.2	+17.9
	0.5	+0.6	+11.5	+24.2

Within the range of the experimental data, there is no overall bias. However, when extrapolation outside the range of experimental data is attempted, considerable differences in the predicted strength of a meaningful sized pillar are obtained. Table 3.9 provides a detailed analysis of the difference in predicted strength for each of the twelve blocks for which data is available.

Table 3.9. Comparison of Optimized Model with Original Salamon Formula

		Strength (MPa)	
Width (m)	Width/Height	Optimized	Salamon original
0,1	1	32,4	11,4
0,1	2	44,2	18,0
0,1	5	66,7	32,9
0,2	1	26,1	9,9
0,2	2	35,7	15,6
0,2	5	53,8	28,6
0,5	1	19,7	8,2
0,5	2	26,8	13,0
0,5	5	40,5	23,8
1,0	1	15,9	7,2
1,0	2	21,7	11,3
1,0	5	32,7	20,8
2,0	1	12,8	6,2
2,0	2	17,5	9,9
2,0	5	26,4	18,1
5,0	1	9,6	5,2
5,0	2	13,1	8,2
5,0	5	19,8	15,0
10,0	5	16,0	13,1
20,0	5	12,9	11,4

In all cases, the "Salamon" model predicts a higher strength for blocks of pillar size than the "optimized" model. While extrapolation outside the range of experimental data is considered to be a dangerous practice, the "optimized" model has the merit of providing what may be regarded as conservative estimates of pillar strength considering that the Salamon and Munro model was derived for large values (greater than 3,0 m, whereas the present laboratory sample range was only 0,025 - 0,3 m.)

It should also be noted that the predictions provided by the "optimized" model are not entirely dissimilar from Salamon's original formula. Salamon's smaller k value coupled with different α and β give rise to a similar function over a range of meaningful pillar sizes. Table 3.9 gives a comparison between the average "optimized" model strength predictions and the original Salamon formula over a range of widths and width to height ratios. At a width of 5,0 m and a width to height ratio of 5, the percentage difference in predicted strength is 32%, at a width of 10 m the difference drops to 22% and at a width of 20 m the difference is 13%.

The reasons for the difference in α and β between the "optimized" model and the "Salamon" model can only be speculated on based on the statistical evidence available. However, possible explanations lie in some or all of the following areas:

- circular cross section blocks follow a different relationship than square section pillars
- preferential extraction of competent elements from within the coal seam lead to different failure modes of the blocks than in bulk composition
- a different form of relationship applies to smaller widths and heights and this cannot be scaled up (e.g. the "scale effect is much stronger for small specimens than it is for pillar sized blocks."
- loading characteristics in experimental tests differ from field loading conditions

In conclusion, the "optimized" α and β model should be preferred to model the failure strength of laboratory sized samples (widths from 0,1 to 0,3 m and width to height ratios from 1,0 to 8,0). However, this does not provide any evidence that the "Salamon" α and β are inappropriate for blocks of realistic pillar size.

The relatively small differences between the predictions obtained from extrapolated laboratory results using the optimized model and Salamon's field data on meaningful size pillars represents encouraging, though not conclusive, evidence that the laboratory strength results could provide a useful input to coal pillar design procedures, in particular to distinguish coals of inherently different strengths. The suggestion is that the original Salamon formula possibly provides conservative strength predictions which may in some circumstances result in an over-design. However, the differences in strength observed between the different seams are relatively small (Figure 3.7) and consequently changes in the design formula would be minor.

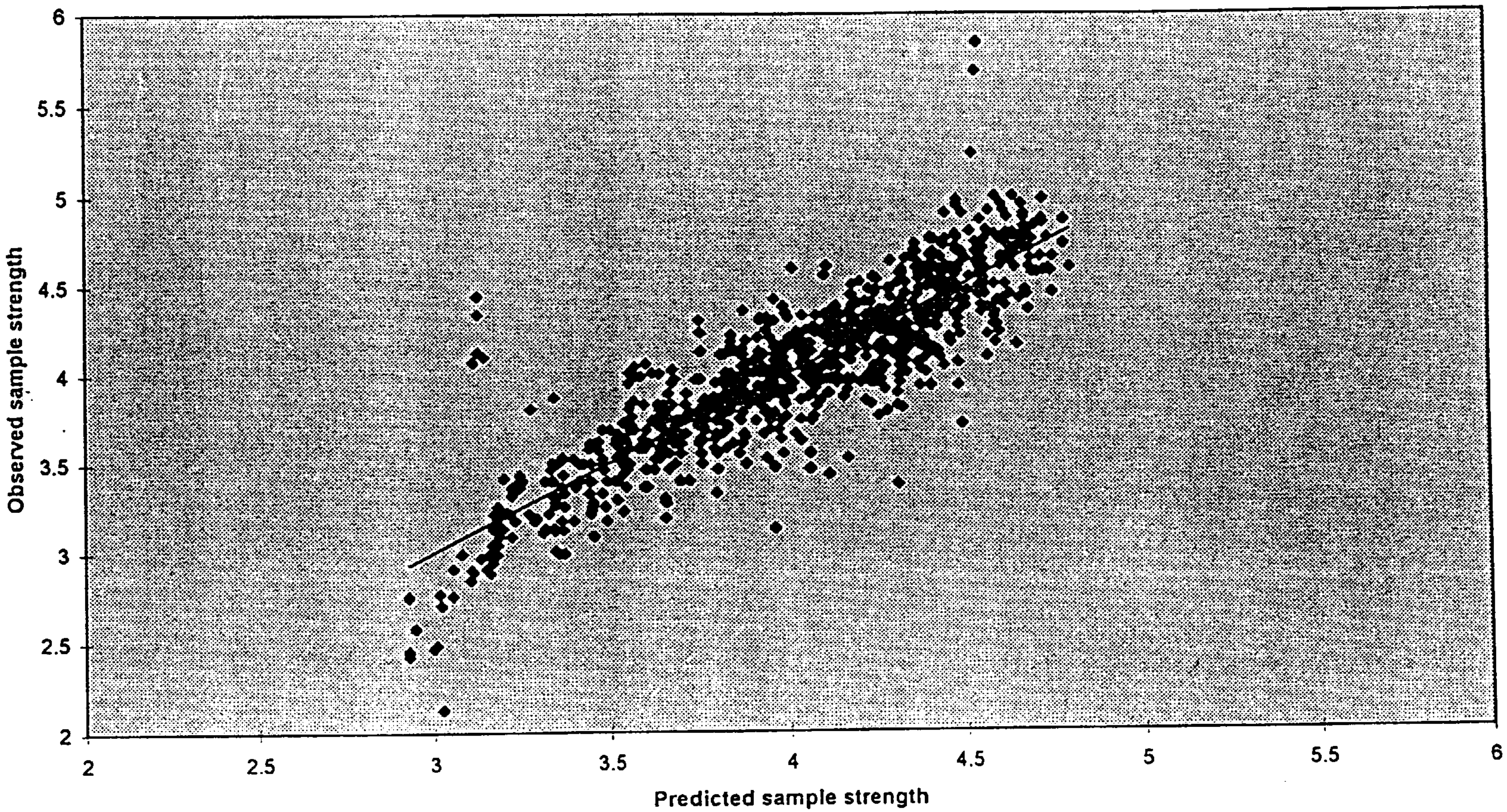


Figure 3.1. Comparison of Predicted and Observed Strengths (Natural Logarithms-Optimized Alpha and Beta)

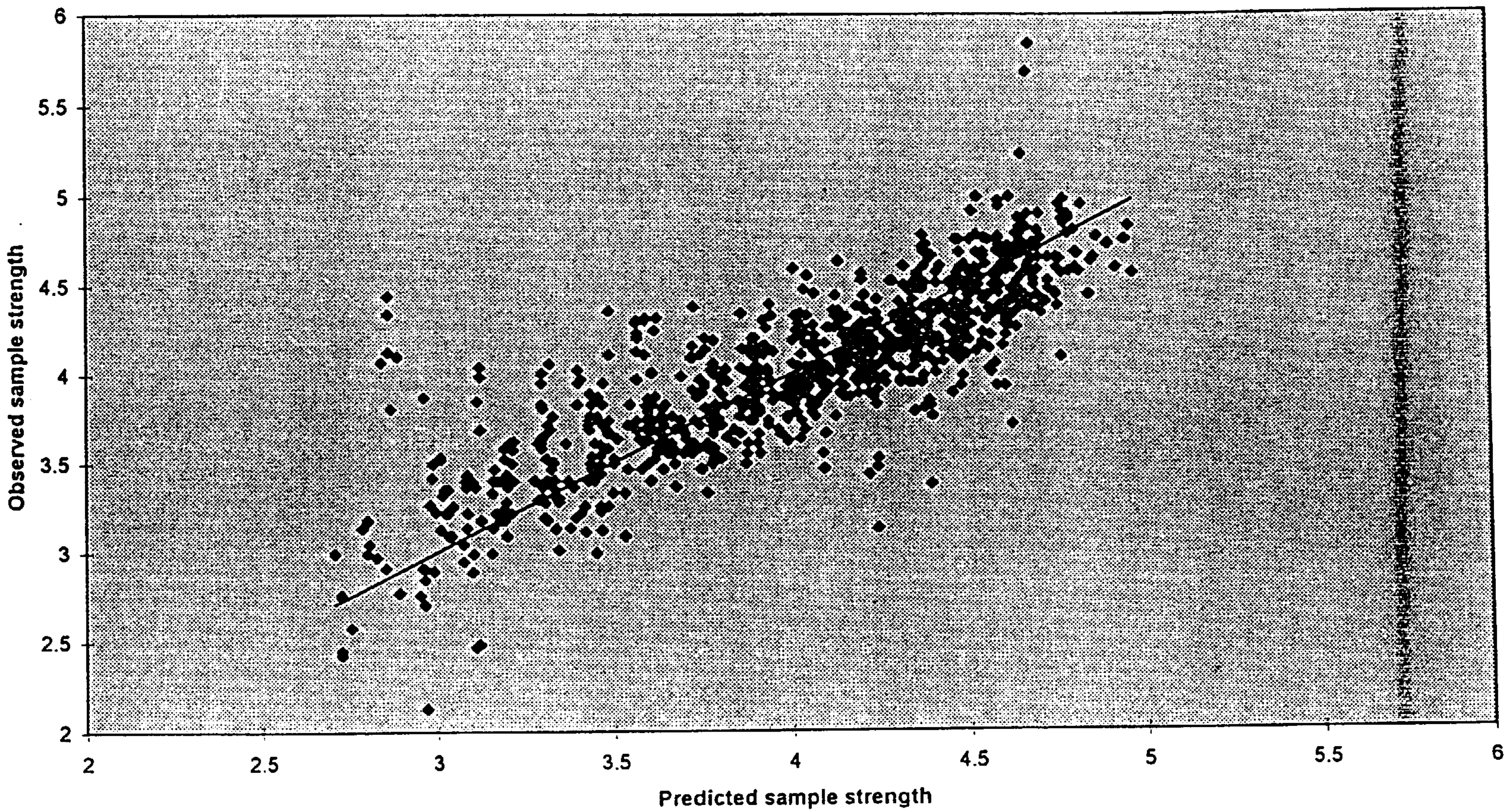


Figure 3.2. Comparison of Predicted and Observed Strengths (Natural Logarithms-Salamon Alpha and Beta)

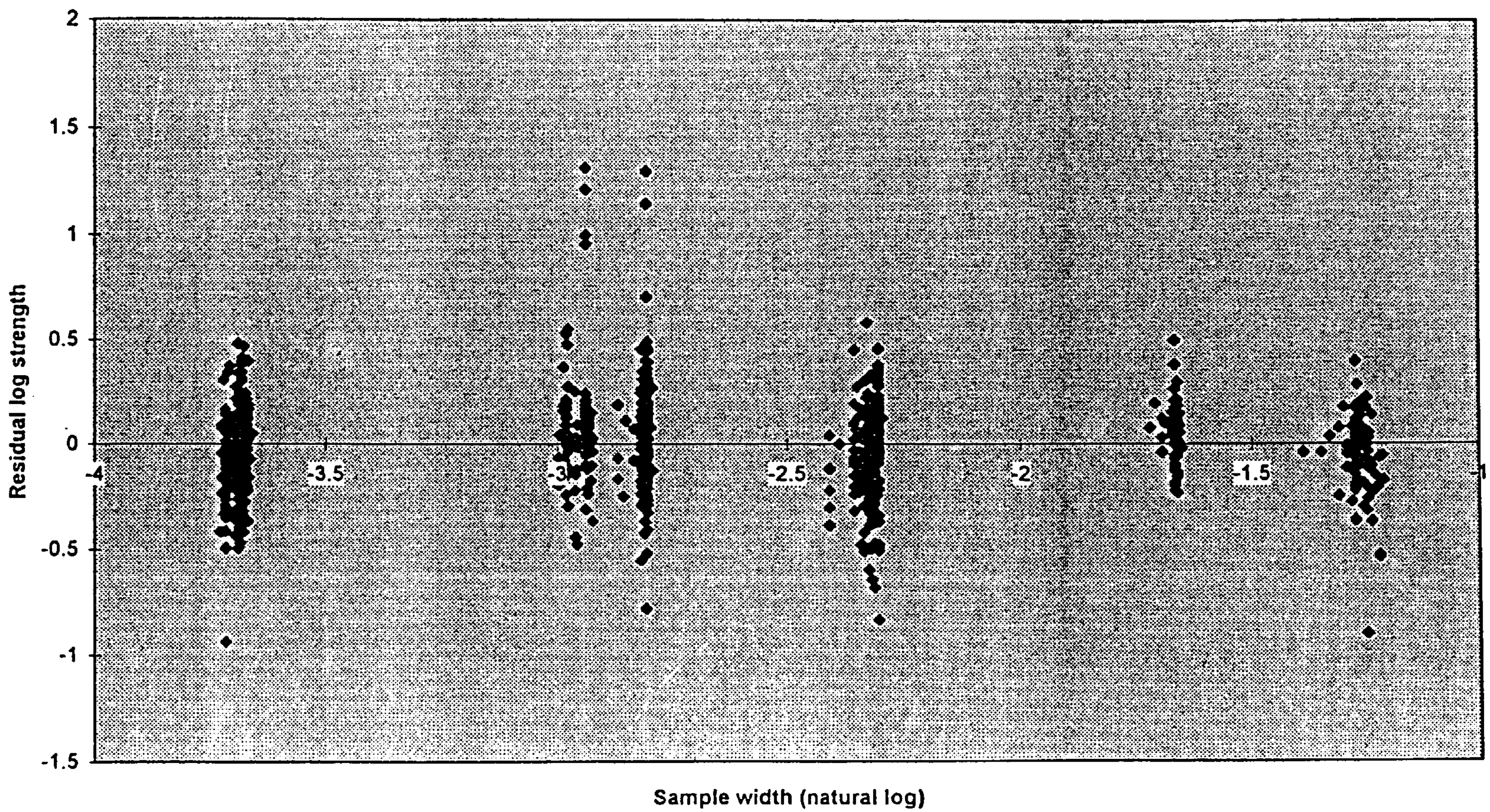


Figure 3.3. Dependence of Residual Strength on Sample Width (Observed Strength-Predicted Strength)-Optimized Alpha and Beta

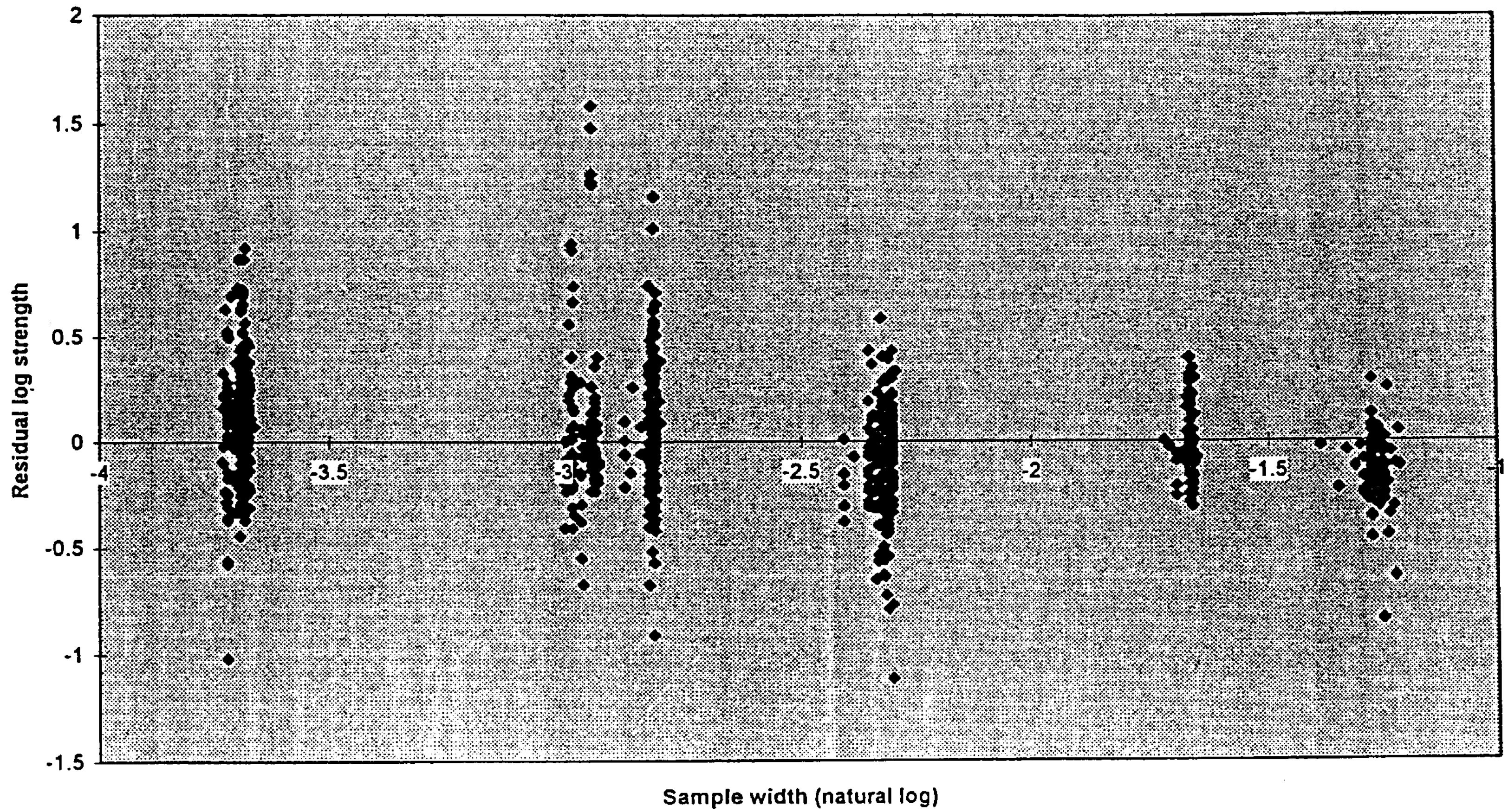


Figure 3.4. Dependence of Residual Strength on Sample Width (Observed-Predicted strength)-Salamon Alpha and Beta

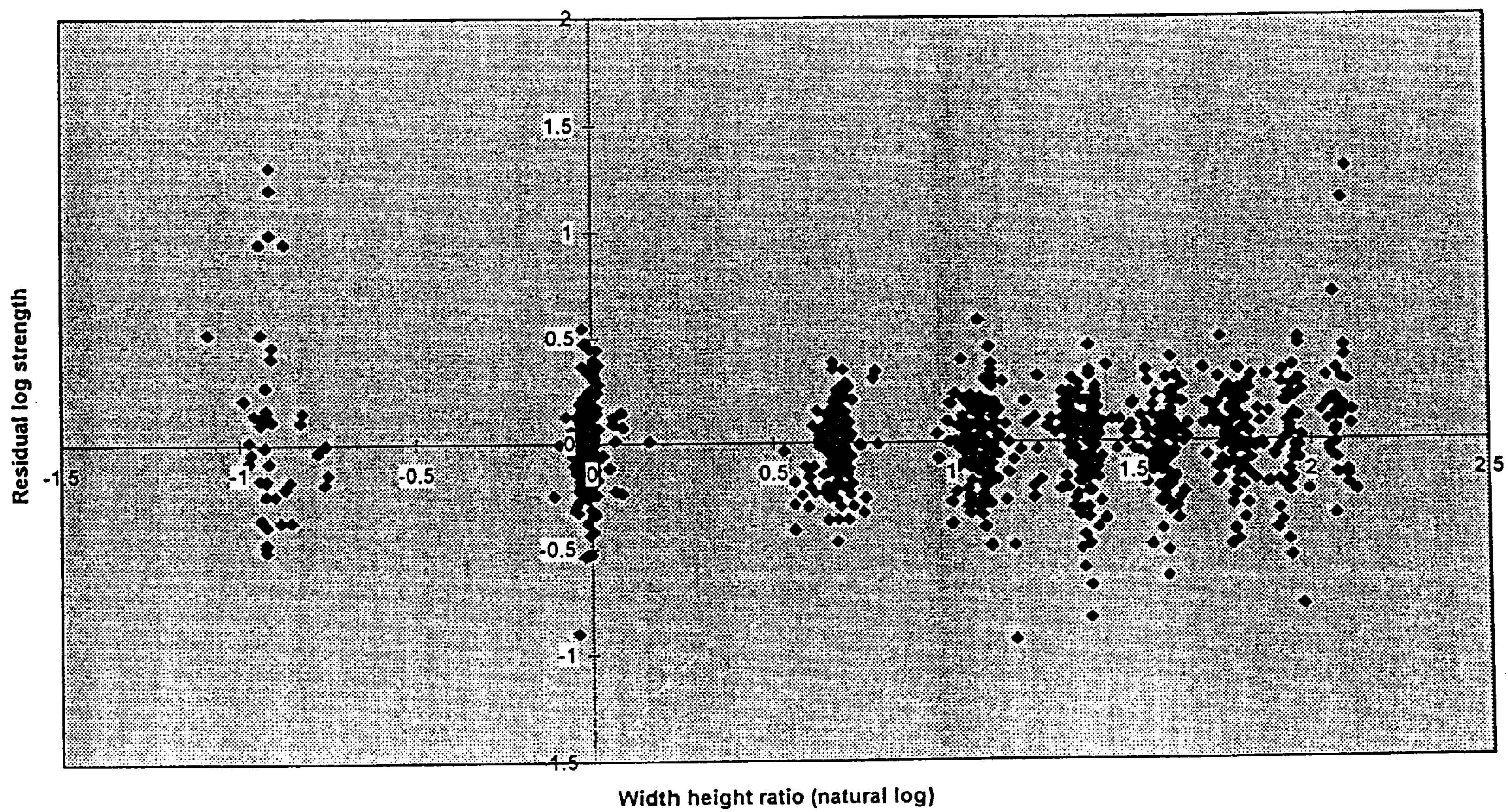


Figure 3.5. Dependence of Residual Strength on Width:Height Ratio (Observed Strength-Predicted Strength)-Optimized Alpha and Beta

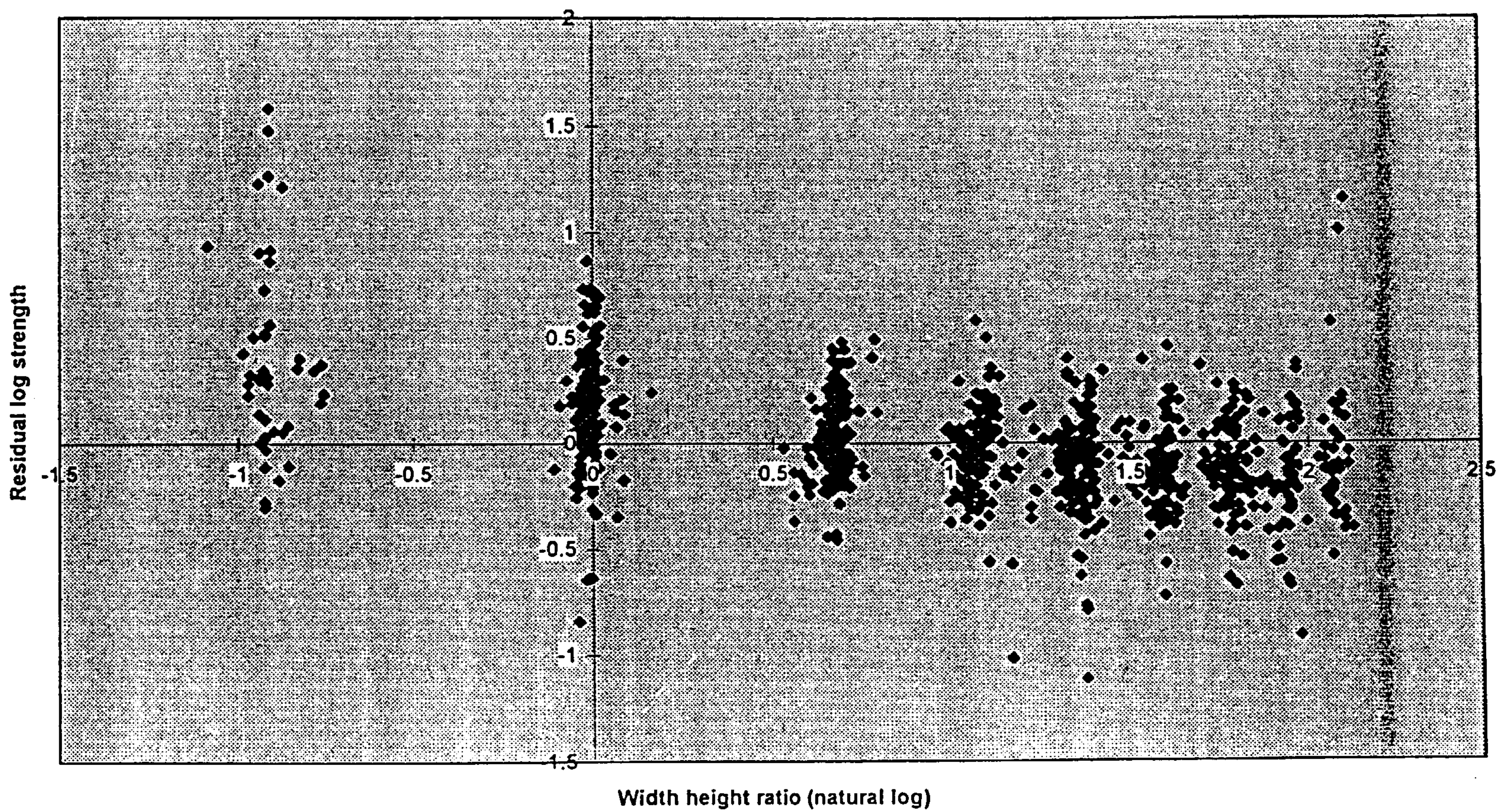


Figure 3.6. Dependence of Residual Strength on Width:Height Ratio (Observed Strength-Predicted Strength)-Salamon Alpha and Beta

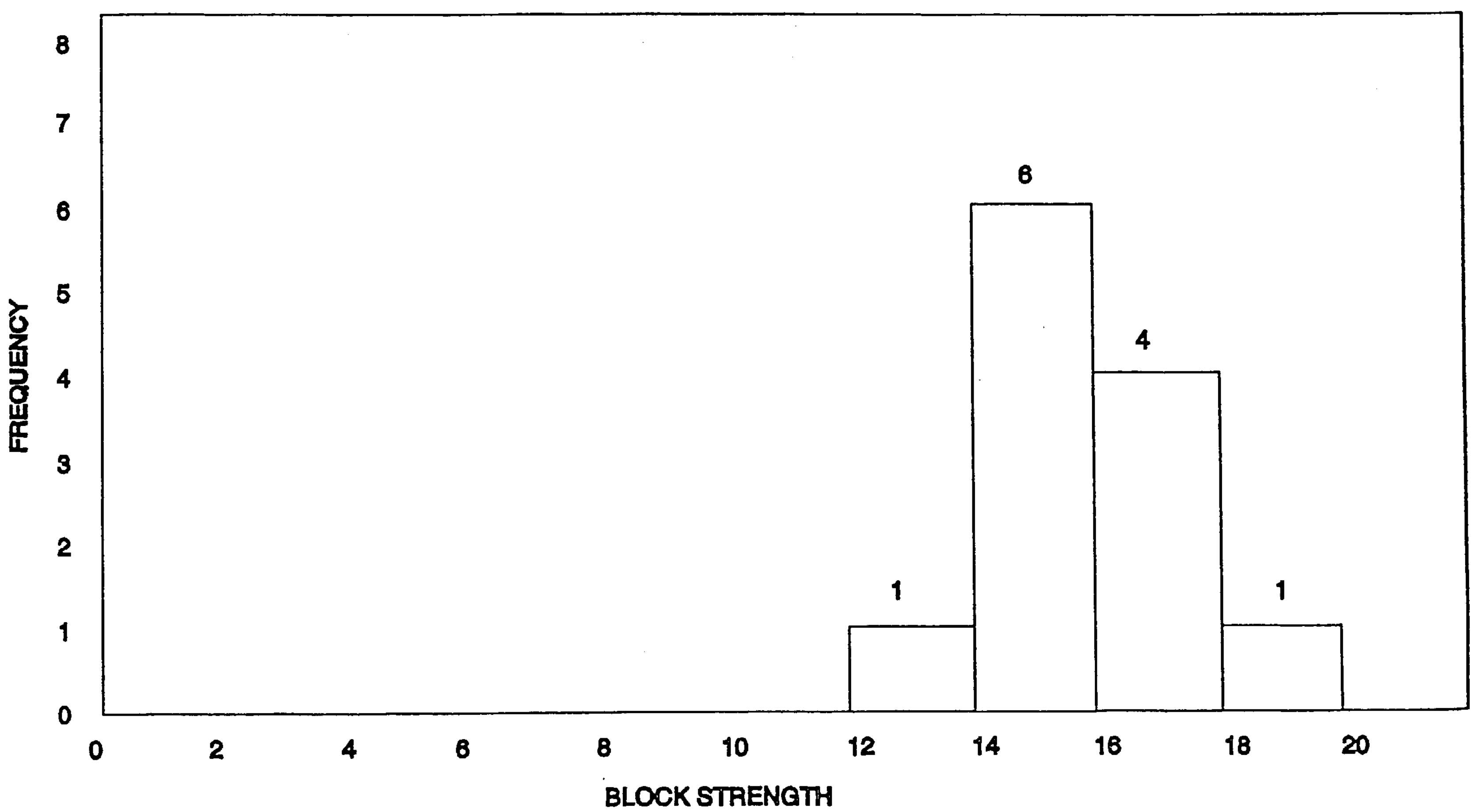


Figure 3.7 Frequency versus Block Strength

3.1.1 Representative Sampling For Pillar Strength Determination

The sampling method to obtain the laboratory samples was a two stage process, firstly involving selection of a block of coal, and secondly involving extraction of test samples from each block. In each of these stages, there is a possibility that the sampling process is not representative.

Non-representative sampling may arise in two fundamentally different ways, as follows:

> A *bias* may arise in the selected samples due to a *consistent* method of non-representative sampling. For example, in the block selection stage of the process, it may only be possible to extract blocks consisting of particularly strong coal, or the blocks may always be extracted from the middle of the seam where particular geological properties apply. It is considered less likely that a bias would arise in extracting samples from each block as this process is under close control. However, an example of such a bias could arise from the extraction of samples of each width to be tested from different coal layers within the block. This could give rise to an incorrect assessment of the effect of width on strength. However, since several blocks were involved, it is unlikely that this same selection bias would be made on all the blocks.
>
> A *variance* will arise in the selected samples due to *inconsistent* departures from the average strength for the coal seam. Examples of pure variances would be inhomogeneities within the coal block being studied or random fluctuations in the instrumentation used to do tests. This type of error can be reduced by taking a greater number of <u>independent</u> samples. However, taking more samples from a block will not reduce any variance associated with the blocks not being representative of coal seam strength (such additional samples would not be independent as they would be from the same block).

It is frequently difficult to identify biases in experimental results on a rigorous basis from the data available. There is not always a benchmark value available against which to identify a bias. Examples of benchmarks which are available to permit unbiasedness to be assured are the calibrations of the instruments used to perform the strength tests. This allows a confident assertion that the recorded sample strengths are an unbiased reflection of the real sample strength. Since there are no other benchmarks with which to test for unbiasedness, careful attention to the design of the experimental work to avoid (or, if unavoidable, enable quantification) of possible biases is important.

A further important consideration about bias is that, if the magnitude of the bias in the experimental results is known, the data can be used to provide an unbiased predictor by using appropriate factors. One such example of this would be the use of a strength downgrading factor to accommodate for the difference in strength between relatively unjointed samples with relatively more heavily jointed or fractured rock in the coal seam.

Although the current work is intended to expand on Salamon's formula by making provision to consider different coal types, Salamon's field strength results give the possibility of indirect validation to determine the overall bias. Such an exercise could be used to provide a factor by which to multiply predicted field strength to obtain an unbiased predictor. The range of coal types considered in the current work is understood not to be too dissimilar from the range in Salamon's data, so the comparison of the entire experimental data set with Salamon's formula for strength is appropriate.

From the experimental test work which was carried out, the variance between samples within a block can be determined precisely and with over 900 degrees of freedom for the residual variability between samples in a block, the value is determined within tight limits. Since the sample strengths are unlikely to be a biased subpopulation, the variance can be accepted as a reasonable estimate for the overall distribution of sample strengths from a block.

The variance between blocks can also be estimated quite precisely as there are 10 degrees of freedom for this statistic. However, there is some concern that the variance statistic may be an underestimate for the overall block strength distribution, as the blocks may share a common attribute, for example being only the higher strength blocks. This is a matter of speculation at this stage, and there is no data to either confirm or deny this. However, since both the mean and the variance of the distribution of pillar strengths are critical inputs into the safety factor calculation, a bias giving rise to too high an estimate of the mean and too low an estimate of the variance will for both reasons give rise to an underdesign. Merely correcting for bias will reduce the extent of the underdesign, but the underestimate of variance will still contribute towards underdesign.

A further note of caution in this area revolves around the possible dependence of variance in strength on the dimensions of the sample (as size is increased from single sample to block to pillar). There is relatively little information available from the data on which to estimate this effect, but it is of great importance for establishment of a reliable safety factor methodology. By considering Salamon's field data on pillar strengths in conjunction with the experimental data, it may be possible to obtain some insights in this area. However, this has not yet been attempted within the scope of the present work.

Based on these concepts on representative sampling, it is possible to address certain key questions.

1 How repeatable are the results obtained from Delmas block No. 1 and Delmas block No. 2?

The two Delmas coal blocks appear to belong to the distribution of strengths for the No. 2 Seam, Witbank Coalfield, or rather there is no meaningful evidence based on only two blocks to suggest that the distribution of strength in Delmas coal is not a sub-population of No. 2 Seam as a whole.

In order to assess whether lateral variations in strength existed across a coal seam, (e.g. different mines have different strength coal), it would be necessary to conduct a comprehensive block sampling campaign, comparing results from several blocks per colliery. Details of the sampling requirements would depend on the smallest strength difference that would need to be determined between areas. The smaller the required difference, the more blocks would be required. Based on the results from the No. 2 Seam where 8 blocks were available, the logarithmic variance between blocks from a seam is 0,471, implying a standard deviation of +/- 7,4% of the average strength for the seam. As discussed above, this variance may represent an underestimate of the variance of blocks drawn at random from the seam. However, provided the same extraction practice from a seam is followed, any biases would remain consistent and this variance could be expected to be repeated. Thus to determine strength to +/- 5% with 99% confidence would require 20 blocks to be analysed. If a +/- 10% range were acceptable, 5 blocks would be required.

2 Is there a comparison between the larger (300 mm) samples?

There is no evidence to suggest that the larger diameter samples are subject to a significantly different logarithmic variance than other diameter samples. Taking this into account, it is considered most appropriate that a comparison between blocks should be based on all samples, as this provides the most powerful test for difference in strength. Using only a sub-set of samples would reduce the significance level of any difference which may exist.

3 Is there a suggestion that as sample diameter increases, the laboratory results approach Salamon's field data?

The laboratory samples actually exhibit a shallower trend in strength with diameter than the relationship based on Salamon's field data. As a result of this, at small sample diameters, the predictors based on laboratory data estimate a far higher strength than extrapolations from Salamon's formula would indicate. At meaningful size pillar widths, the difference between the predictors is much smaller.

One possible explanation is that the weaker width relationship obtained from laboratory data incorporates a mathematical representation of "downgrading" in the lower width exponent. If Salamon's exponents are applied to the laboratory data, it is necessary to apply a significantly higher downgrading to the delta strength factor to obtain agreement with field observations. However, with the optimized exponents, the downgrading is relatively smaller. The observed factor with the optimized relationship could possibly be to correct for selection biases in extracting blocks. There is, however, no evidence to prove this contention over many other possible explanations.

Figure 3.8 shows the laboratory and field strength versus pillar width for a constant width to height ratio of 2,0 on a log scale. Similarly Figure 3.9 shows strength versus pillar height, again with a constant width to height ratio of 2,0. It is interesting to note that similar variation between the laboratory and the field was found in an extensive study by Martin (1995) on Canadian granite, Figure 3.10. Martin stated that "these results demonstrate that there is not an unique strength-scaling law that can be applied to both laboratory and in situ failure." The down-grading from the laboratory to the field is contained in the exponents for width and height. Figures 3.11 and 3.12 show the comparison between laboratory and field predictions. The aspect of down grading can be of significance in that each data set may be calibrated with each other, allowing comparative assessment of seams via laboratory testing.

laboratory and field data

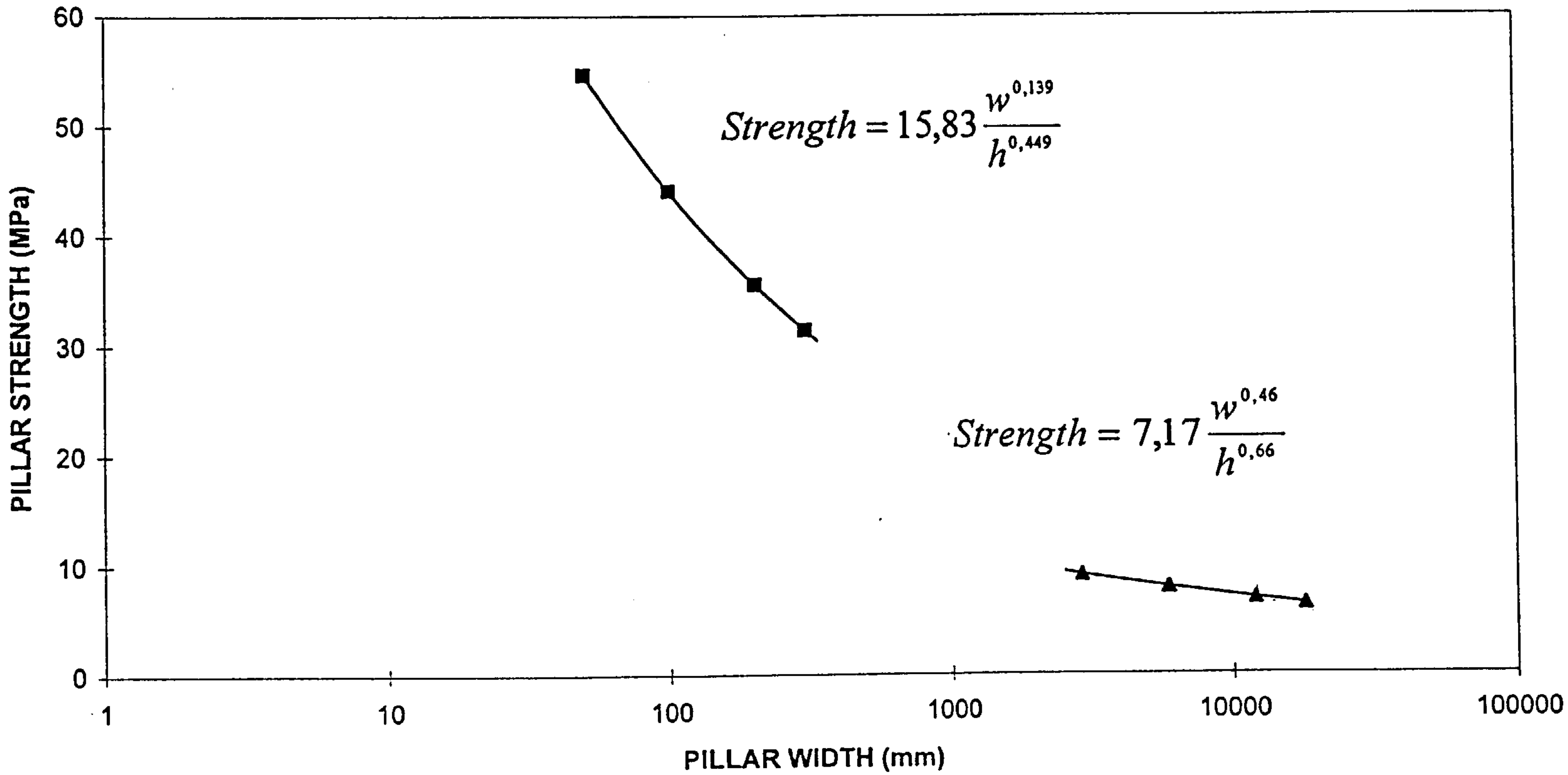


Figure 3.8 The Laboratory and Field Strength versus Pillar Width for a Constant Width to Height Ratio of 2,0 on a Log Scale

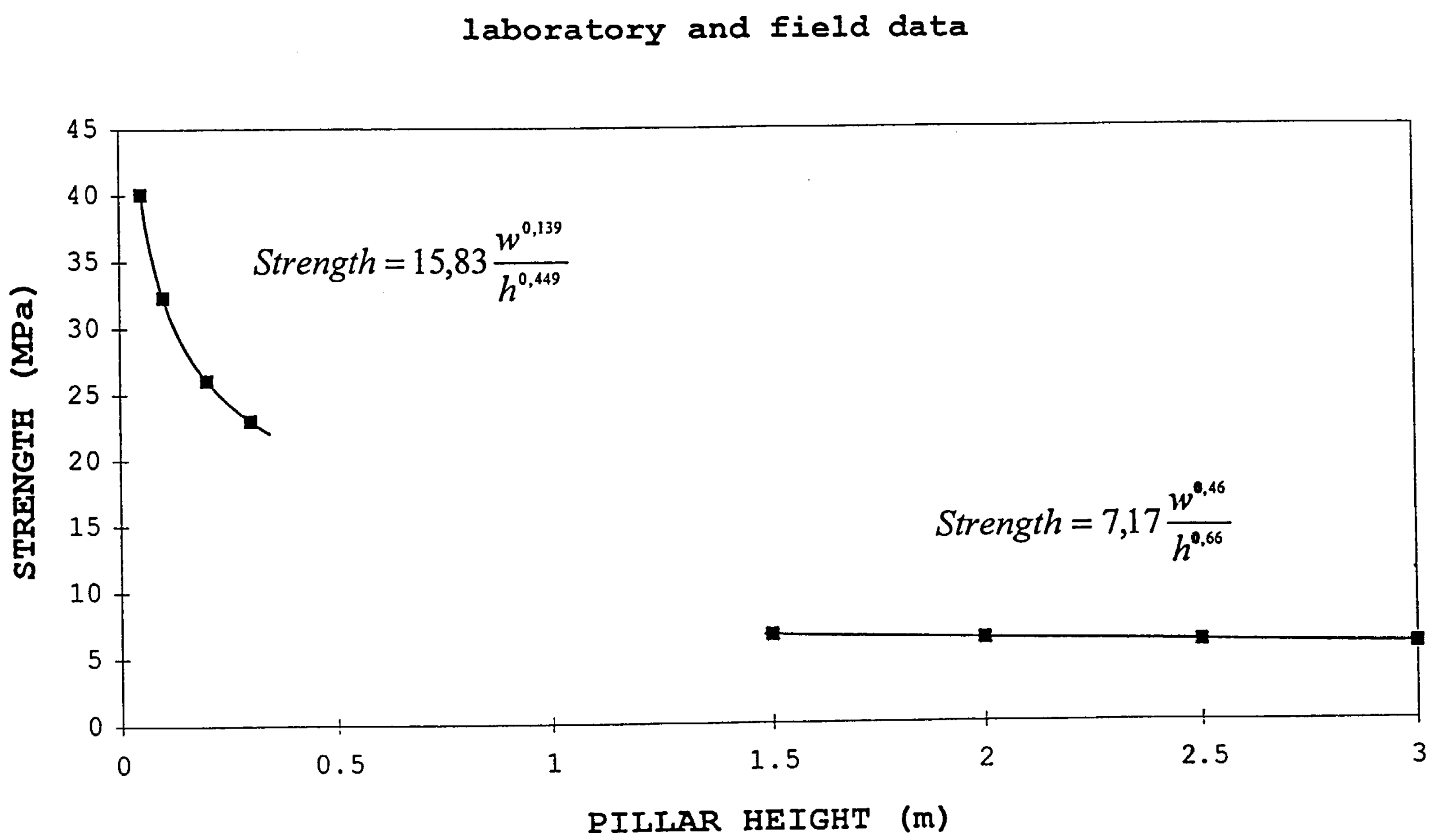


Figure 3.9 Strength versus Pillar Height, Constant Width to Height Ratio of 2,0

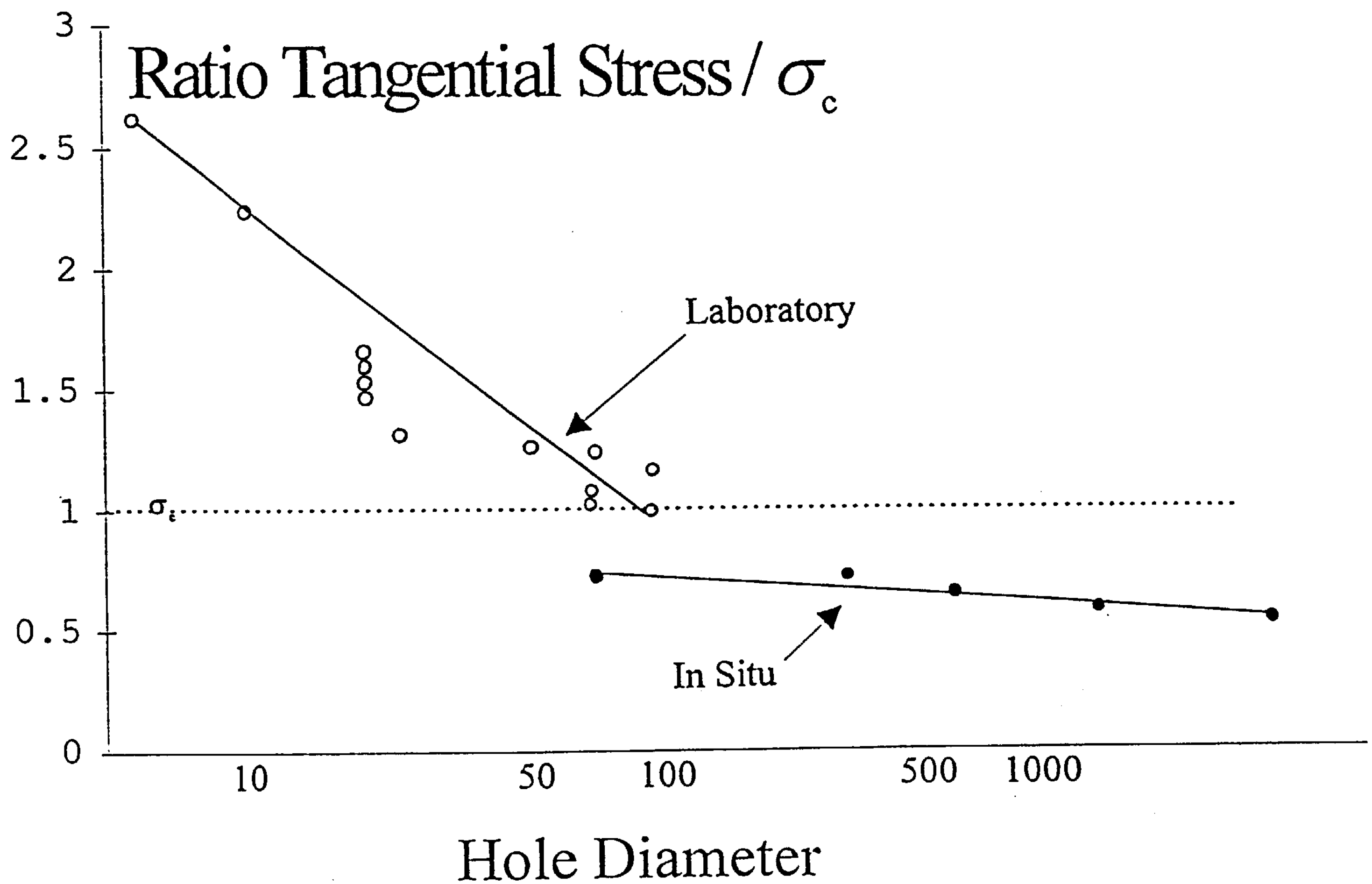
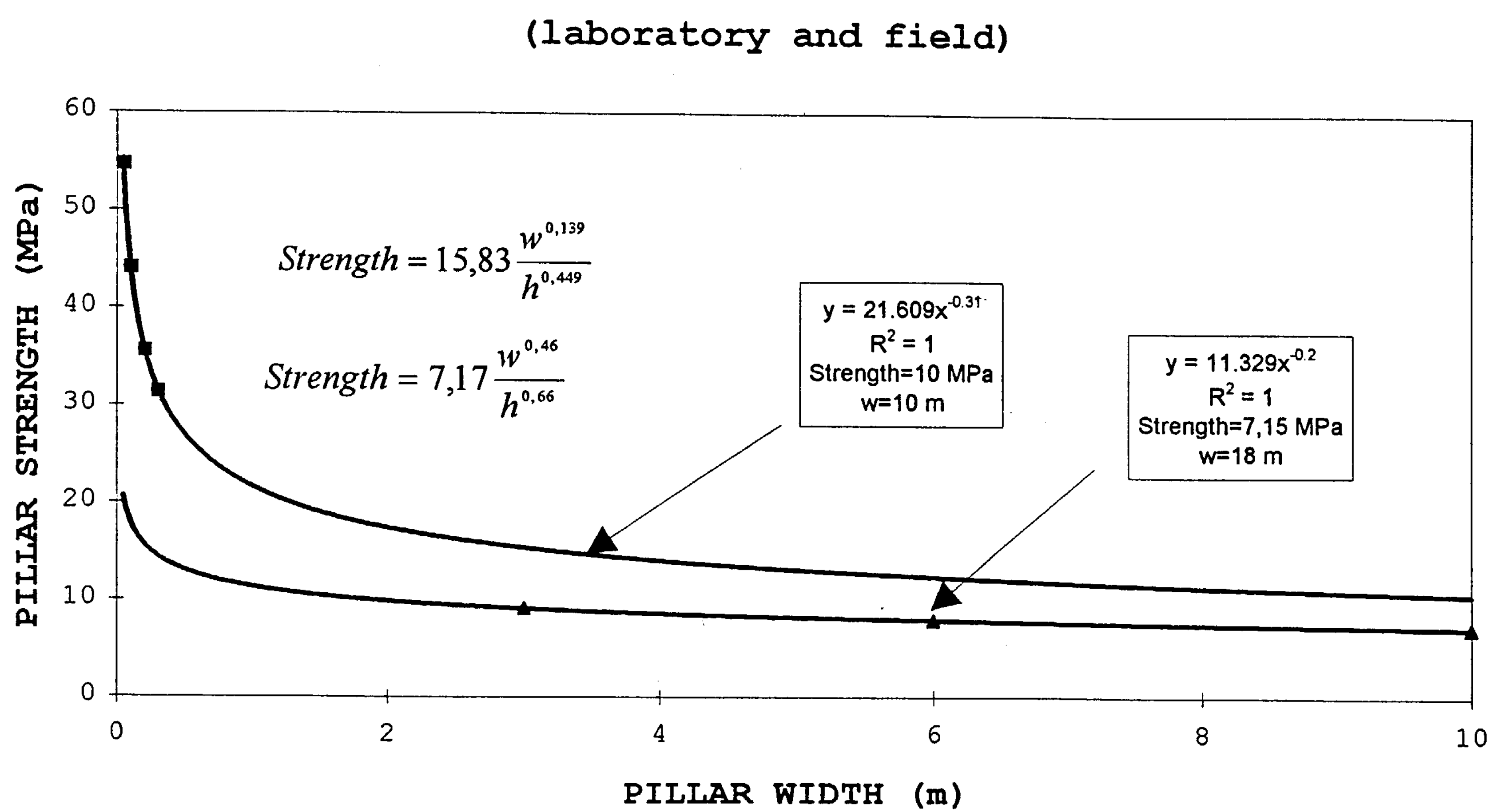
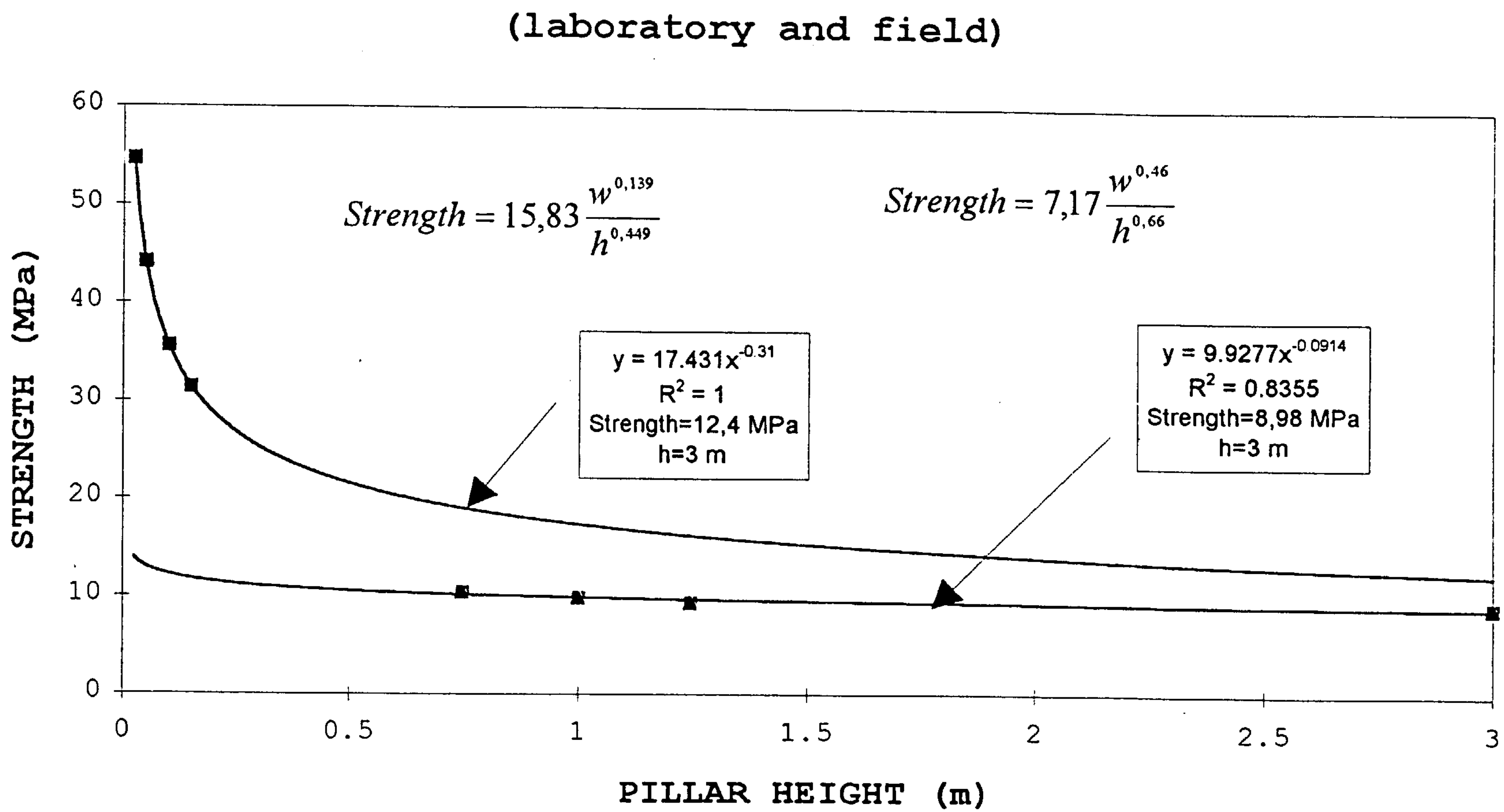


Figure 3.10. Strength versus Hole Diameter (Log Scale), after Martin (1995)



Figures 3.11 Comparison between Laboratory and Field Predictions



Figures 3.12 Comparison between Laboratory and Field Predictions

4 Does the statement of a "tight range indicating that a single strength is not unreasonable for all coals" still hold?

There is statistical evidence in the data that blocks from the No 2 Seam have a higher strength than blocks from other seams. It is also understood that there is some field evidence that other seams suffer from a greater percentage of pillar failures than the safety factor calculation based on a common strength relationship would indicate. It could therefore be validly proposed that the No 2 Seam should be regarded as having a higher strength than the other seams tested. However, the number of blocks tested from other seams is relatively low and the difference in strength is relatively small.

5 How would laboratory testing be used to determine whether a new coal type had a different strength than current coals, and what sample size would be required?

The answer to this question was partly provided in 1 above. The standard deviation between blocks within a seam is estimated at 7,4% of the mean (when extracted using the selection scheme adopted for these experiments), and the standard deviation between samples from within a block is estimated at 21,4% of the block mean. As discussed above, a strategy involving the testing of 5 blocks, analysing 25 samples per block would give rise to a 95% confidence range between 92,4% and 107,6% of the mean for the seam. This may be an appropriate basis on which to estimate the distribution of pillar strength which would be expected.

A number of observations are extremely important in this context.

Firstly, although the current experimental results displayed a greater degree of homogeneity within the No 2 Seam than for the total experimental dataset, it is suggested that consideration should be given to the geological characteristics of the coal to see whether there are differences in the physical attributes of the coal which would give rise to a relatively consistent strength within the seam, with other seams displaying different physical attributes. Such modelling exercises based on physical relationships can be used with greater confidence than purely empirical data. Another seam's behaviour may be different because of different geological processes responsible for its formation etc. With more mechanistic approaches, this problem can be overcome. Development of such a model would require the recording of geological attributes of the coal blocks when conducting the test programme.

Secondly, on a new coal seam, it is strongly advised that borehole data alone would not provide a valid comparison with the current set of results. Any biases in the block selection process would be radically changed with the result that the factor to apply to strength in order to scale up to pillar size would be unknown. It appears that the selection process which was adopted in extracting the blocks would probably have resulted in overstrength blocks being used. Testing of borehole samples would probably therefore lead to lower (unbiased) strength results. Application of a factor to reduce strength estimates would then lead to the true strength of the coal being undervalued, thereby resulting in overdesign of pillars. The potential error is probably a conservative situation, but it is recommended that extreme caution should be adopted in this area with appropriate statistical expertise being used to evaluate the results.

Thirdly, the more blocks and samples that are processed, the more tightly can coal strength of the coal be determined, such that it may become possible to base the safety factor calculations on a higher average strength of the coal. (It would be necessary to be conservative in the calculation to take the confidence limit as the average strength) in order to achieve the same predicted probability of pillar failure, based on more comprehensive testing. The statistical calculations underlying this are not trivial to optimize the extent of testwork which should be carried out.

The results from the testing of coal blocks supports the contention that the average strength of coal is within a fairly tight band, with the possible exception indicated by Madden (1985) and van der Merwe (1993) where the Vaal Basin Coal was shown to be significantly weaker.

3.2.1 Australian Research Into Coal Strength And Pillar Design Formulae

Recent investigations into the back analysis of collapsed and intact pillar geometries in Australian collieries have been conducted by Galvin (1995).

This investigation is considered highly significant as the results from various coalfields and seams in Australia yielded similar results to those obtained in the South African back analysis of full sized pillars. Back analysis using case histories has significant advantages over laboratory based experiments as the variables of size, discontinuities, moisture content, platen contacts, loading rate and extrapolation of results are eliminated.

Significant extracts from Galvin's report are included between pages 56 and 68.

"Development of a statistically based pillar design formula for Australian conditions relies upon the assumption that reliable data of stable and collapsed pillars exists. Without this information, it is impossible to define a design formula.

During the study, selected examples of stable cases, concentrating on cases with low to moderate factors of safety were also collected.

Close attention was paid in all cases to the accuracy of mining dimensions presented and possible adverse influence of geological anomalies. If geological features were believed to have influenced behaviour then these cases were rejected. Similarly cases where good roof and floor contacts did not exist were not included.

Those cases of pillar collapse of bord and pillar workings are detailed (18 in all) in Table 3.10. The data spans most of the major coal fields, in the states of New South Wales and Queensland. The relevant dimensions of the bord and pillar workings are contained in Table 3.10 along with grade and age at collapse and general comments.

Mine site data on stable bord and pillar workings for NSW and Qld are detailed in Table 3.11. Only those cases with a moderate factor of safety are listed and the numerous number of cases of stable bord and pillar workings of very high factors of safety have been omitted. Those stable cases of a very high factor of safety have minimal influence on any statistical analysis. In all, fifteen (15) stable cases are tabled with mine dimensions, grade, age of pillar, etc. given in Table 3.11. As was the case for the collapsed cases, the loads acting on the stable pillars were calculated by the tributary area method.

The range of depth is shown in Table 3.12 and pillar width to height ratios for stable and collapsed pillars is detailed in Table 3.13. It is important to note that pillars with high width to height ratios (in one case 8,0) can fail if, as in this case, partial extraction results in loads exceeding pillar strength.

Most stable cases have moderate to high width to height ratios (from say 4.5 to 10). Only a minority of stable pillars have low width to height ratios and these occur with low loads. Low seam height also plays a role in stable low width to height pillars. It has long been known that the nature of pillar collapse is a function of pillar width to height ratio. In South Africa, violent or uncontrolled collapses have been associated with width to height ratios of 4 or less. The Australian data base supports this observation, with the range displayed in Table 3.14. Controlled (creeplike) failures were observed to occur in cases where width to height ratios exceeded 4.5, again this was in broad agreement with international experience.

The transition from uncontrolled to controlled collapses appears to be around the width to height range of 4 to 6. However, due to the limited size of the data base, no certainty can be placed on this observation.

As can be seen from Table 3.15, over 50% of collapses occur within six months of the pillar being formed and almost three quarters of failed cases occurred within 12 months. Therefore, it is most likely that mining was still occurring in the area where collapses occurred. This fact was borne out in a number of cases where only luck prevented the workforce from being buried in the collapse.

The table also shows that time is no guarantee of stability. Collapses do occur many years after pillars have been formed.

A study of Table 3.11 indicates that provided the appropriate conditions are fulfilled, pillars may remain stable for many decades. Case FS4 has remained stable for 170 years.

Note that the Australian data correlates with time whilst the South African cases did not.

Table 3.10. Australian Collapse Cases

CASE	DEPTH (m)	PILLAR DIMENSIONS (m)	MIN. PILLAR WIDTH (m)	PILLAR HEIGHT (m)	w/h	BORD WIDTH (m)	LOAD (MPa)	AGE AT COLLAPSE (yrs)	COMMENT
FC1	140	15X15 A	15	5D	3.0	7A	7.5	2.0	
FC2	80	7.5X18 D	7.5	7D	1.07	7D	5.4	0.5	
FC3	80	6X7.5 D	6	3D	2.0	7D	8.4	0.5	
FC4	135	7X20 D	7	5D	1.4	6D	8.15	0.5	
FC5	100	10X10 D	10	6D	1.66	5.5D	6.0	1.0	
FC6	120	10X20 D	10	4.5D	2.22	7D	6.9	18.0	General grade 1 in 8. Local grades 1 in 5. Pillar height at mining 3.2 m. 1.3 m coal tops
FC7	95	3.5X12 A	3.5	1.75D	2.0	6.2A	10.0	unknown	
FC8	70	9X9 A	9	6A	1.5	8A	6.24	0.25	
FC9	250	15X27 D	15	3D	5.0	15A	19.44	0.5	Initial bords 7 m. Then stripped an extra 8 m of 2 sides of pillars.
FC10	115	10.5X22 A	10.5	5A	2.1	8A	6.53	0.25	
FC11	145	11.25X22.5 D	11.25	7D	1.6	6.25D	7.2	4.0	Grade 1 in 8.5 along pillar length. Grade 1 in 11 along pillar width.
FC12	336	13.5X28.5 D	13.5	3D	4.5	6.3D	15.04	0.5	
FC13	185	21X21 D	21	4.9D	4.28	6.1D	7.7	unknown	
FC14	240	22X22 D	22	4.9D	4.48	6.1D	9.78	unknown	
FC15	152	25X30 D	20	9.2D	2.17	5.5D	6.33	10.0	Diamond pillars. Minimum width determined by reduction of pillar points. Load determined by extraction ratio.
SC1	75	8X8 D	8	4.5D	1.77	6.5D	6.2	1.0	
SC2	58	3.65X54 D	3.65	2.7D	1.35	7.3D	6.0	1.0	
SC3	170	20X20 A	20	2.5D	8	5.5/70D	24.5	0.25	Partial extraction panel

A= Actual Dimension D= Design Dimension

Table 3.11. Australian Stable Cases

CASE	DEPTH (m)	PILLAR DIMENSIONS (m)	MIN. PILLAR WIDTH (m)	PILLAR HEIGHT (m)	w/h	BORD WIDTH (m)	LOAD (MPa)	AGE OF PILLAR (yrs)	COMMENT
FS1	80	23.5X23.5 A	23.5	2.1 A	11.19	7 A	3.37	33	
FS2	122	7.6X30.5 A	7.6	1.5 A	5.07	6 A	6.53	85	
FS3	40	10X10 A	10	3.2 A	3.13	6 A	2.56	1.5	
FS4	22.5	2X4 A	2	1.0 A	2	5 A	4.43	170	Convict Era
FS5	58	5X30 D	5	1.7 D	2.94	6 D	3.83	85	
FS6	70	14X14 A	14	2.5 A	5.6	6.5 A	3.75	43	
FS7	510	23.5X44.5 D	23.5	2.85 D	8.23	7 D	19.15	5	
FS8	500	23.5X44.5 D	23.5	2.75 D	8.56	7 D	18.78	5	
FS9	470	23.5X44.5 D	23.5	2.65 D	8.87	7 D	17.65	5	
FS10	450	23.5X44.5 D	23.5	2.55 D	9.22	7 D	16.9	5	
FS11	150	16X18 A	16	3.5 A	4.6	7 A	7.49	30	
FS12	75	4X11 A	4	1.0 A	4	5 A	6.14	40	
FS13	50	10X10 A	10	6 A	1.67	5.5 A	3	4	
FS14	430	32X36 D	32	3 D	10.66	6 D	14.9	unknown	
FS15	75	15X15 D	15	3 D	5	15 D	7.5	6	Bord width 7 m. Then stipped an extra 8 m. of 2 sides of pillars.

A= Actual Dimension D= Design Dimension

Table 3.12. Australian Range of Depths of Cover

	DEPTHS OF COVER (M)	% < 200M
COLLAPSED CASES	58 - 336	83
STABLE CASES	22.5 - 510	66

Table 3.13. Australian Range of Pillar Width/Height Ratios

	WIDTH TO HEIGHT RATIOS
COLLAPSED CASES	1,07 - 8,0
STABLE CASES	2,0 - 11,19

Table 3.14. Australian Nature of Collapses

	WIDTH TO HEIGHT RATIOS
UNCONTROLLED COLLAPSE	1,07 - 3,0
CONTROLLED COLLAPSE	4,5 - 8,0

Table 3.15. Australian Time to Collapse

TIME TO COLLAPSE	% OF TOTAL CASES	CUMULATIVE %
< 3 MONTHS	20	20
3 - 6 MONTHS	34	54
6 - 12 MONTHS	20	74
1 - 5 YEARS	13	87
> 5 YEARS	13	100

3.2.2 Australian Pillar Strength Formulae

Strength formulae developed from laboratory testing find limited application because they fail to account for factors such as rock mass variations, geological imperfections, pillar volume and pillar shape which affect pillar strength in the field.

Quantification of field strength therefore, needs to be based on analyses of the actual field performance of full scale pillars, **both failed and unfailed**. Even then, it is still not practical nor realistic to measure all rock mass and geological variations and input them into the design. The established engineering design methodology in these circumstances is to utilise probabilistic statistical methods to quantify the degree of variability and uncertainty in the design due to these factors (major geological disturbances or features still need to be assessed separately).

A probabilistic analysis of collapsed and stable bord and pillar working has been performed on field data from NSW and Queensland coal mines. Australian parameters for the two most universal forms of pillar strength formulae, namely the linear form (e.g. Bieniawski) and the power law form (e.g. Salamon and Munro) were quantified from this probabilistic analysis.

The analysis is based on calculating the pillar working load at the time of failure.

The process is analogous to placing a large water tank on top of a pillar and progressively filling the tank with water until the load i.e. the pillar working load, just exceeds the pillar strength. Theoretically, pillar failure occurs the instant pillar working load just exceeds pillar strength.

The statistical analysis involves fitting values to variables contained in the pillar strength formulae that obey the principles of empirical research (i.e. they include the primary variables that control the physical process e.g.

equations (3.1) and (3.2)) until the scatter of predicted strength values is minimised with respect to computed failure loads.

Stable as well as collapsed cases are weighted in the statistical analysis. The standard deviation is the measure of the degree of fit (or conversely, scatter).

The parameters which yield the maximum likelihood of predicting the field performance within the range of the Australian database (w/h = 1.07 to 10.6) are :

Linear Formula :

$$S_p = 5{,}36\left(0{,}64 + 0{,}36\left(\frac{w}{h}\right)\right) \quad \text{MPa} \qquad (3.3)$$

(Standard Deviation = 0.0863)

Power Law Formula

(a) For w/h < 5

$$S_p = 7{,}4\frac{w^{0{,}46}}{h^{0{,}66}} \quad \text{MPa} \qquad (3.4)$$

(b) For w/h ≥5

(Squat Pillars)

$$S_p = 19{,}24\frac{\left(0{,}2373\left(\left(\frac{w}{5h}\right)^{2{,}5} - 1\right) + 1\right)}{w^{0{,}1334}h^{0{,}0667}} \quad \text{MPa} \qquad (3.5)$$

(Standard Deviation = 0.0735)

The power law formula fits the Australian database slightly better than the linear formula as shown by the lower standard deviation. This is primarily because, unlike the linear formula, the power law formula takes into account :

- the exponential increase in lateral confinement generated within a pillar as w/h increases;
- the effect of pillar volume on pillar strength.

The probabilistic analysis has shown that **pillar strength in the field is only marginally dependent on the material strength of the coal seam once the pillar width to mining height (w/h) exceeds 2,0.** This behaviour is consistent with field strength being dominated by the w/h ratio and the associated lateral constraint that this parameter generates.

It is also consistent with the laboratory findings of the Mining Research Establishment of the National Coal Board (U.K.) reported by Evans and Pomeroy in 1957 :

"... at atmosphere pressure (no lateral confinement) there is a 14:1 ratio between the strongest and weakest coals while at 5,000 lb./in² (34 MPa) confining pressure the ratio is only 2:1. This latter ratio is reduced to about 1.25:1 if the results for anthracite are excluded. In other words, the strengths of almost all coals are virtually indistinguishable at high confining pressures"

The Australian database is moderately small (30 cases) and it is plausible that some critical factors may be absent. The largest database of collapsed and stable pillars assembled is that of Salamon and Munro in South Africa (125 cases). Because pillar strength is largely independent of coal seam material strength at moderate to high w/h ratios, it is reasonable to compare and to combine both databases in order to check for inconsistencies.

Close correlations were obtained in both cases reaffirming that pillar strength is largely independent of coal seam material strength. The strength parameters for the linear and the power law formulae differed by only 5% and 3% respectively. In all cases, a lower standard deviation was associated with the power law formulae (0.0689 to 0.0735) than the linear law formulae (0.0776 to 0.0863).

Probabilistic analysis is a far more rigorous mathematical process than curve fitting. It assigns a statistical significance to stable as well as to collapsed cases, although collapsed cases carry a much higher weighting.
Neither the Australian nor the South African database contain a collapsed case above a w/h ratio of 8.2, hence there are no checks on the upper limits to which the formulae find application.

Either the linear or power formula may be used up to a w/h ratio of 8 although the power law form is more flexible and preferred on the basis of statistical trends and conformity to physical principles.

The power law formula is recommended for w/h ratios greater than 8 recognising however, that there is a lack of data to validate either formula at w/h ratios greater than 8.2 and especially, greater than 10.6 (upper stable case).

The pillar strength law formula given by equations (3.3) and (3.4) are based on the strength of square pillars. No definitive method exists for calculating the strength of rectangular shape pillars.

Rectangular pillars could be expected to be stronger than square pillars of the same minimum side length since there is a greater contact area at the roof and floor contacts and more coal available to confine the core. However, this assumption would not be valid when the pillar is narrow since failure can quickly propagate through to the centre of the pillar, irrespective of whether pillars are rectangular.

Until more data becomes available, it is recommended that the minimum pillar width be used when calculating the strength of all rectangular pillars.

3.2.3 Advanced Statistical And Probabilistic Analysis

The outcomes of the statistical analysis presented are based on fixing the constants A and B in Bieniawski's equation and α and β in Salamon and Munro's equation and permitting the values of k1 and k2 to float.

However, the research also evaluated statistically, the maximum likelihood parameters associated with:

- fixing k1 and letting A and B float and fixing k2 and letting α and β float.
- letting all parameters float.

These analyses produced very similar values for each parameter (within 3%) in all cases except one. In this latter case, all three variables were allowed to float, resulting in an increase in one variable being offset by a decrease in another.

In all cases, the standard deviation was only either marginally better or marginally worse than that associated with equations (3.3) and (3.4). In all cases, the standard deviation was marginally higher for the linear formula than for the power law formula.

Figure 3.13(a) shows a comparison of the pillar strength predicted by the linear formula, equation (3.3), and the power formula, equation (3.4), using the combined Australian and South African databases. Small circles represent stable cases and large circles represent failed cases. The straight line represents situations where both formulae give the same strength value.

Figure 3.13(b) is a replot of Figure 3.13(a) but with the symbol size proportional to w/h ratio. The figure reveals that the deviation between the two formula at the top end of the straight line is due to the higher strength calculated by the squat pillar power law formula for pillars of w/h >5."

Galvin's research is of significance, as the question of the strength of different seams from different coalfields may be only one of many when considering pillar stability.

According to Galvin, the influence of individual seam strengths may only be significant in low pillar width to mining height ratios, along with the effects of discontinuities and influences due to mining.

Of particular interest, is the rather close agreement obtained by Galvin between the behaviour of Australian coal pillars and those analysed in South Africa, the differences in strength and in other parameters being only a few per cent. Figure 3.13 also illustrates the interesting point that the "linear" and "power law" formulae actually fit the data equally well excepting the cases of squat pillars.

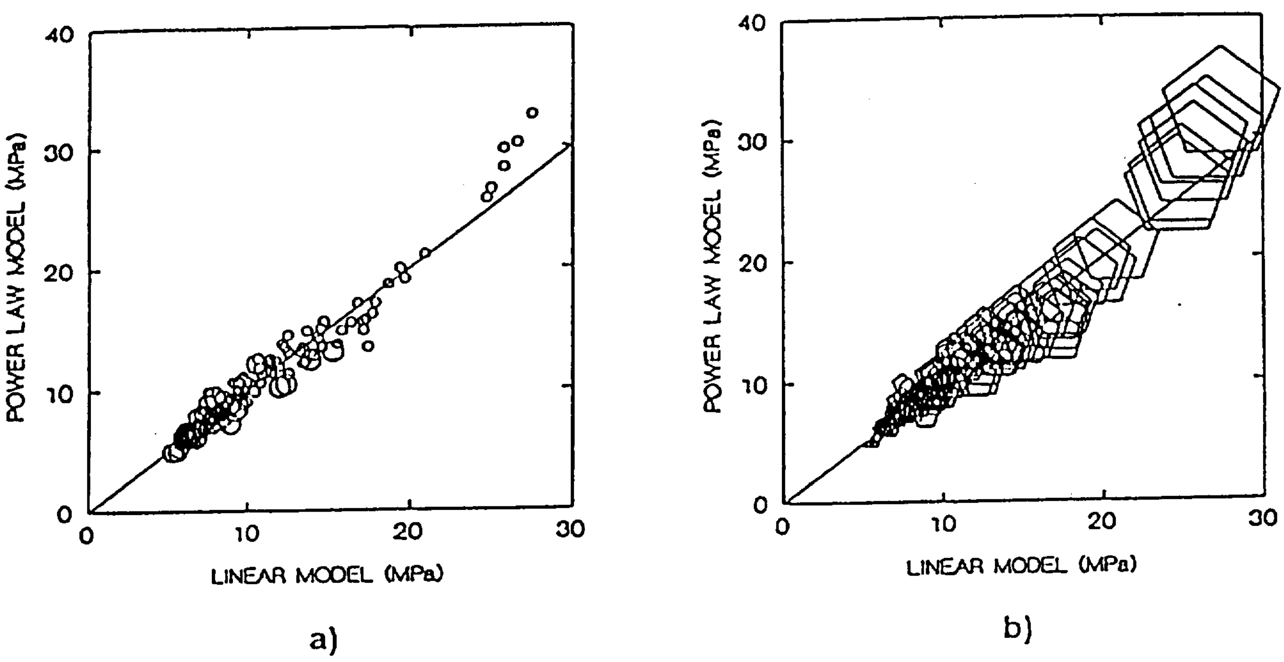


Figure 3.13. Comparison of Univ. of NSW Linear and Power Law Formulae for Combined Australian and South African Data Bases

3.3 Discussion

The significance of the Australian research programme was briefly reviewed at the end of the previous section. In the South African pillar SIMRAC project, seam strengths based the laboratory testing programme conducted at CSIR-Miningtek were re-analysed in two forms. In the first the parameter Delta, representing strength, and the exponents for height and width were allowed to float, and in the second analysis the exponents for height and width were fixed to -0,66 and 0,46 respectively.

In both analyses each groups of tests was assessed individually as well as collectively. The results of the analysis where both the exponents were fixed and the strength allowed to float yielded best results with a scatter of only about ± 7 percent when all test samples were included together. The results showed statistically significant variation within a fairly tight range. The variation between the blocks showed that there was a tighter distribution between the No. 2 Seam blocks than within blocks from other seams. Eight of the 12 blocks came from the No. 2 Seam while only one block each came from the Main Seam at ZAC, No. 4 Seam, Witbank Coalfield and Sigma Colliery, Vaal Basin. There may be a fundamental bias of the laboratory tests to the upper strength range due to the block selection and sampling procedures necessarily used, while Salamon's back analysis results are relatively unbiased since they are based on actual failed and intact in situ pillars.

It is interesting to note that the form of the laboratory test results was

$$\text{Strength} = \delta\, w^{0,139}/h^{0,449} \qquad \text{(MPa)} \qquad (3.1)$$

while the form found by Bieniawski (1967) after an extensive program of laboratory tests on South African coal from the No. 2 Seam, Witbank Coalfield was

$$\text{Strength} = \delta\, w^{0,16}/h^{0,55} \qquad \text{(MPa)} \qquad (3.6)$$

Bieniawski's results were obtained in a laboratory installed underground within the section where the samples were obtained. This procedure was conducted to overcome the potential difficulties of transportation, the effects of moisture content and the time between sample collection and testing.

The major difficulty with laboratory testing is the extrapolation of results to full size pillars, and how to account for the variability of strength in the layers within the coal seam and the effects of cleats and discontinuities.

An assessment of the influence of discontinuities has been proposed by Esterhuitzen (1995) whereby the amount and type of discontinuity occurring within the coal pillar can be classified by a simple mapping technique. The importance of the technique is that the influence of discontinuities, particularly slips, can drastically reduce pillar strength. This effect is significant at low pillar width to height ratios of say less than 3,0. As the pillar geometry changes and the pillar width to mining height ratio increases the pillar strength is increasingly determined by the increased surface contacts between the coal and surrounding strata as well as the triaxial effects within the pillar. The material strength and effects of discontinuities become less significant as the pillar width to mining height increase. Esterhuizen (1995) found that the pillar strength of a pillar with a pillar width to height ratio of 2,0 can be reduced by 77 per cent due to joints dipping at 45°, while the same joints reduce the strength of a pillar with a width to mining height ratio of 6,0 by only 17 per cent.

Considering that between 100 000 and 200 000 pillars are formed annually in South African collieries the performance of these pillars gives the best assessment of the design. The significance of a large empirical data base is that as the number of observations increase the confidence in the predicted value also increases. This is in terms of anomalies as well as in satisfactory performance.

The examination of anomalies is vital to delineate the limits of the assumed design parameters. Rating and classifying pillar condition as suggested by Esterhuizen (1995) is a convenient method of re-evaluating the significance of discontinuities. In this way the potential of forming an under strength pillar can be accounted for due to the influence of discontinuities.

It should be noted that Salamon (1967), Madden (1990) and Galvin (1995) all excluded collapsed cases where coal was not considered the weakest element in the system. Thus pillar collapses attributed to weak floor or roof or where discontinuities may have contributed to the failure of the system were excluded as were cases where the influence of multiseam workings may have contributed to the collapse.

The result is a design formula suitable to be applied where a good roof and floor are present. In these situations the formula developed by Salamon (1967) has been shown to work very well. One of the primary aims of Salamon was to design against another Coalbrook type pillar collapse and this has been achieved.

However, pillar collapses still occur. It has to be remembered that of the 50 cases available to Salamon only 27 fulfilled the criteria outlined above. Madden (1991) found 31 collapsed cases between 1967 and 1988 but excluded 14 cases on the same grounds. Therefore of the 81 collapsed cases up to 1988 only 44 or 54 per cent were included in the analysis. While some of these cases were excluded due to unreliable data there is a need to re-examine the information to highlight where potential difficulties may be found when mining.

It is imperative to obtain all possible information with regard to collapsed cases that have occurred since 1988 and to thoroughly review all collapses cases.

4.0 THREE DIMENSIONAL COMPUTER MODELS

A report entitled " An Overview of Some Numerical Models for Strata Control Applications in Coal Mining" by T. T. Ozan (1994), Internal Report RE 3/94 summarised the available numerical codes and their suitability to coal mine research. In the project FLAC was used for 2-dimensional modelling and MINLAY was used for 3-dimensional modelling.

5.0 PILLAR CLASSIFICATION RATING

Over 300 panels were visited and the pillars rated according to the system developed by Madden (1985) noting the amount of pillar scaling, the influence of discontinuities, weakness in the pillar and age of the workings The results of the pillar rating classification are shown in Appendix B. Each seam, 19 in total, was plotted, however in some seams limited classification ratings were conducted. Figure 5.1 shows the plot of the average rating for each of the nineteen seams versus the average skin stress.

A wide scatter of results was obtained within seams, with the ratings within a colliery also showing wide variations. The scatter can be seen when examining the No. 2 Seam, Witbank Colliery, Figure 5.2. The No. 2 Seam is the most extensively mined seam in the country; likewise, most classifications of pillar condition were in the No. 2 Seam, Witbank Coalfield.

The rating system was used to compare collieries within one seam, however, no distinct trend was evident. The classification methodology is sensitive to skin stress, and the rating system perhaps needs to highlight the parameter of discontinuities to a greater extent. Discontinuity effect on the pillar is included in the present classification rating system however; the type, as suggested by Esterhuizen (1995), may be of greater benefit.

Pillar classification using the rating system supports the contention that the seams vary within a colliery and coalfield. The conclusion as to their relative strength rating is shown in Figure 5.1 and summarised by the following order:-

Top-Bottom	Klip River
Bottom	Klip River
Alfred	Vryheid
C	Eastern Transvaal
Dundas	Utrecht
No 7	Soutpansberg
No 4	Highveld
Gus	Utrecht
No 2	Highveld
No 1	Witbank
No 2	Witbank
No 5	Highveld
No 5	Witbank
No 2A	Vereeniging
No 2B	Vereeniging
Main	Zululand
No 3	Vereeniging
Alfred	Utrecht

The relative strength should be treated with caution.

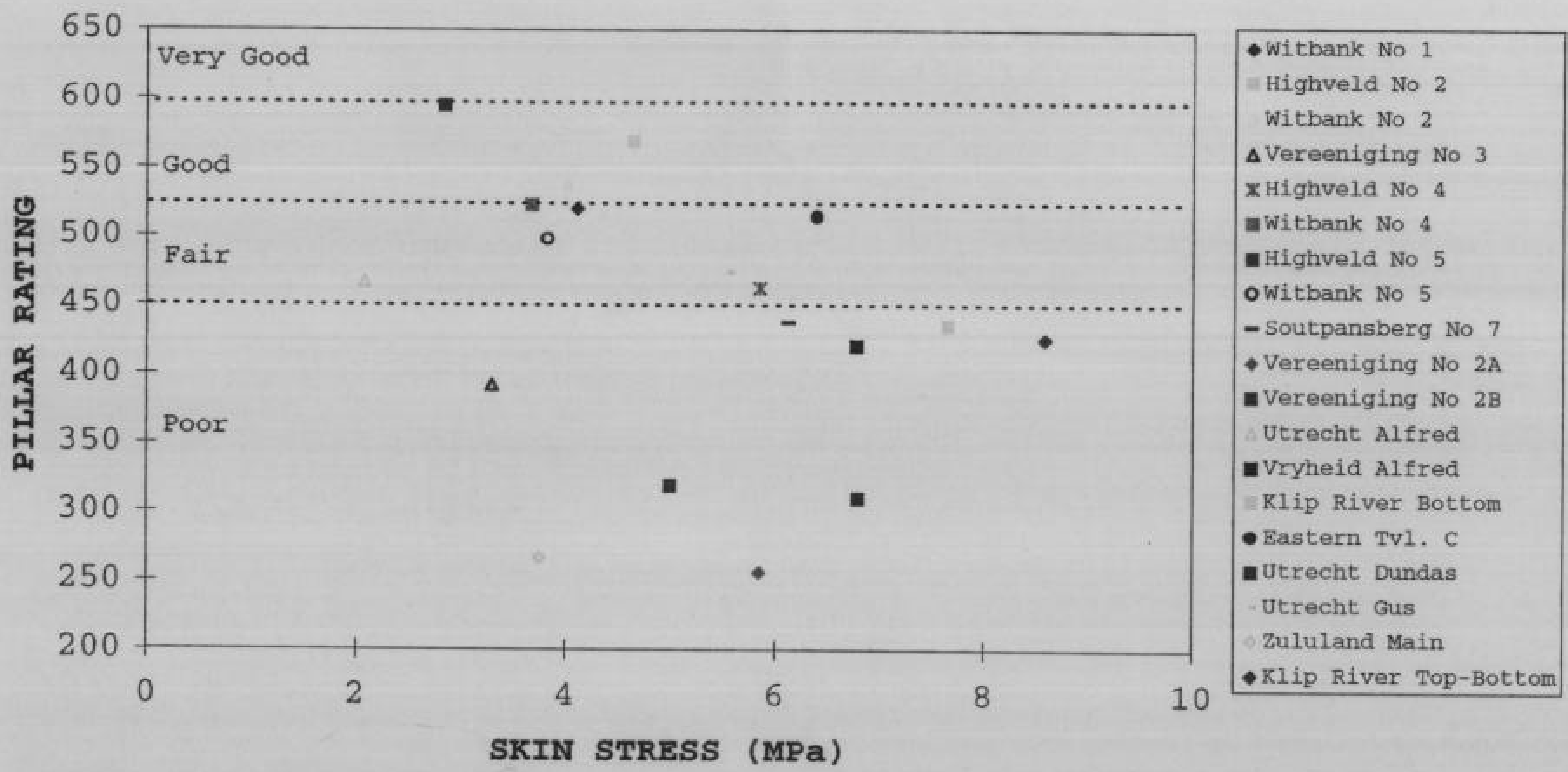


Figure 5.1. Average Pillar Rating versus Skin Stress

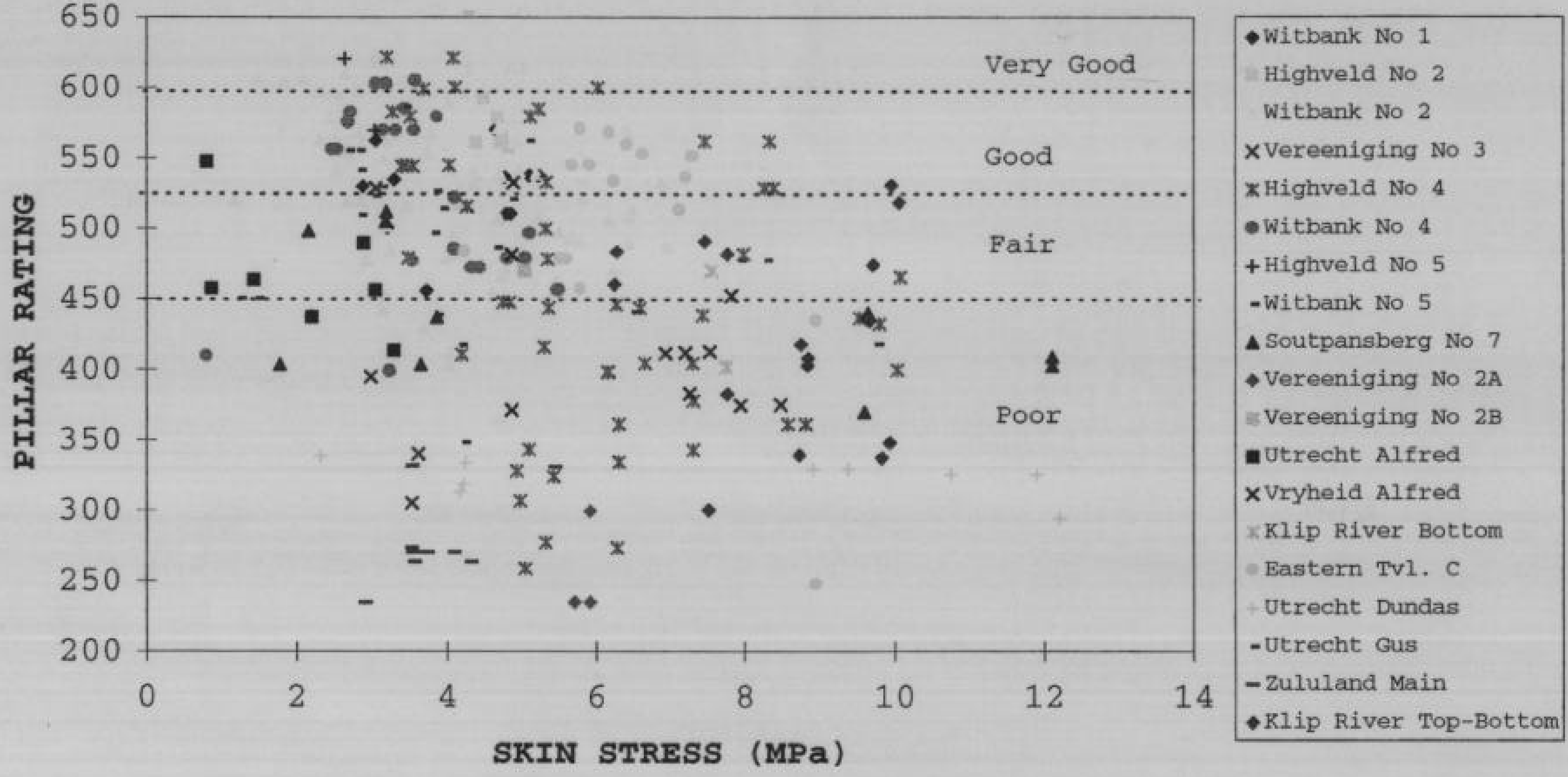


Figure 5.2. Pillar Rating versus Skin Stress

6.0 COLLAPSED PILLAR CASES

Madden and Hardman (1992) published tables of the dimensions of collapsed and intact pillar cases and details pertaining to the following: -

- -Collapsed cases used by Salamon and Munro (1967)
- -Intact pillar geometries used by Salamon and Munro (1967)
- -Dimensions of collapsed pillar cases recorded but not used by Salamon and Munro (1967)
- -Pillar collapses 1965 - 1988 Geometries used in a re-assessment of coal pillar design, Madden (1991)
- -Collapsed Pillar Geometries recorded but not used in the re-assessment by Madden (1991).

During the period since 1988 at least 12 collapsed cases have been recorded. Detailed information on nine cases have been collected. The information will be analysed and reported in full when all cases have been established. Publication of the current information may give a false impression as to the occurrence of pillar collapses due to the bias of the available data.

Obtaining information on known pillar collapses has the potential to increase the fundamental understanding of pillar performance. Questions as to the effect of time on stability may be addressed by examining the past history of pillar collapses. Therefore, greater manpower was and is required to obtain the information than was originally anticipated. Information on all known collapses is being collected. This includes the collation of previously known information.

Even if the collapse has no effect in terms of loss of life or disruption to the surface, understanding each collapse will assist in future design decisions that may involve risk to personnel, for example extraction of previously formed pillars.

An additional benefit of examining past pillar collapses is an increased study of the anomalies and limitation, of current design methods. Increased knowledge leads to greater confidence that men and equipment are not exposed to unnecessary risk.

The Matla No.1 Colliery, 5 Seam collapse is a case worthy of consideration. The design was in excess of the current industry accepted standard for the design of main developments, that is in terms of minimum pillar width, pillar width to height ratio, safety factor and percentage extraction. The mode of failure, however, was not one where the pillar was the weakest element within the system. Salamon's design formula is strictly for where the pillar is the weakest element. In the Matla No.5 Seam, the floor was the weakest element, subsequently foundation failure caused the pillar system, comprising the roof, pillar and floor, to fail. The result was disruption to the surface and co-incidenctly, effecting a gas pipe line and damaging a power line.

Were the collapse to have occurred in the access way of an operating mine, isolation of personnel would have been a real possibility.

Examination of previous collapses is essential to make possible the prevention of future foreseeable problems.

7.0 IN-SITU STRENGTH TRIALS

During the project duration, four experiments were conducted into the behaviour of selected pillars within a panel. Two pillar extraction sites were monitored at Bosjesspruit Colliery, and one at Greenside Colliery. A further site at Khutala Colliery was observed during pillar splitting on the retreat. No instrumentation was installed at the latter site.

The first site at Bosjesspruit Colliery was reported by Jack and Prohaska (1994) Rock Engineering Internal Note, RE10/94. Difficulties were experienced with the installation of the stress cells and it was concluded that

Glotzl cells would be preferred for future monitoring. The instrumented pillar did not fail due to its large dimensions; however, calibration of the model MINLAY was conducted from results of the surface extensometers whereby very close correlation was obtained for the seam compression.

The second test pillar was reported by Jack and Prohaska (1995), Rock Engineering Internal Note, RE 4/95. At this site the pillar selected for instrumentation ended up next to a row of solid pillars due to a change in extraction sequencing. In addition, an unplanned intact pillar was left next to the pillar in the previous row. Despite these pillars being left, pillar spalling was observed to occur some 60 m from the goaf line.

Experience gained from the experiment concluded that the surface borehole needs to be surveyed along its length to establish its precise position at the seam horizon.

To gain information on the loading of a pillar, a selected pillar at Greenside Colliery was instrumented and reduced in size prior to extraction. Glotzl cells were installed together with roof-floor closure meters and roof deflection levelling. In addition a surface borehole was instrumented with five extensometers. The results were detailed by Jack and Prohaska (1995), Rock Engineering Internal Note RE2/95. The pillar did not fail, and goaf surrounded the pillar on at least two sides. The Glotzl cells did not perform as anticipated. Several aspects of the investigation were highlighted including the need for continuous monitoring of instrumentation, a site where the pillar can be loaded to failure, and reliable monitoring of instruments properly protected from damage from goafing and the mining operations.

The site at Khutala Colliery was not monitored by instrumentation. Pillar splitting had resulted in nominal low safety factors over a limited area. Slight signs of pillar deterioration were observed on same pillars during a visual inspection some 12 months after splitting. It is suggested that this site continue to be observed as the important, and largely unknown parameter of time effects on pillar performance may be gained.

As a result of the field trials a new site has been selected at Delmas Colliery. The site will be dedicated to the experiment of obtaining information regarding loading a pillar to failure. Intrinsically safe continuous monitoring data loggers have been calibrated and passed for intrinsic safety by the SABS. The question of the influence of the grout modulus on the results obtained from the Glotzl cells is currently being investigated. The experiment will be conducted once all instruments have been calibrated. It was considered paramount to solve the technical problems relating to the instruments first, rather than to hastily conduct a further experiment. It is important that the time and effort already expended by both the researchers and the colliery should yield results that will significantly further the knowledge of the behaviour of coal pillars.

8.0 PILLAR FOUNDATION FAILURE

The coal pillar element does not always comprise the weakest link in the pillar system.

Empirical pillar strength formulae can result in pillars of adequate strength but the pillar system may become unstable because the roof or floor strata cannot support the load.

Foundation failure or bearing capacity failure can take a number of forms, depending on the strength, thickness and location of weak strata within the roof or floor horizons.

During the project several sites were visited and data collected relating to mining dimensions. Sites visited included Durban Navigation Colliery, Natal Anthracite, Hlobane and Piet Retief Collieries. Two monitoring sites were established at Natal Anthracite and New Denmark Collieries and were reported in a paper by Ozan and Budavari (1993).

One of the most recent cases of pillar failure occurred at Matla Colliery in January 1995. Although the collapse was in the proximity of dykes, the pillars were designed to a safety factor of about 2,65. This particular No. 5 Seam panel was 120 m wide and had been developed into virgin ground in 1981. Both the bord and pillar widths were approximately 5,5 m with a depth below surface of about 35,5 m. The average mining height was 2,2 m. In terms of the mining lease area the pillar failure area is insignificant. However as fate would have it, it was not the size but position that was critical, with both a pipe line and power line traversing the subsidence area.

With a view to documenting as much information as possible a visit to the panel in question was undertaken on 29th November 1995. Mapping of the panel was carried out from the collapsed area back towards the shaft (north east). The condition of the roof, pillars and floor were recorded and photographed. Full details are contained in the report "Matla Colliery No. 5 Seam Pillar Collapse" by Jack and Madden (in publication).

Roof falls could be seen to extend back in excess of 250 m from the main collapsed area. Although not always the case, the majority of the falls covering a larger area had broken away at the sandstone interface, some 0,3-0,4 m into the roof. The patterns appeared to be of a random nature with no one direction having any preference over another. In conjunction with the roof falls there was another distinct class of roof damage. This occurred as what may best be described as channels that had broken away from the roof. The widths of these channels appeared, in many instances, to be governed by the roof bolt spacing. They were fairly widespread throughout much the same area as the roof falls and did not seem to be more prevalent around the edge of the collapsed area. Their trend appeared random in nature and changes in direction of up to 90 degrees were not uncommon. In a high percentage of the cases, a strong shear component appeared to have been involved as part of the mechanism responsible for disturbing and dislodging the roof material.

In the rows of pillars immediately adjacent to the collapse cracks could be seen penetrating the main body of the pillar. These cracks initiate at the base of the pillar. This type of damage is consistent with pillar punching and foundation failure

Very severely damaged pillars tended to be concentrated around the edges of the collapsed area. Moderate and severe damage continued along the panel up to 200 m back from the main collapse area and occurred along the centre of the panel span. The pillar conditions did not improve in an anywhere near linear manner relative to the distance way from the main collapse area.

Large areas of the floor were covered with fallen roof material making it very difficult to positively identify floor heave. Areas where it was detected, either as a hollowness of the floor or the more spectacular thrust mounds, occurred throughout the panel.

Having observed the damage in the collapsed panel (mined in 1981), the panel to the north of it, running at 90 degrees to it and mined in 1983 was visited. This panel was 130 m wide, about 40 m deep with centres of 12 m compared to 11 m in the collapsed panel. These dimensions result in a safety factor of about 3,0.

The transition from the one panel to the other was quite abrupt, the conditions changing to near perfect, with continuous miner cutter marks clearly visible on the pillar sides, 100 percent stonedust and no slabbing or roof falls. However, minor floor heave was observed throughout the panel. Compared to the collapsed panel the other panel was 10% deeper. However due to the extra metre in pillar width there was a reduction in stress levels of approximately 5% at the pillar edges and 6% at the pillar centres.

No significant damage was observed when the No. 4 Seam directly below the No. 5 Seam collapsed area was visited on the 29th of November.

Over the period 16th and 17th December heavy rains fell in the area (probably in excess of 100 mm), during the underground visit two days later there was no evidence of any water inflow into the workings.

The failure at Matla Colliery in the No. 5 Seam was attributed to foundation failure by Buddery (1995) and this is agreed with by the authors of this report. Geotechnical logging of the core and index tests, including durability tests, highlighted the potential weakness of the foundation strata beneath the pillars.

Floor heave is also a major problem in Australian collieries and research has been conducted for a number of years and is still being conducted today. It is interesting to observe the similarity between conditions at Natal Anthracite and a particular Australian Colliery. Note the floor heave and the tilting of the timber props in both photographs, Figure 8.1. Vertical fractures in the coal pillar were also observed in the pillars next to the collapsed area in the Matla No.5 Seam. Severe floor heave can destabilise an area and the mechanism has been stated (Galvin 1992) to be one or a combination of the following factors :

1. Drainage of floor strata with time.
2. Swelling of clay minerals when exposed to moisture.
3. Time-dependent creep of floor strata under stress.
4. Foundation bearing failure of floor strata immediately under the pillar.
5. Failure of floor strata in bords caused by induced horizontal stresses at the pillar/floor contact.

While the floor characteristics will ultimately determine the final behaviour high stress levels are thought likely to exaggerate the situation.

A substantial amount of research has been conducted into floor heave throughout the world. It is widely recognised that heave is determined by two groups of factors, namely :

(i) The physical properties of the floor, including structure
(ii) The mining geometry.

The generation of a database of the physical properties has been undertaken by several researchers overseas.

Wuest (1992) recognises two fundamental types of floor heave :

(i) Buckling of the floor beam.
(ii) Plastic flow of the floor material.

" Buckling of the floor beam can be due to pillar punching or high horizontal stress. Plastic flow of material can be due to bearing capacity failure leading to lateral extrusion of material from beneath the pillar or swell in the case of mudstone and claystone. Buckling is usually associated with stronger floor bands, whereas plastic flow is usually associated with weak material, such as claystone. However, the different mechanisms often work in conjunction, the distinctions can be very subtle and the net results can be virtually identical.

Identifying the controlling factors can therefore be a major challenge. Although it is possible to conduct a basic analysis by borrowing and adopting simple bearing capacity concepts from civil engineering, it should be noted that the mining situation constitutes a far more complex problem, due to :

(i) The much greater area of floor exposed than in civil engineering projects, the potential for greater variation in conditions and the proportionally smaller budget for geotechnical testing and analysis.

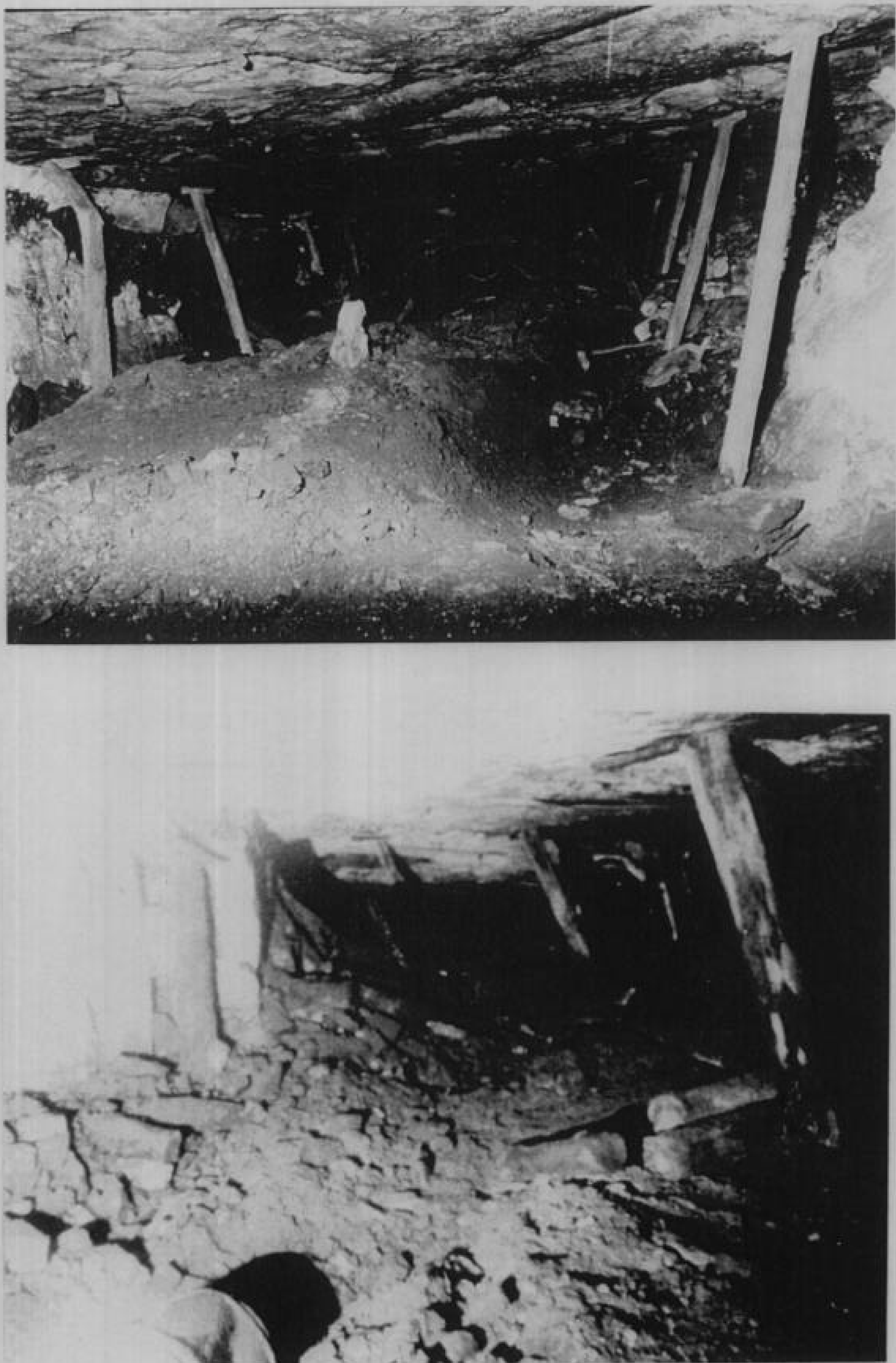


Figure 8.1. Floor Heave Natal and at an Australian Colliery

(ii) The loading system is more complicated than in foundation design, being a function of the strength and stiffness of all the elements in the system, the pillars, the immediate and upper roof and the immediate and deeper floor. In mining, deformation leads to a significant redistribution of stress; monitoring, analysis and design should therefore attempt to involve the total system."

Galvin (1995) summarised foundation failure and included the example

- where the floor material has only marginally lower in situ strength properties than the pillar.

In this case, high loads can be generated in the pillar prior to the onset of failure. Bearing capacity failure develops around the edges of the pillar because:

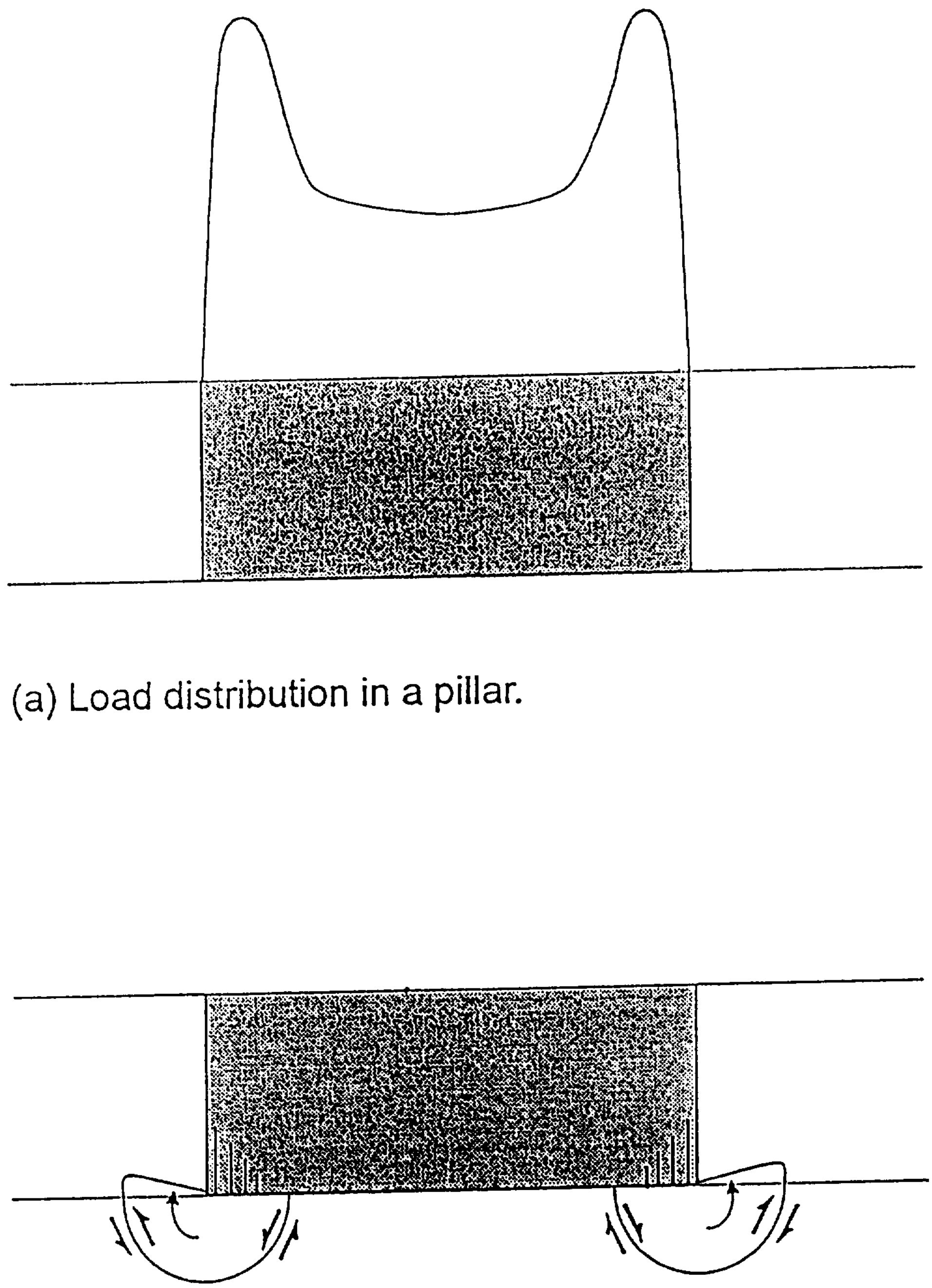
i) Peak pillar loads occur close to the pillar edges, Figure 8.2(a).

ii) The floor strata are weaker in these regions due to the removal of the vertical confinement.

iii) As a result of removing the vertical confinement, the floor strata near the pillar edges are both loaded in shear and free to fail in shear, Figure 8.2(b). Since the shear strength of coal measure rocks is typically only half that of their compressive strength, the floor strata may fail in shear in the vicinity of pillar edges. This can cause blocks of floor strata to rotate out from under the pillar edges into the roadway.

- Loss of bearing capacity around the edges of a pillar has two effects:

i) Load previously supported by the failed foundation is transferred further into the pillar.



(a) Load distribution in a pillar.

(b) One.type of bearing capacity failure
of the floor.

Figure 8.2 Shear Foundation Failure of a Moderate Strength Floor

ii) Pillar strength is reduced due to a reduction in the surface area of the end constraints.

- Three factors interact to determine whether stability can be re-established:

i) Is the confinement that builds up as failure progresses under (or over) the pillar sufficient to arrest the foundation failure?

ii) Is the increase in pillar load and the reduction in pillar strength that results from foundation failure sufficient to indirectly induce pillar failure?

iii) What effect does the resulting increase in effective bord width have on roadway stability and, if roof control is lost, what effect does the increase in effective pillar height have on pillar strength?

- A feature of bearing capacity failure in strata of moderate or higher shear strength is that it tends to progress gradually rather than suddenly. Resistance to the process can build up as it progresses. Energy (load) has to be continuously added to the pillar system to overcome resistance and drive the process. Unless the situation is one of pure dead-weight loading (load controlled, low stiffness system) and the load can "chase" displacement, this energy is not immediately available. In mining situations, one is usually dealing with a displacement controlled system where load input is governed by displacement of the roof strata. The stiffness of the roof strata controls the rate of loading into the pillar system. Hence, failure usually develops over time.

- As pillar width increases, greater confinement is provided to the failing foundation and there is increased probability of arresting foundation failure and maintaining pillar stability. However, whilst the pillar may function as an effective regional support element, roadways may become unserviceable due to very poor local conditions.

In civil engineering practice, Terzaghi's method is widely applied for the calculation of the bearing capacity of a foundation. The method has also been extensively used for determining the strength of floors beneath pillars in underground mines.

In Figure 8.3, the foundation strength qu is given by :

$$q_u = c\,N_c + q\,N_q + \lambda b N_\lambda \qquad (8.1)$$

where :

c = cohesion (MPa)

q will normally be zero unless the failure is likely to take place in a weak bed some distance, z, below or above the floor or roof contact.
λ = specific weight of the material, MN/m^3
b = half the pillar width, m

N_c, N_q and N_λ are bearing capacity factors which depend on the angle of friction of the material. Approximate values of these factors can be read off the graph in Figure 8.4.

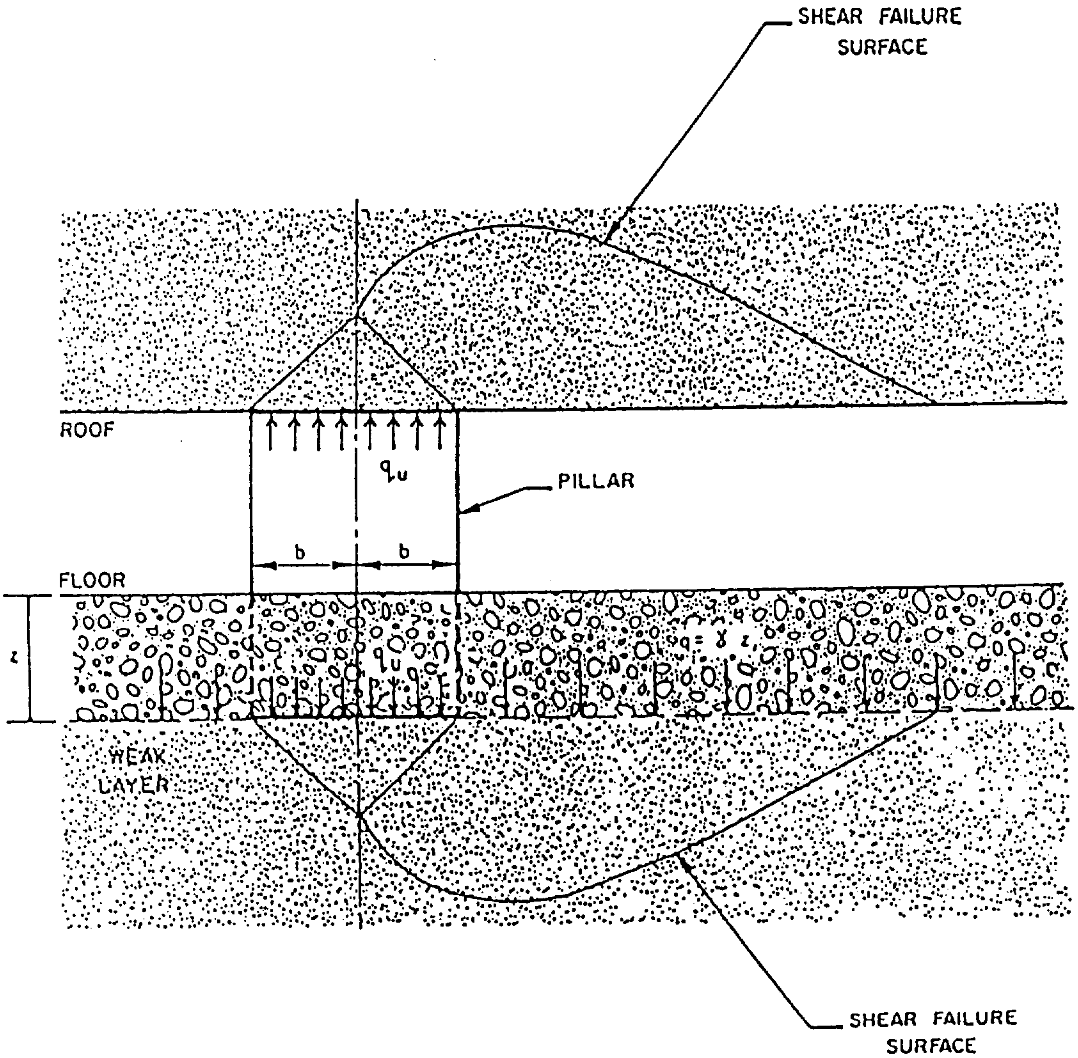


Figure 8.3 Foundation Strength for Pillar Stability

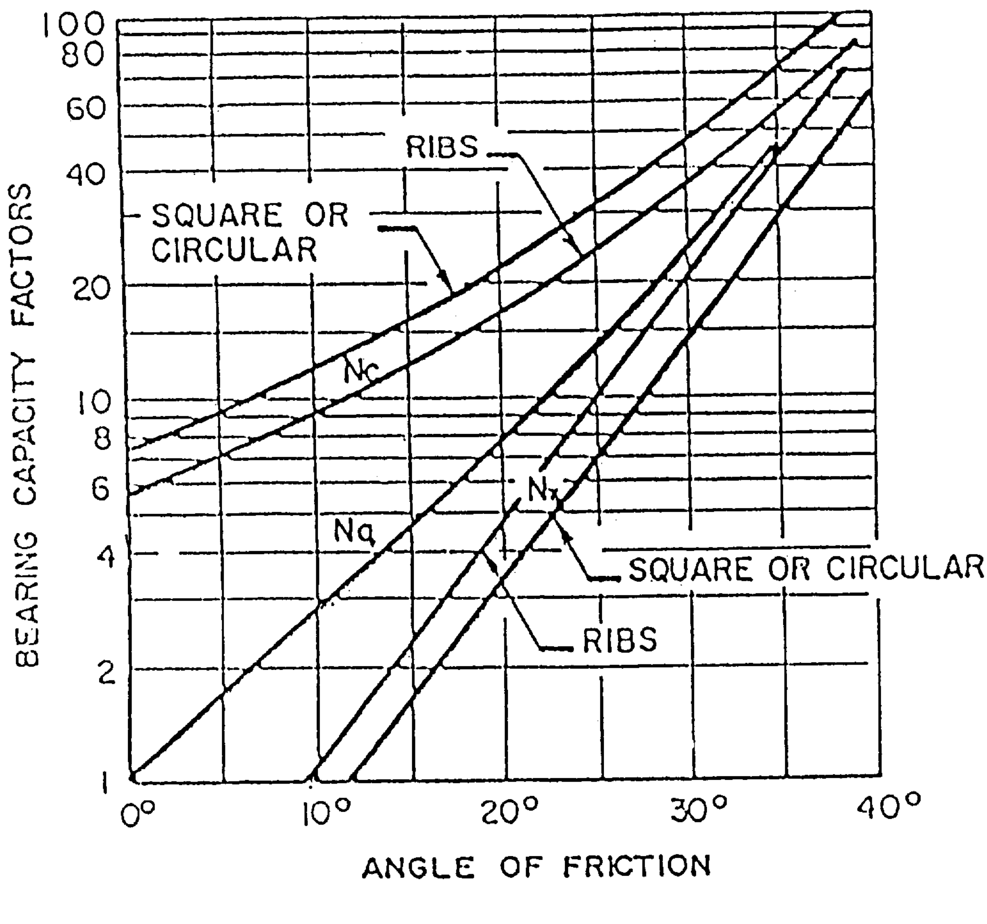


Figure 8.4 Foundation Bearing Capacity Factors

The significance of the above relationship is that the bearing capacity of the foundation of a pillar is directly proportional to the width of that pillar. Measures adopted to improve pillar stability that effectively result in an increase in pillar width will therefore also have a positive effect on foundation stability.

Unfortunately, there are problems associated with the application of Terzaghi's formula. Firstly, the bulk material properties are difficult to define precisely, and bearing capacity is particularly sensitive to friction angle. Secondly, pillar loading is more complex than the regular loading of a footing; depending on the geometry and the stiffness of the strata, the pillar edge stresses can be significantly greater than the average stress.

The Terzaghi method also ignores the thickness of the weak layer , whereas in practice, the bearing capacity of a foundation is found to be inversely proportional to the thickness of the weak layer in the floor. As the thickness of the weak layer decreases, the constraint (due to friction) increases at the interfaces with adjacent stronger layers.

Therefore :

Bearing Capacity = function (pillar width/weak layer thickness)

The USBM is conducting research into the bearing capacity of weak, layered floors [Chugh et al (1990)] but currently, no proven analytical approach is available.

Australian experience has shown that strata that are potentially susceptible to floor heave can be indicated by a series of index tests. These include Moisture Content, Slake Durability Index, Density, Swell Index, and clay type and proportion. A data base of geotechnical tests has been collected and compared to the conditions experienced during mining. This method has built a data base used in the prediction of likely conditions given the thickness of strata, its geotechnical index characteristics, the pillar geometry and the stress regime.

Galvin (1995) states that

- Pillar behaviour in the presence of a soft stratum is a complex issue which requires further research.

- Whilst many of the fundamental mechanistic principles have been established in civil engineering foundation theory, they do not find direct application to mining.

- Some reasons for this include:

i) The properties of the engineering materials in mining are both more complex and more vaguely defined or known.

ii) Civil engineering foundation theory is incapable of dealing with many of the geometries encountered in mining e.g. interaction between foundations.

iii) Civil engineers have the capability to engineer the problem out of the design e.g. by adding reinforcement to concrete to improve its tensile performance or by excavating sub standard material, rather than having to modify the design to control the problem.

- In the interim, operators should be aware that:

1 On the basis of soil and foundation engineering principles, the ratio of pillar width, w, to weak floor thickness, l, i.e. w/l has a major controlling influence on the development of bearing capacity failure. Figure 8.5 shows that the bearing pressure capacity of the floor decreases almost four fold when the thickness of weak floor layers increases from 0.25 m to 2 m under a 24 m wide pillar. Increasing pillar width, w, in weak strata environments offers many

advantages in that it results in an increase in pillar strength, a reduction in pillar load and an increase in safety factor against bearing capacity failure.

2 Some soft strata e.g. claystones undergo significant consolidation over time under the effects of pillar load. Differential floor displacement and resulting surface subsidence due to consolidation should not be taken as indicators of pillar failure or bearing capacity failure.

3 One of the most vexing issues still to be addressed in a soft strata environment is the effects of water on the strength of the foundation material in the long term.

The significance of shear strength in Figure 8.5 should be examined to assess the suitability of the criteria to South African conditions.

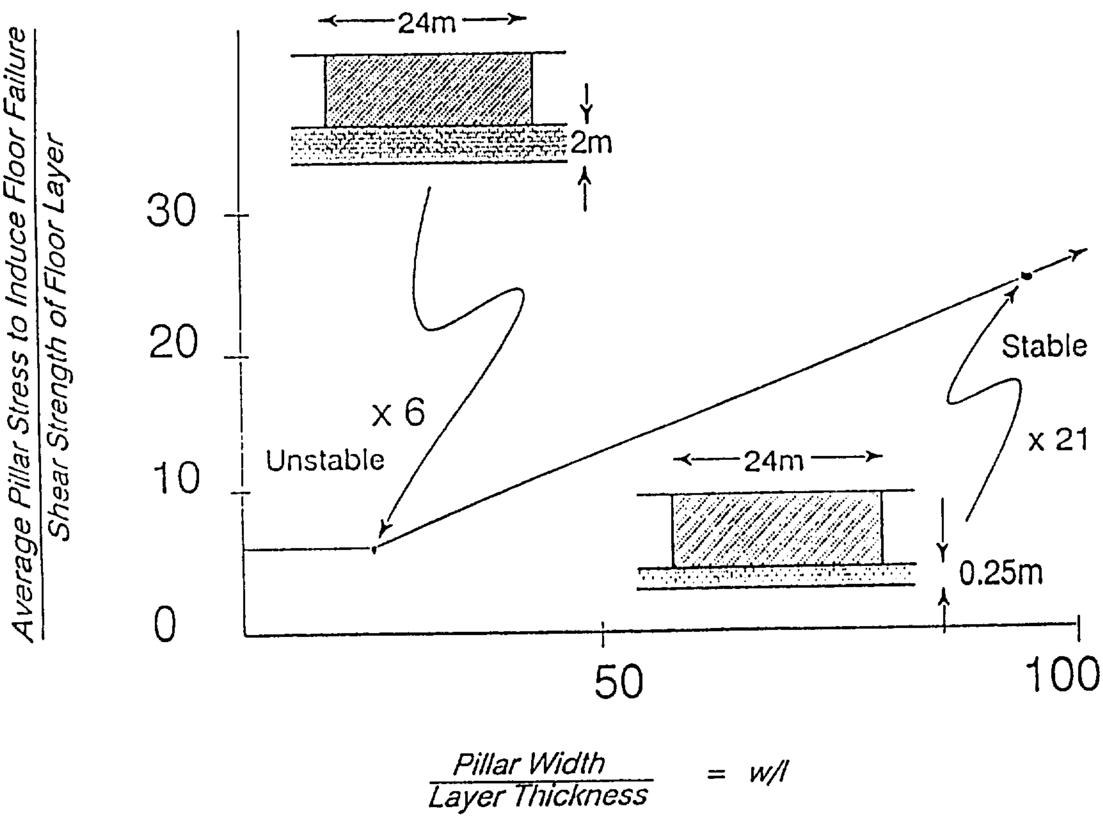


Figure 8.5 Variation in Soft Floor Bearing Capacity with Thickness of Soft Floor

9.0 CONCLUSIONS

Based on the research conducted during the project the following can be concluded under the enabling outputs:-

Individual seam strength

Pillar rating of over 300 panels was conducted. The classification system highlighted a method of ranking the relative performance of coal pillars. This was shown to be effective in the assessment of panel conditions at Tavistock Colliery where pillar deterioration had occurred.

Pillar classification can provide an assessment of the suitability for intended pillar extraction of pillars formed previously. The attractiveness of extracting long-standing pillars will increase in the future.

The classification system indicated that there was a variation in effective strength within collieries and across the coalfields. The relative ranking showed that the seams, ordered from strongest to weakest were:

Top-Bottom	Klip River
Bottom	Klip River
Alfred	Vryheid
C	Eastern Transvaal
Dundas	Utrecht
No 7	Soutpansberg
No 4	Highveld
Gus	Utrecht
No 2	Highveld
No 1	Witbank
No 2	Witbank
No 5	Highveld
No 5	Witbank

No 2A	Vereeniging
No 2B	Vereeniging
Main	Zululand
No 3	Vereeniging
Alfred	Utrecht

An improvement to the classification system would be the incorporation of the extent of structural discontinuities within the coal pillar.

The extensive laboratory testing program provided valuable information on the laboratory strength of South African coals. The statistical re-analysis showed that the strength of the eight blocks from the No 2 Seam, Witbank Coalfield occurred in a fairly tight strength range; and that laboratory coal strengths from individual seams or mines could deviate to a significant although relatively small extent from the overall average.

While the laboratory results cannot be directly applied to the field a methodology for the estimation of relative strength between coal seams has been established. This could be of significance when mining a greenfield region.

Results from an investigation into the back analysis of collapsed and intact pillar cases in Australian collieries were summarized. These results are considered highly significant as the resulting design formula is extremely similar to that obtained from South African coalfields. It was concluded that the strength of a pillar, once the pillar width to height ratio exceeds about 2,0, is increasingly determined by geometry and coal strata contact friction, and less by intrinsic coal strength.

Three dimensional computer models were evaluated for their suitability to coal mining situations. The model MINLAY was found to provide suitable linear-elastic solutions and was widely used in the analysis of seam convergence and skin and average stresses.

Remote and continuous monitoring of field instrumentation is essential for the further understanding of pillar behaviour. The field trials conducted highlighted this requirement and systems have been developed to meet this.

Pillar Foundation Failure

Given the potential seriousness of foundation failure on pillar stability the collection of index tests on suspect strata, for example at the No. 5 Seam Witbank Coalfield, should be initiated. While this mode of failure may only occur some time after mining the failure has the potential to result in the loss of access ways as experienced by the trapping of miners at Emaswati Colliery. Two sites were monitored for floor behaviour during the project. These field trials aimed to establish the deformations based on instrumentation results and to enable a comparison with numerical modelling results.

Foundation stability is dependent on the floor strata properties and thickness in relation to the pillar width. Overseas research has suggested possible design parameters. To obtain these parameters an investigation of shear strength and frictional properties is indicated.

Current index tests and geotechnical logging can highlight potentially susceptible foundation layers. Additional research is required before design criteria can be applied.

Techniques and Guidelines for Safe and Stable Pillar Design

- Applying the Salamon design formulae
- Applying the classification system will confirm the performance of pillars formed in a colliery.
- Incorporating the evaluation of discontinuities (after Esterhuizen) will examine the extent of structural influences.
- Testing of the floor for geoduribility by sampling and conducting index and laboratory tests. Should the floor material be found to be susceptible to

foundation failure the design of foundation stability becomes a paramount requirement.

10.0 FUTURE DIRECTIONS

The further collection and evaluation of collapsed pillar cases is required. This will highlight any anomalies in current design procedures and point to any significant missing design parameters. Further laboratory testing is required to expand on the existing data base. Strata material properties and their relation to index tests will assist in the establishing of the type of mining environment and the potential for foundation failure. The incorporation of structural discontinuities into the classification system for improved assessment of similar geotechnical areas could be a major contribution to the design of safe pillar systems in South Africa.

ACKNOWLEDGEMENTS

The project team wish to thank the management and personnel of the South African collieries and the staff of the Department of Minerals and Energy Affairs for their co-operation and assistance during the project. Without the assistance of the collieries the research effort would be impossible. The discussions and encouragement from the SIMRAC committee is also appreciated. Messrs T. Ozan, K. Akermann and G. Ashworth are thanked for their contribution during the project.

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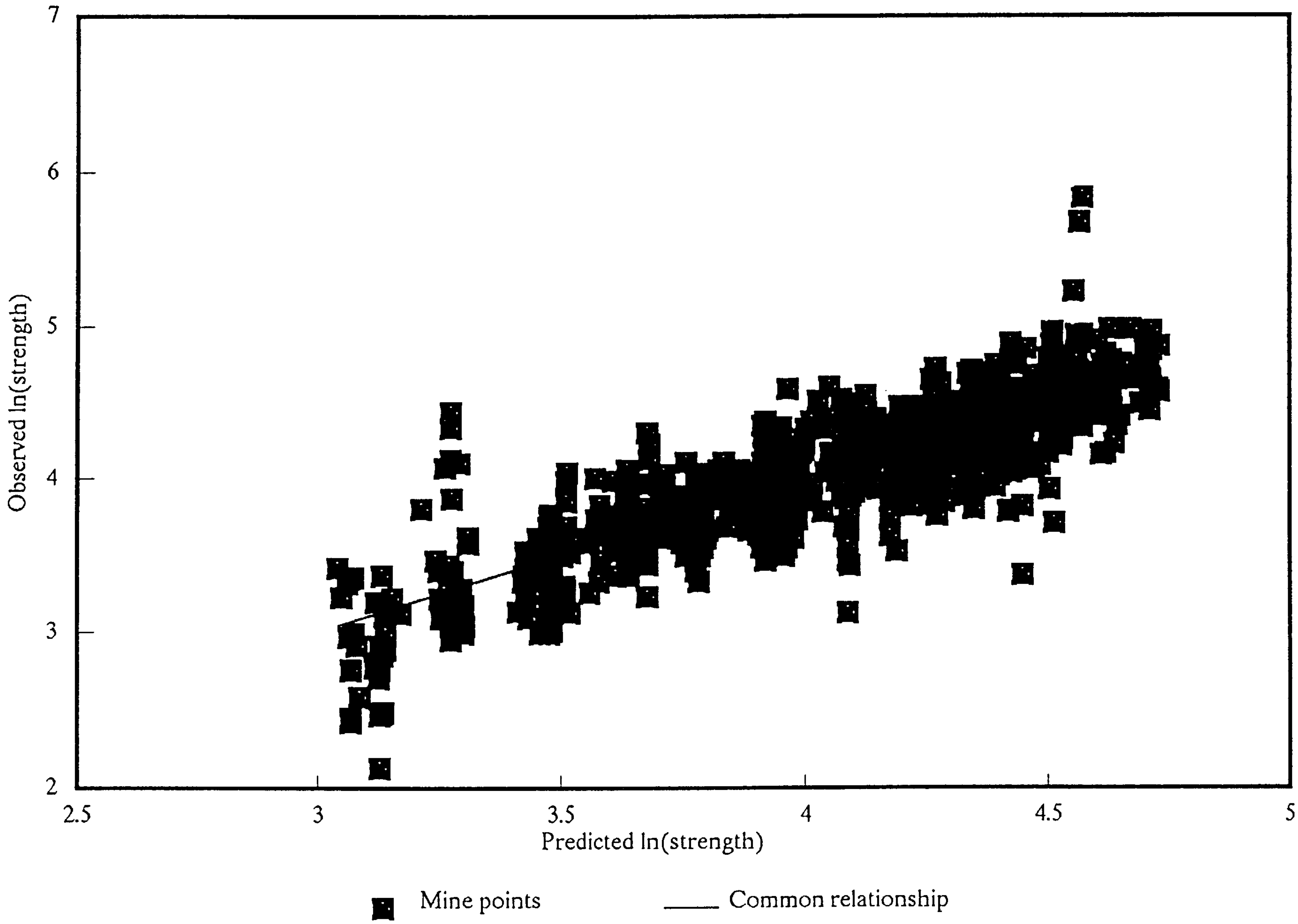
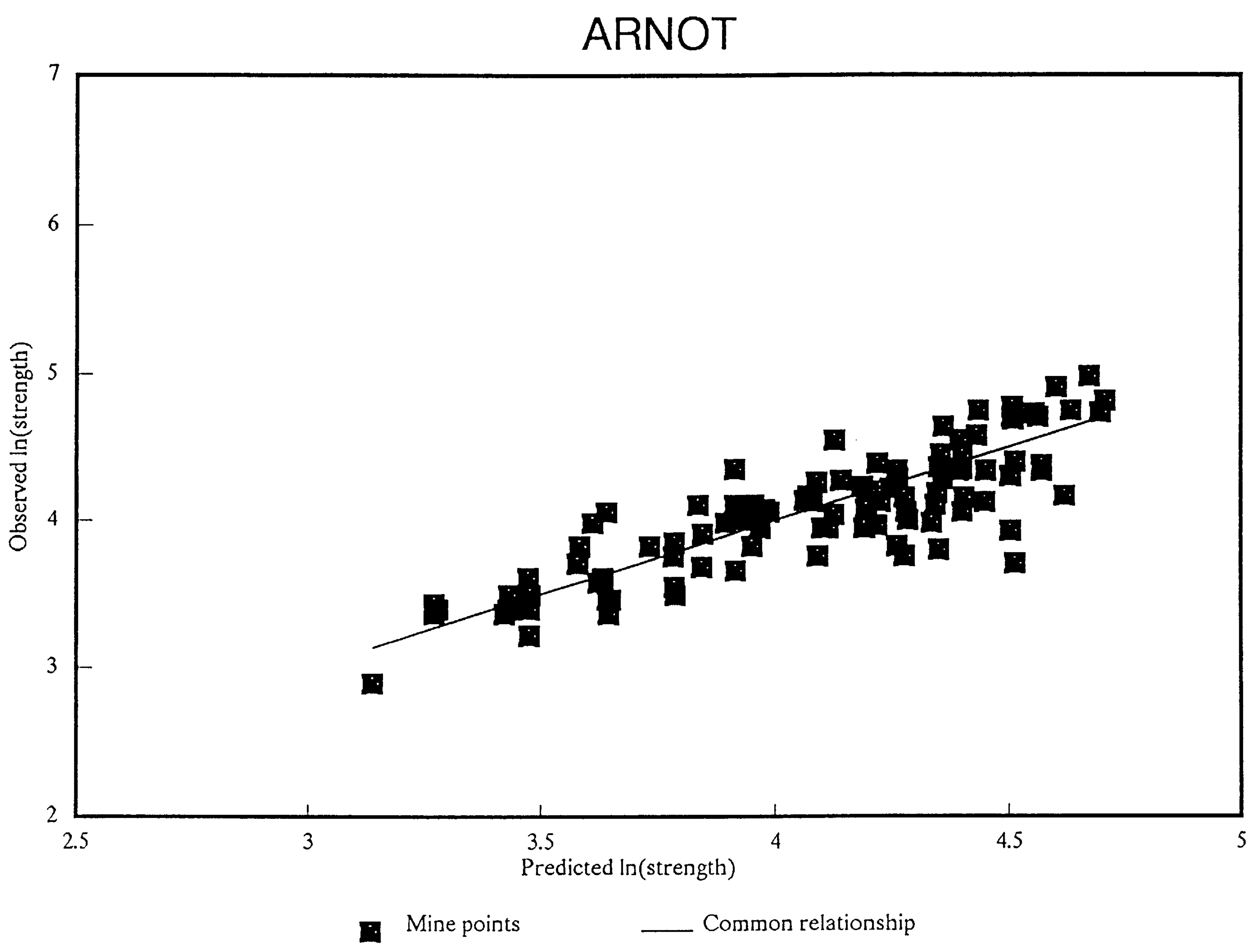
APPENDIX A

Results from the Statistical Analysis

ALL MINES



7
6
5
4
3
2
Observed ln(strength)
2.5
3
3.5
4
4.5
5
Predicted ln(strength)
Mine points
Common relationship


ARNOT
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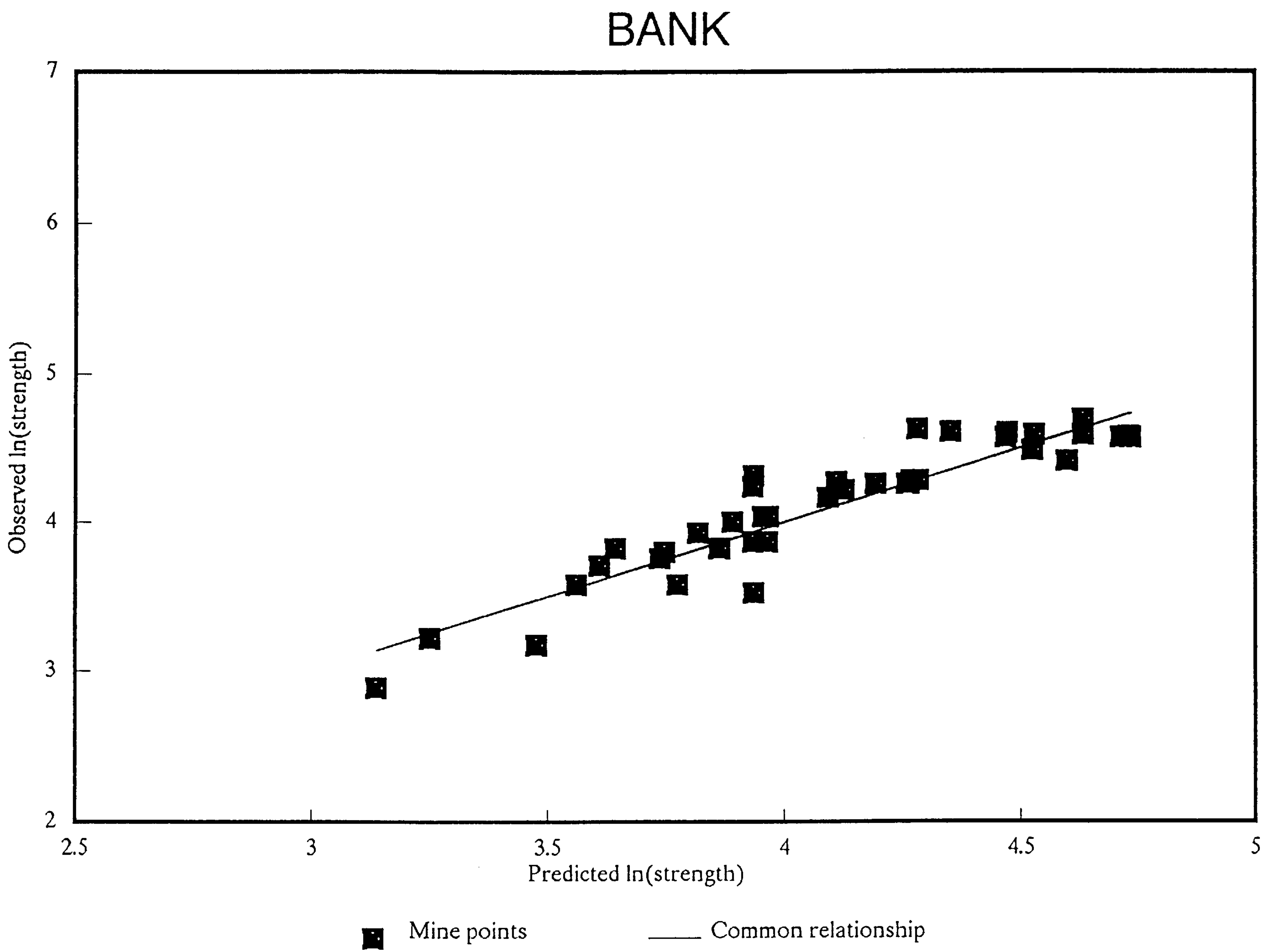
Predicted ln(strength)
Mine points
Common relationship



BANK
7
6
5
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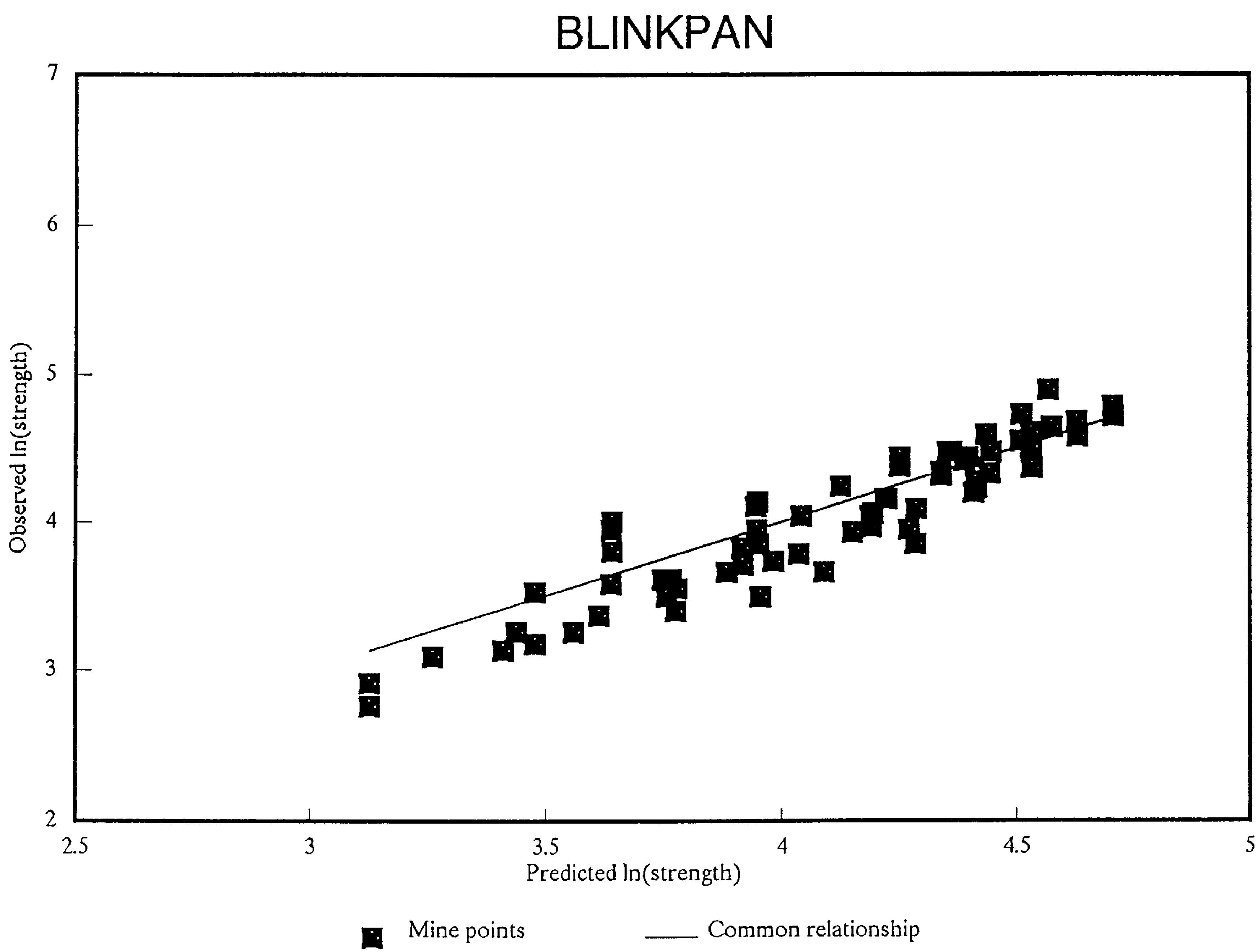
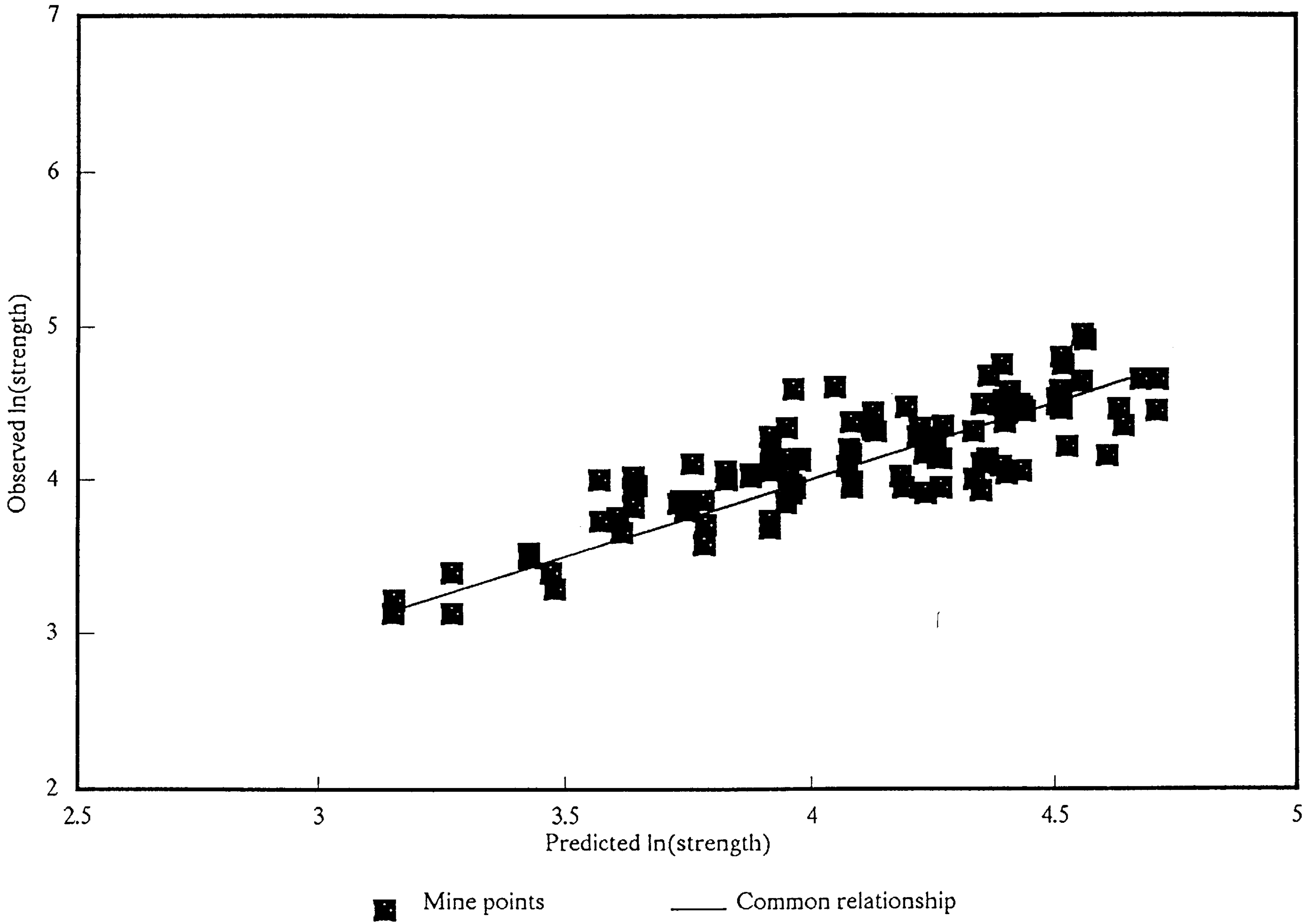
2
Observed ln(strength)
2.5
3
3.5
4
4.5
5
Predicted ln(strength)
Mine points
Common relationship


BLINKPAN
Observed ln(strength)
7
6
5
4
3
2
2.5
3
3.5
4
4.5
5
Predicted ln(strength)
Mine points
Common relationship

DELMAS Block 2



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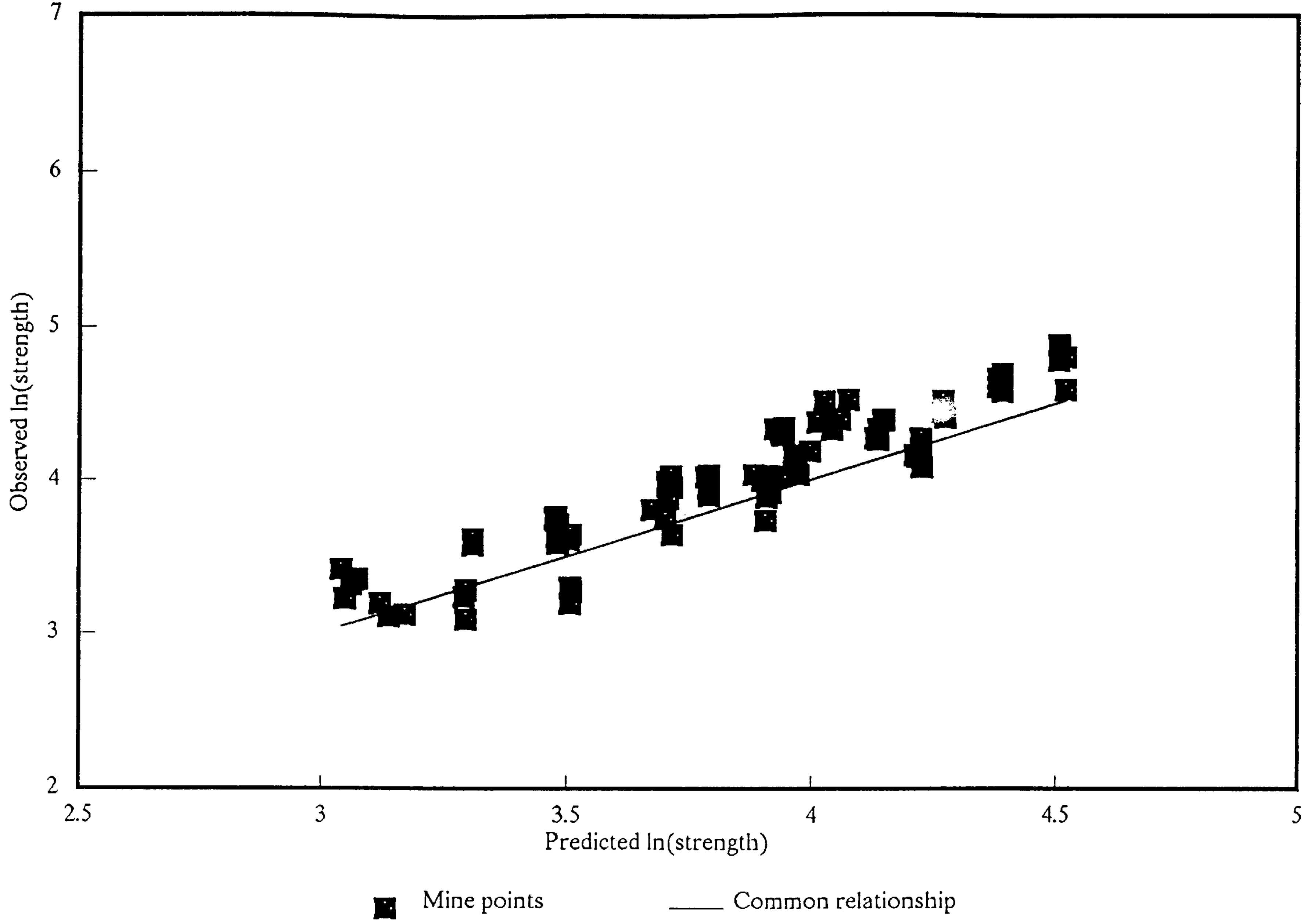

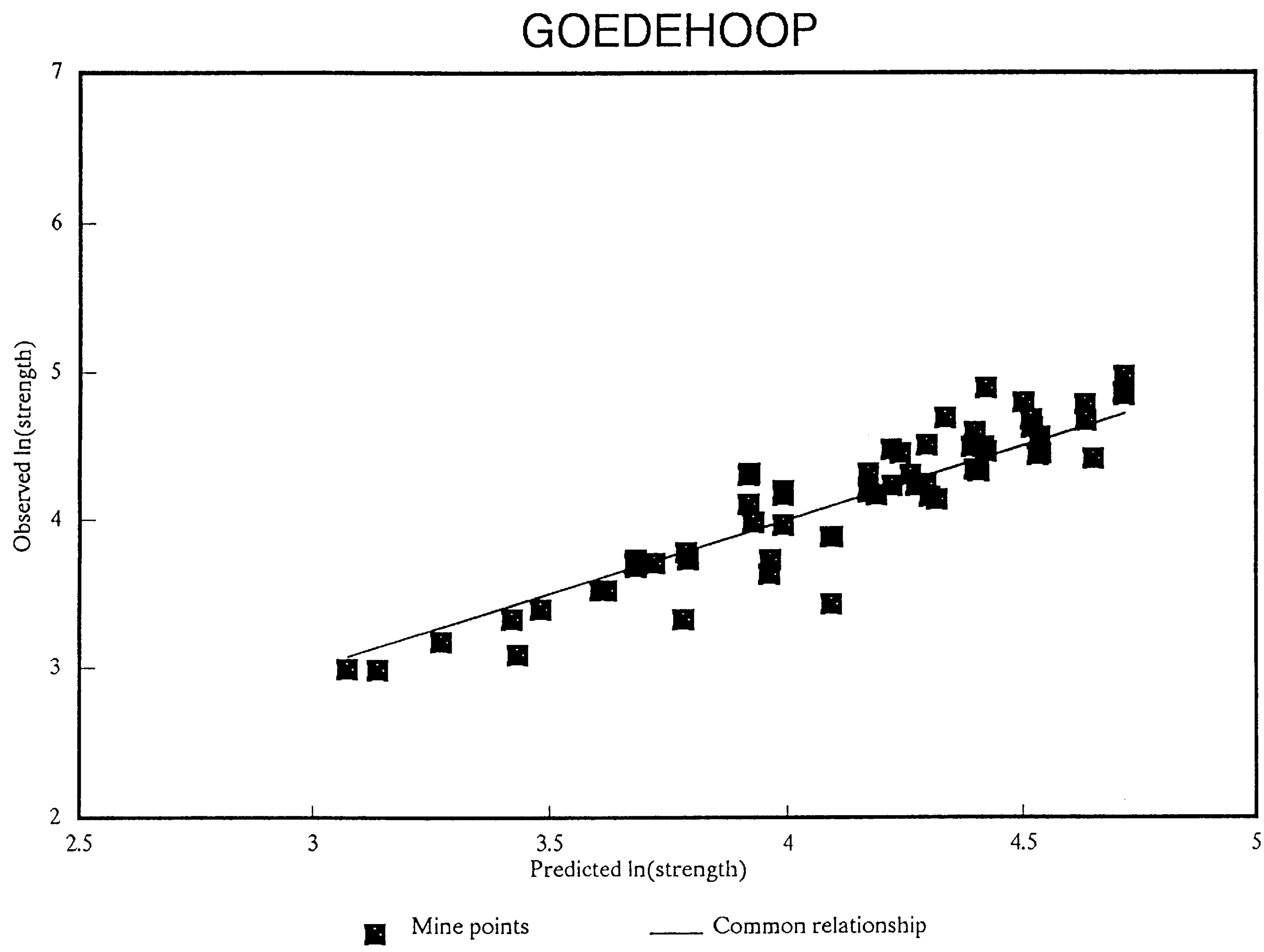
GOEDEHOOP
Observed ln(strength)
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6
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3
2
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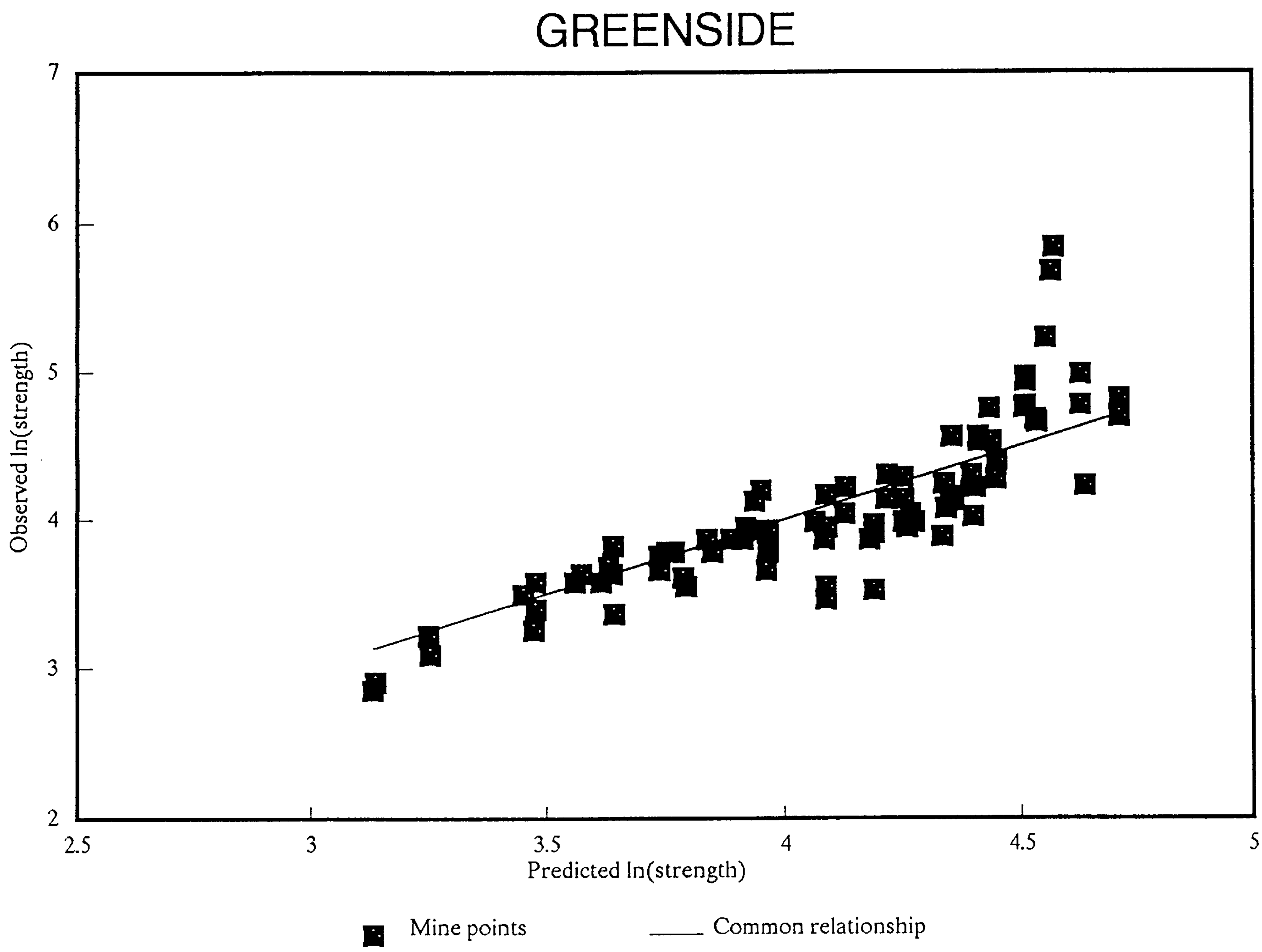
3
3.5
4
4.5
5
Predicted ln(strength)
Mine points
Common relationship


GREENSIDE
Observed ln(strength)
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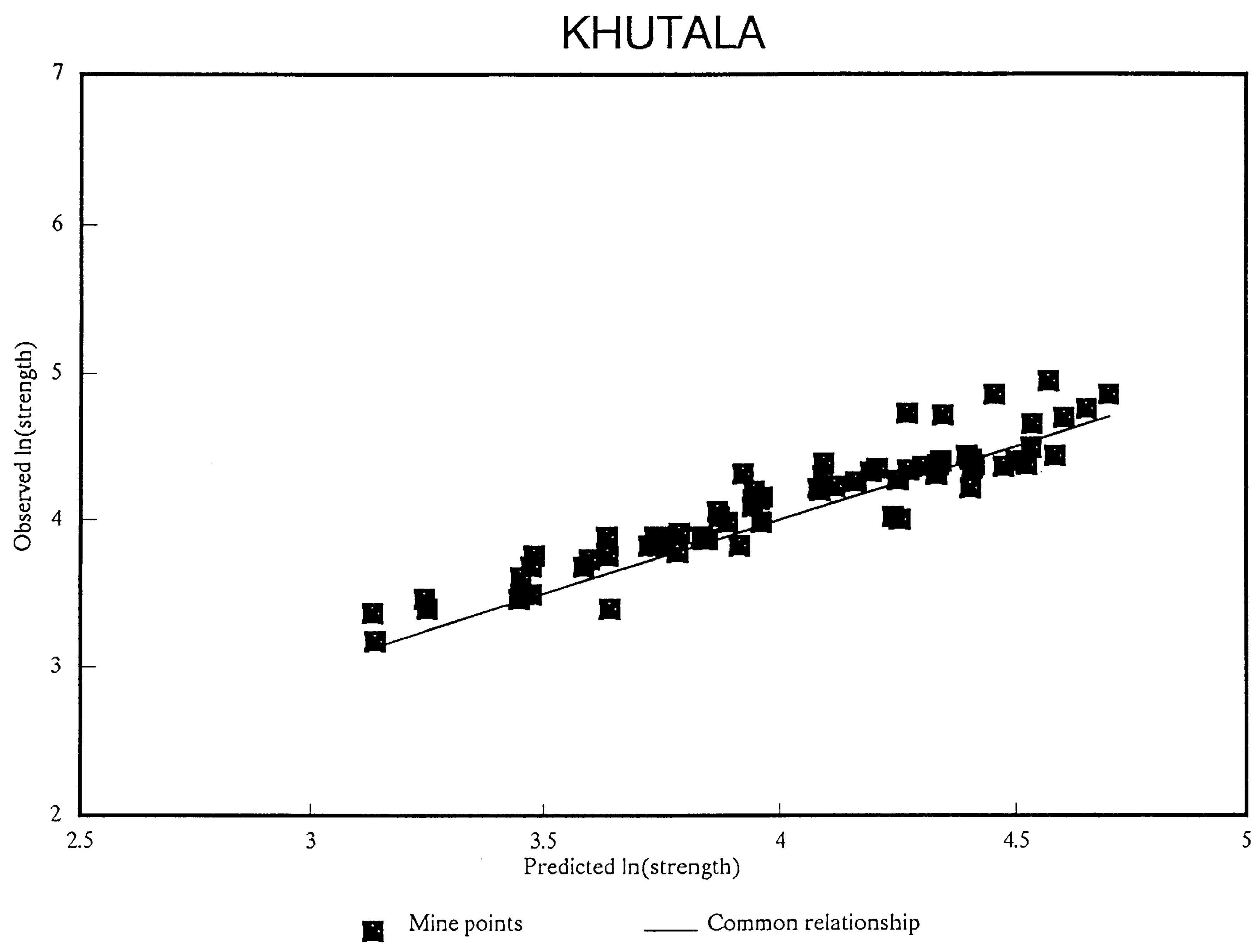
6
5
4
3
2
2.5
3
3.5
4
4.5
5
Predicted ln(strength)
Mine points
Common relationship


KHUTALA
Observed ln(strength)
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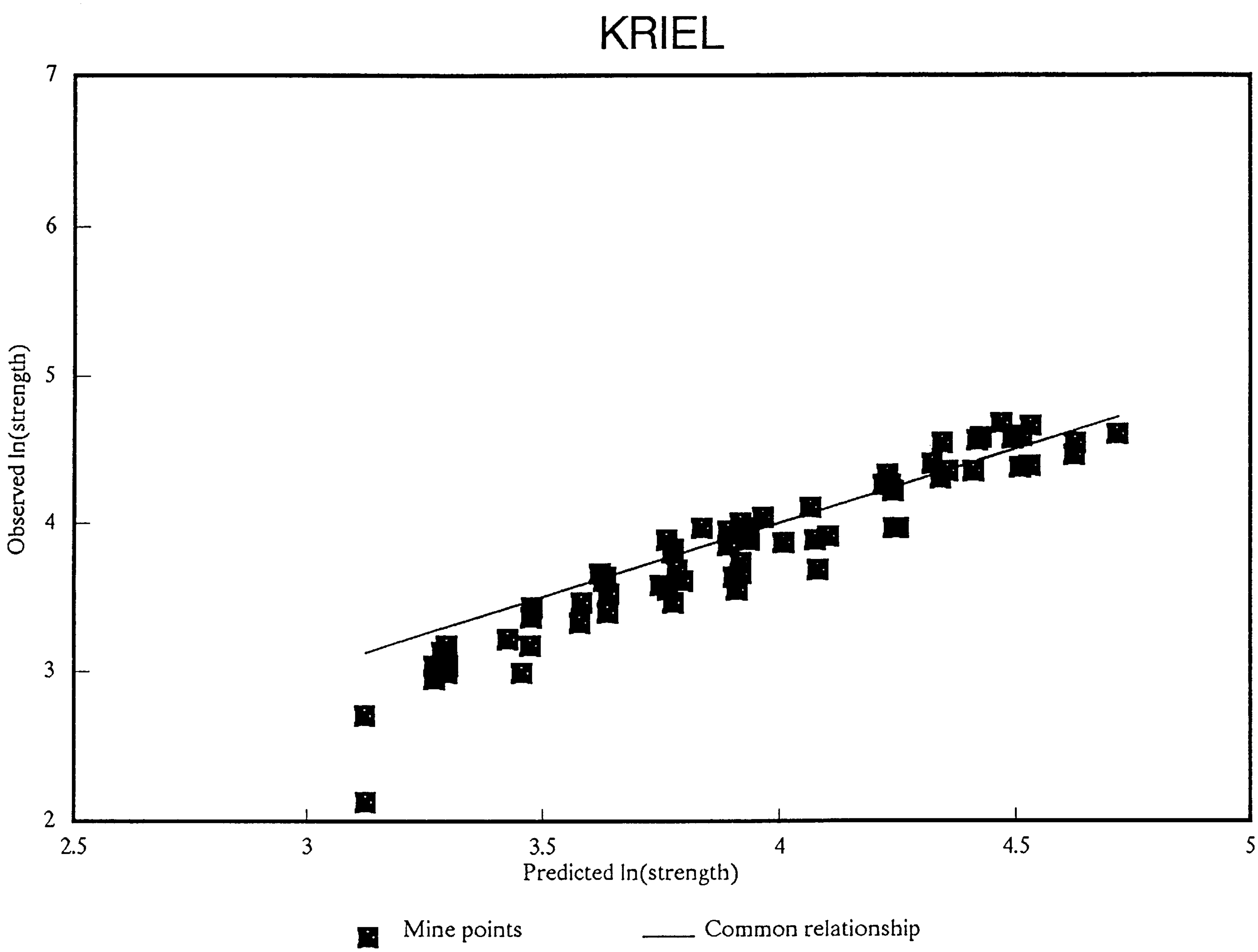
6
5
4
3
2
2.5
3
3.5
4
4.5
5
Predicted ln(strength)
Mine points
Common relationship


KRIEL
Observed ln(strength)
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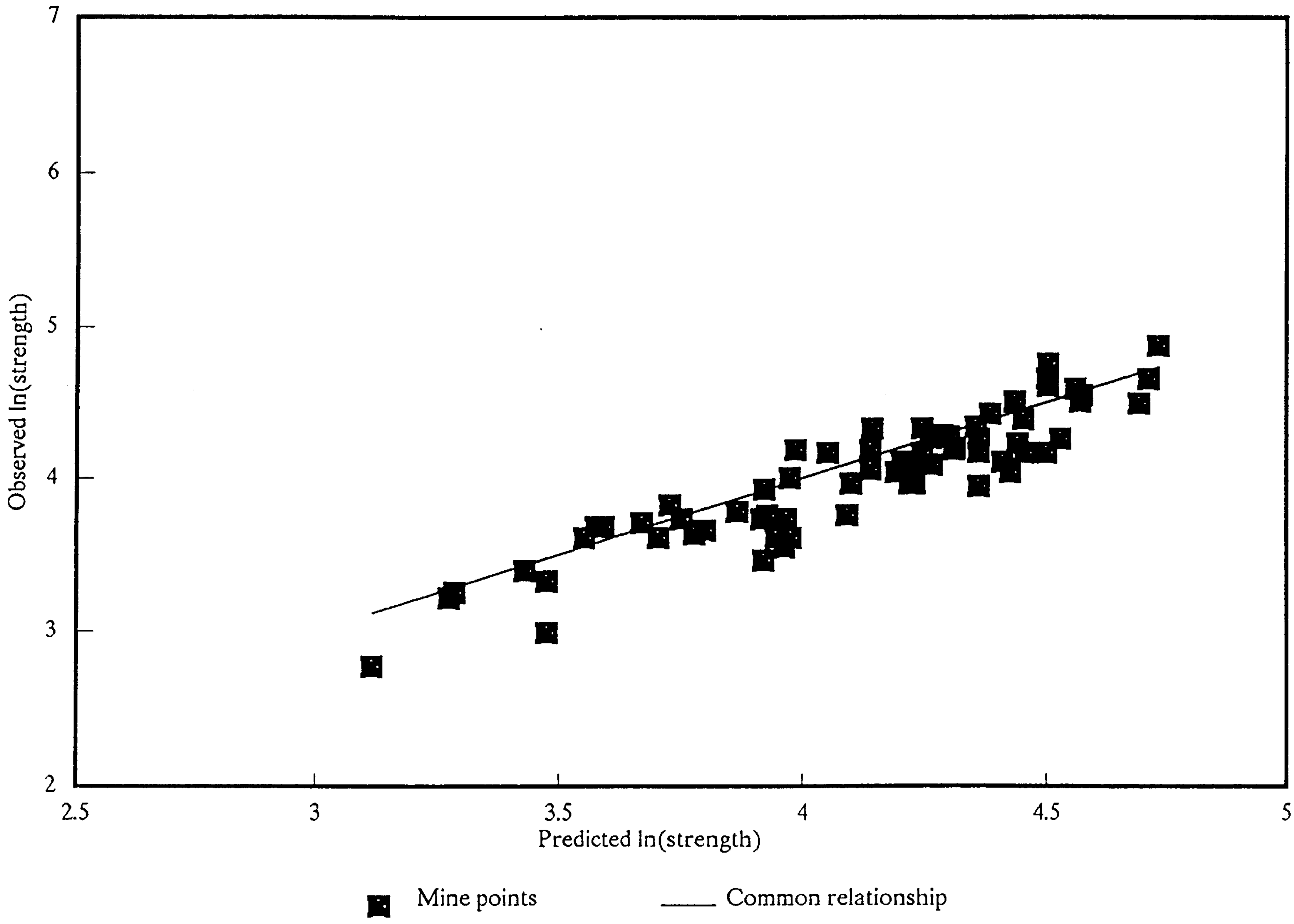
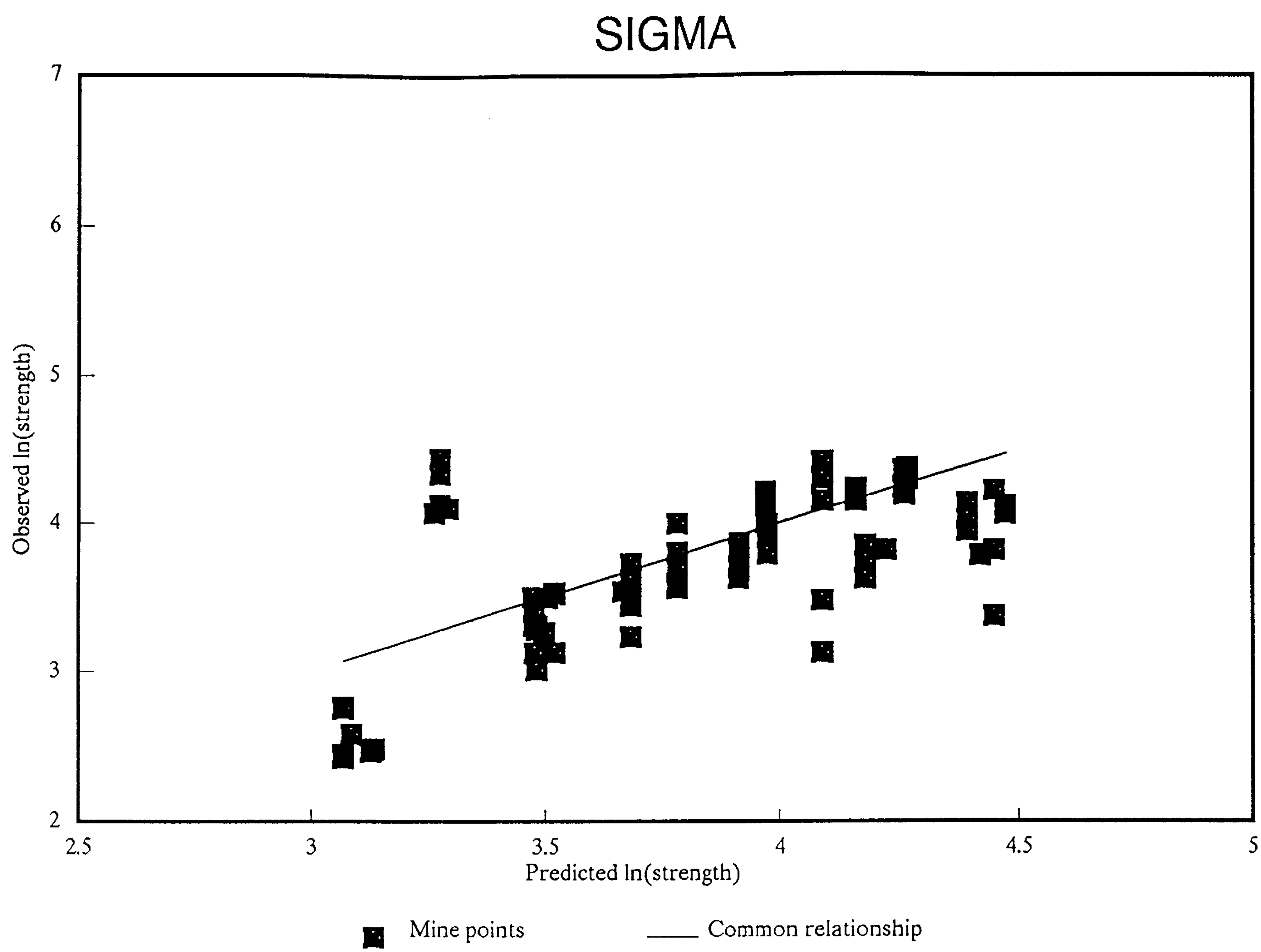
3
2
2.5
3
3.5
4
4.5
5
Predicted ln(strength)
Mine points
Common relationship

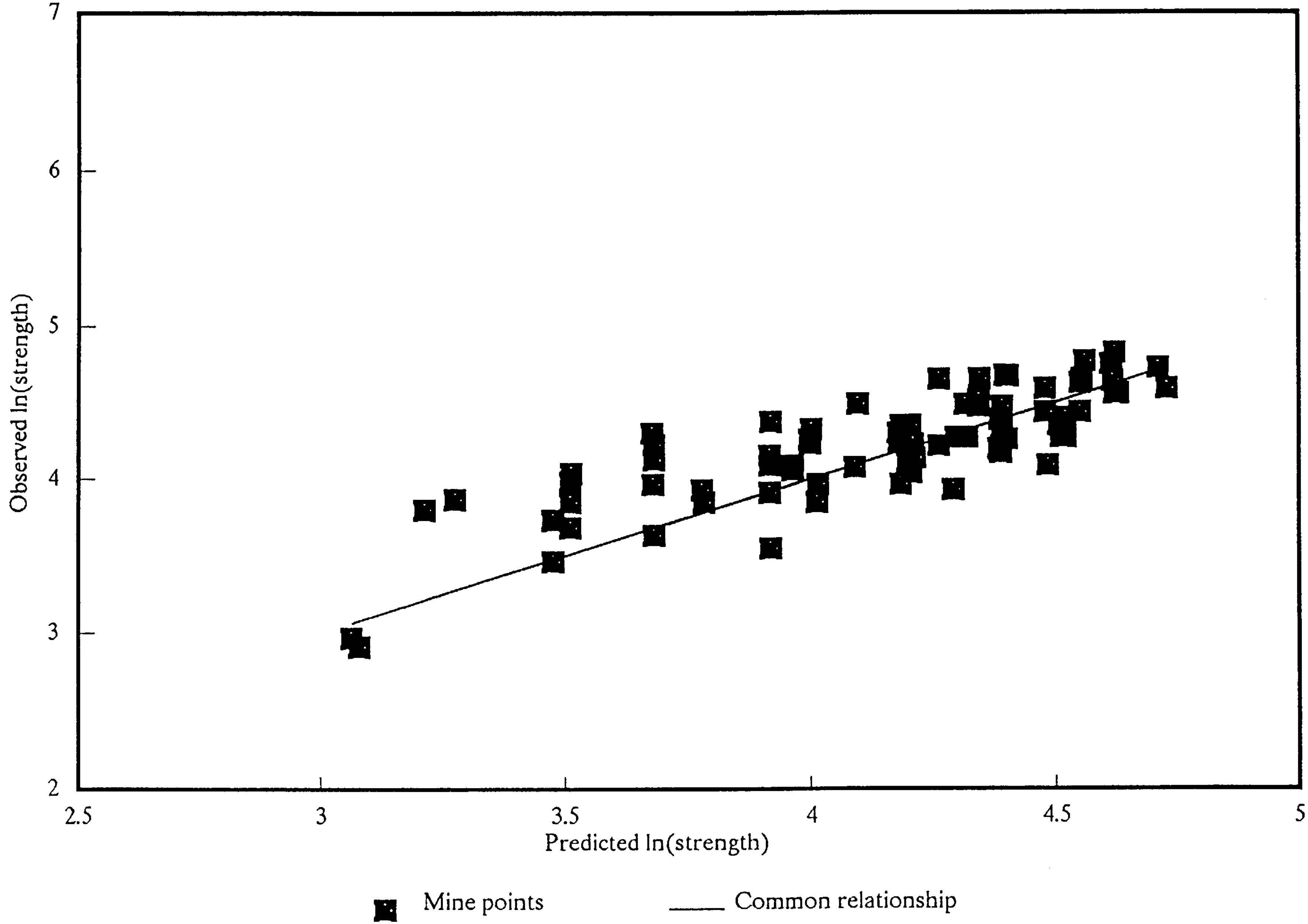
SECUNDA



7
6
5
4
3
2
Observed ln(strength)
2.5
3
3.5
4
4.5
5
Predicted ln(strength)
Mine points
Common relationship


SIGMA
Observed ln(strength)
7
6
5
4
3
2
2.5
3
3.5
4
4.5
5
Predicted ln(strength)
Mine points
Common relationship

ZULULAND ANTHRACITE



Coefficient	Term	Variance 1m block (w/h=1)		Variance 0.1m block (w/h=1)		Variance 0.1m block (w/h=3)	
		Line	Observation	Line	Observation	Line	Observation
2.845	intercept						
−0.310	ln(width)						
0.449	ln(width/height)						
−0.106	ARNOT	0.00132	0.0541	0.00067	0.0535	0.00051	0.0533
−0.060	BANK	0.00226	0.0551	0.00147	0.0543	0.00134	0.0542
−0.157	BLINKPAN	0.00167	0.0545	0.00096	0.0538	0.00081	0.0536
−0.035	DELMAS Block 2	0.00134	0.0541	0.00067	0.0535	0.00050	0.0533
0.091	DELMAS Block 1	0.00153	0.0543	0.00066	0.0535	0.00071	0.0535
−0.066	GOEDEHOOP	0.00181	0.0546	0.00103	0.0538	0.00089	0.0537
−0.113	GREENSIDE	0.00144	0.0542	0.00081	0.0536	0.00063	0.0534
0.010	KHUTALA	0.00147	0.0543	0.00086	0.0537	0.00071	0.0535
−0.187	KRIEL	0.00152	0.0543	0.00087	0.0537	0.00079	0.0536
−0.183	SECUNDA	0.00167	0.0545	0.00095	0.0538	0.00079	0.0536
−0.213	SIGMA	0.00154	0.0543	0.00066	0.0535	0.00071	0.0535
0.000	ZULULAND ANTHRACITE	0.00176	0.0546	0.00081	0.0536	0.00072	0.0535

Coefficient	Term	sd 1m block (w/h=1)		sd 0.1m block (w/h=1)		sd 0.1m block (w/h=3)	
		Line	Observation	Line	Observation	Line	Observation
2.845	intercept						
−0.310	ln(width)						
0.449	ln(width/height)						
−0.106	ARNOT	0.0364	0.233	0.0259	0.231	0.0225	0.231
−0.060	BANK	0.0475	0.235	0.0383	0.233	0.0367	0.233
−0.157	BLINKPAN	0.0408	0.233	0.0310	0.232	0.0285	0.232
−0.035	DELMAS Block 2	0.0366	0.233	0.0259	0.231	0.0225	0.231
0.091	DELMAS Block 1	0.0391	0.233	0.0257	0.231	0.0267	0.231
−0.066	GOEDEHOOP	0.0426	0.234	0.0320	0.232	0.0298	0.232
−0.113	GREENSIDE	0.0379	0.233	0.0285	0.232	0.0250	0.231
0.010	KHUTALA	0.0383	0.233	0.0294	0.232	0.0267	0.231
−0.187	KRIEL	0.0390	0.233	0.0295	0.232	0.0280	0.232
−0.183	SECUNDA	0.0409	0.233	0.0308	0.232	0.0281	0.232
−0.213	SIGMA	0.0392	0.233	0.0257	0.231	0.0267	0.231
0.000	ZULULAND ANTHRACITE	0.0419	0.234	0.0284	0.232	0.0267	0.231

Mine	Logarithmic relationship			Un-logged relationship			Salamon formula equivalent		
	Intercept	Diam (mm)	Diam/length	Strength	Width (m)	Height (m)	Strength	Ratio	Volume
ARN	4.914	−0.303	0.394	16.8	0.092	0.394	16.4	0.294	0.101
BAN	4.943	−0.318	0.465	15.6	0.147	0.465	15.2	0.359	0.106
BLI	5.244	−0.413	0.473	10.9	0.060	0.473	10.6	0.335	0.138
DE2	4.666	−0.236	0.416	20.8	0.181	0.416	20.5	0.338	0.079
DEL	4.726	−0.230	0.537	23.0	0.307	0.537	22.6	0.460	0.077
GOE	5.342	−0.416	0.466	11.8	0.049	0.466	11.4	0.327	0.139
GRE	5.165	−0.394	0.513	11.5	0.119	0.513	11.2	0.382	0.131
KHU	4.865	−0.276	0.431	19.2	0.155	0.431	18.8	0.339	0.092
KRI	4.924	−0.363	0.573	11.2	0.210	0.573	10.9	0.452	0.121
SEC	4.421	−0.244	0.534	15.4	0.290	0.534	15.1	0.452	0.081
SIG	4.723	−0.289	0.356	15.3	0.067	0.356	14.9	0.259	0.096
ZAC	4.593	−0.179	0.336	28.6	0.157	0.336	28.2	0.277	0.060
Overall	4.941	−0.318	0.444	15.6	0.127	0.444	15.2	0.338	0.106

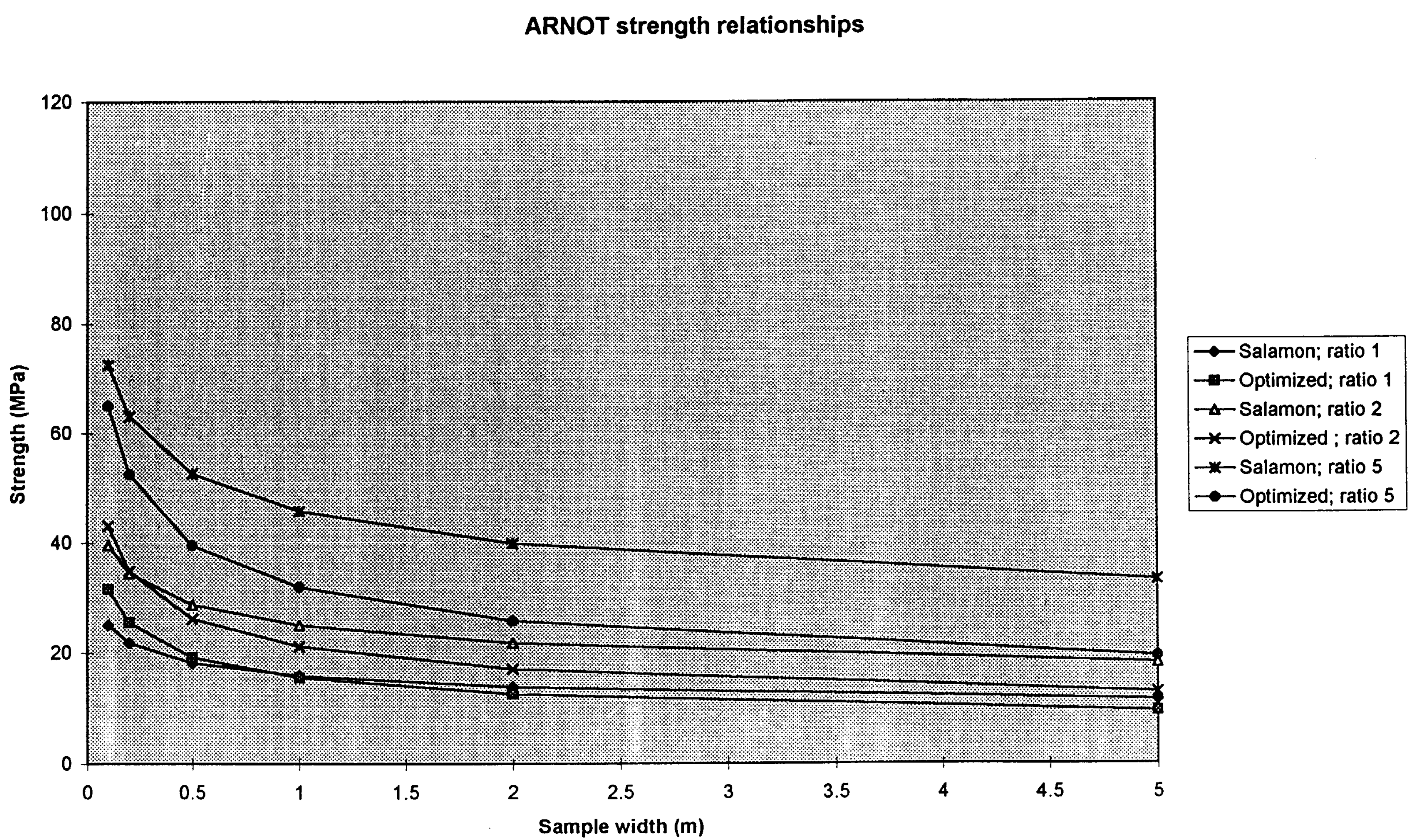
Block	Seam	Nobs	Mean	sd	t statistic	% diff str
ARN	2	105	−0.022	0.052	−0.425	−2.2%
BAN	2	40	0.021	0.027	0.777	2.1%
BLI	2	66	−0.074	0.037	−1.988	−7.1%
DE2	2	106	0.050	0.047	1.054	5.1%
DEL	2	87	0.168	0.027	6.331	18.3%
GOE	2	61	0.016	0.041	0.393	1.6%
GRE	2	85	−0.028	0.089	−0.310	−2.7%
KHU	2	74	0.095	0.021	4.581	10.0%
KRI	4	68	−0.104	0.038	−2.759	−9.9%
SEC	4C	68	−0.099	0.029	−3.383	−9.5%
SIG	2A	87	−0.136	0.146	−0.928	−12.7%
ZAC	Main	82	0.078	0.054	1.454	8.2%

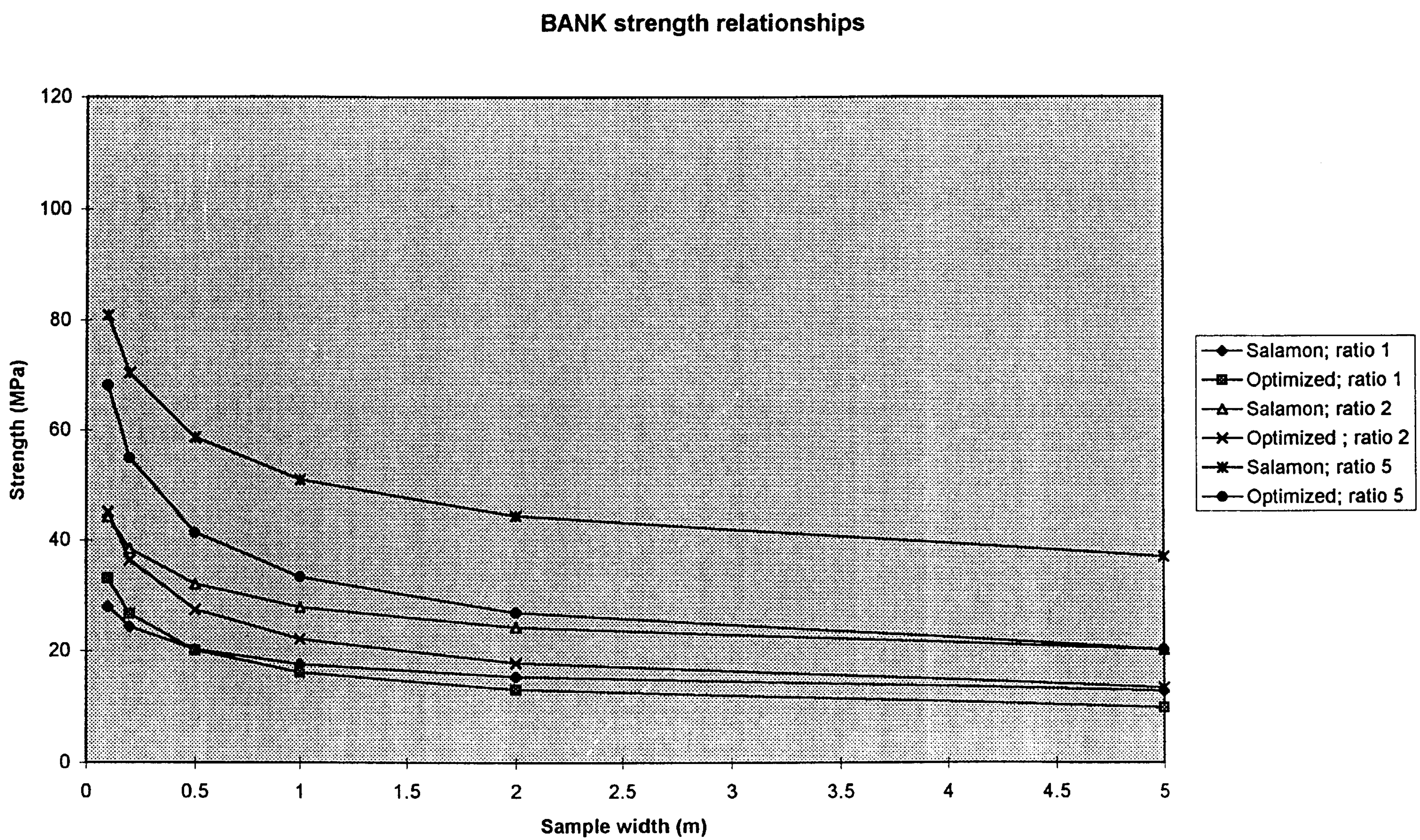
Mine	Width	Width/height	Strength (MPa) Salamon	Optimized
Arnot	0.1	1	25.1	31.6
	0.2	1	21.8	25.5
	0.5	1	18.2	19.2
	1	1	15.8	15.5
	2	1	13.8	12.5
	5	1	11.5	9.4
	0.1	2	39.6	43.1
	0.2	2	34.5	34.8
	0.5	2	28.7	26.2
	1	2	25.0	21.1
	2	2	21.7	17.0
	5	2	18.1	12.8
	0.1	5	72.5	65.0
	0.2	5	63.1	52.5
	0.5	5	52.5	39.5
	1	5	45.7	31.9
	2	5	39.8	25.7
	5	5	33.1	19.3
Bank	0.1	1	28.0	33.1
	0.2	1	24.3	26.7
	0.5	1	20.3	20.1
	1	1	17.6	16.2
	2	1	15.4	13.1
	5	1	12.8	9.8
	0.1	2	44.2	45.1
	0.2	2	38.5	36.4
	0.5	2	32.0	27.4
	1	2	27.9	22.1
	2	2	24.3	17.8
	5	2	20.2	13.4
	0.1	5	80.9	68.1
	0.2	5	70.4	54.9
	0.5	5	58.6	41.4
	1	5	51.0	33.4
	2	5	44.4	26.9
	5	5	37.0	20.3
Blinkpan	0.1	1	24.3	30.0
	0.2	1	21.2	24.2
	0.5	1	17.6	18.2
	1	1	15.3	14.7
	2	1	13.3	11.9
	5	1	11.1	8.9
	0.1	2	38.4	41.0
	0.2	2	33.4	33.0
	0.5	2	27.8	24.9
	1	2	24.2	20.1
	2	2	21.1	16.2
	5	2	17.6	12.2
	0.1	5	70.3	61.8
	0.2	5	61.2	49.9
	0.5	5	50.9	37.5
	1	5	44.4	30.3
	2	5	38.6	24.4
	5	5	32.1	18.4

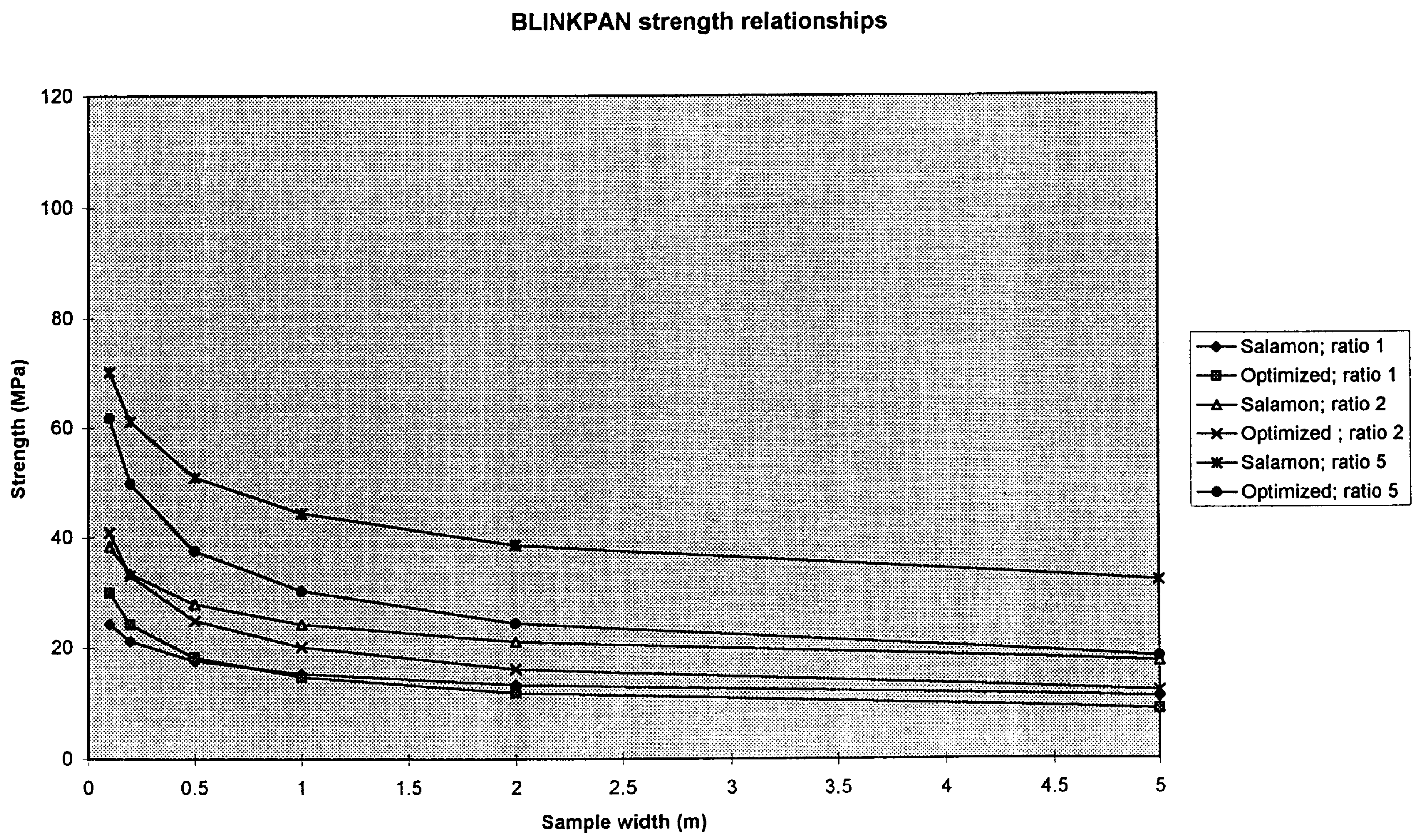
Mine	Width	Width/height	Strength (MPa) Salamon	Optimized
Delmas 2	0.1	1	27.0	33.9
	0.2	1	23.5	27.4
	0.5	1	19.6	20.6
	1	1	17.0	16.6
	2	1	14.8	13.4
	5	1	12.4	10.1
	0.1	2	42.7	46.3
	0.2	2	37.2	37.3
	0.5	2	30.9	28.1
	1	2	26.9	22.7
	2	2	23.4	18.3
	5	2	19.5	13.8
	0.1	5	78.1	69.9
	0.2	5	68.0	56.4
	0.5	5	56.6	42.4
	1	5	49.3	34.2
	2	5	42.9	27.6
	5	5	35.7	20.8
Delmas	0.1	1	38.0	38.4
	0.2	1	33.1	31.0
	0.5	1	27.6	23.3
	1	1	24.0	18.8
	2	1	20.9	15.2
	5	1	17.4	11.4
	0.1	2	60.1	52.5
	0.2	2	52.3	42.3
	0.5	2	43.6	31.9
	1	2	37.9	25.7
	2	2	33.0	20.7
	5	2	27.5	15.6
	0.1	5	110.0	79.2
	0.2	5	95.8	63.9
	0.5	5	79.8	48.1
	1	5	69.4	38.8
	2	5	60.4	31.3
	5	5	50.3	23.6
Goedehoep	0.1	1	27.5	32.9
	0.2	1	23.9	26.5
	0.5	1	19.9	20.0
	1	1	17.3	16.1
	2	1	15.1	13.0
	5	1	12.6	9.8
	0.1	2	43.4	44.9
	0.2	2	37.8	36.2
	0.5	2	31.4	27.3
	1	2	27.4	22.0
	2	2	23.8	17.7
	5	2	19.8	13.3
	0.1	5	79.4	67.7
	0.2	5	69.1	54.6
	0.5	5	57.6	41.1
	1	5	50.1	33.2
	2	5	43.6	26.8
	5	5	36.3	20.1

Mine	Width	Width/height	Strength (MPa) Salamon	Optimized
Greenside	0.1	1	24.4	31.4
	0.2	1	21.2	25.3
	0.5	1	17.7	19.0
	1	1	15.4	15.4
	2	1	13.4	12.4
	5	1	11.1	9.3
	0.1	2	38.5	42.8
	0.2	2	33.5	34.5
	0.5	2	27.9	26.0
	1	2	24.3	21.0
	2	2	21.1	16.9
	5	2	17.6	12.7
	0.1	5	70.5	64.6
	0.2	5	61.4	52.1
	0.5	5	51.1	39.2
	1	5	44.5	31.7
	2	5	38.7	25.5
	5	5	32.2	19.2
Khutala	0.1	1	28.2	35.5
	0.2	1	24.6	28.6
	0.5	1	20.5	21.5
	1	1	17.8	17.4
	2	1	15.5	14.0
	5	1	12.9	10.5
	0.1	2	44.6	48.4
	0.2	2	38.9	39.1
	0.5	2	32.3	29.4
	1	2	28.2	23.7
	2	2	24.5	19.1
	5	2	20.4	14.4
	0.1	5	81.7	73.1
	0.2	5	71.1	59.0
	0.5	5	59.2	44.4
	1	5	51.6	35.8
	2	5	44.9	28.9
	5	5	37.4	21.7
Kriel	0.1	1	24.7	29.1
	0.2	1	21.5	23.5
	0.5	1	17.9	17.7
	1	1	15.6	14.3
	2	1	13.6	11.5
	5	1	11.3	8.7
	0.1	2	39.0	39.8
	0.2	2	34.0	32.1
	0.5	2	28.3	24.1
	1	2	24.6	19.5
	2	2	21.4	15.7
	5	2	17.8	11.8
	0.1	5	71.4	60.0
	0.2	5	62.2	48.4
	0.5	5	51.8	36.4
	1	5	45.1	29.4
	2	5	39.2	23.7
	5	5	32.7	17.8

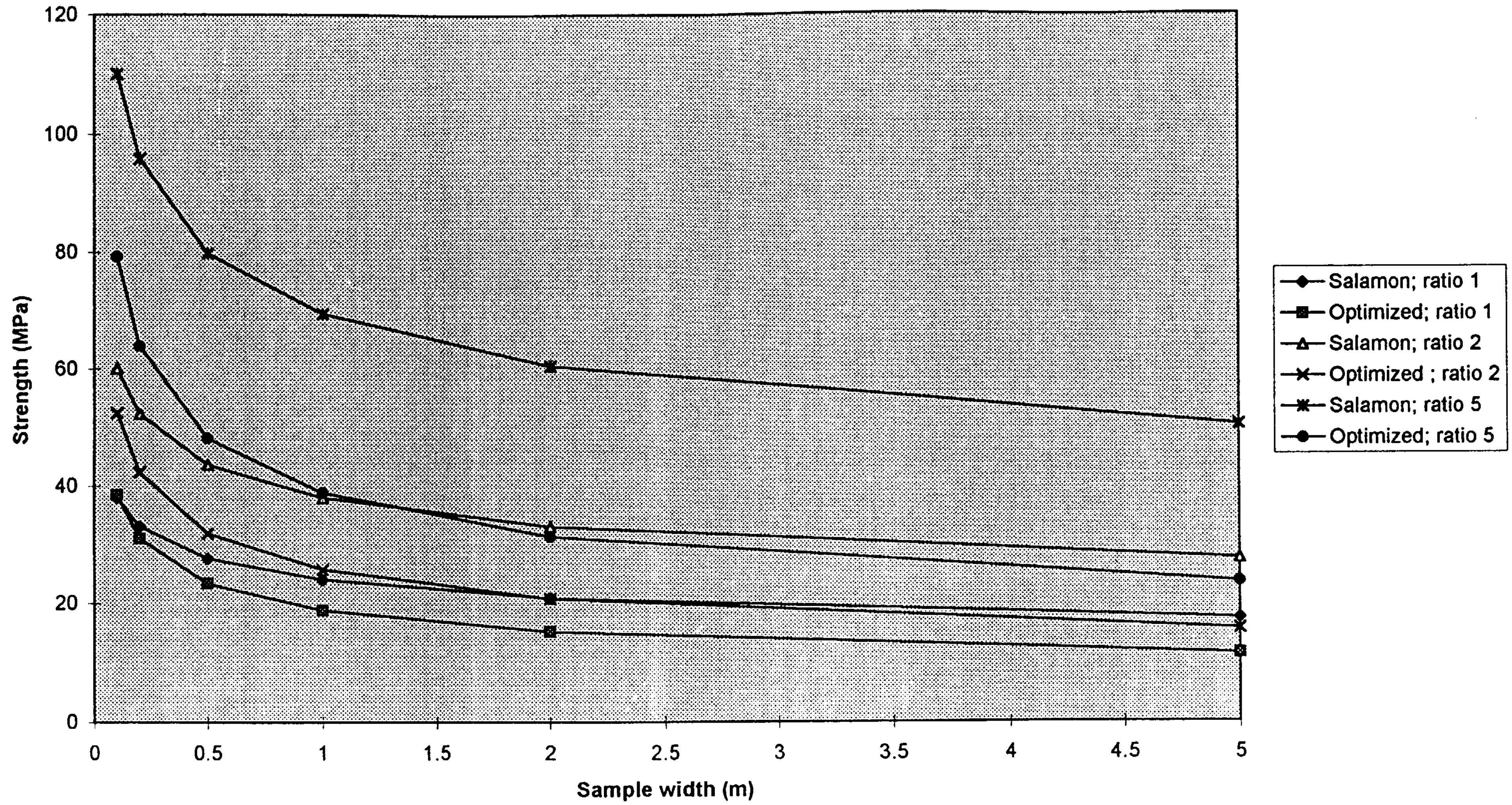
Mine	Width	Width/height	Strength (MPa) Salamon	Strength (MPa) Optimized
Secunda	0.1	1	23.6	29.2
	0.2	1	20.6	23.6
	0.5	1	17.1	17.8
	1	1	14.9	14.3
	2	1	13.0	11.6
	5	1	10.8	8.7
	0.1	2	37.3	39.9
	0.2	2	32.5	32.2
	0.5	2	27.0	24.2
	1	2	23.5	19.6
	2	2	20.5	15.8
	5	2	17.1	11.9
	0.1	5	68.3	60.3
	0.2	5	59.5	48.6
	0.5	5	49.5	36.6
	1	5	43.1	29.5
	2	5	37.5	23.8
	5	5	31.2	17.9
Sigma	0.1	1	28.2	28.4
	0.2	1	24.5	22.9
	0.5	1	20.4	17.2
	1	1	17.8	13.9
	2	1	15.5	11.2
	5	1	12.9	8.4
	0.1	2	44.5	38.7
	0.2	2	38.8	31.3
	0.5	2	32.3	23.5
	1	2	28.1	19.0
	2	2	24.5	15.3
	5	2	20.4	11.5
	0.1	5	81.5	58.5
	0.2	5	71.0	47.2
	0.5	5	59.1	35.5
	1	5	51.4	28.6
	2	5	44.8	23.1
	5	5	37.3	17.4
Zululand Anthracite	0.1	1	31.5	35.1
	0.2	1	27.4	28.3
	0.5	1	22.8	21.3
	1	1	19.9	17.2
	2	1	17.3	13.9
	5	1	14.4	10.4
	0.1	2	49.7	47.9
	0.2	2	43.3	38.7
	0.5	2	36.0	29.1
	1	2	31.4	23.5
	2	2	27.3	18.9
	5	2	22.7	14.3
	0.1	5	91.1	72.3
	0.2	5	79.3	58.3
	0.5	5	66.0	43.9
	1	5	57.4	35.4
	2	5	50.0	28.6
	5	5	41.6	21.5


ARNOT strength relationships
Strength (MPa)
120
100
80
60
40
20
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Sample width (m)
Salamon; ratio 1
Optimized; ratio 1
Salamon; ratio 2
Optimized ; ratio 2
Salamon; ratio 5
Optimized; ratio 5

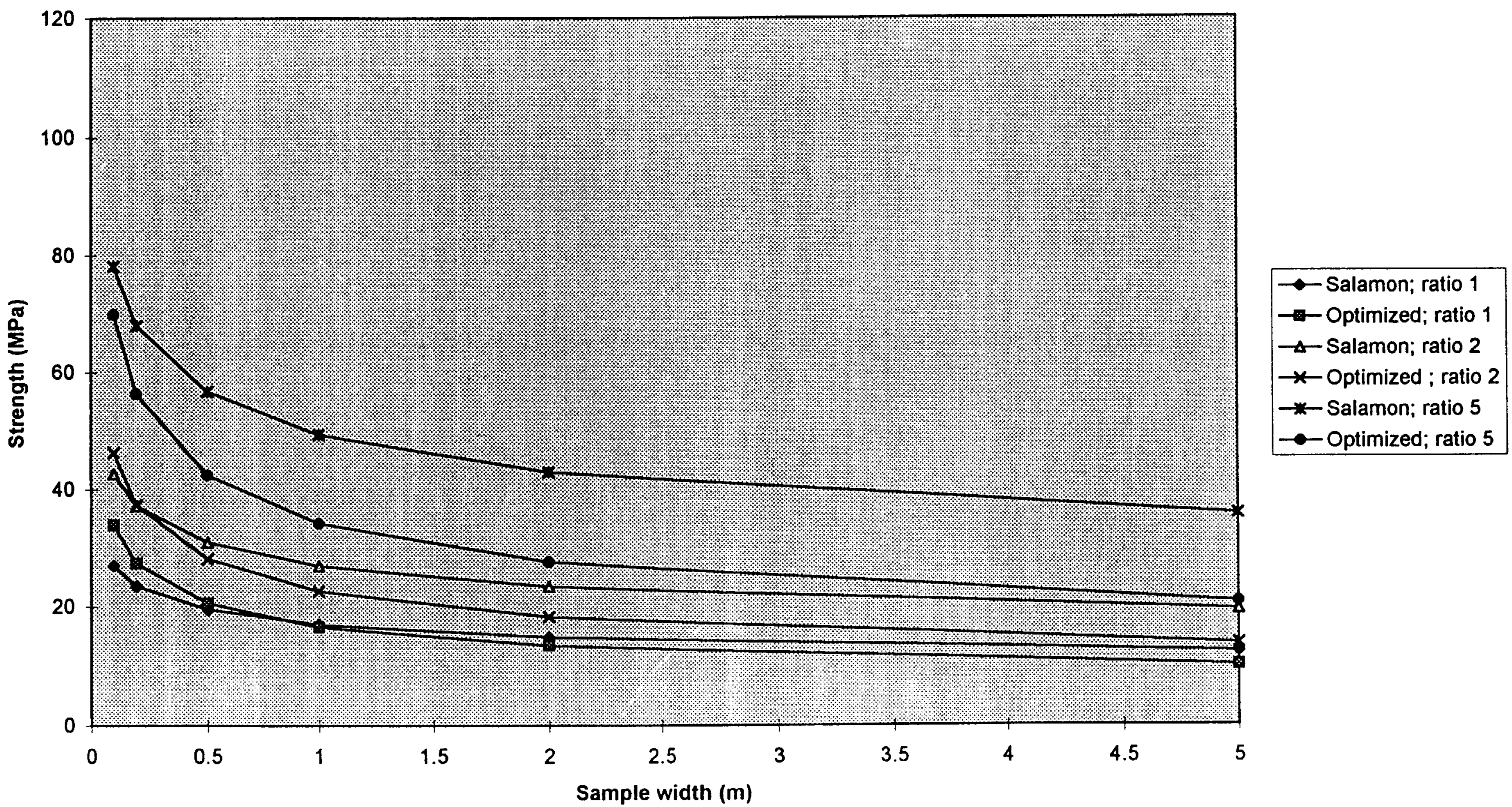

BANK strength relationships
Strength (MPa)
120
100
80
60
40
20
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Sample width (m)
Salamon; ratio 1
Optimized; ratio 1
Salamon; ratio 2
Optimized ; ratio 2
Salamon; ratio 5
Optimized; ratio 5

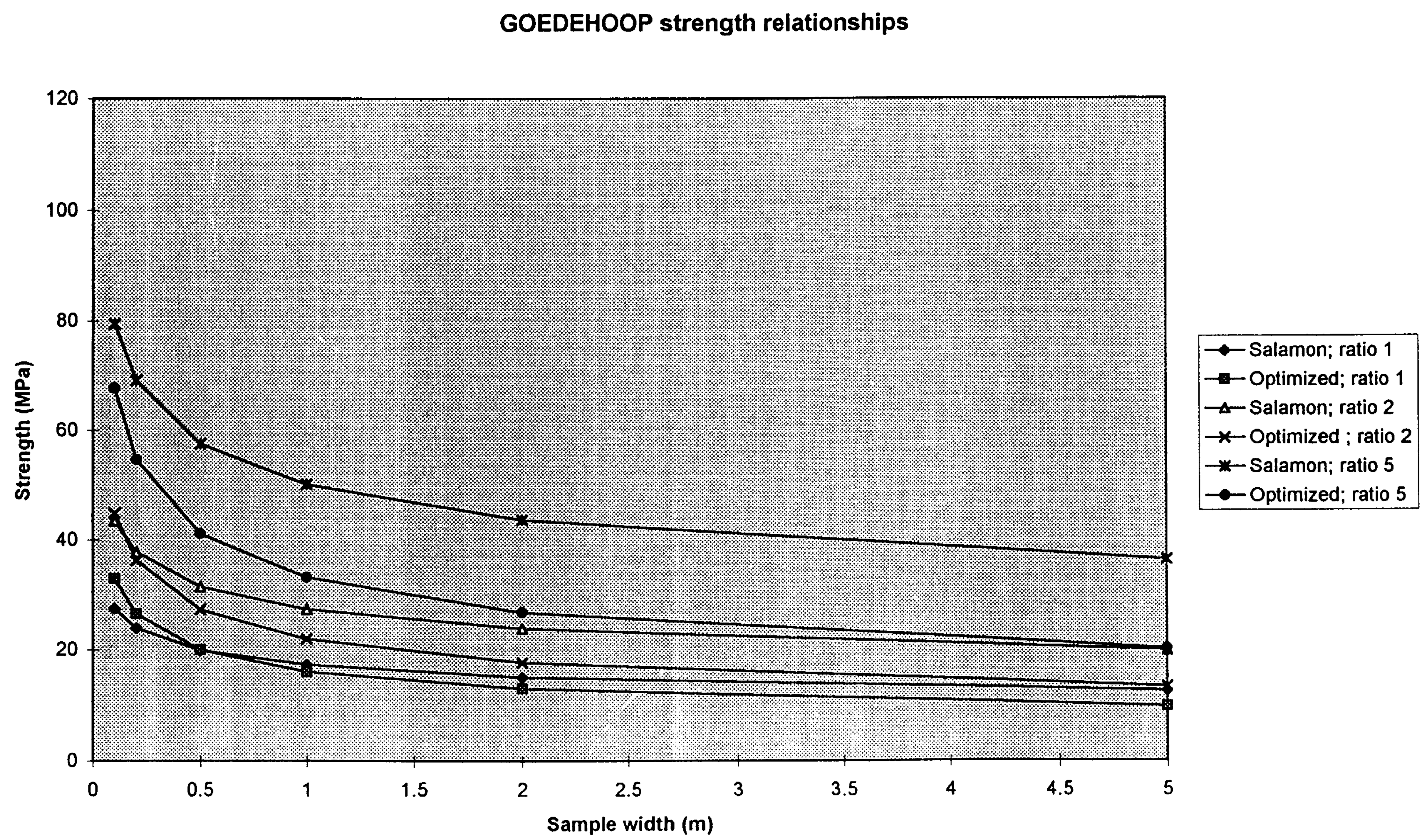

BLINKPAN strength relationships
Strength (MPa)
120
100
80
60
40
20
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Sample width (m)
Salamon; ratio 1
Optimized; ratio 1
Salamon; ratio 2
Optimized ; ratio 2
Salamon; ratio 5
Optimized; ratio 5

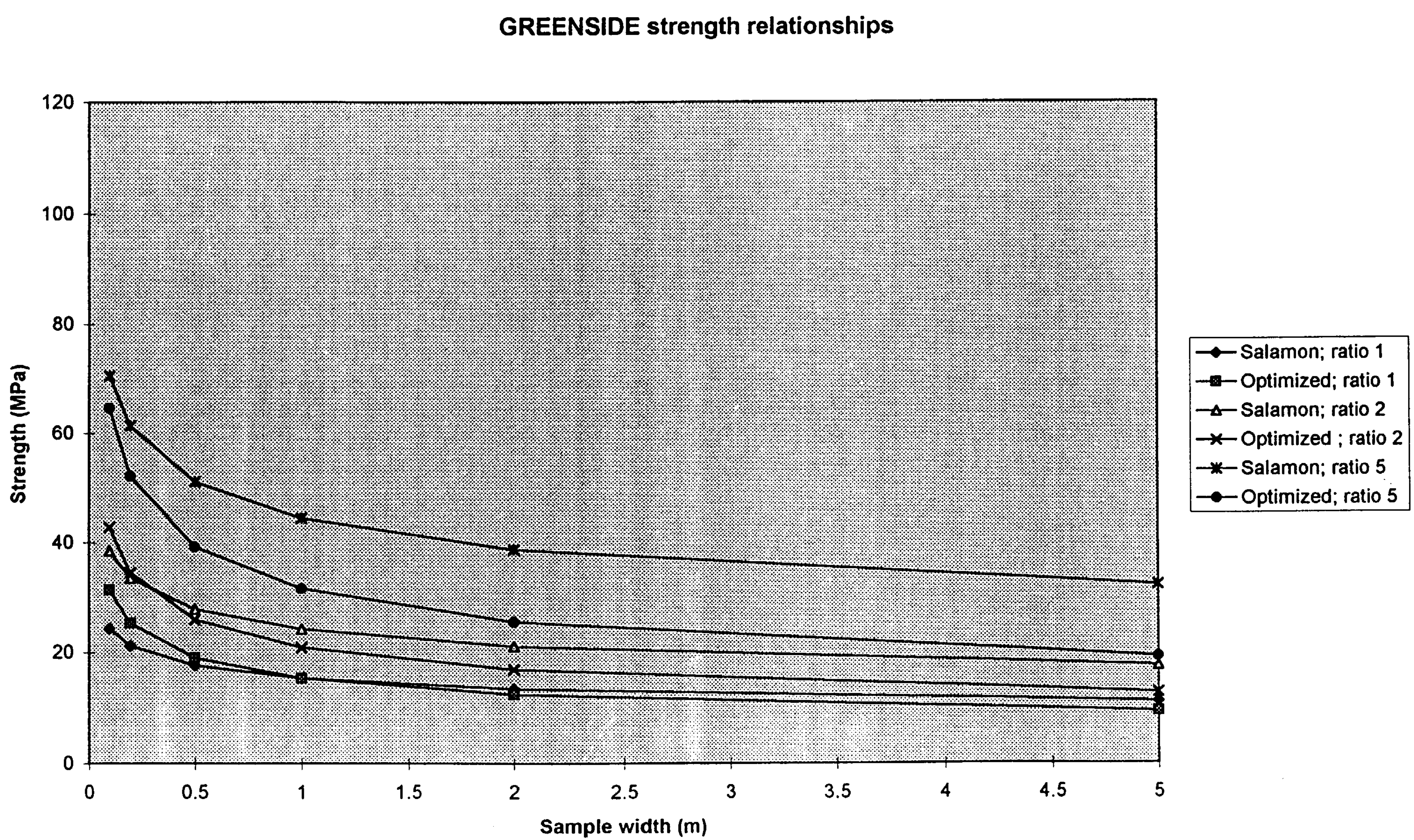
DELMAS strength relationships

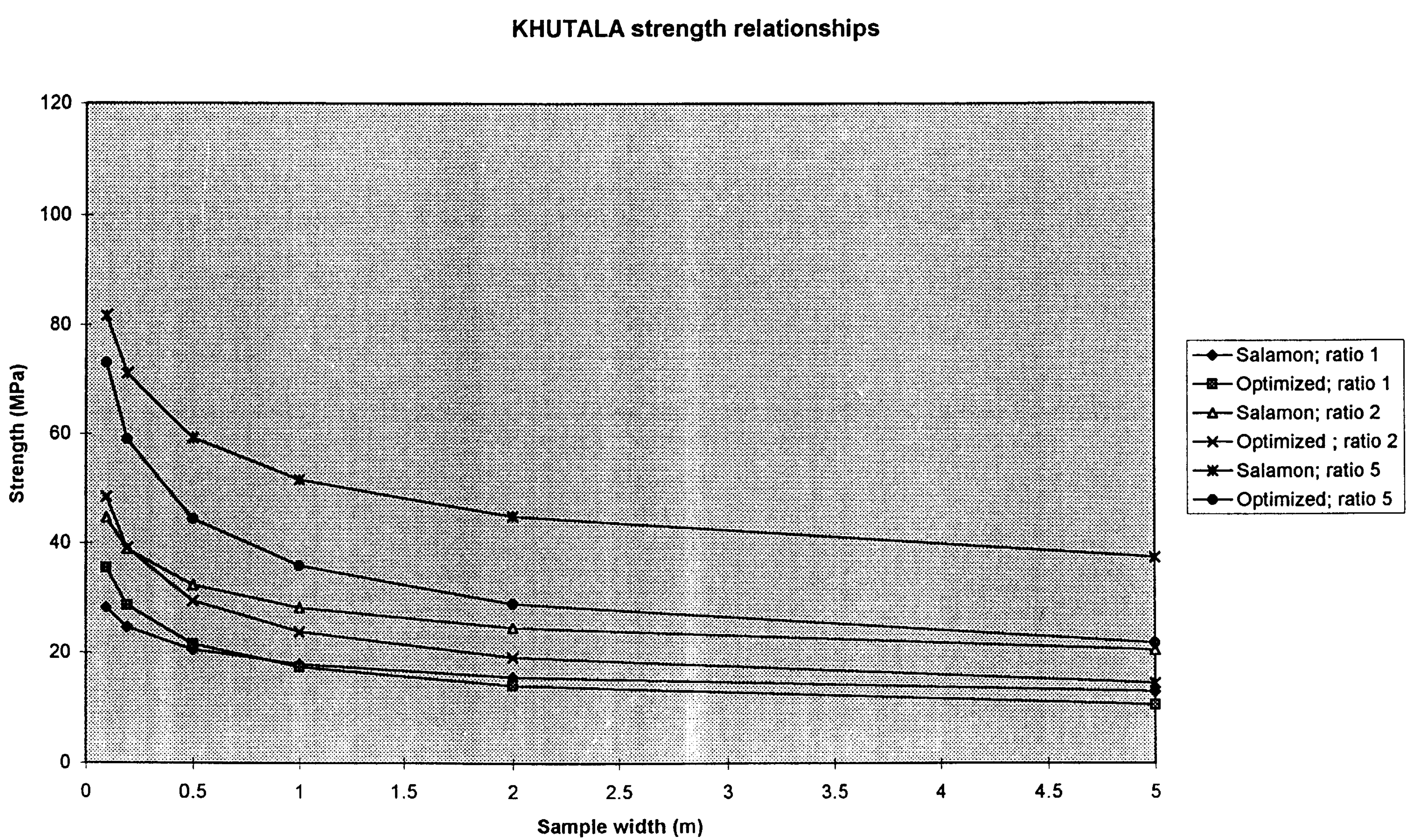


DELMAS BLOCK 2 strength relationships

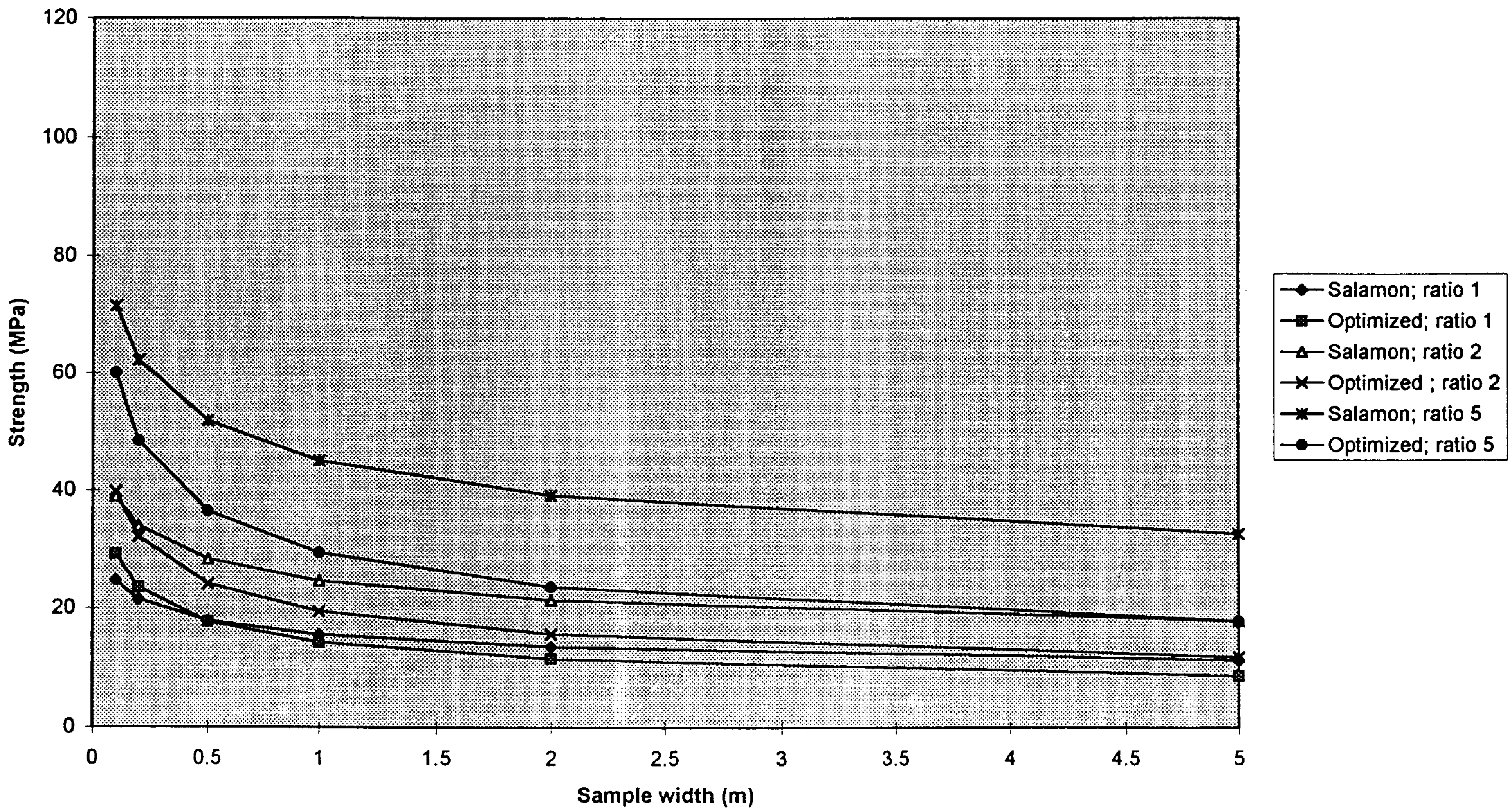


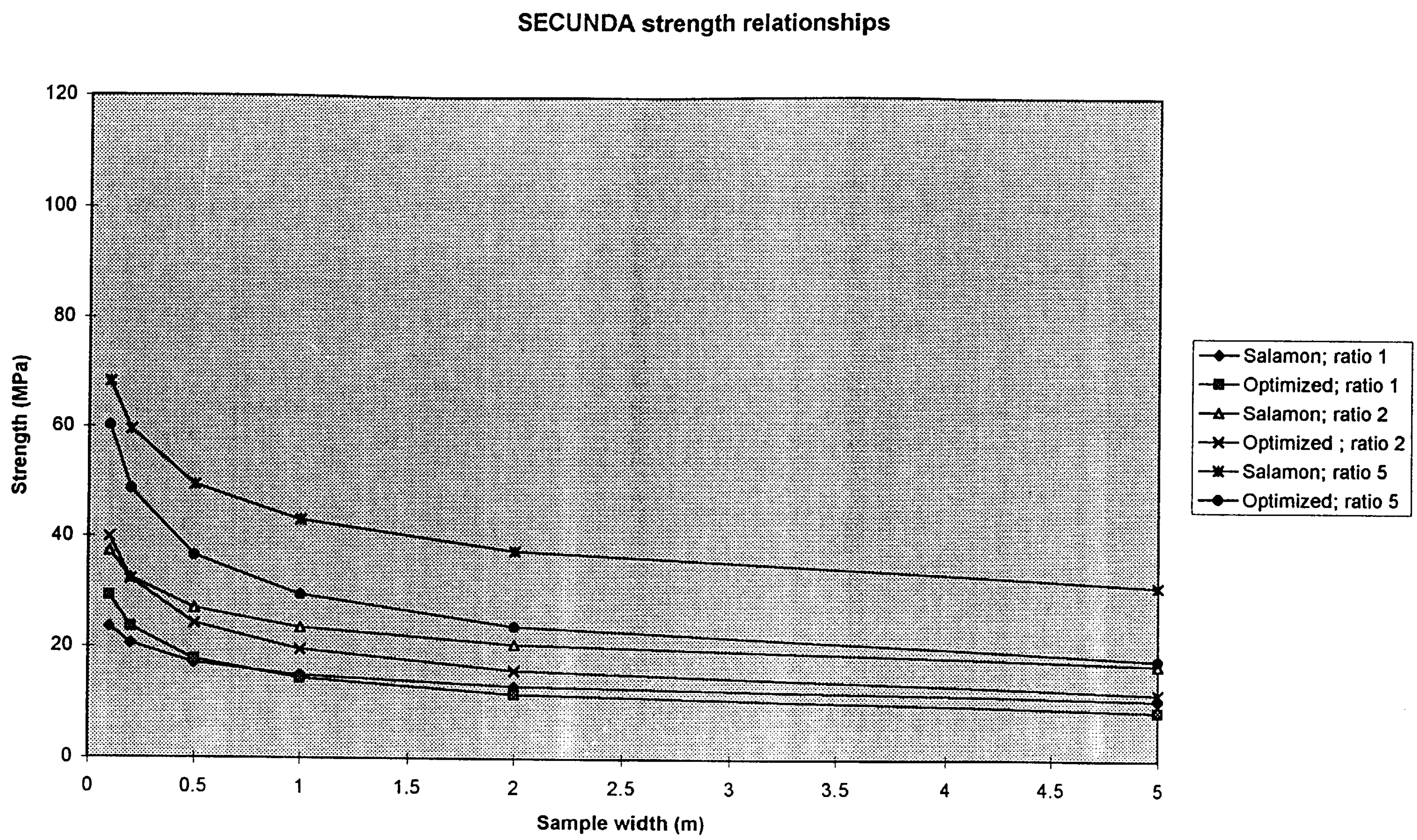

GOEDEHOOP strength relationships
Strength (MPa)
120
100
80
60
40
20
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Sample width (m)
Salamon; ratio 1
Optimized; ratio 1
Salamon; ratio 2
Optimized ; ratio 2
Salamon; ratio 5
Optimized; ratio 5


GREENSIDE strength relationships
Strength (MPa)
0
20
40
60
80
100
120
Sample width (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Salamon; ratio 1
Optimized; ratio 1
Salamon; ratio 2
Optimized ; ratio 2
Salamon; ratio 5
Optimized; ratio 5

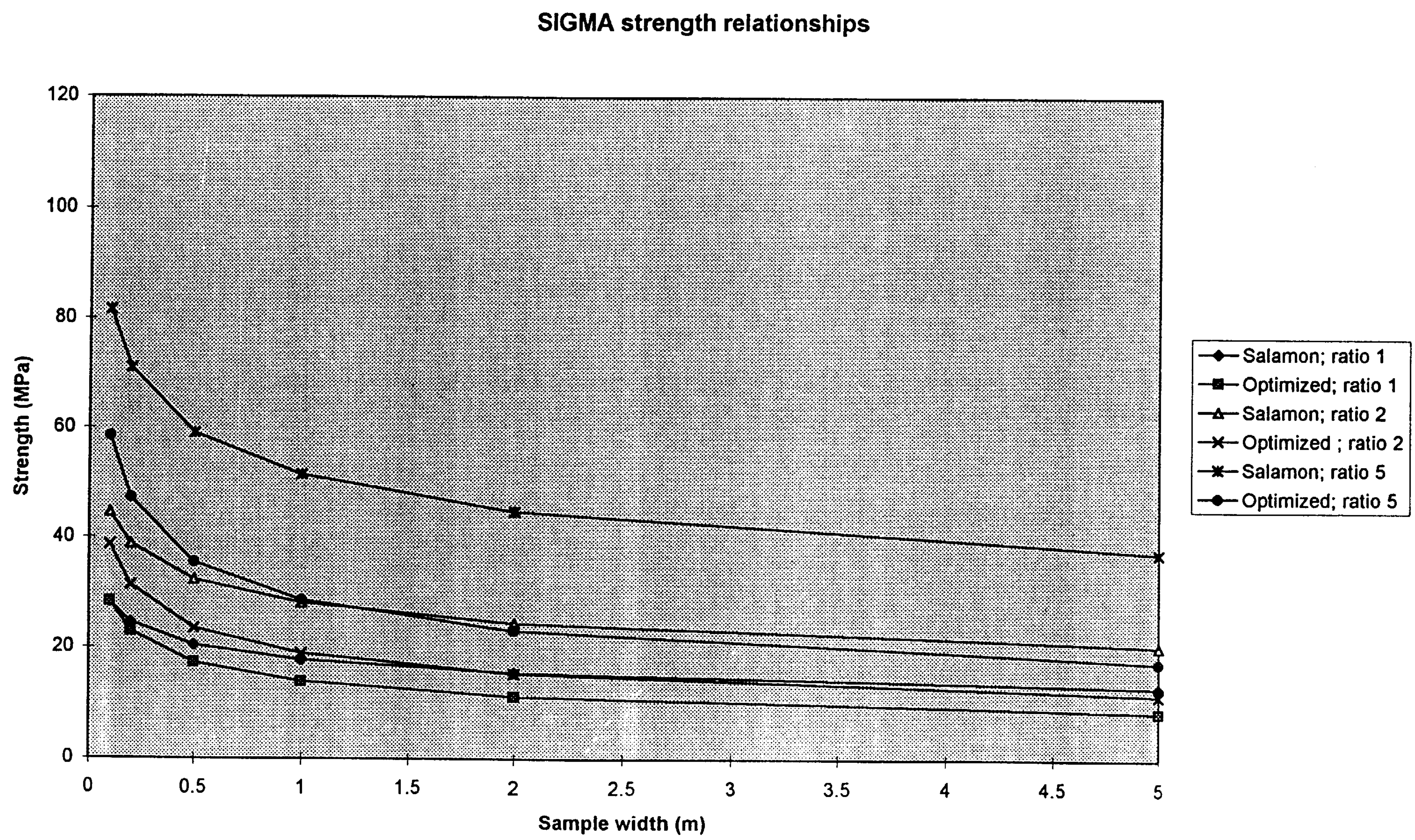

KHUTALA strength relationships
Strength (MPa)
120
100
80
60
40
20
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Sample width (m)
Salamon; ratio 1
Optimized; ratio 1
Salamon; ratio 2
Optimized ; ratio 2
Salamon; ratio 5
Optimized; ratio 5

KRIEL strength relationships





SECUNDA strength relationships
Strength (MPa)
120
100
80
60
40
20
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Sample width (m)
Salamon; ratio 1
Optimized; ratio 1
Salamon; ratio 2
Optimized ; ratio 2
Salamon; ratio 5
Optimized; ratio 5


SIGMA strength relationships
Strength (MPa)
120
100
80
60
40
20
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Sample width (m)
Salamon; ratio 1
Optimized; ratio 1
Salamon; ratio 2
Optimized ; ratio 2
Salamon; ratio 5
Optimized; ratio 5

ZULULAND ANTHRACITE strength relationships

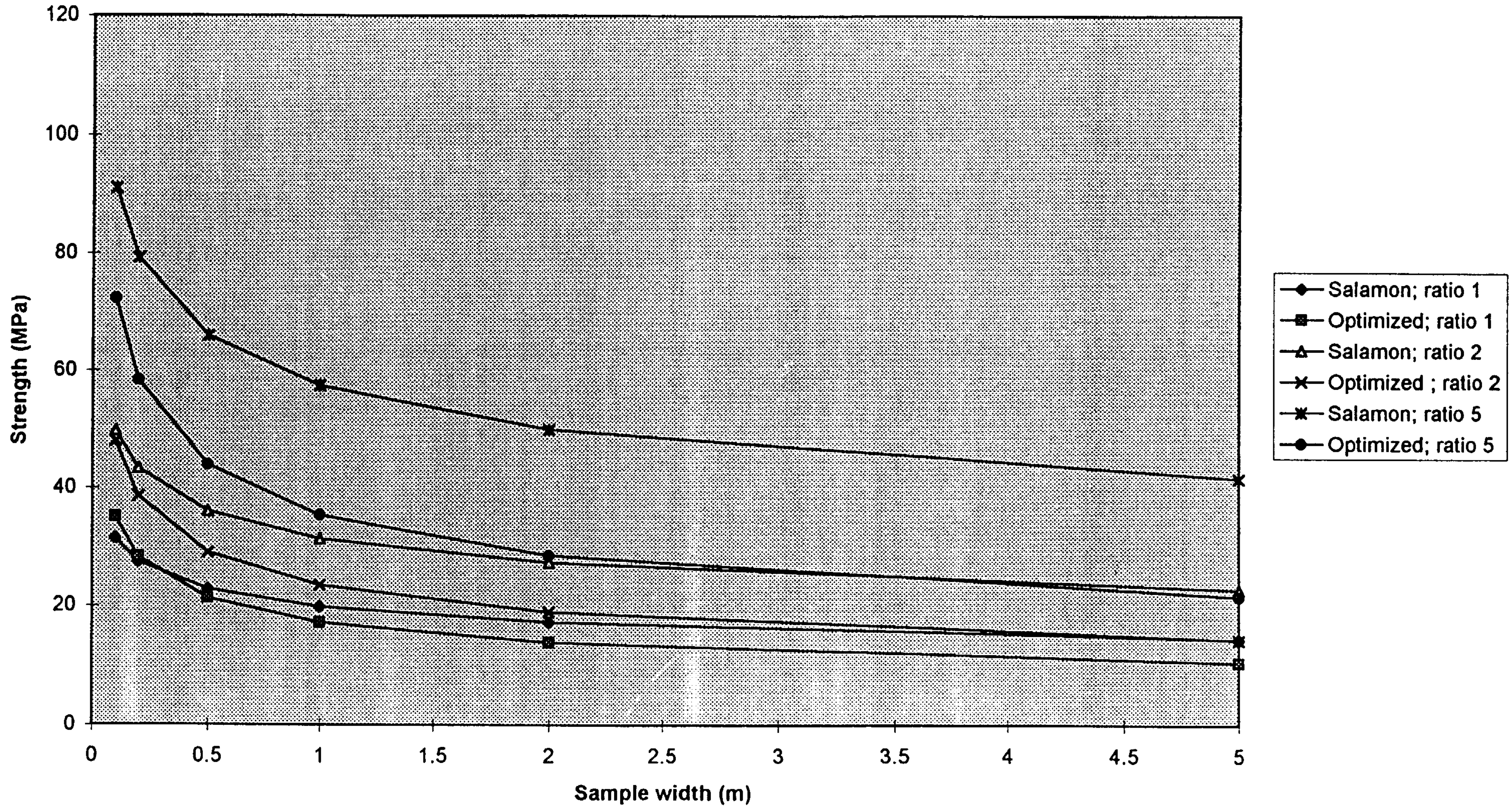


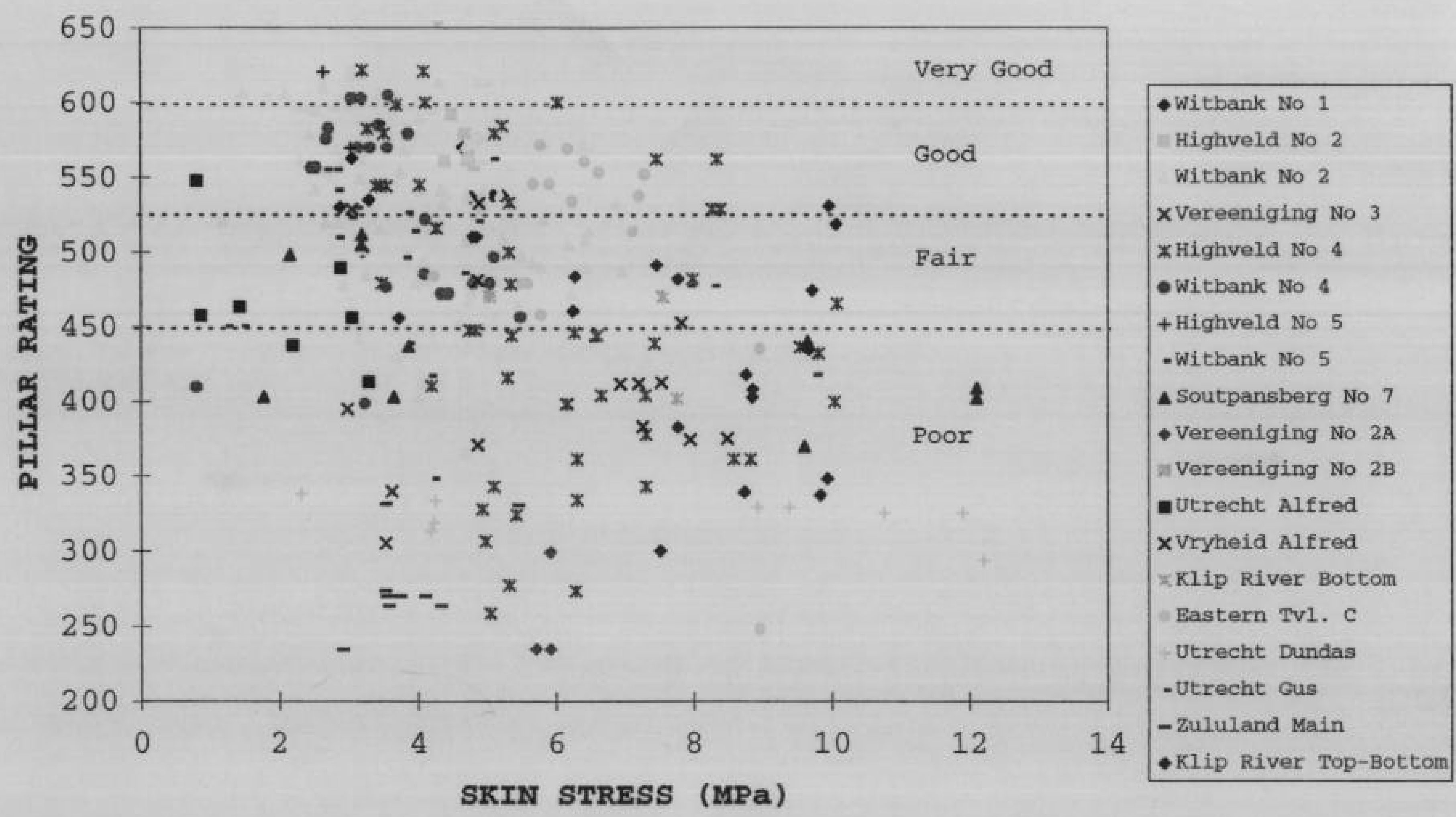
Table 4 Comparison of optimized model with original Salamon formula

Width (m)	Width/height	Strength (MPa) Optimized	Strength (MPa) Salamon original
0.1	1	32.4	11.4
0.1	2	44.2	18.0
0.1	5	66.7	32.9
0.2	1	26.1	9.9
0.2	2	35.7	15.6
0.2	5	53.8	28.6
0.5	1	19.7	8.2
0.5	2	26.8	13.0
0.5	5	40.5	23.8
1.0	1	15.9	7.2
1.0	2	21.7	11.3
1.0	5	32.7	20.8
2.0	1	12.8	6.2
2.0	2	17.5	9.9
2.0	5	26.4	18.1
5.0	1	9.6	5.2
5.0	2	13.1	8.2
5.0	5	19.8	15.0
10.0	5	16.0	13.1
20.0	5	12.9	11.4

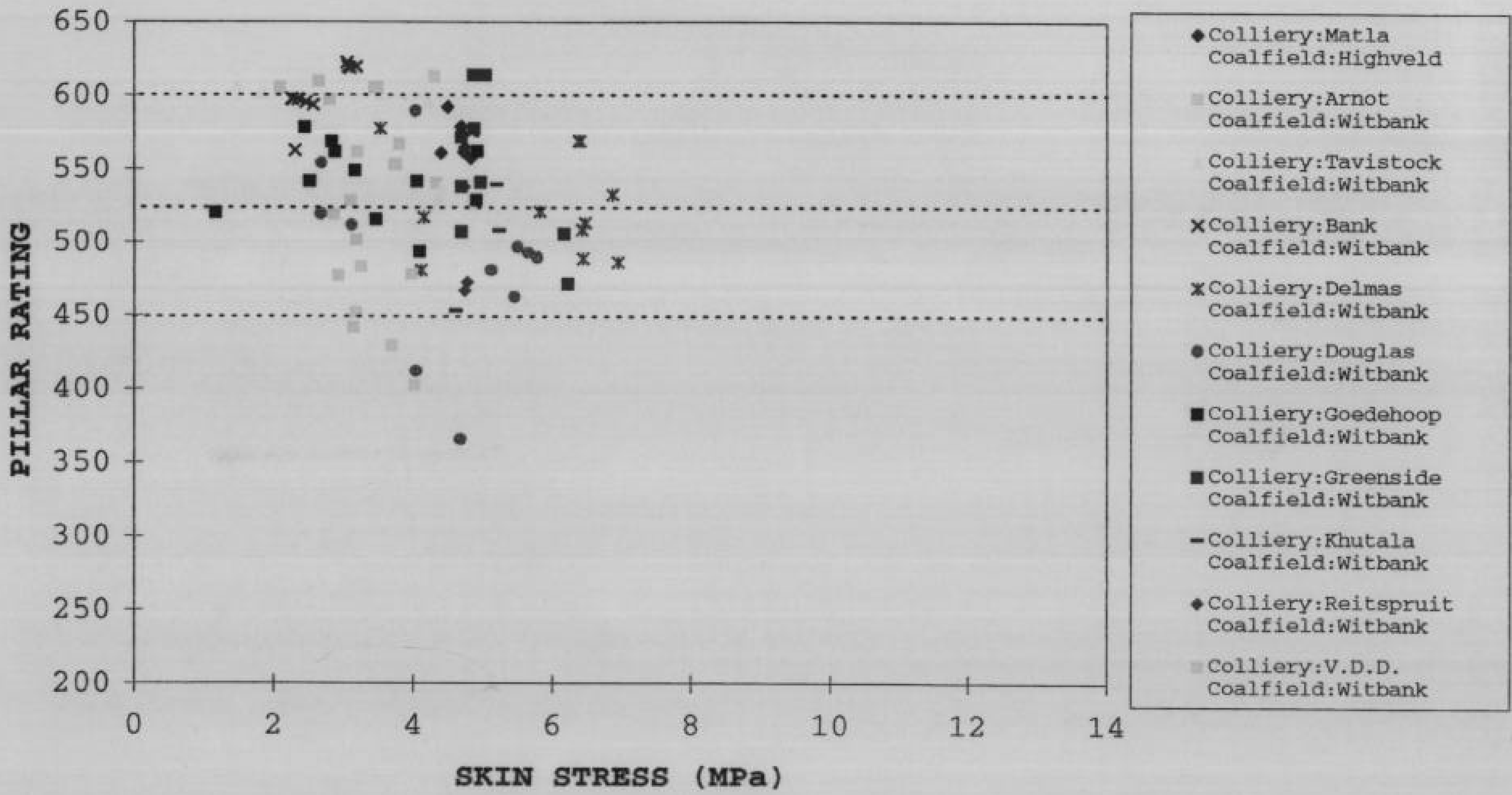
APPENDIX B

Results of Pillar Classification Rating

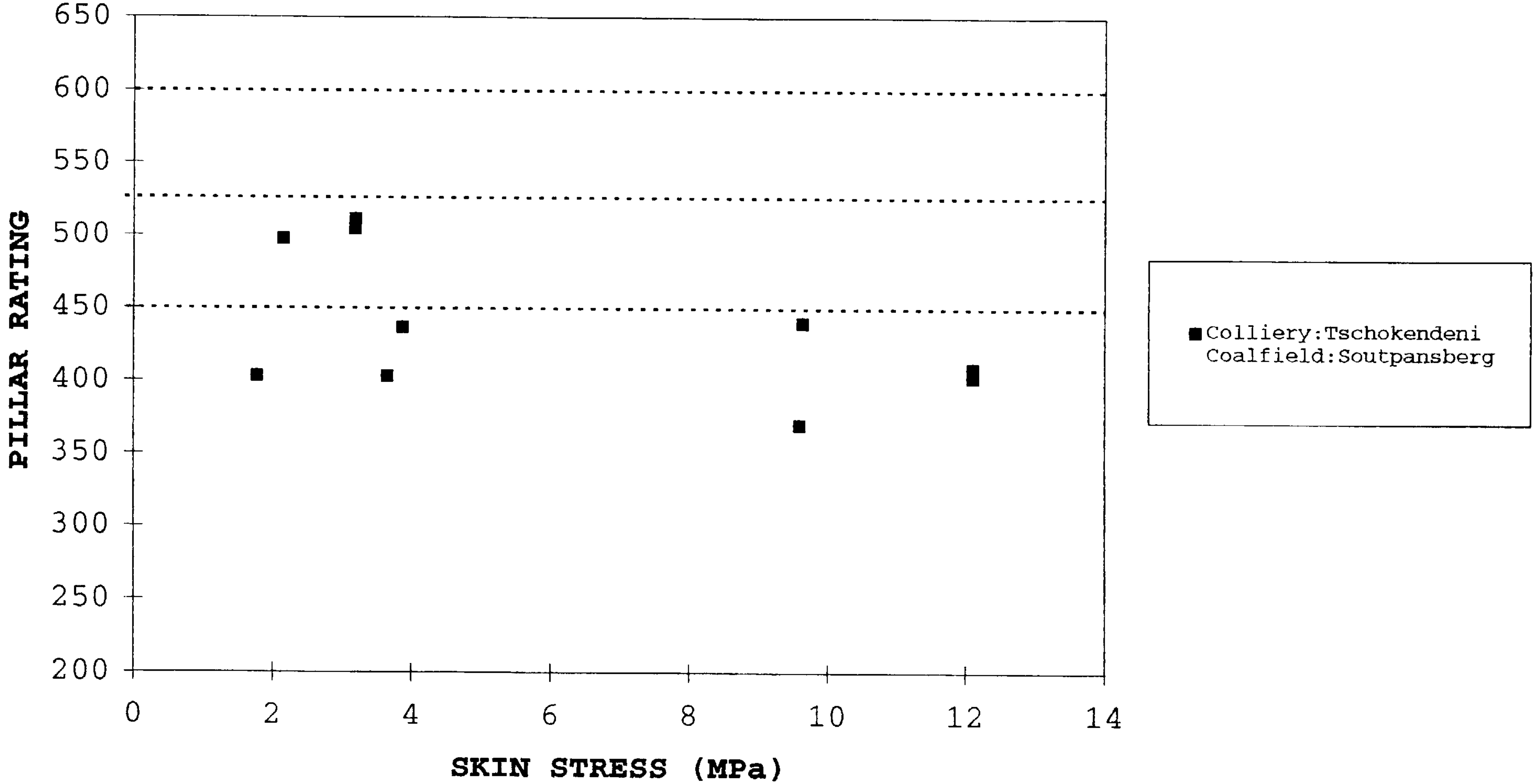
PILLAR RATING VERSUS SKIN STRESS



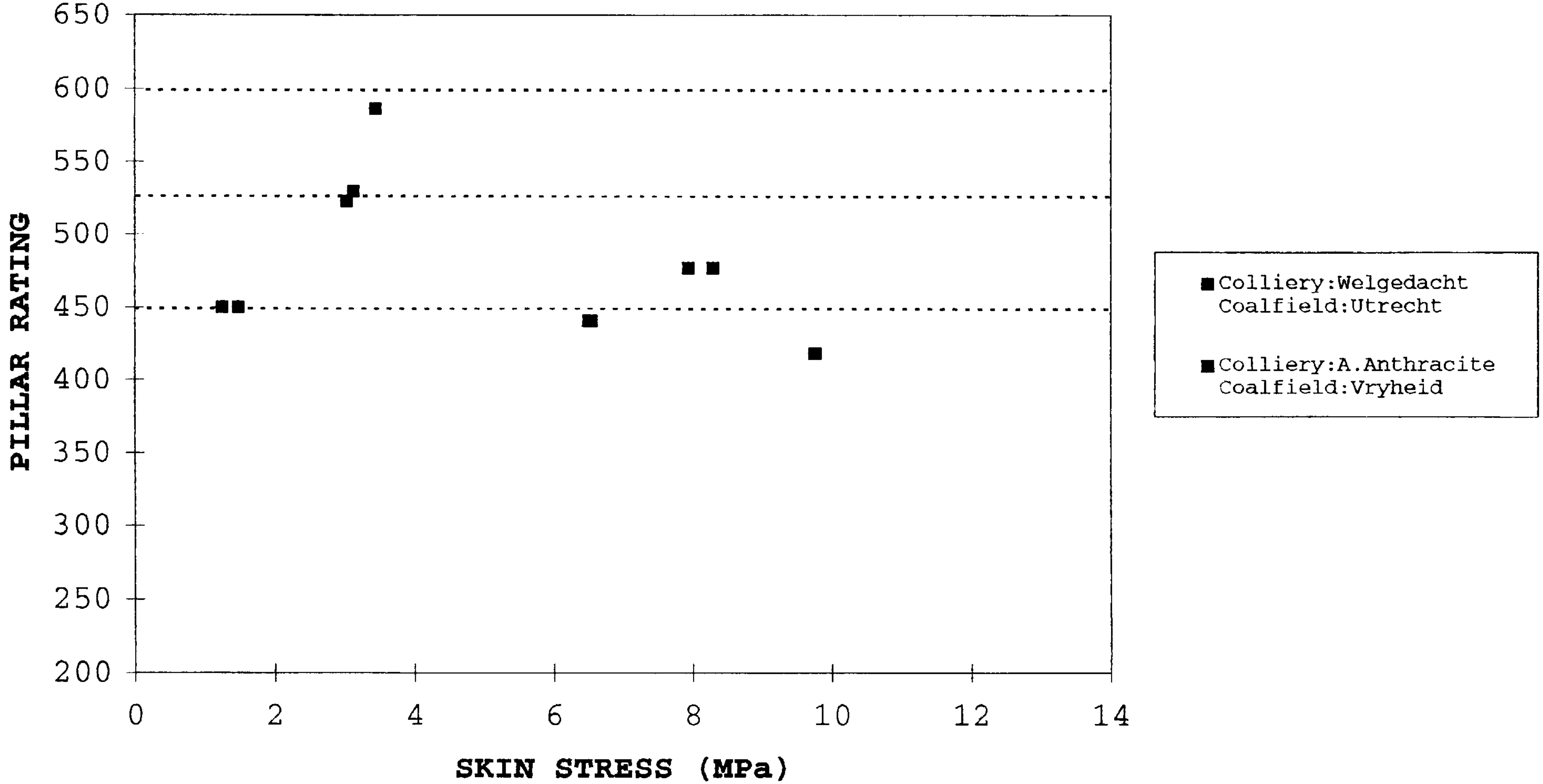
PILLAR RATING VERSUS SKIN STRESS
(No 2 SEAM)



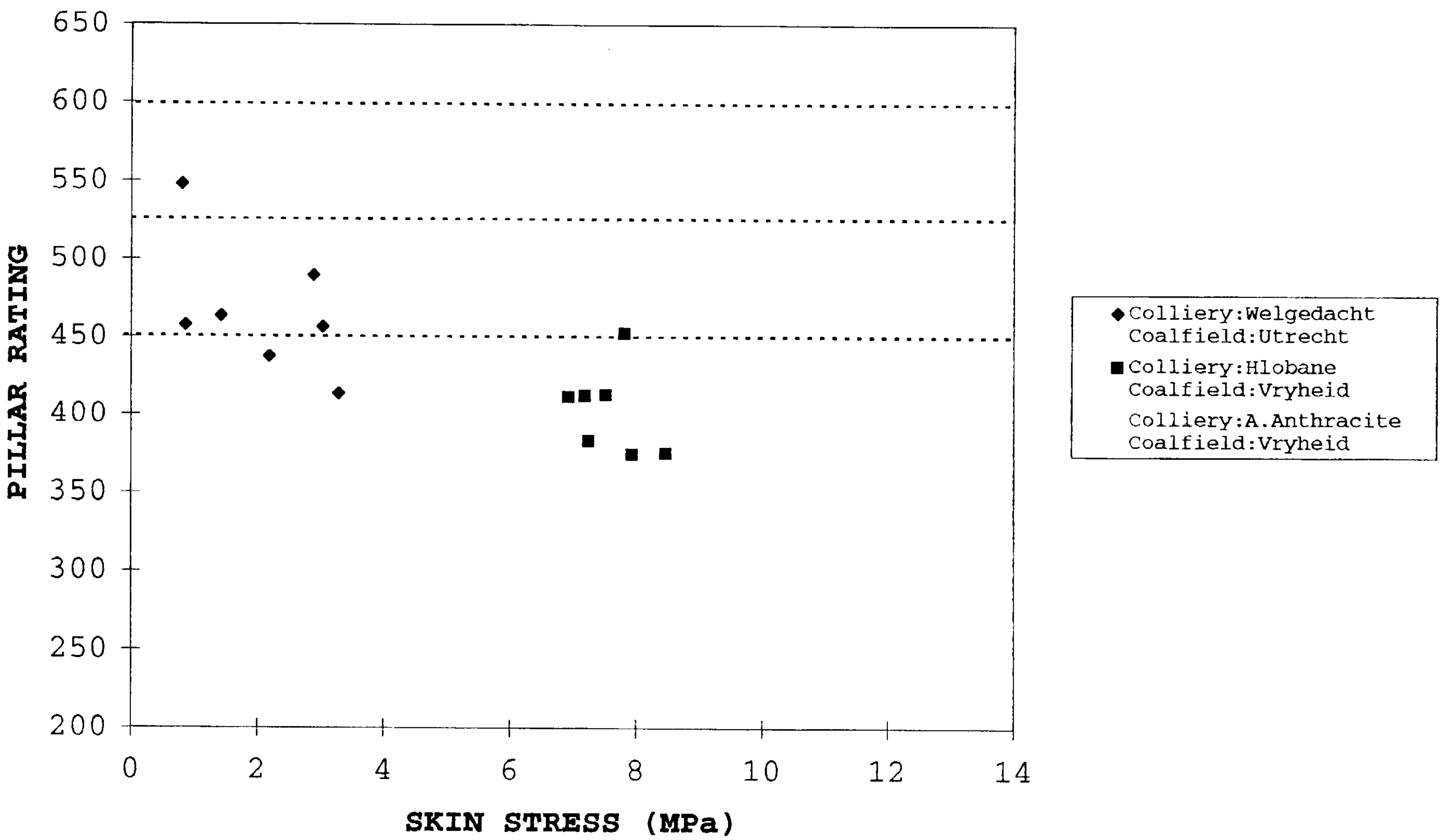
PILLAR RATING VERSUS SKIN STRESS
(No 7 SEAM)



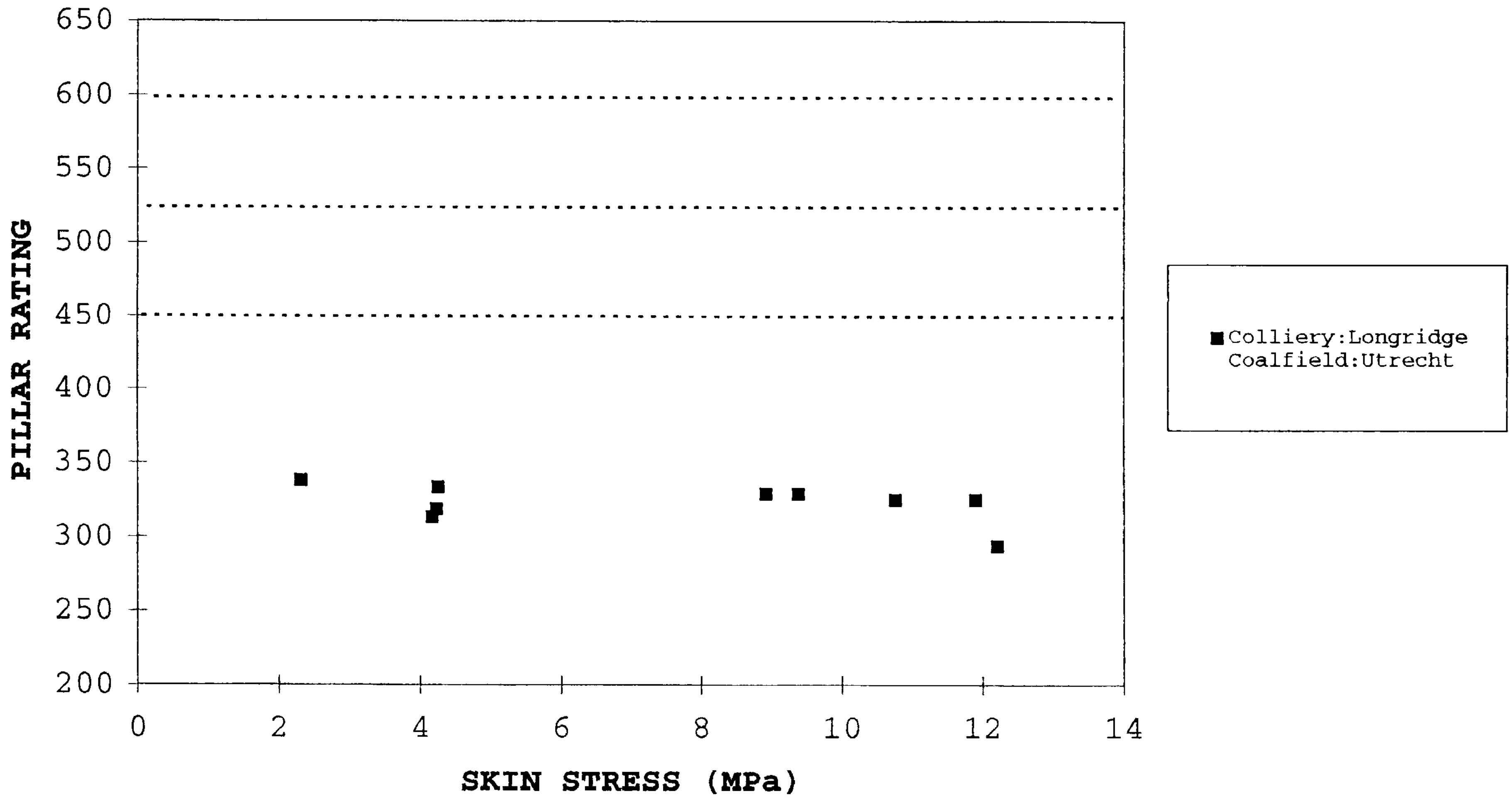
PILLAR RATING VERSUS SKIN STRESS
(GUS SEAM)



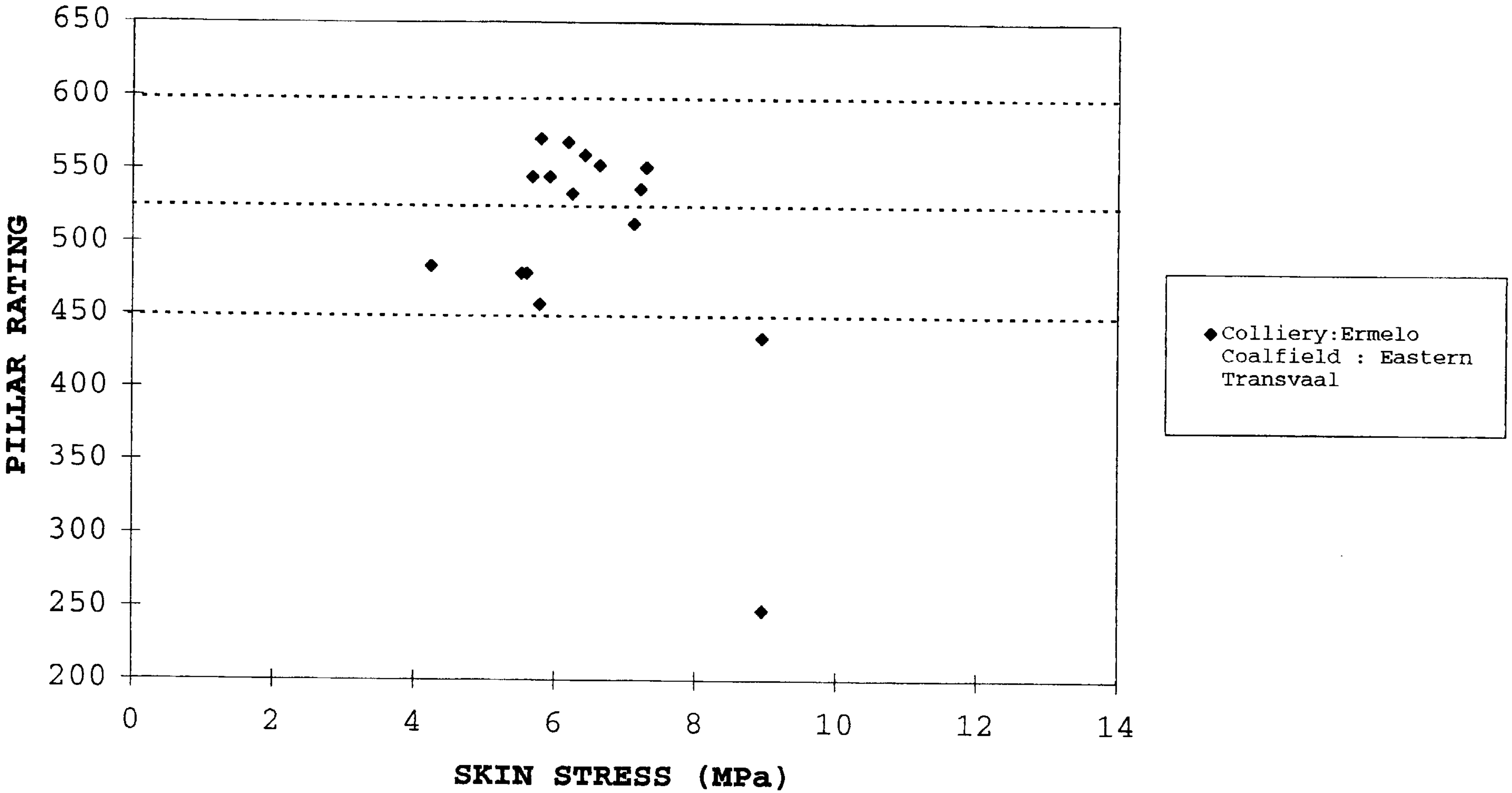
PILLAR RATING VERSUS SKIN STRESS
(ALFRED SEAM)

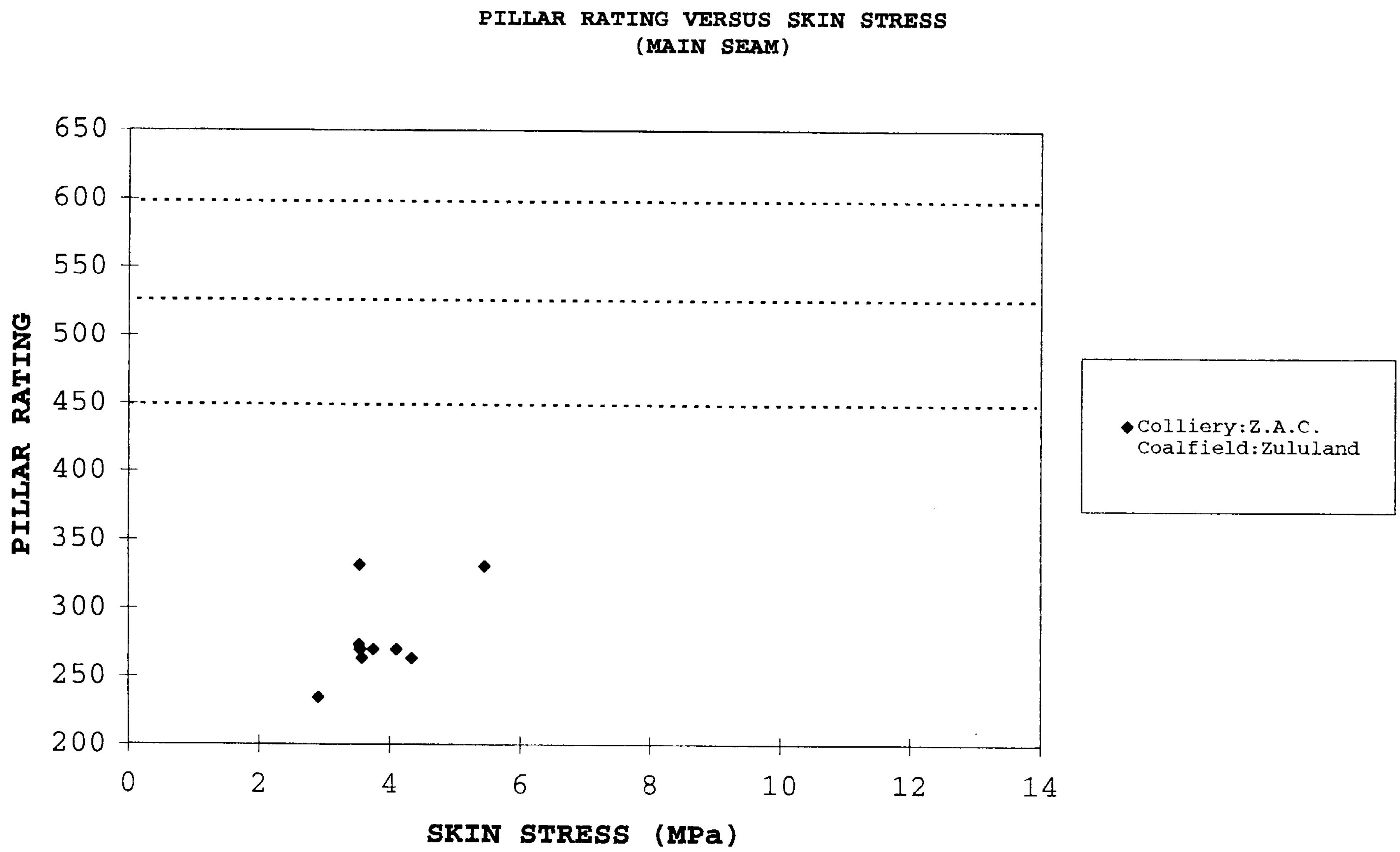


PILLAR RATING VERSUS SKIN STRESS
(DUNDAS SEAM)

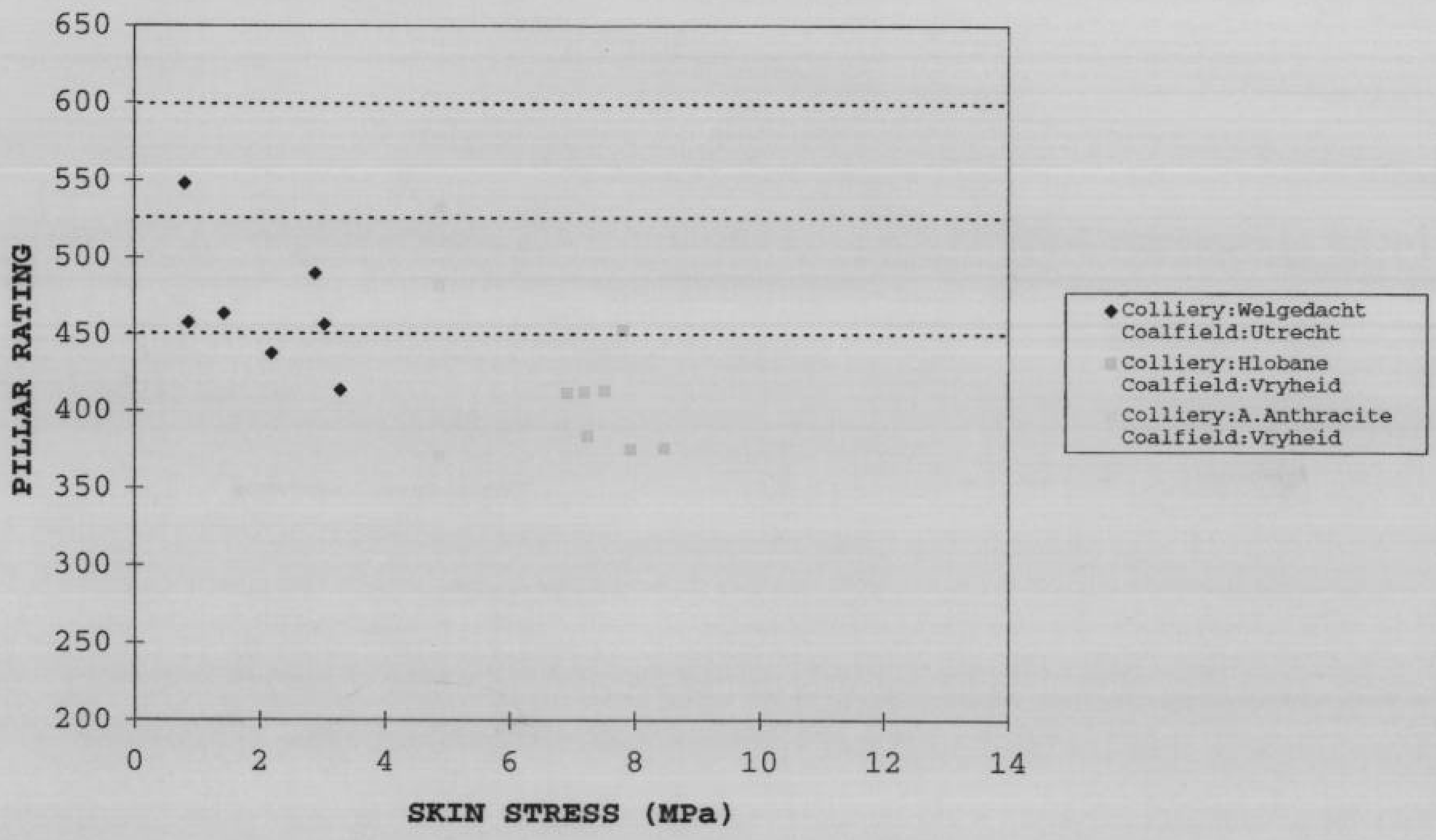


PILLAR RATING VERSUS SKIN STRESS
(C SEAM)

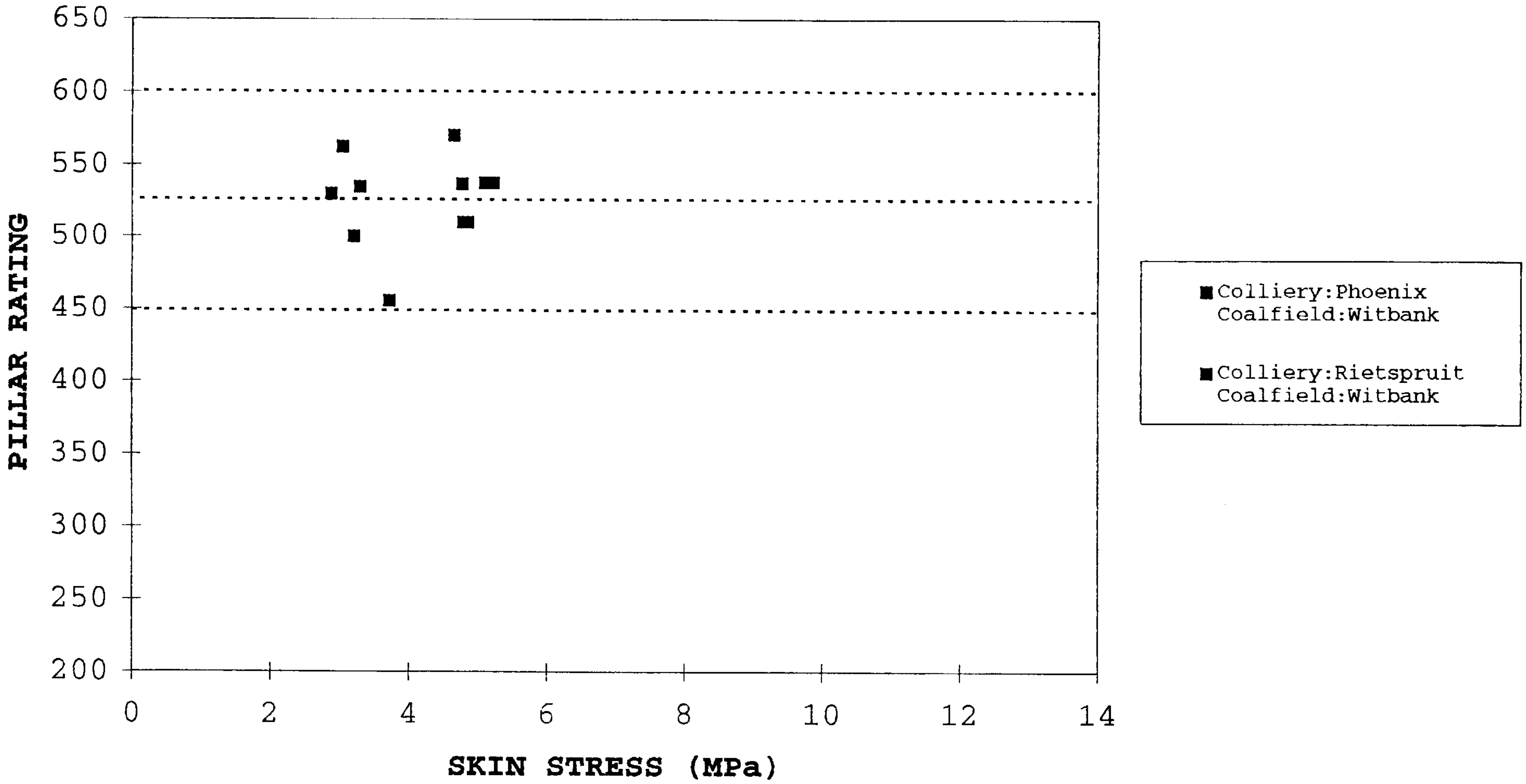



PILLAR RATING VERSUS SKIN STRESS
(MAIN SEAM)
PILLAR RATING
650
600
550
500
450
400
350
300
250
200
0
2
4
6
8
10
12
14
SKIN STRESS (MPa)
Colliery:Z.A.C.
Coalfield:Zululand

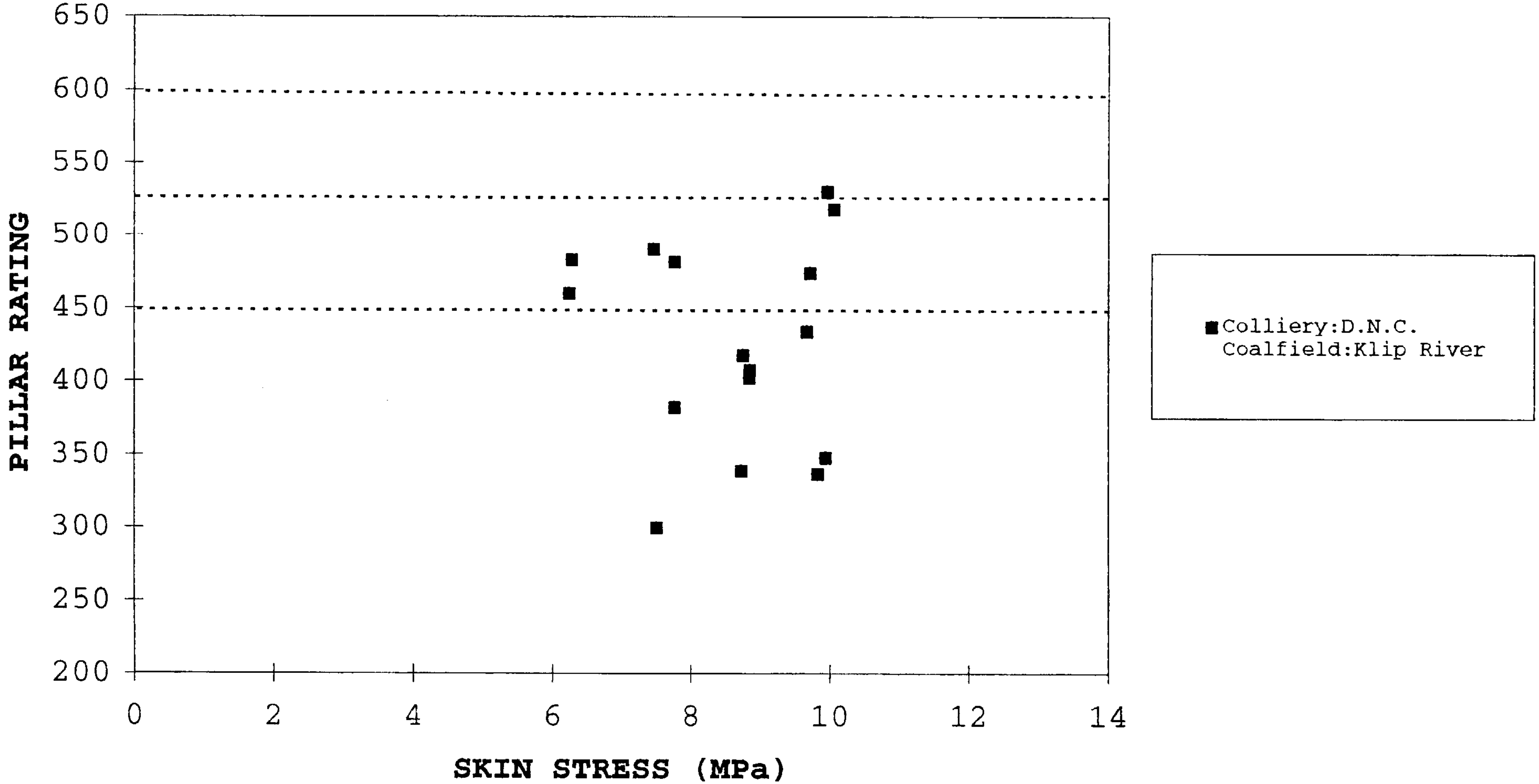
PILLAR RATING VERSUS SKIN STRESS (ALFRED SEAM)



PILLAR RATING VERSUS SKIN STRESS
(No 1 SEAM)



PILLAR RATING VERSUS SKIN STRESS
(TOP-BOTTOM SEAM)



PILLAR RATING VERSUS SKIN STRESS
(No 4 SEAM)