

# **Determination of the factors affecting the performance of grout packs**

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## DECLARATION

I declare that this research report is my own, unaided work. It is being submitted for the Degree of Master of Science to the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other University.

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## Abstract

In tabular mining, common in South African gold and platinum mines, the removal of the tabular ore body by mining operations leaves behind excavations known as stopes. These stopes form the production areas of a mine and have to be supported in order that a safe working environment is created. Stopes generally have widths of close to a metre but, in some areas and on certain reefs, may be much wider. Prior to the 1980s, a combination of in-stope pillars and timber was used to support these stopes, but innovations from the 1970s have produced grout packs as a viable support option. These packs are cast *in situ* through the use of cemented classified tailings gravitated from surface and placed in reinforced geotextile bags at the stope face. As these packs cure and become rigid they are able to bear load when compressed by stope closure. In this way, the packs keep the working areas open.

To quantify the load-bearing capacity of grout packs, a range of sizes and designs was tested in a laboratory press and, thereafter, a select few were tested underground. Initially, two aspects of grout packs that had not been adequately quantified previously were addressed. These were: the *in situ* load / compression characteristics of different forms of grout packs; and the relationship between laboratory test results and *in situ* performance. The laboratory test programme was extended to allow for an investigation into methods of improving the yieldability of grout packs and the possibility of using them to replace in-stope pillars.

It was found that the factors that most affect the initial strength and post-failure characteristics of a grout pack are: the grout strength; the amount and type of steel reinforcement; the inclusion of ancillary columnar support; and the height and diameter of the pack. It was also found that grout packs could be used to replace in-stope pillars, but that pack strength and spacing should be conservatively calculated before implementation.

A provisional relationship between the behaviour of packs tested in a press and those placed underground was determined.

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# 1 Introduction

Providing in-stope support is an essential part of South African tabular mining operations. Prior to the 1980's, different methods of supporting the hangingwall were used. These included pillars, waste packing (a labour-intensive operation not favoured because of potential ore loss) and using timber poles and packs. Because modern mines may have as many as 120 operational stope faces per shaft, a large quantity of timber would have to be transported into the mine if timber was used exclusively as the in-stope support, which would be both expensive and time consuming. To reduce these problems grout packs are often used in shallower operations.

Grout packs were first introduced to the South African mining industry in 1975. The concept was motivated by a need to reduce labour-intensive and costly material handling in the stopes and bottlenecks in the shafts. A tailings-based support system transported in pipe ranges and pumped over long distances was, therefore, attractive to the industry. Using tailings pumped from surface was a logical extension to the cement / water batching plant technology that had been developed for the purpose of water control in Stilfontein, Buffelsfontein and Harmony Gold Mines in the mid 1960s. In 1975, tailings were sent to Dowell Schlumberger (France) for the evaluation of the pumpability of tailings over long distances with or without dispersing agents. The results were extremely encouraging. Friction losses of 0.518 pounds per square inch / foot (psi/ft) or less were recorded in a 3 inch pipe while pumping 210 US gallons per minute without dispersing agents and 0.156 psi/ft with them (Heinz 2001).

Following these tests the grout pack support system was conceived. First used at Randfontein and Elsburg Gold Mines, grout packs proved to be a very effective, stiff and strong support, particularly for high stoping width and multi-reef excavations (Heinz 2001). The first grout-based packs were designed with the lower portion consisting of a 1200 mm square outer skeleton for containing the bag, and three or four layers of solid slabs for blocking it against the hangingwall. A bag capable of containing cemented grout was suspended inside the skeleton portion. Different types of bag material were tried; for example, hessian and various polypropylenes. An open-weave polypropylene became the preferred grout bag fabric because it retains almost all the solids while allowing the water to permeate out (Coetzee 1975).

In shallow mining applications, with low stope convergence, the support performance criteria were for stiff and strong support that did not fail under specified loading conditions. Grout packs met these criteria more than adequately if they were properly installed. It is estimated that several hundreds of thousands of grout packs have been installed in successful support systems over the years. However, there is one deadweight situation under which grout-pack-based support systems are inadequate. This condition results from the phenomenon known

as back-break, where tens of metres of hangingwall strata are detached from the rock mass, and their weight is transferred to the support system. No artificial support system except backfill can control this phenomenon. Shortly after the introduction of grout packs, the potential occurrence of back break under particular geological conditions led to the introduction of in-stope pillars as the primary stope support and grout packs as secondary support to prevent back breaks on some mines.

The concept of grout packs was carried over to the platinum mines in the early 1980s, where they were initially used as breaker-line support. At present, 12 000 to 14 000 grout packs are placed each month within the South African platinum industry where they are variously used as the main element of local support, gully support or breaker-lines in combination with timber props.

In recent years, the platinum mining industry has focused on changing its mining method from “drill-and-blast” mining with crush pillar to more efficient mechanised mining. Fully mechanised mining would not allow pillars to be left in the face. For this reason, there is a need to establish alternative support strategies. It was determined that a high percentage of backfill could be used to stabilise faces of up to 100 m in length. It was also found that grout packs could supply the required support resistance, depending on the height of the hangingwall material to be supported, the support characteristics of the grout packs, and the pack spacing (Roberts et al 2005).

Recent measurements conducted on in-stope 4 m x 3 m crush pillars in platinum mines have shown residual strengths of between 15 MPa and 20 MPa (Watson 2006). A strong motivation for this project was, therefore, to determine whether or not a practical, cost-effective system of grout packs could replace in-stope pillars where the potential fallout thickness, defined by a consistent parting, was 12 m. The required support resistance for 12 m of hangingwall rock is 340 kN/m<sup>2</sup>.

The methodology adopted for this project was to determine *in situ* the load / deformation characteristics of different designs and sizes of grout packs and the relationship between laboratory tests results and *in situ* performance. The former is required for the engineering design of support systems in terms of support resistance and stiffness. The latter will allow a better estimate of the underground performance of new designs of grout packs to be made from tests carried out in the laboratory. Using laboratory test results to determine the characteristics of *in situ* grout packs would be easier, quicker, and less expensive than obtaining the information from instrumented packs placed underground. A programme of full-scale laboratory testing and *in situ* measurements was therefore undertaken to establish a relationship between packs tested in these two different environments. In addition, as mines extend deeper, or if pillars are not cut, stope closure rates will increase, with the result that

greater yieldability of grout packs will be required. Methods for providing this yieldability were therefore also investigated.

Prior to this investigation, a number of other grout-pack-testing programmes have been undertaken by suppliers and users, many of which show favourable grout pack strength and stiffness when compared with timber packs of similar dimensions. Matjhabeng (formally Western Holdings, Welkom) and Great Nologwa (formally Vaal Reefs 8 shaft) undertook a series of laboratory tests between 1998 and 2001 upon implementing “pack in the pipe” grout packs as part of their stope support standard. The tests were primarily used to adjust their grout mix to achieve the desired support resistance, stiffnesses and yieldability. The press tests showed that grout packs could supply a cost effective alternative to timber packs, previously used exclusively on these mines, while dynamic testing showed that they were capable of withstanding the type of seismic events possibly expected at these mines (Trevarthen and Van Wyk 1998).

Reported evidence of successful *in situ* grout pack tests is scarce, which is surprising because several mentions are made in the literature of the likely difference between press strengths and strengths underground as a result of the *in situ* packs curing under closure. A program of grout pack monitoring was however undertaken in 2003 at Great Nologwa in a collaborative effort between CSIR Miningtek and the mine. Electronic instrumentation installed in grout packs and in continuous closure monitors proved unreliable and the results were inconclusive (Coetzer 2003).

Before beginning the current test programme, a literature survey was undertaken to determine whether or not a standard pack testing procedure existed. Several of the support suppliers implement testing procedures that apply to their own products, but there is currently no standard methodology that applies to all variations of pack types. Following formal meetings between suppliers, industry consultants and users, a simple testing method was proposed:

- Packs to be constructed according to manufacturer’s standard (including aspects such as base configuration and pre-stressing loads).
- The packs should be tested to within 15% of the pack’s aspect ratio (height: width) as specified by the supplier.
- Standard loading rates should be set at 30 mm/min. However results are acceptable from 15 –30 mm/min as long as the rate is recorded and is reflected as such in the database.
- Every variation in pack size and design needs to be evaluated (generic types are not included in this specification).
- The pack should be compressed at least 50% or 600 mm whichever is the least.

- The average of at least 3 test results should be used for design purposes (Kullmann 1999).

The above test procedure was adhered to during the course of this press testing program with the exception that the compression rate was set at 15 mm/min as it was felt that the slower rate would allow more focus on the initial stiffness of the packs. In addition, very few packs were tested beyond 40% strain as the packs proved to be well past their peak strengths and only low residual strengths remained.

The results of the laboratory and *in situ* tests conducted on grout packs are described in this report. Chapter 2 details the grout strength testing program. The methodology used, and results of laboratory tests are given in Chapters 3 and 4. In Chapter 5, flatjack type loadcells are tested for reliability and are calibrated. Access to a reliable loadcell provided a methodology for underground testing of grout packs which is discussed in Chapter 6. An analysis of factors affecting the performance of grout packs is presented in Chapter 7. Chapters 8 and 9 look at the difference between underground tests and press tests and show a potential relationship between the test results from these two different environments. Conclusions and Recommendations are presented in Chapters 10 and 11.

## 2 Grout testing

The background to the testing of grout packs was given in Chapter 1. In this chapter three methods of grout testing are described and the results of a grout testing programme, conducted concurrently with the pack tests, are given. A number of grout cubes and cylinders were formed as the grout packs were being cast for press testing, or within a day of the *in situ* packs being placed underground. The cylinders were tested for Unconfined Compressive Strength (UCS) and the cubes were tested for compressive strength at intervals for up to 56 days. The purpose of the tests was to determine the increasing strength of the grout during the curing process for each batch of packs tests.

### 2.1 Grout sample preparation

Three different techniques were used for the preparation of specimens for grout strength testing. The contractors who operate the grout plant regularly make grout cubes using 150 mm cube moulds (Figure 2.2). CSIR also undertook a grout strength testing programme, casting the grout into 43 mm x 84 mm cylinders giving a width : height (w:h) ratio of 1:2 (Figure 2.1).



Figure 2.1 Preparation of 43/84 mm grout cylinders



Figure 2.2 Preparation of 150 mm grout cubes

**Cube tests (Mine)** At the time that the packs were pumped, a sample of grout was pumped directly into a spare grout bag by the plant operators. The spare bag was then twisted to squeeze water from the grout. After several minutes the grout was shovelled into lubricated cubic moulds. The top part of each moulded cube was trimmed with a spade and the cubes were left to cure outside with no cover. The cubes were removed from their moulds after one day and were placed in a barrel of water until tested.

**Cylinder tests (CSIR)** While the packs were being pumped, a sample of grout was collected in a bucket. Care was taken to avoid spilling so as not to adversely affect the percentage of water present in the mixture. The grout was agitated and then decanted into paper-lined cylinder moulds placed on a flat lubricated plate.

Water was allowed to egress slowly from between the mould and the surface plate while grout was continually added to the mould to compensate for the water loss. The cylinders were cast indoors and then sealed in large plastic bags to reduce evaporation during curing. The cylinders were trimmed the following day, removed from their moulds, and left to cure indoors.

For the purpose of testing the grout used in the underground packs, grout cylinders were made up in the laboratory using tailings, water, cement and fly-ash collected from the mine, which were then mixed together in the same ratios as are used on the mine.

The third method of preparing cylinders for UCS testing involved removing lumps of grout from underground packs, and then drilling core samples from them, in a laboratory.

All the 43/84 mm cylinders produced from the above methods were then tested for UCS in a press.

## **2.2 Testing procedure**

Altogether four different grout recipes were used during the pack testing programme. The reasons for this are:

- The press tested packs were cast at Lonmin's Karee 3 shaft using a 10 MPa grout.
- Underground tests were carried out at Union Section's Spud shaft and Lonmin's Rowland shaft, where 7 MPa and 14 MPa grouts were used respectively.

In addition, a weaker, 3 MPa grout was made up at Karee 3 shaft to test the effect of altering the grout strength.

Two of the cubes made by the grout operators were press tested to determine their strength. The four grout recipes used are given in Table 2-1.

Table 2-1 Recipes for the four strengths of grout used (percent by mass)

	Karee shaft 8 to 10 MPa grout	Rowland shaft 14 MPa grout	Karee shaft 3 MPa grout	Spud shaft 6 to 8 MPa grout recipe
Cement (OPC)	10	14.5	4	12
Fly ash	16	17.5	17	13
Tailings	50	44	57	42
Water	24	24	22	33

UCS testing was also conducted on a one day old grout cylinder that had been strained 4% in a load bearing capacity test, put aside, and then retested after 56 days, and on two cylinders that had been drilled from a sample of grout taken from a pack at the Rowland shaft site. The former was done to simulate grout curing under closure, a condition that may occur when packs are cast in an advancing stope, while the latter tested the UCS of *in situ* grout. The results are included in Figure 2.6.

Before grout testing began, a simple visual examination was undertaken by an experienced technician to assess the quality of each cube or cylinder, looking at compaction, excess voidage, aggregate uniformity, size distribution and the presence of cracking. Once the sample had passed the visual test, it was then loaded between the two steel platens of the pre-calibrated press, and load was then applied at a constant predetermined rate. Load and displacement were recorded throughout the test, which continued until the sample no longer provided any significant load resistance. (Figure 2.3, 2.4 and 2.5)



Figure 2.3 UCS testing



Figure 2.4 Grout cylinder first tested after one day undergoing re-testing after 56 days



Figure 2.5 Grout cube undergoing strength testing

## 2.3 Results

The results from cylinder and cube tests for various curing times are presented in Figure 2.6. and summarised in Table 2-2. It is evident from the Karee tests that the cubes have strengths two times greater than those of the cylinders over the first 28-day period, largely because of the factors listed below.

- The cube strength is generally higher than the UCS owing to differences in the effective w:h ratio (Ozbay, 1987).
- Water is squeezed from the grout before it is placed in the cube moulds, whereas the cylinders are un-drained and therefore have higher water content.

The retested cylinder, which was strained 4% after one day was then left to continue curing. It regained 47% of its strength by day 56 (3.5 MPa compared with 7.5 MPa for previously untested specimens of the same age).

The UCS test conducted on a sample taken from the top portion of an underground pack at Rowland Shaft shows a weaker than expected result. This is possibly the result of cement settling from the top towards the bottom of the pack during the casting stage, or the result of inefficiencies during the hydraulic transportation of the slurry, thus weakening the upper portion of the pack.

Cube tests are commonly used in the construction industry and are often used by grout plants on mines to measure the strength of concrete against the contractors own index; however UCS tests are more regularly used in rock engineering to directly determine fundamental characteristics of the rock. The UCS tests are direct indicators of grout strength, while the cube tests are useful in that they may be more representative of the grout as poured. Cube tests are also commonly used in the industry, allowing for a direct comparison with typical results.

The different grout recipes used at the three shafts (Table 2-1), resulted in a wide variation in the strength of grout (Figure 2.6 and Table 2-2). Generally the mines choose grout recipes with the minimum amount of water possible in the slurry, while maintaining hydraulic transportability.

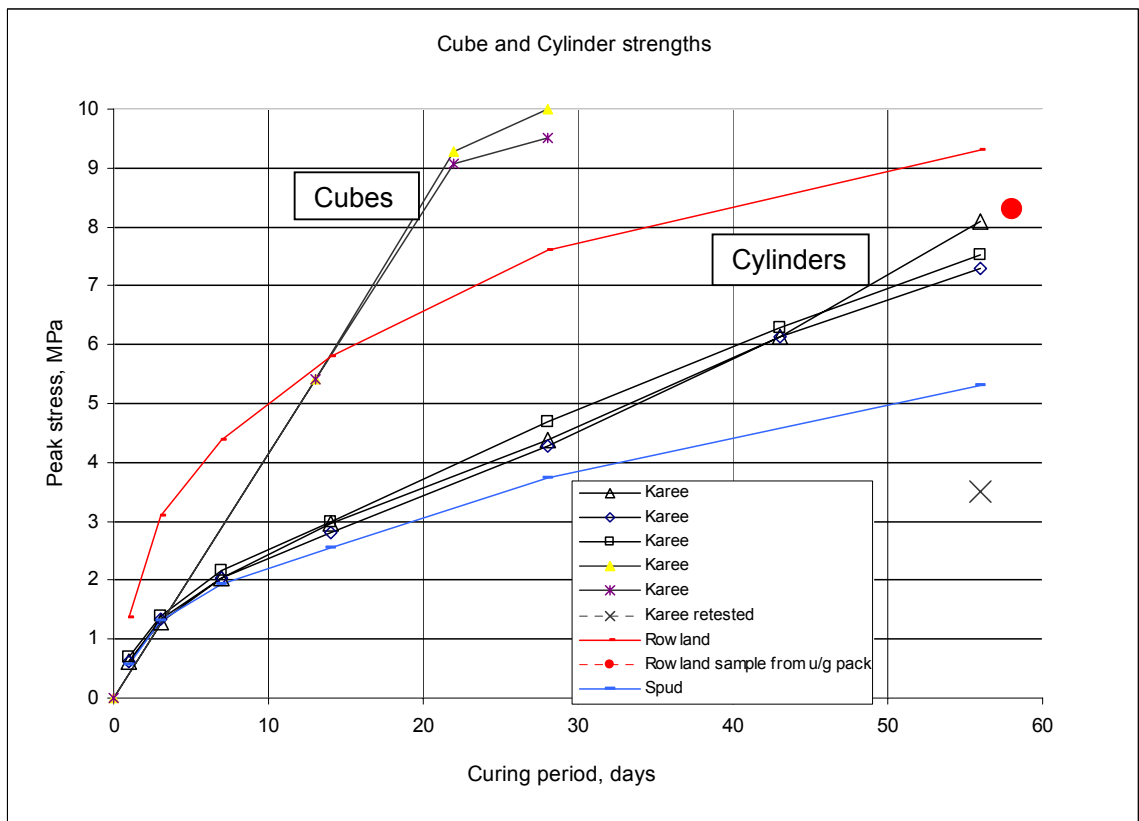


Figure 2.6 Strength tests conducted on cubes and cylinders

Table 2-2 Grout strengths (MPa) vs. days

	1 day	3 day	7 day	14 day	28 day	56 day
Karee cylinders: UCS	0.7	1.2	2.0	2.9	4.6	7.7
Karee cylinder retested: UCS						3.5
Karee cubes				5.4	9.5	
Rowland cylinders: UCS	1.5	3.1	4.5	5.8	7.7	9.3
Rowland cylinder retested: UCS						8.3
Spud cylinders: UCS	0.7	1.2	2.0	2.5	3.8	5.4

## 2.4 Summary

The grout testing programme illustrated significant differences in the strength of grout used on different mines. It also showed significant increase in grout strength with respect to time, which continued for the full 56 days of testing. This was an important finding because the stiffness and initial load bearing capacity of packs, prior to the steel support becoming active is entirely dependent of the strength of the grout. In the following chapter the load bearing capacity and stiffness of press tested grout packs are determined after 28 days of curing, while in Chapter 6 the results of 3 months of monitoring *in situ* packs are presented.

### **3 Grout pack testing**

In Chapter 3 the specifications of the press tested packs and the method used for testing them are presented. The results of the pack press tests are presented in the following Chapter.

Fifty press tests were conducted on eight different types of grout packs with variations in diameter; reinforcement; strength of grout; height and the inclusion of mine poles. The packs were cast at Lonmin's Karee 3 shaft grout plant and the tests were conducted using the CSIR's 10 MN press at Cottesloe.

Originally five variations in the w:h ratio of a standard RSS pack were to be determined. Other variables, such as grout strength and reinforcement characteristics were excluded from the tests so that only the effect of w:h ratio could be better understood. Having completed four sets of tests on different diameter RSS packs, it became evident that many critical aspects of pack design required investigation. Evidence from the tests showed that the amount and type of reinforcement included in the pack had a significant influence on the post-failure behaviour of the packs. This was identified as an area of interest to the mines as packs may be required to provide substantial support resistance up to and beyond 30% closure. The scope of the test work was subsequently expanded to include variations in pack design, with particular emphasis on the reinforcement used to enhance post-peak-load performance.

#### **3.1 Casting of packs for laboratory testing**

The availability of pumpable grout in an area where access to mechanical loading equipment was possible was a priority in selecting the site for casting the packs. The Karee 3 shaft grout plant fulfilled all the requirements and, accordingly, a frame was built on surface at the plant to contain the packs as they were poured. A photograph of the frame is presented in Figure 3.1. The frame was designed so that packs could be cast with heights of up to 2 m, however, except where otherwise stated; all packs were cast with 1.2 m height, using a grout with 28 day cube strength of 10 MPa. The 1.2 m height was chosen as it represented a typical stoping width for Platinum mines, while the 10 MPa grout used had the same strength as grout pumped underground at Karee 3 shaft.

Reinforced grout bags were attached to the upper surface of the frame and grout was then pumped into them at the same rate that packs are pumped underground. After allowing approximately 30 minutes for seepage, the packs were re-pressurised, by pumping more grout into them. This forced some excess water out of the bags, and forced them to lie tightly against the upper frame. In many cases the pressurisation led to bending of the frame, which resulted in uneven and non-parallel pack surfaces. However, load cells sensitive to pressures

of 100 kPa, installed at the pack and frame interface, were unable to record any pre-stressing load.

After casting and allowing the packs to cure for at least two weeks, they were transported by truck to the CSIR testing facility at Cottesloe. Grout cubes and cylinders were cast simultaneously with the packs as described in Chapter 2 and grout strengths were recorded.



Figure 3.1 The frame at Karee 3 Shaft grout plant where the packs were cast

### 3.1.1 Press tests

After 28 days of curing, each pack was loaded on the test platen, centralised and then wheeled into the 10 MN press. Once correctly positioned, the presses top platen was lowered onto the pack and testing commenced. A 15 mm / minute compression rate was applied to all packs tested in the press.

The deformation and applied load were recorded digitally by the press instrumentation and the data were stored on a computer. In addition, the following aspects of the test were monitored:

- Initial pack dimensions;
- Reinforcement configuration;
- Internal and external load cell readings;
- Change in circumference (from which pack dilation could be calculated).

Photographs were also taken at regular intervals to record the sequence and mode of failure.

A summary of the number tested, dimensions and reinforcement used on each of the packs is presented in Table 3-1. Six tests were conducted on 550 mm diameter RSS packs at varying heights (Section 4.3), while all other packs were cast with a height of 1.2 m.

Table 3-1 A summary of the number and specifications of the packs tested in the press

Type	Diameter / dimension mm	Number tested	Reinforcement	Pack volume m <sup>3</sup>	Percentage of steel by mass
RSS	1400	4	The bag is contained in a nylon net with a 75 mm x 75 mm x 3 mm diamond pattern. Seven 12 mm diameter 504 MPa mild steel bars formed into rings, with a tensile strength of 57 kN each and capable of 33% elongation. The rings are spaced 200 mm apart from top to bottom.	1.85	0.81
	1200	3		1.36	0.92
	900	4		0.76	1.23
	750	9		0.53	1.48
	550	8		0.29	1.98
Yield pack	750	5	Ten steel rings as above. Seven at 750 mm diameter and three at 900 mm diameter.	0.53	2.20
	750	2	Ten steel rings as above but including two 200 mm mine poles.		
Tau pack	620 x 620	1	200 mm diameter mine pole.	0.46	0
	900 x 900	2		0.97	0
Cartwheel	810	1	Two 30 mm x 5 mm mild steel cross braced bands + steel mesh.	0.62	0.90
	810	1	Three 30 mm x 5 mm mild steel cross braced bands + steel mesh.	0.62	1.35
Cempack	1000 x1000	1	Six layers of 3 mm x 60 mm x 60 mm annealed weld-mesh.	1.2	1.00
	1000 x1000	1	Eight layers of 3 mm x 60 mm x 60 mm annealed weld-mesh.	1.2	1.33
Mesh Pack	750	4	Four 25 mm x 4 mm galvanised steel straps + 3.15 mm x100 mm x 100 mm diamond steel mesh containment.	0.53	1.04
Bamba pack Mk 1	750	2	Six 6 mm steel rings + 2 x 50 mm x 50 mm steel mesh containment.	0.53	0.60
Bamba pack Mk 2	750	2	Six 12 mm steel rings + 2 x 50 mm x 50 mm steel mesh containment.	0.53	0.90

## **4 Grout pack test results**

In this chapter, the results of the press tests are presented in graphical format and are discussed in each subsection. Photographs, depicting the mode of failure, and a summary table are presented for each set of tests. Also included in the table are the support resistances at maximum load and at 200 mm and 300 mm compression for each pack design, assuming a two metre skin-to-skin pack spacing.

Where possible, repeated tests on any particular design of pack were cast in successive batches at the grout plant. This was done so that any variations in the grout strength, or curing conditions between successive batches, would not adversely affect the results obtained from one particular design. RSS packs are the most common grout packs used in the industry and were thus investigated in the most detail.

### **4.1 RSS packs of varying diameter**

Five different diameters of RSS packs were tested. The smaller, 750 mm and 550 mm diameter packs are the most commonly used in the industry, however, an investigation into the load bearing capacity of large packs was necessary as they are considered a potential substitute for crush pillars in mechanised mining stopes. The first pack tested was the 1400 mm diameter RSS pack.

#### **4.1.1 1400 mm diameter RSS packs**

The load-displacement characteristics for four 1400 mm diameter packs are presented in Figure 4.1. The best performing pack reached a peak load of 8.4 MN which, using the nominal pack diameter to calculate the area, translates to peak strength of 5 MPa.

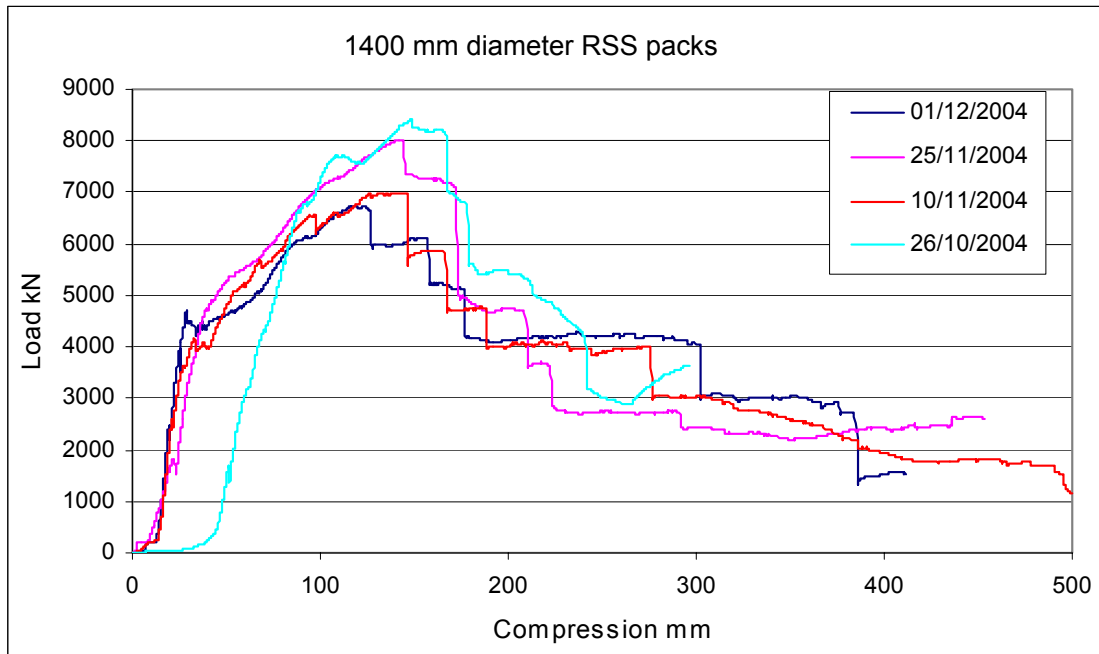


Figure 4.1 Press test results of 1400 mm diameter RSS packs

Table 4-1 1400 mm diameter RSS pack results

Average initial stiffness kN / mm	Average maximum		Average at 200 mm compression		Average at 300 mm compression kN	
	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup> /	Load	Support resistance kN/m <sup>2</sup>
135	7200	623	4740	410	3025	261

Except for the test conducted on 26/10/2004 the packs showed an initial stiffness, up to 30 mm (2.5 % strain), of 135 kN/mm. During this period no cracks were evident on the pack surfaces. A period of strain hardening followed during which cracks first appeared on the outer surfaces of the packs. Peak loads were achieved at approximately 140 mm compression, or 11% strain at which stage substantial evidence of peripheral slabbing and the formation of a diagonal shear plane were evident. The pack tested on 26/10/2004 was cast with non parallel ends, resulting in asymmetric press loading and subsequent early yielding, which continued until the face/platen contact had improved under compression. Photographs taken at different stages of a 1400 mm diameter pack test are presented in Figures 4.2 and 4.3.

The typical failure mode of all the press tested grout packs was peripheral slabbing, followed by the formation of a diagonal shear plane as the packs expanded laterally (Figures 4.2 and 4.3).

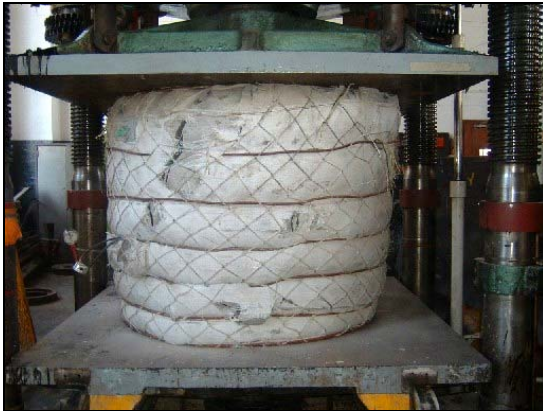


Figure 4.2 After 16 % compression



Figure 4.3 After 28 % compression

The sudden drops in load evident in Figure 4.4 are associated with failure of the steel rings. In most cases the upper rings failed before the lower rings. In order to quantify the extent of the observed lateral expansion under load, a trace wire with a section of measuring tape was tightened around the circumference of the pack at its mid height and readings taken as the dilation occurred. The readings continued until the circumference had increased by 630 mm indicating an average radial increase of 100.2 mm, after which peripheral damage to the pack made further readings impossible. Figure 4.4 shows the relationship between axial compression and radial dilation of the measured pack under load. Note that sudden increases in dilation correspond approximately to ring failures. It is noted that Poisson's ratio ( $\nu$ ) exceeds 0.5 after 30 mm compression (using the radial dilation curve) indicating evidence of early failure of the grout in the load compression cycle.

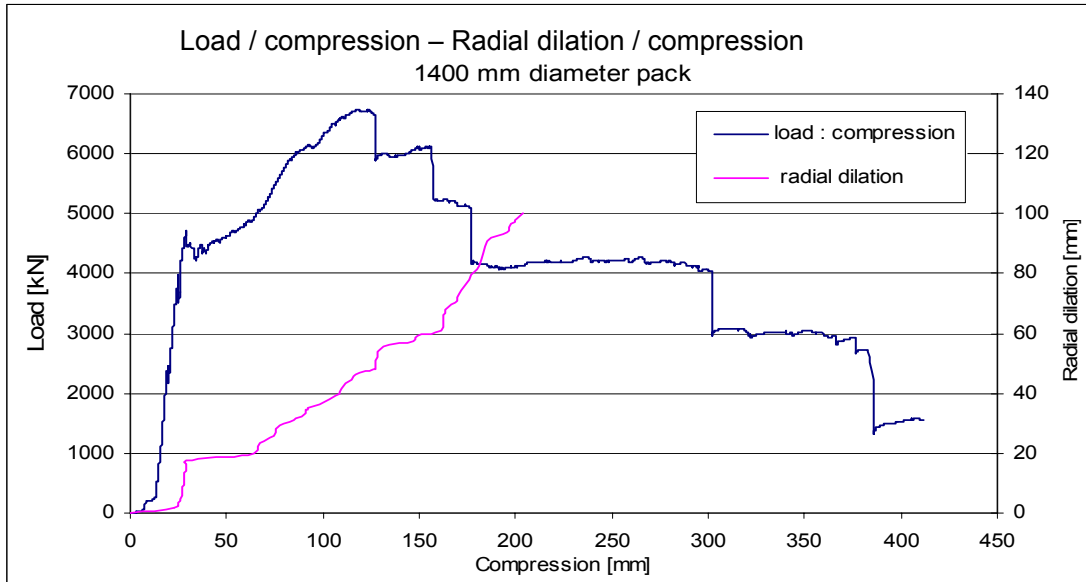


Figure 4.4 Compression vs. Load and Radial dilation of the pack

#### 4.1.2 1200 mm diameter RSS packs

The results of three tests of the 1200 mm diameter RSS packs are presented in Figure 4.5 and summarised in Table 4-2. A maximum load of 5.5 MN was recorded, which translates to a strength of 4.86 MPa. Uneven loading conditions (indicated by substantial tilting of the loading platen) were observed for the test on 1/12/04. This may account for the relatively low strength and post peak performance indicated. In all cases, an approximate 40% drop in load was recorded following failure of the first ring, which occurred between 13% and 18% strain. A photograph taken during the press test (Figure 4.6) shows a large amount of lateral dilation and associated failure of the upper rings.

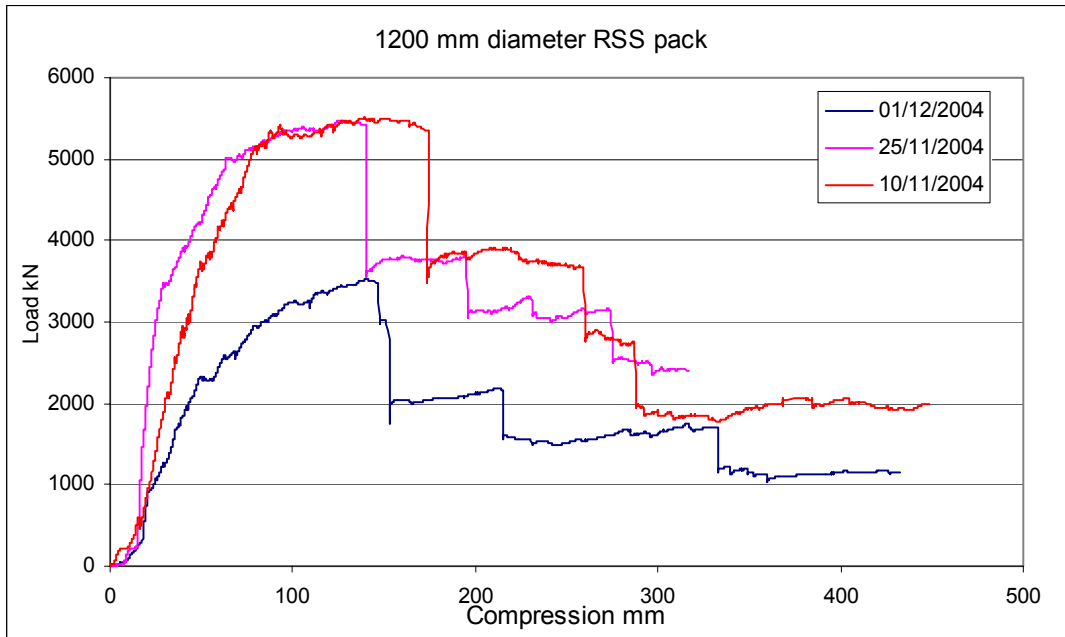


Figure 4.5 Press test results of 1200 mm diameter RSS packs

Table 4-2 1200 mm RSS pack results (the result from 1/12/2004 is not included)

Average initial stiffness kN/mm	Average maximum		Average at 200 mm compression		Average at 300 mm compression	
	Load kN	Max support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>
106	5484	536	3740	463	2325	365



Figure 4.6 1200 mm diameter RSS pack showing lateral dilation and failure of the upper rings

### 4.1.3 900 mm diameter RSS packs

Tests on the 900 mm diameter RSS grout packs indicated maximum press load bearing capacity of 3.6 MN (5.66 MPa) and an average peak load bearing capacity of 3.5 MN (5.5 MPa). The load-compression curves from the tests are presented in Figure 4.7 and a summary of the results is presented in Table 4-3. Rapid drops in load associated with ring failure are evident as with previous tests. Horizontal dilation, resulting in vertical slabbing of the grout and progressive downward rupturing of the rings is evident in Figure 4.8.

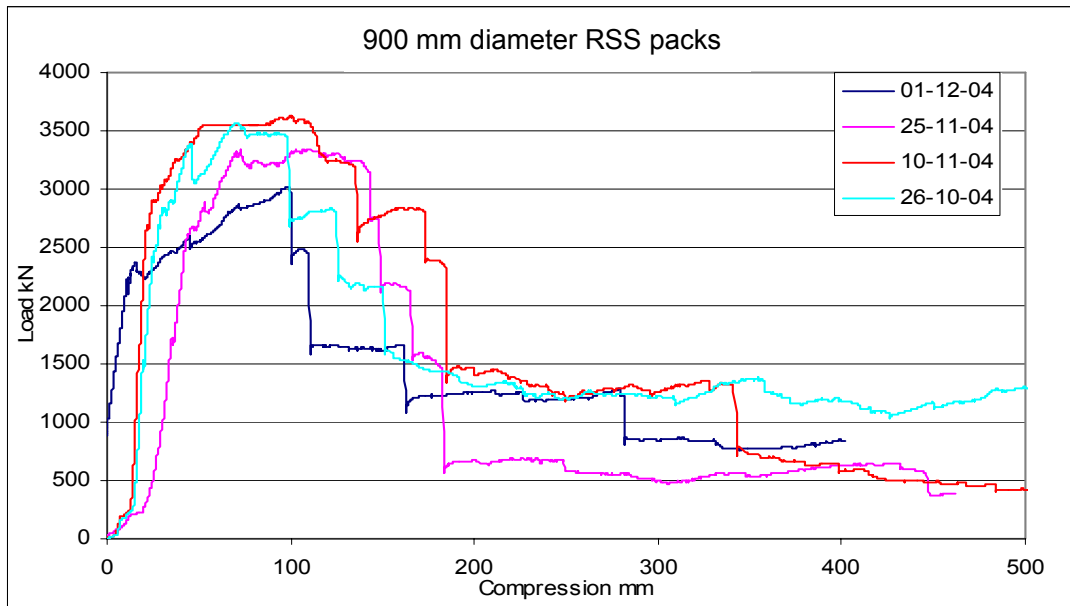


Figure 4.7 Press test results of the 900 mm diameter RSS packs

Table 4-3 900 mm RSS pack test results

Average initial stiffness kN / mm	Average maximum		Average at 200 mm compression		Average at 300 mm compression	
	Load kN	Max support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>
92	3324	395	1314	156	1000	119



Figure 4.8 900 mm diameter pack at 36% compression

#### 4.1.4 750 mm diameter RSS packs

The load-compression curves for 750 mm diameter packs are presented in Figure 4.9. To investigate the effect of grout strengths on pack performances, both 3 MPa and 10 MPa grout strengths was used in these tests. The 10 MPa grout pack tests indicated an average peak load of 2.67 MN, which translates to a peak strength of 6.05 MPa. An average peak load of 1.47 MN (nominal 3.4 MPa) was recorded for packs cast using the weaker (3 MPa) grout. It is evident from Figure 4.9 that the 3 MPa grout pack peak load was about half that of the stronger grout, but the post-failure behaviour (beyond 12% strain) for the two grout strengths, was similar. The implication is that the stronger grout is more brittle than the weaker grout and breaks down more rapidly under load. After 12% strain, the cement in both packs is mostly pulverised and residual strength, which is reliant on the internal friction angle, is similar for both grout strengths. A summary of the test results is presented in Table 4–4.

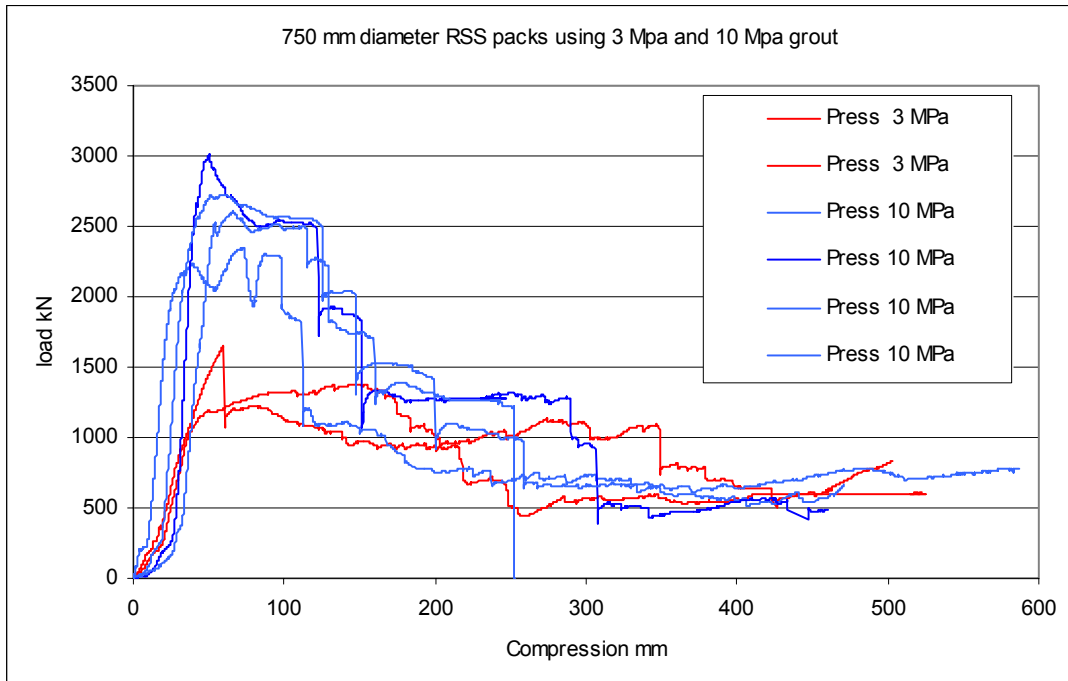


Figure 4.9 750 mm diameter RSS packs with 3 MPa and 10 MPa grout

Table 4-4 Summary of 750 mm RSS pack results

Grout strength MPa	Initial stiffness kN / mm	Maximum		At 200 mm compression		At 300 mm compression	
		Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>
3	32	1470	194	990	130	800	105
10	62	2550	337	1270	168	860	113

Figure 4.10 shows 750 mm diameter RSS packs undergoing press testing while Figure 4.11 shows a pack of the same diameter and similar height installed in an underground stope. Both packs have been compressed approximately 150 mm (12% strain) and both have a broken 'second from the top' ring (the top ring is flush against the upper surface and is not visible in either photograph). More grout has broken off the outer surface of the press tested pack, possibly as a result of its grout being more stiff and therefore, more brittle. The underground test results are presented in Chapter 6.



Figure 4.10 A 750 mm diameter RSS pack in the press at 150 mm compression



Figure 4.11 A 750 mm diameter RSS pack installed at Spud shaft with 150 mm closure

#### 4.1.5 550 mm diameter RSS packs

The 550 mm diameter grout packs were the smallest diameter packs to be tested. Figures 4.12 and 4.13 show a 550 mm diameter RSS pack before and after press testing. The typical sequence of failure of an RSS pack was peripheral slabbing followed by the failure of the upper rings and the formation of a diagonal shear plane. The end result is evident in Figure 4.13 where a 550 mm diameter RSS pack, originally 1.2 m high has been compressed by 400 mm and a large amount of the cemented grout has broken away. Figure 4.14 shows the resulting load-compression characteristics. The average peak load for the press tests was 1.69 MN (nominal 7.1 MPa). Initial ring failure occurred on the press tested packs between 100 mm and 200 mm compression.



Figure 4.12 550 mm diameter RSS pack before a press test



Figure 4.13 550 mm diameter RSS pack after a press test

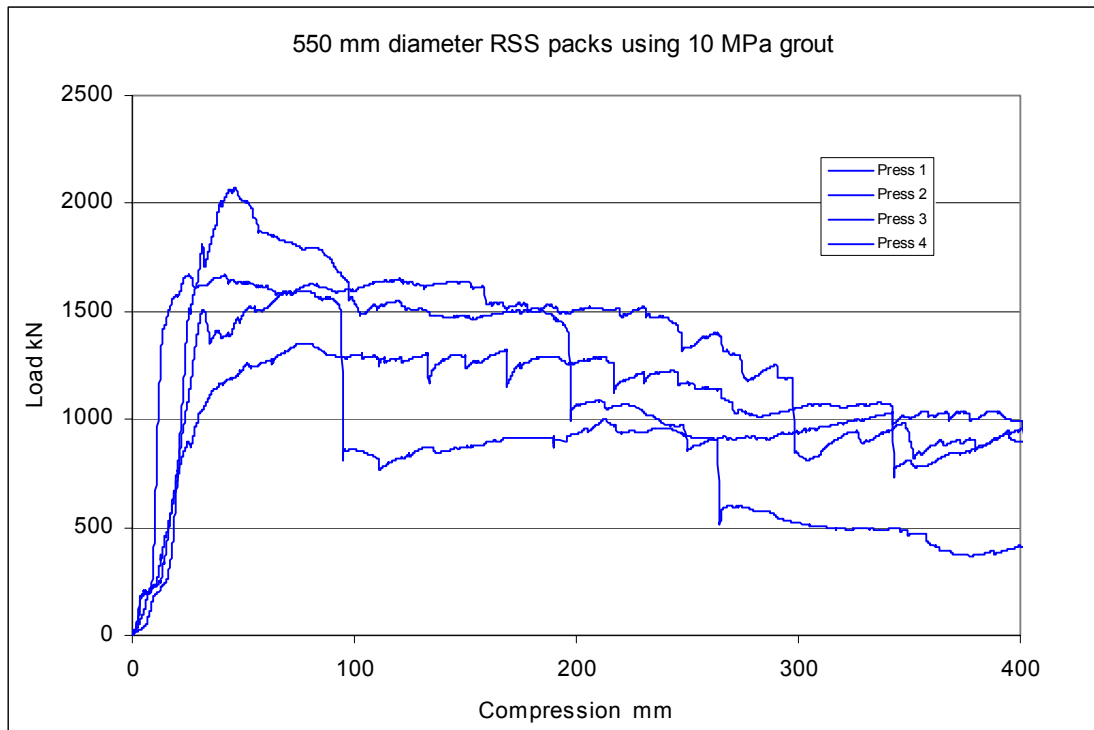


Figure 4.14 Press test results of 550 mm diameter RSS packs

Table 4-5 550 mm RSS pack results

Average initial stiffness kN/mm	Average maximum		Average at 200 mm compression		Average at 300 mm compression /	
	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>
41	1656	255	1263	192	934	143

## 4.2 RSS packs of varying height : width ratio

A number of press tests, conducted on 550 mm diameter RSS packs with the same grout strength and ring spacing, but cast with different heights, are shown in Figures 4.15 and 4.16. Figure 4.17 shows that increasing the height from 0.8 m to 1.55 m (1.5 to 2.8 h:w ratio) did not substantially affect the load-bearing capacity of the pack, despite almost doubling the h:w ratio. However the peak loads were about half those of the 0.8:1 h:w packs. The 0.45 m high packs (0.8:1 h:w ratio) show re-compaction and associated strain-hardening behaviour beyond approximately 230 mm compression (around 50% strain). The 0.33 m high pack (0.6:1) did not fail at all. A different platen (of unspecified strength) was used in this test due to the limited stroke of the press and the shortness of the pack. The drop in load at approximately 180 mm compression is associated with failure of this platen.

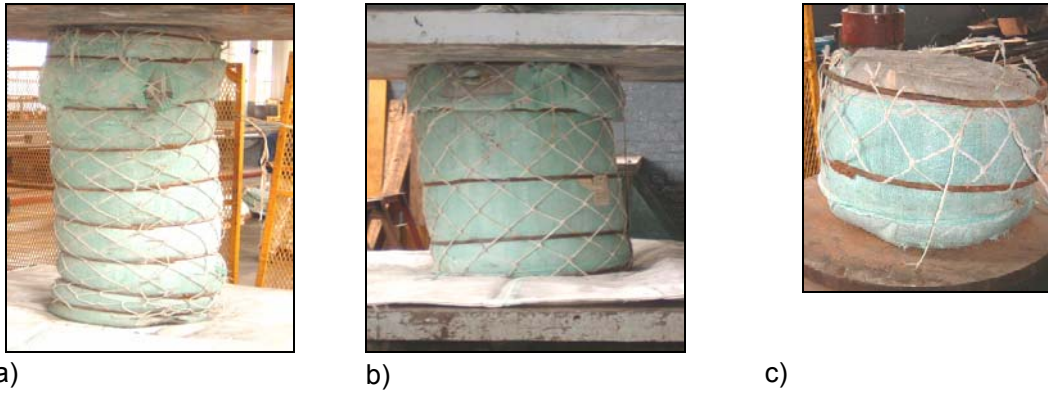


Figure 4.15 550 mm diameter RSS packs of heights: (a) 1530 mm (b) 450 mm (c) 330 mm in the press

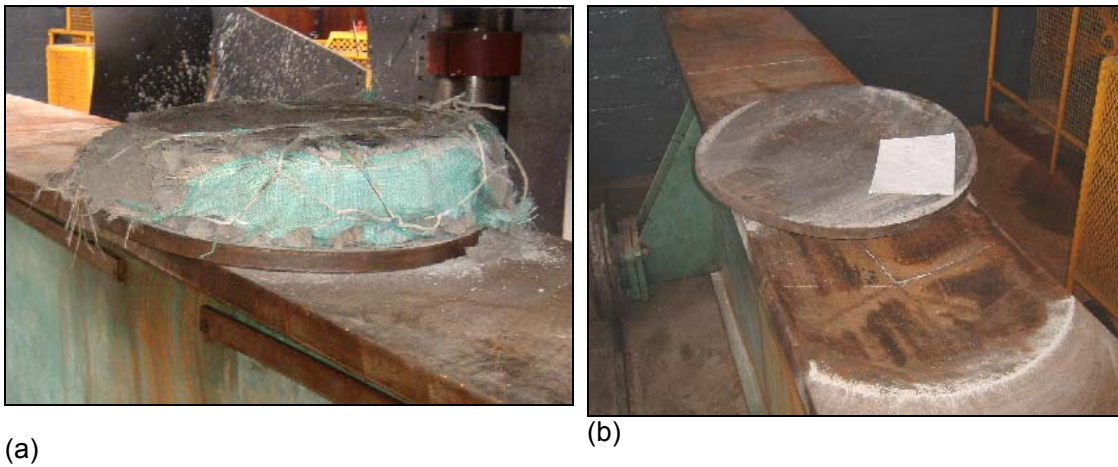


Figure 4.16 (a) 330 mm high RSS pack after 62% compression and (b) damaged lower platen.

Examination of the platen showed major plastic deformation and tensile cracks which penetrated through the thickness of the loaded surface. This pack behaved much like a “squat” pillar, with no failure indicated in the pack, beyond some spalling of the grout skin.

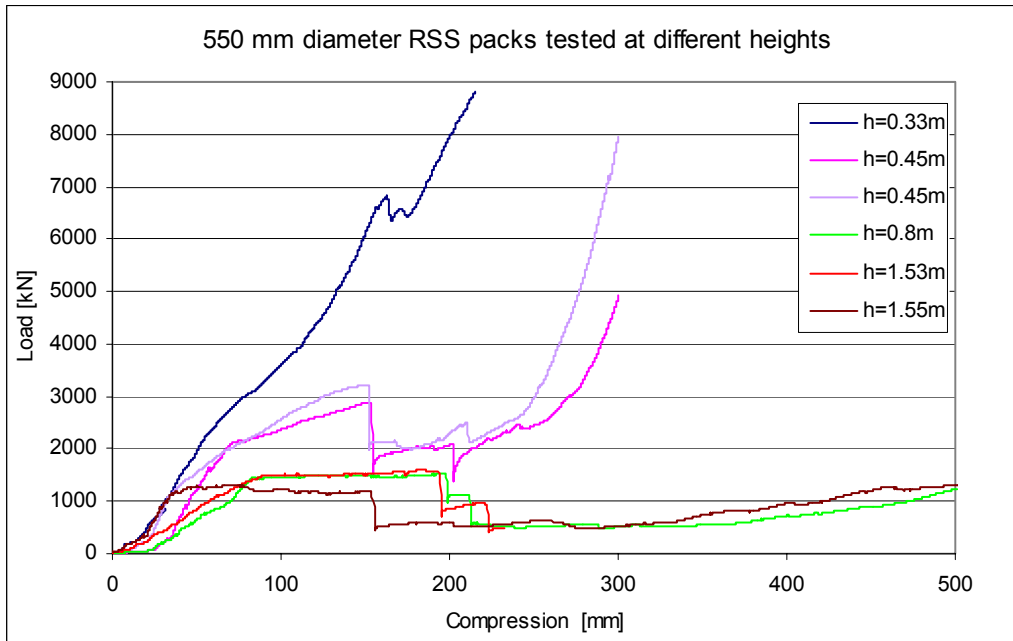


Figure 4.17 Press test results of 550 mm diameter RSS packs of different heights

Figure 4.18 shows the effect of altering the h:w ratio of 550 mm diameter RSS packs. The load bearing capacity decreases rapidly as the h:w ratio increases beyond about 1.2, however, above a ratio of about 1.2, the effect of increasing the height is not significant. These results indicate that 550 mm diameter packs should not be installed where the stopping widths are greater than 1.2 m and perhaps even 1 m.

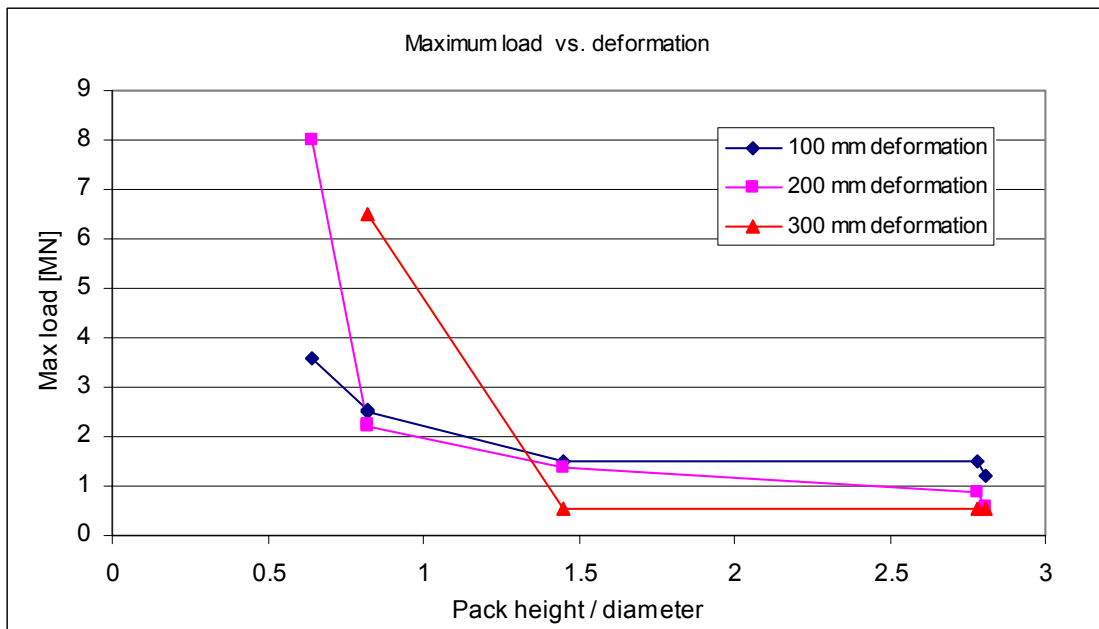


Figure 4.18 Pack loads with different height to diameter ratios

### 4.3 Taupacks

A Taupack is made up from a stack of square bags, each with a 200 mm diameter hollow centre, such that they can be wrapped around a previously installed mine pole, making installation quick and easy (Figure 4.19). Early stiffness is normally provided by the mine pole while the grout portion will take up increasing load as it cures and becomes compressed. It will also assist to contain the pole laterally, thus restricting the pole from buckling. Once the pack has strained beyond about 5%, the pole is likely to have failed and the grout alone will continue to provide support. The Taupacks however, do not have any external support, making them susceptible to lateral failure under closure. Figure 4.19 shows a 900 mm square Taupack being press tested and includes a photograph of the damaged internal mine pole. The test results are presented in Figure 4.20 and are summarised in Table 4-6.



Figure 4.19 A 900 mm square Taupack at different stages of a compression test with the mine pole that was contained within

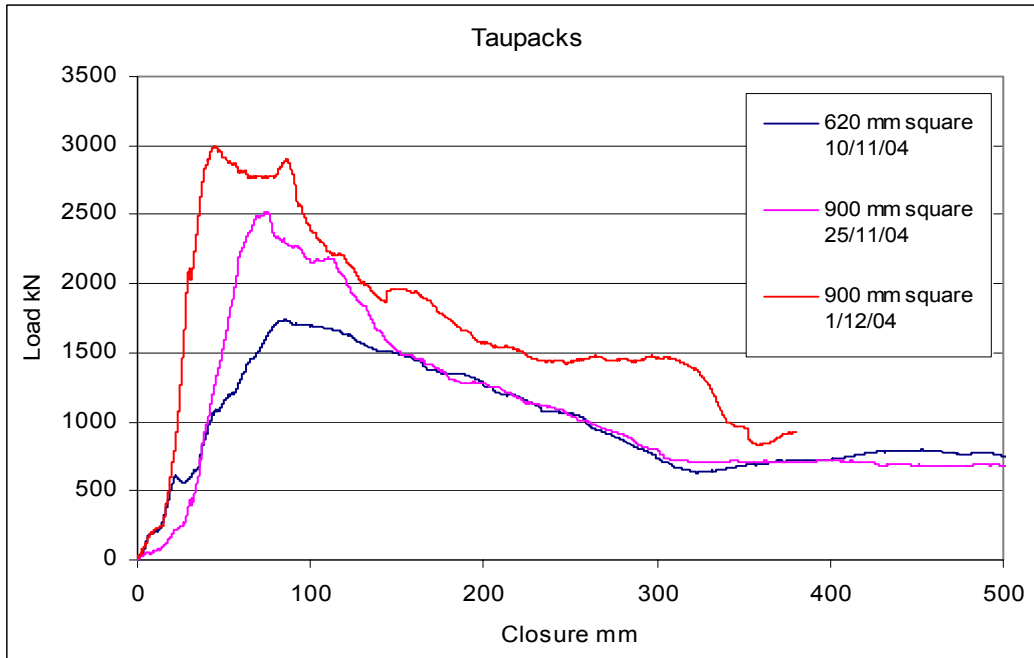


Figure 4.20 Results of the compression test conducted on two sizes of Taupacks

Table 4-6 Taupack test results

Pack cross sectional dimensions mm	Average. initial stiffness kN/mm	Average. maximum		Average at 200 mm compression		Average at 300 mm compression	
		Load kN	Support resistance kN/m <sup>2</sup>	load kN	Support resistance kN/m <sup>2</sup>	load kN	Support resistance kN/m <sup>2</sup>
900 x 900	51	2750	326	1433	170	1137	135
620 x 620	22	1705	203	1262	150	762	90

#### 4.4 750 mm Mesh packs

The mesh pack is similar to the RSS pack in design but with slightly altered shuttering. Instead of the nylon net and steel bars of circular cross section, the shuttering on a mesh pack consists of 100 mm x 100 mm x 3.15 mm diamond steel mesh or similar, reinforced by 30 mm x 5 mm flat bar straps, typically spaced 400 mm apart around the perimeter of the bag. The top and bottom straps have a smaller cross section (8 mm x 5 mm steel bars), as these straps are not expected to contribute significantly to the load bearing capacity of the pack. Figure 4.21 shows two mesh packs installed in a stope. In both cases the fabric of the bag has been damaged by blasting. As with all grout packs, the roll of the polypropylene bag is to give form to the grout pack while containing most of the fines and allowing for water seepage during the early stages of curing. The strength of the pack would therefore not have been compromised by blast damage to the bag, provided that sufficient curing of the grout had

occurred before the blast. A delay period of two hours is generally considered sufficient for this to occur.

The first photograph was taken close to the face where relatively little closure had occurred. The second shows a mesh pack in the back area where lateral dilation and failure of the straps is evident as a result of stope closure. The results of the load-compression tests conducted on the mesh packs are presented in Figure 4.22 and are summarised in Table 4-7. The larger drops in load observed in Figure 4.22 are associated with strap failure and the smaller drops with successive strands of mesh taking up the load, and then failing. The post-failure slope is moderated by the load-bearing capacity of the mesh.



Figure 4.21 Two mesh packs installed in a stope (a) soon after installation and (b) with lateral dilation and strap failure

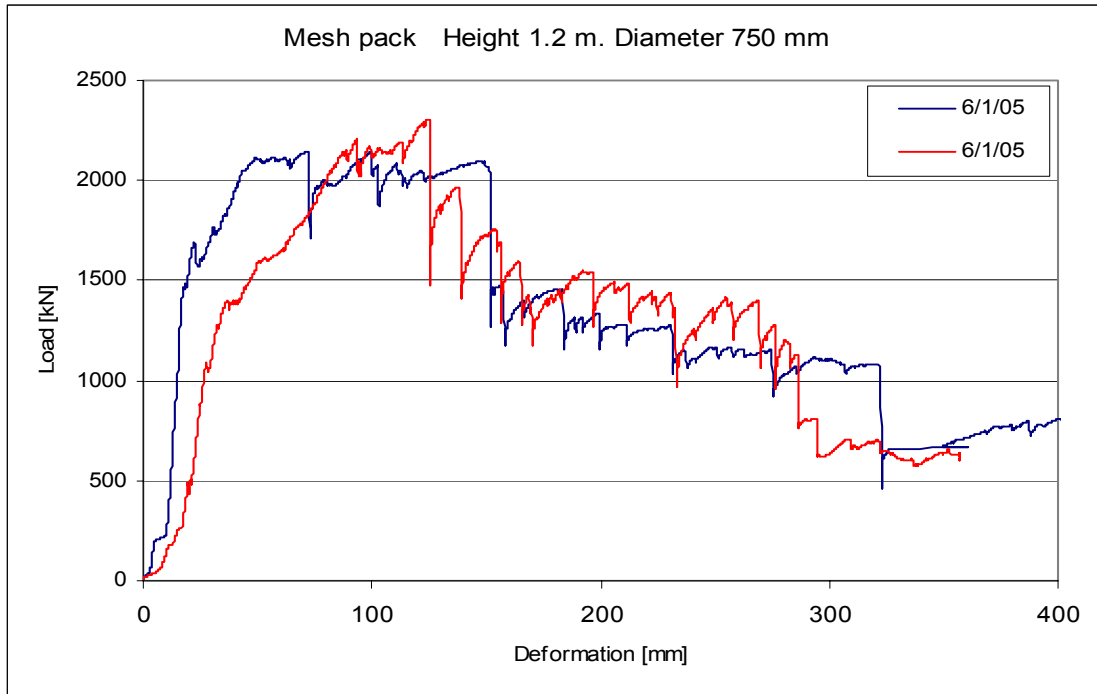


Figure 4.22 Results of the compression test conducted on mesh packs

Table 4-7 Mesh pack test results

Initial stiffness kN / mm	Average maximum		Average at 200 mm compression		Average at 300 mm compression	
	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>
56	2220	293	1412	186	872	115

## 4.5 750 mm Bamba pack

The Bamba pack Mk1 (shown in Figure 4.23) is a 750 mm diameter pack reinforced with 6 mm diameter steel rings spaced 240 mm apart, and with 70 mm x 80 mm x 2 mm wire mesh. The wire mesh is included mainly to help form the pack as it is cast, but will also provide some reinforcement to the packs as they dilate under axial load. The Mk2 version uses the same wire mesh as the Mk1, but the diameter of the rings is increased from 6 mm to 12 mm. Both the Mk1 and Mk2 packs include two mine poles placed outside the fabric, but contained within the steel rings. These will support the packs as they cure and increase their initial stiffness under compression.

The results of the Mk1 and Mk2 Bamba pack tests are presented in Figure 4.24 and are summarised in Table 4-8. The average peak loads achieved by the Mk1 and Mk2 packs were

1450 kN and 2050 kN respectively which translate to nominal strengths of 3.3 MPa and 4.6 MPa.



Figure 4.23 Bamba pack Mk1

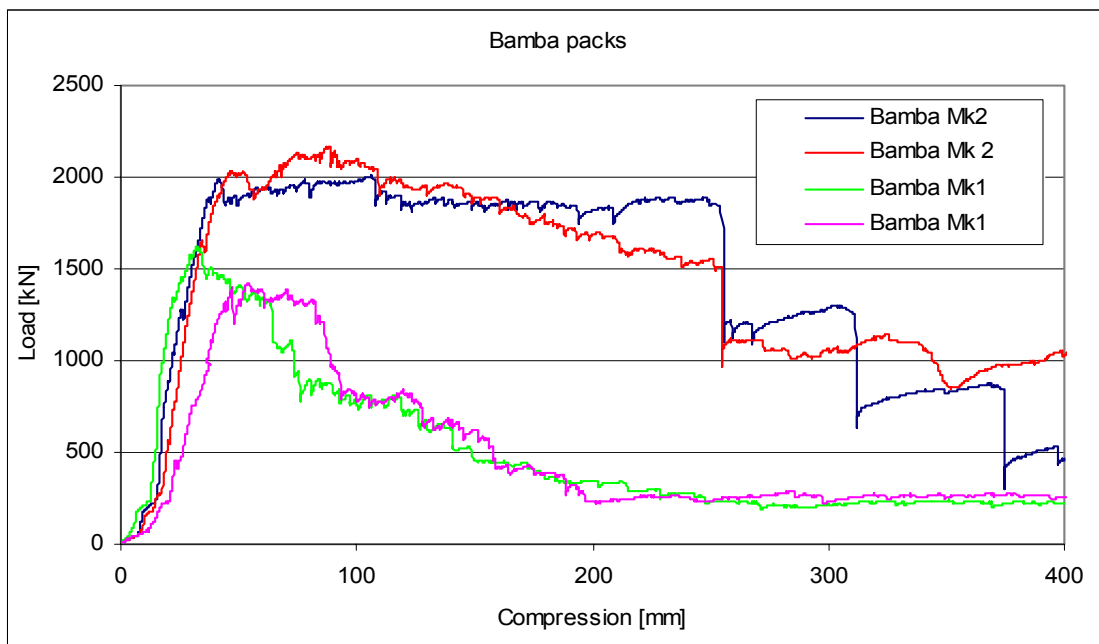


Figure 4.24 Results of compression tests conducted on two types of Bamba packs

Table 4-8 Bamba pack test results

Pack	Average. initial stiffness kN / mm	Average maximum		Average at 200 mm compression		Average. at 300 mm compression	
		Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>
Mk 1	38	1450	191	348	46	224	30
Mk2	46	2050	271	1807	238	1184	157

Comparing the test results of the Bamba pack Mk 1 and Mk 2 in the above results clearly illustrates the significant contribution made by lateral support. Replacing the five 6 mm diameter rings with the same number of 12 mm diameter rings on the otherwise identical packs, improved the stiffness by 20%, the peak load by 40% and the post failure load at 200 mm compression by a factor of five.

## 4.6 Cartwheel pack

The cartwheel pack is a 750 mm diameter grout pack reinforced by two or three 30 mm x 5 mm steel rings, cross-braced by two diameter length rods, set at right angles to each other (Figure 4.25 (c)). The inclusion of wire mesh (75 mm x 75 mm x 2 mm) will provide lateral support to the pack while curing and later, under load. Because of the rods, it is necessary to position the cross braced rings inside the bag which would otherwise have to be perforated to accommodate them. The mesh, which relies on bracing from the rings, must therefore also be installed inside the bag. To stop the bag from bulging PVC straps are tightened around the outer circumference, however this measure was found to be inadequate when pumping the bags which tended to bulge excessively, resulting in a much bigger pack than was originally intended. When tested, the outer, unsupported grout broke away in layers at relatively low axial loads and made no further contribution to the strength of the pack. The inner core, which was supported by only two rings, compared with five for other pack designs, broke up under relatively low axial loads. The result was a pack with lower load bearing capacity than an RSS pack of the same dimensions. A third ring was added during the second test to reduce the lateral support spacing but this measure merely increased the initial yielding prior to ring failure. Figure 4.25 (a) and (b) shows a cartwheel pack undergoing press testing with the results presented in Figure 4.26 and summarised in Table 4-9.

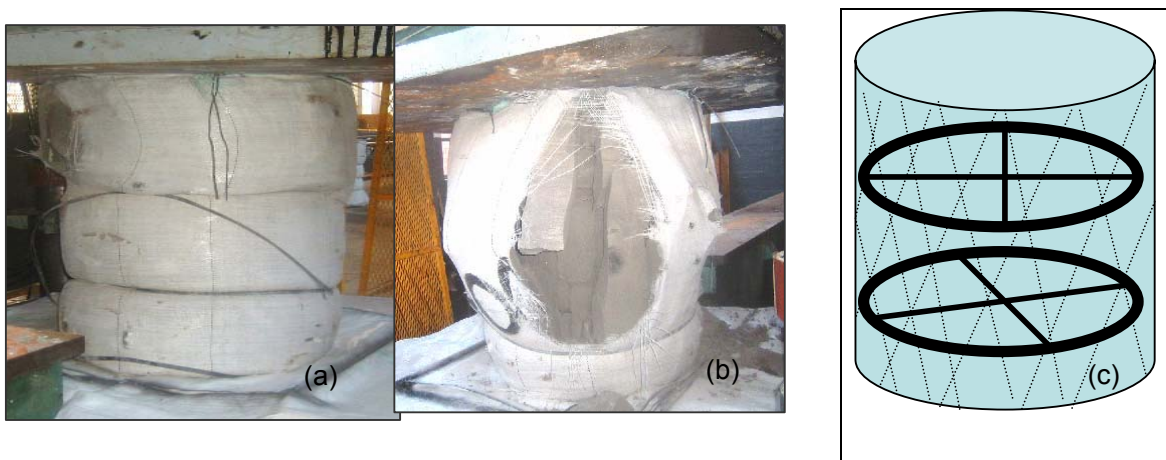


Figure 4.25 A cartwheel pack (a) after installation and (b) after 20% closure (c) the design

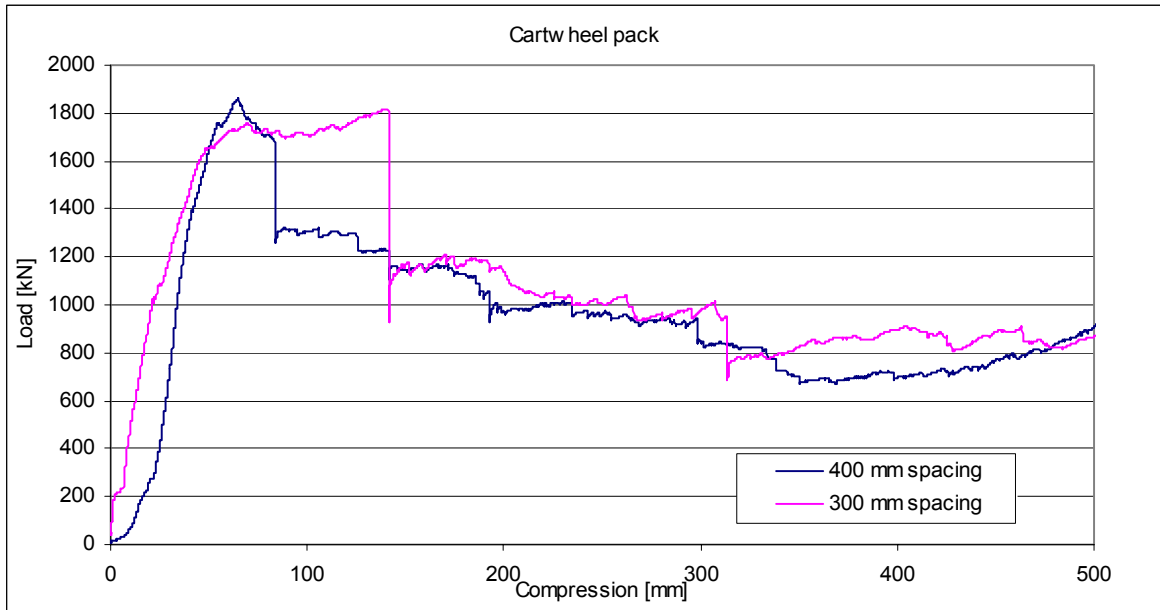


Figure 4.26 Press test results of two cartwheel packs, with two and three rings respectively

Table 4-9 750 mm diameter Cartwheel pack results

Initial stiffness kN / mm	Maximum		At 200 mm compression		At 300 mm compression	
	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>
31	1810	239	1041	138	850	112

## 4.7 Yielding packs

As mining is expected to go deeper in the future and stope spans are likely to increase with increasing mechanisation, it was considered necessary to test the yield characteristics of packs that are designed to maintain a high proportion of their initial load bearing capacity up to and beyond 30% closure. In addition, a new yielding pack concept was developed during the course of the test program that was considered worth testing. In this section, two types of yielding packs are investigated: The Cempack, which was developed by Reinforced Earth (Pty) Ltd in the 1980's and the more recently developed Yield pack, developed by Afrinet (Pty) Ltd. The effect of including mine poles in the Yield pack was also investigated.

### 4.7.1 Cempack

The original concept of yielding grout packs comes from the development of reinforced earth systems. These systems use internal reinforcing in the form of straps or fibres to provide lateral constraint, significantly improving the load-bearing capabilities of cohesionless materials.

The packs use internal reinforcement in the form of annealed 50 mm x 50 mm x 4 mm welded mesh distributed evenly in layers throughout the pack. The steel will strain by approximately 30% before failure (Dison and Blight, 1988). The annealed welded mesh gives lateral constraint throughout the pack and inhibits shear planes forming in the granular failed material.

The initial yield strength of the pack is determined principally by the strength of the cementitious material. To obtain an optimum yielding characteristic with reasonable stiffness, it is essential that the grout used in the Cempack should be relatively weak, typically with a UCS of 3 to 4 MPa. A high initial strength will adversely affect the residual yield strength and increase the difference between the peak and residual strengths.

The Cempack has been developed as a modular pack for ease of construction. The pack is built up using a number of “pillows”: permeable geotextile bags that contain the pumped grout. These are separated from each other by single layers of weld-mesh until the desired height is reached. Each layer of mesh is connected to the next using high tensile steel clips to ensure correct spacing, prevent bulging of the bags and improve stability of the pack (King and Jager, 1992). A photograph of a Cempack undergoing press testing is presented in Figure 4.27.



Figure 4.27 A Cempack being press tested

Figure 4.27 shows a shear plane forming diagonally across the front face of a Cempack which had eight layers of annealed weld-mesh included. The weld-mesh inhibited the packs disintegration and allowed it to maintain a high percentage of its peak load over an extended range of compression.

Both packs tested were made up of 3 MPa grout. Two configurations were tested, one using six layers of mesh between seven bags, and the other using eight layers of mesh between nine bags. The test results are presented in Figure 4.28 and are summarised in Table 4-10.

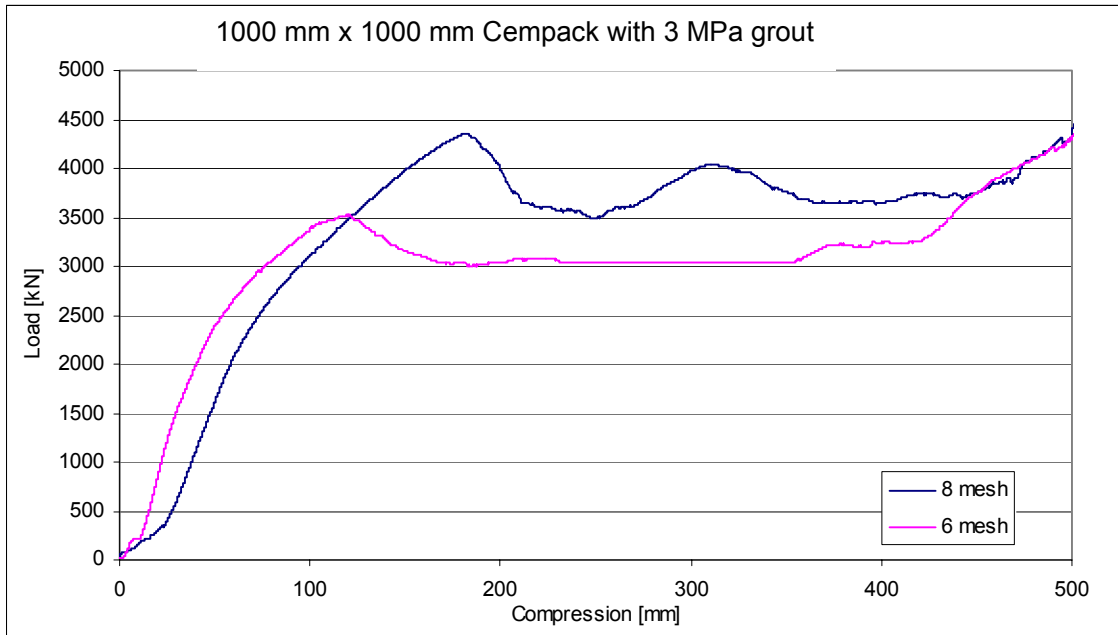


Figure 4.28 Results of Cempack press tests

Table 4-10 Results of 1000 mm x 1000 mm Cempacks

Reinforcement	Initial stiffness kN / mm	Average at Maximum		At 200 mm compression kN		At 300 mm compression	
		Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>
8 mesh layers	31	4314	479	3883	431	3970	441
6 mesh layers	45	3522	391	3040	337	3090	343

Three important characteristics of Cempacks are illustrated in Table 4-10:

- They are not as stiff as packs made from stronger grout.
- They maintained a high percentage of their peak load under extended compression.
- Both designs of Cempacks bettered the 340 kN/m<sup>2</sup> support resistance criterion set for this project, up to 500 mm compression.

#### 4.7.2 Yield packs

During the course of the test program a modification was made to the standard RSS pack to improve post-failure performance. Dubbed the Yield pack, it incorporates three extra 900 mm diameter rings suspended from the smaller 750 mm diameter rings by strings. These larger rings provide continued lateral constraint as the grout dilates and the inner rings begin to yield

and fail. Three stages of the construction of a Yield pack in a stope are shown in Figure 4.29. (a) the outer shuttering is deployed against the hangingwall in the front row of support with timber poles (b) the polypropylene bag is inserted into the shuttering and then pumped with grout (through a 30 mm diameter hose pipe) and (c) the filled pack forms part of the stopes local support (having withstood a blast).

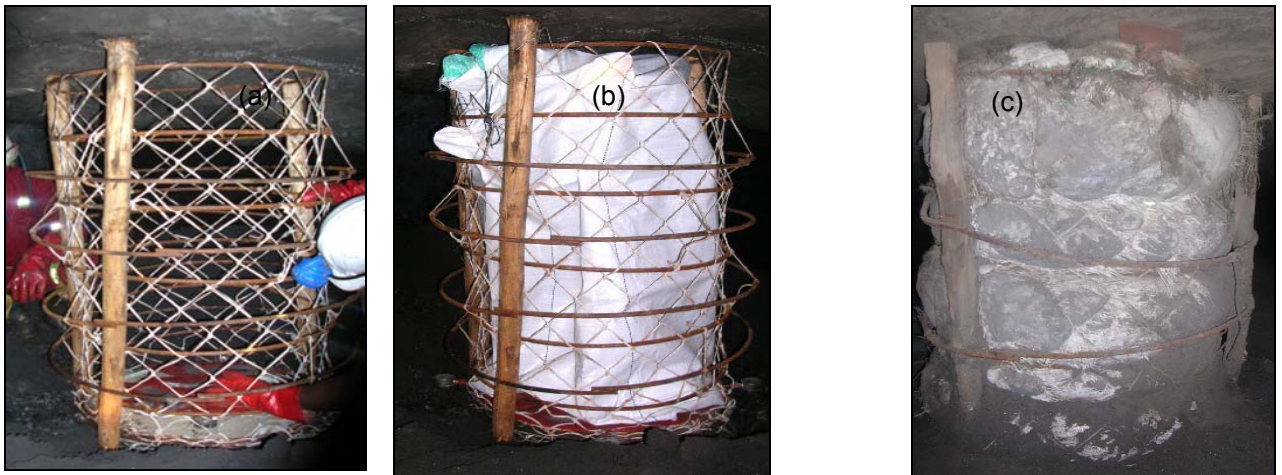


Figure 4.29 (a) and (b) A Yield pack being constructed in a stope and (c) after a face blast

The results of the laboratory tests conducted on a 750 mm diameter Yield pack, made with 3 MPa grout, are presented in Figure 4.30 and are summarised in Table 4-11. The yield packs tested, remained within 34%, 25% and 37% respectively of peak load over a 33% compression range.

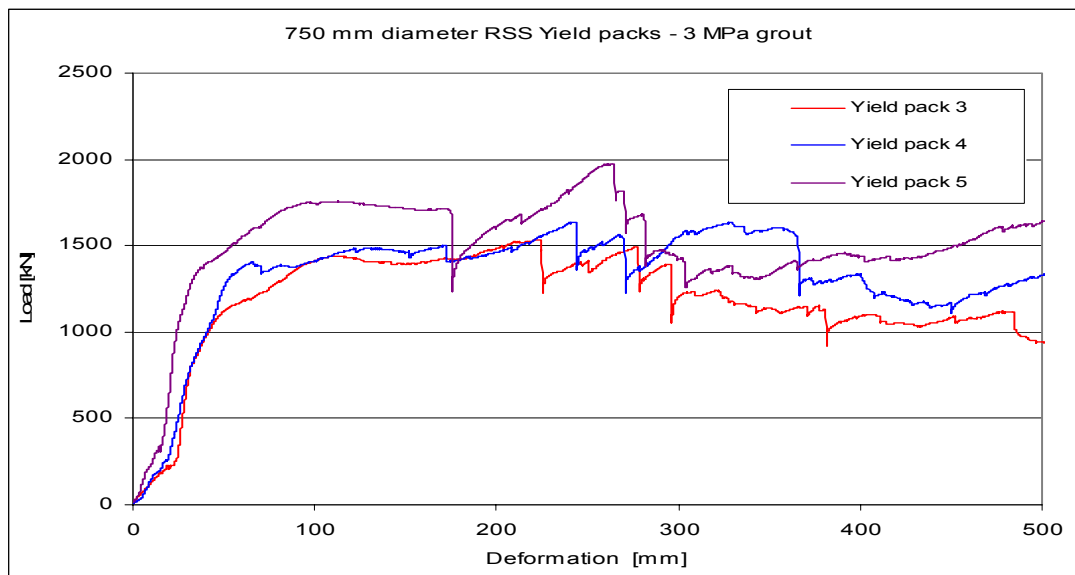


Figure 4.30 Press test results of 750 mm diameter Yield packs

Table 4-11 Results of 750 mm diameter Yield pack press tests

Average initial stiffness kN/mm	Average. maximum		Average at 200 mm compression		Average at 300 mm compression	
	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>
31	1650	218	1580	209	1400	185

To illustrate the enhanced yielding performance of Yield packs, a standard RSS pack and a Yield pack were cast and tested together using 3 MPa grout. The results are presented in Figure 4.31. The Figure shows an improved performance for the Yield pack after approximately 50 mm compression. It also shows that the Yield pack maintains a high percentage of its initial strength over an extended compression range.

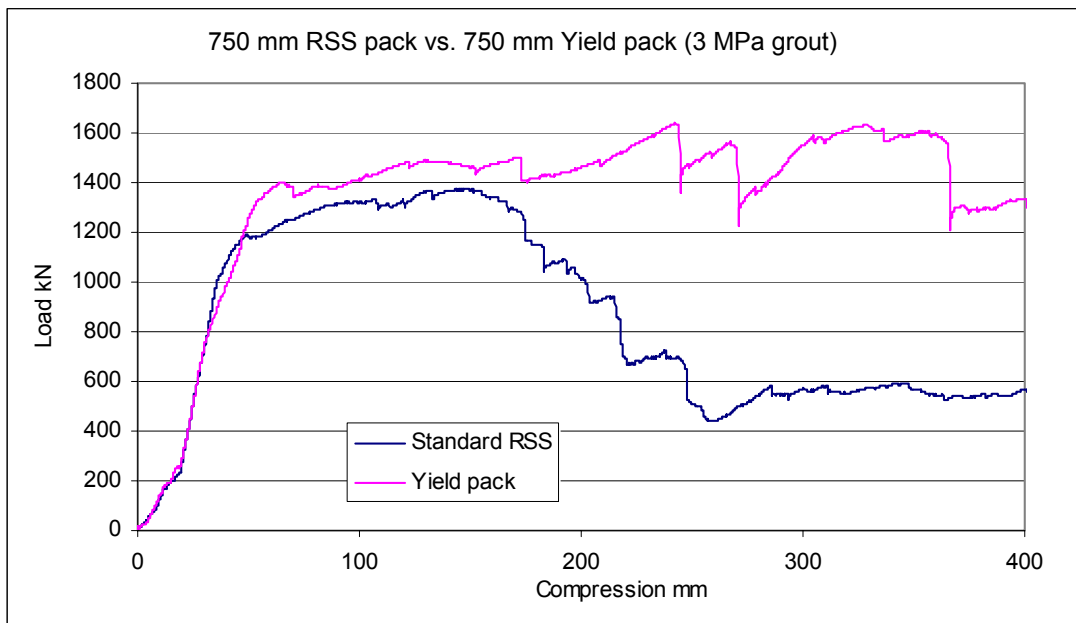


Figure 4.31 Results of press tests carried out on a 750 mm diameter Yield pack and a standard RSS pack, both with 3 MPa grout

### 4.7.3 Yield packs with mine poles

The effect of incorporating mine poles in a Yield pack was tested by casting two Yield packs each with two poles, placed within their rings, and two without poles. In addition, a standard RSS pack was included in the suite of tests so that the results could be compared against a known benchmark. All packs were cast from the same 10 MPa grout batch. Figure 4.32

shows the testing configuration. Load compression curves are provided in Figure 4.33 and summarised in Table 4-12.



Figure 4.32 A Yield pack with two mine poles in the press

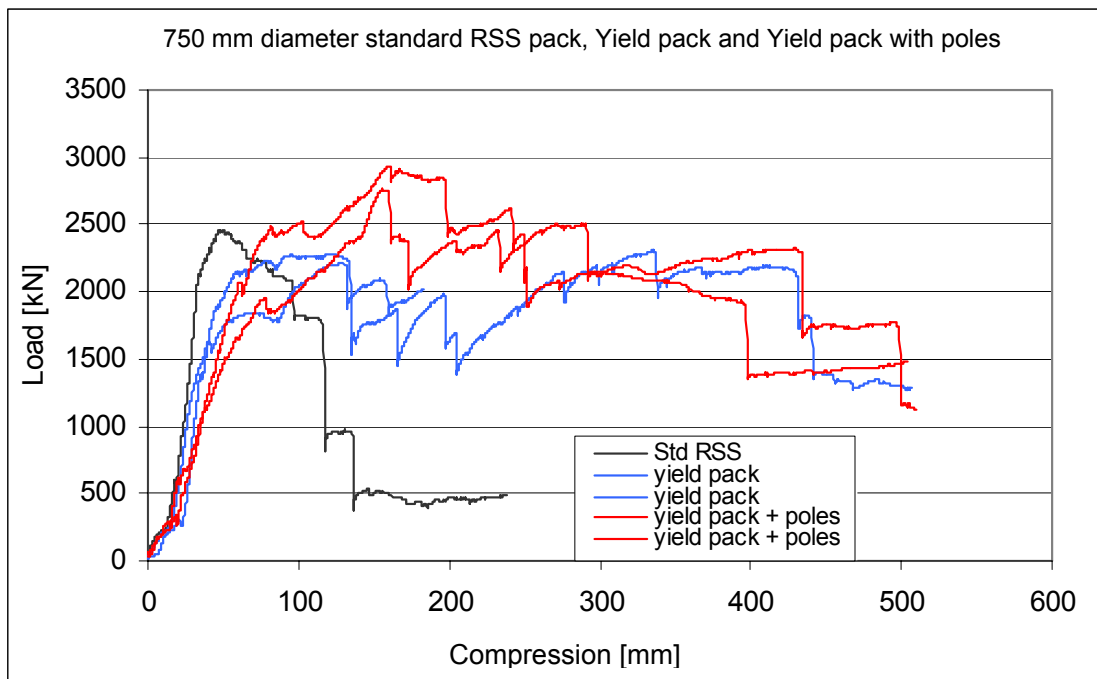


Figure 4.33 Press test results comparing 750 mm diameter standard RSS pack, Yield packs and Yield packs with poles

Table 4-12 750 mm diameter Yield packs and Yield packs with mine poles

Pack type	Average initial stiffness kN / mm	Average maximum		Average at 200 mm compression		Average at 300 mm compression	
		Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>
Yield pack	53	2250	297	1314	156	2250	297
Yield pack + 2 poles	38	2850	377	2425	321	2250	297

The incorporation of mine poles reduced the initial stiffness by 40% but enhanced the peak strength of the pack by 26%. This result was surprising, as it was anticipated that mine poles would enhance the stiffness of the packs. Between 100 mm and 300 mm compression (8.3% - 25% strain), the packs with mine poles recorded higher loads than those without. The implication is that the poles have better load bearing capacity in this range than does the grout they displaced. Beyond 300 mm compression, the performance of packs with and without mine poles was similar.

## 4.8 Summary of test results

The average support resistance using 2 m skin-to-skin support spacing, at 200 mm and 300 mm compression is similar for all RSS packs between 900 mm and 550 mm diameter, with the smaller 550 mm pack giving the best results in this range as illustrated in Figure 4.34. This counter intuitive result is a result of the higher steel to grout ratio in the smaller packs, which boosts their relative load bearing capacity. Of the five sizes of RSS packs tested, only the 1200 mm diameter standard RSS packs exceeded the minimum support resistance (340 kN/m<sup>2</sup>) requirement set for this project up to 300 mm compression. In addition it should be noted that during the course of the project it was agreed by mine personnel and researchers that 1400 mm diameter packs would be too bulky, difficult to install and impractical to implement in a mine. These packs also did not meet the criterion beyond 250 mm compression.

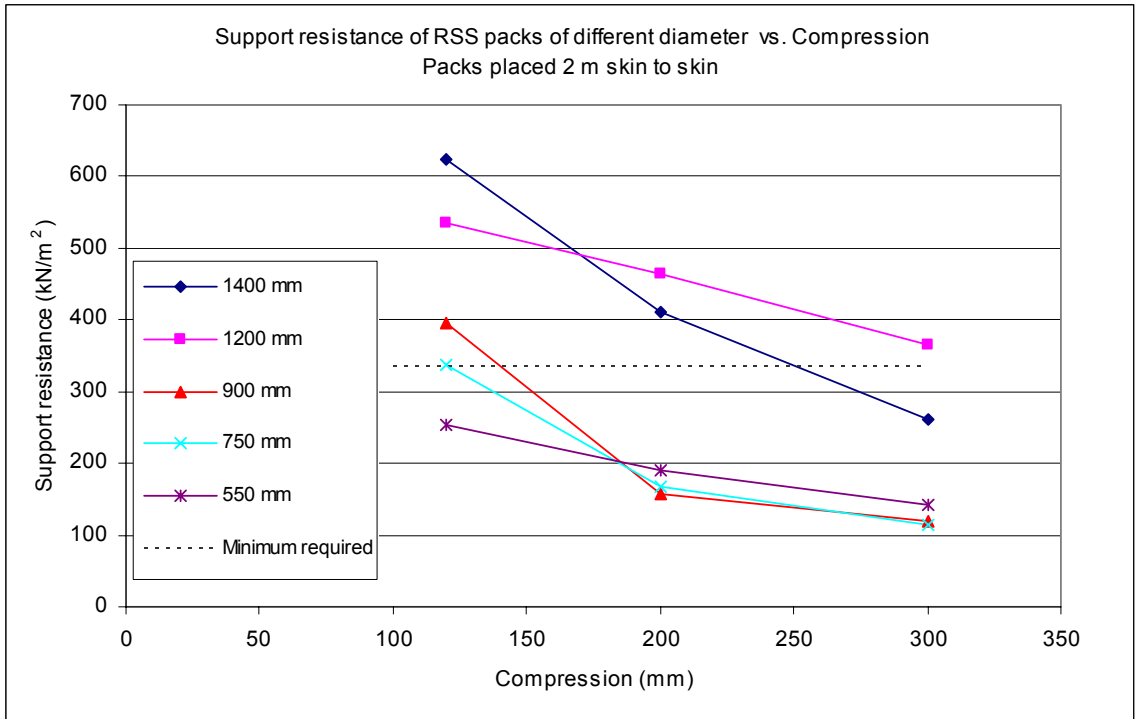


Figure 4.34 Support resistance of systems based on different diameter RSS packs placed 2 m skin-to-skin

A summary of the press test results is presented in Table 4-13. In terms of support resistance the table shows that the large RSS packs (1200 mm and 1400 mm diameter) provide the highest initial support resistance for a “2 m skin-to-skin” support pattern, however, beyond 300 mm closure (25 % strain) they are surpassed by the Cempacks, which notably, use a weaker grout.

All packs that exceeded 340 kN/m<sup>2</sup> support resistance are highlighted in Table 4-13.

Table 4-13 A summary of press test results

Type	Diameter / dimension mm ( <i>number tested</i> )	Av. Initial stiffness kN/mm	Peak load kN.		Peak strength kPa.		Average support resistance placed 2 m skin to skin kN/m <sup>2</sup>		
			Max	Ave	Max	Ave	Max	200 mm closure	300 mm closure
RSS	1400 (4)	135	8400	7200	5456	4400	623	410	261
	1200 (3)	106	5484	5400	4849	4775	536	463	365
	900 (4)	92	3620	3324	5660	5500	395	156	119
	750 (9)	62	3000	2550	6.79	6050	337	168	113
	550 (8)	41	2100	1656	8840	7100	254	189	143
Yield pack (3 MPa grout)	750 (3)	31	1950	1650	4413	3734	284	163	142
Yield pack (10 MPa grout)	750 (2)	53	2250	2240	5092	5070	297	156	297
Yield pack (10 MPa +2 mine poles)	750 (2)	38	2950	2750	6677	6216	377	321	197
Tau pack	620 x 620 (1)	22	1705	1705	3859	3859	203	150	90
	900 x 900 (2)	51	3000	2750	3703	3395	287	170	135
Cart wheel (2 rings)	810 (1)	30	1820		4100	4.1	239	137	112
Cart wheel (3 rings)	810 (1)	31	1850		4200	4200	242	164	123
Cempack (6 mesh)	1000 x 1000 (1)	45	3500		3500		391	337	343
Cempack (8 mesh)	1000 x 1000 (1)	31	4400		4400		471	431	441
Mesh Pack	750 (4)	56	2320	2250	5251	5092	293	186	115
Bamba pack Mk 1	750 (2)	38	1610	1550	3644	3.4	191	46	30
Bamba pack Mk 2	750 (2)	46	2060	2050	4662	4640	271	238	151

Figure 4.34 illustrates that the large diameter (1400 mm and 1200 mm) packs provide a higher support resistance for a 2 m skin-to-skin support pattern than do smaller diameter packs, however, they require considerably more grout to do so as illustrated in Figure 4.35. For example, 1200 mm diameter packs will provide between 2.1 and 2.5 times the support resistance offered by 550 mm diameter packs with the same skin-to-skin spacing, but will require 4.6 times as much grout to do so. Using smaller packs (with reduced spacing) to create the same support resistance would therefore reduce the amount of grout required.

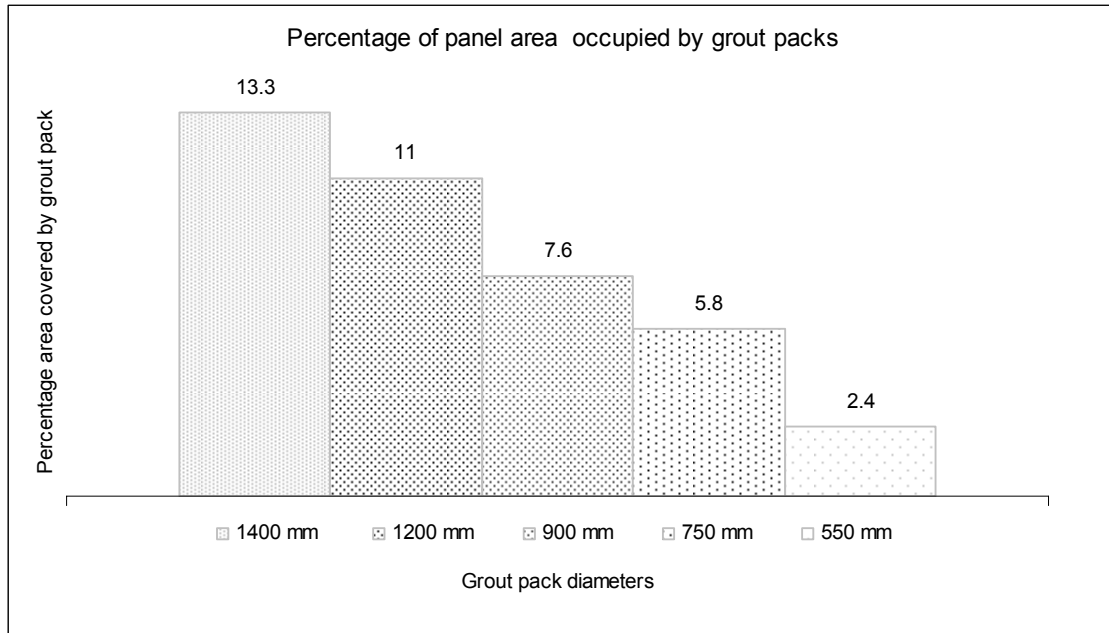


Figure 4.35 Percentage surface area of a panel occupied by grout packs when using a 2 m skin-to-skin support pattern

## 4.9 Summary

The press testing programme has highlighted important design characteristics of grout packs. These include the effects that size, height, reinforcement, and strength of grout has on the stiffness, load bearing capacity and yieldability of grout packs. Before conducting *in-situ* pack testing, suitable loadcells had to be sourced. Various commercially available loadcells were therefore tested with the packs to determine their suitability for underground use. Aspects of the *in situ* test programme are dealt with in the next chapter.

## 5 Investigation of load cells for underground monitoring

In the previous chapter, the results of laboratory tests, conducted on 28 day old grout packs were presented. An important aspect of this investigation has been to establish a relationship between the performance of packs tested in a laboratory and those placed in a working stope underground. In the past, monitoring of grout packs had poor success due to failure of either the load cells or the pressure monitoring system. It was therefore decided to investigate the performance and reliability of different designs of flat jack type load cells as part of the press testing programme and correlate the cell pressures against the press loads. The results would be important in the selection of load cells to be used in the underground investigation. This chapter looks at the performance of these loadcells and discusses the design chosen for the underground investigation.

### 5.1 Loadcell testing

Initially, flat circular load cells of 170 mm diameter and 370 mm x 370 mm square cells were evaluated. The two loadcell types are shown in Figure 5.1 both with 6 MPa gauges attached via a reinforced steel tube.

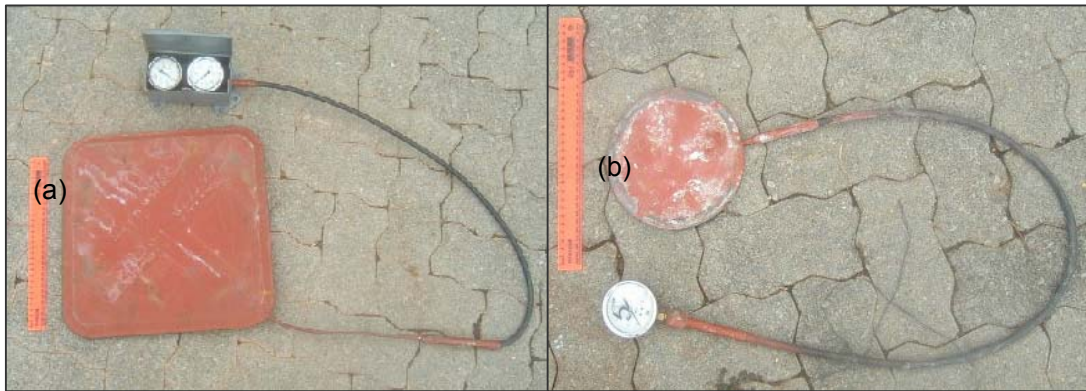


Figure 5.1 (a) 370 mm square flatjack and (b) 170 mm diameter flatjack loadcells

The loadcells were incorporated into the grout of RSS packs, ranging in diameter from 1400 mm to 550 mm as they cured. To facilitate the placement of the smaller 170 mm diameter load cells, a sleeved pocket was incorporated into the fabric of bags at about half height. As the bags were being filled, the process was temporarily stopped so that load cells could be placed in the slurry. The loadcells were positioned near the centre of the packs and at various positions along the pack radius so that the stress distribution across the pack area could be determined as the packs were compressed. Once they were correctly positioned in the slurry, filling was resumed.

The larger square 370 mm x 370 mm load cells were tested by placing them at the lower platen / bag interface before press loading commenced. The positioning of the loadcells is illustrated in Figure 5.2.

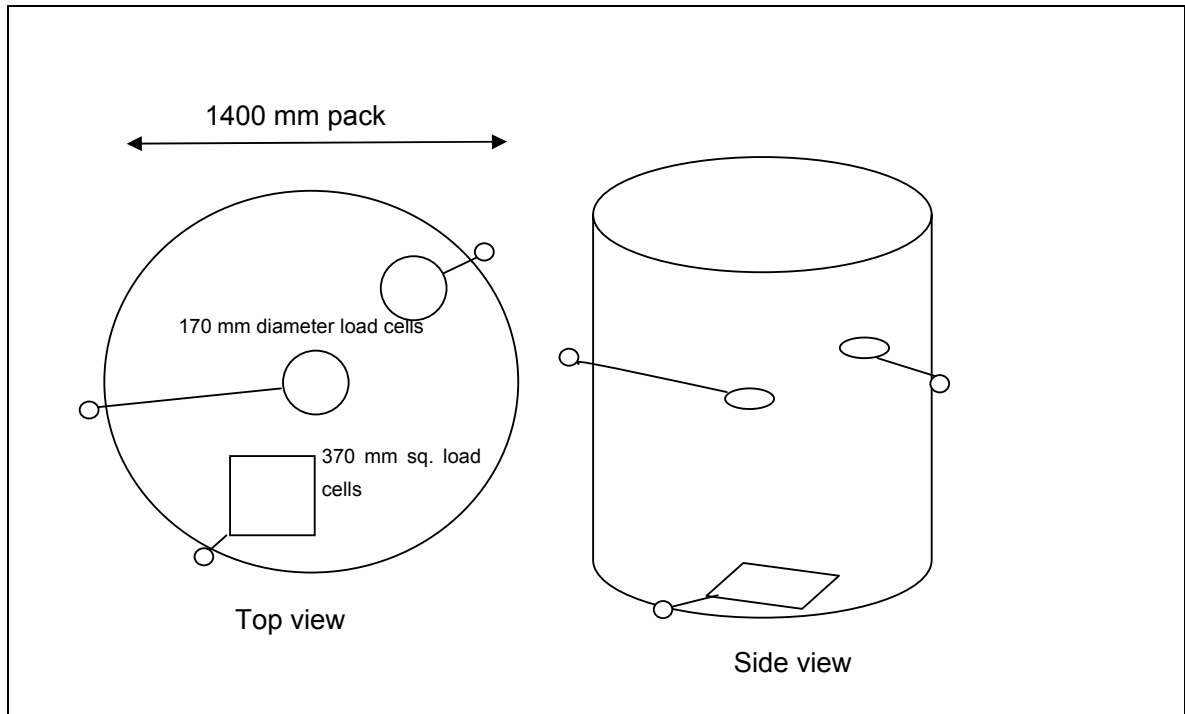


Figure 5.2 Load cell positions in the packs during testing

The loads calculated from the load cell readings were generally lower than the loads recorded from the press. An additional curve is therefore presented with some of the results in Figures 5.3, to 5.6. This curve represents the readings taken from the load cells multiplied by a factor, which normalised the recorded pressures with respect to the peak load measured by the press. The factor was determined graphically as follows; the points on the loadcell graph were all multiplied by a common factor until its peak load coincided with that of the press. For example, in Figure 5.3 the load cell result was multiplied by a factor of 1.549, thus bringing the loadcell graph closer to press graph.

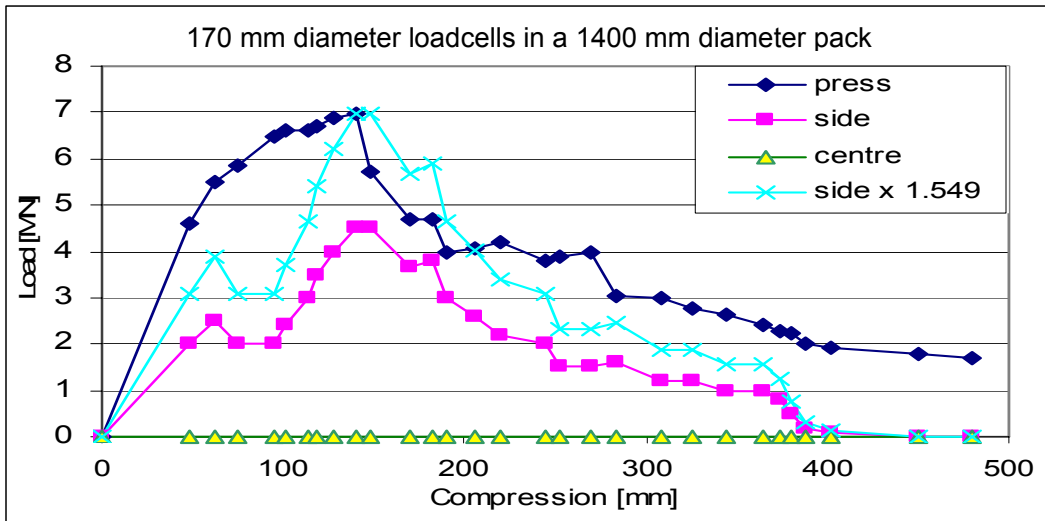


Figure 5.3 Press and 170 mm diameter load cell readings with a scaled curve

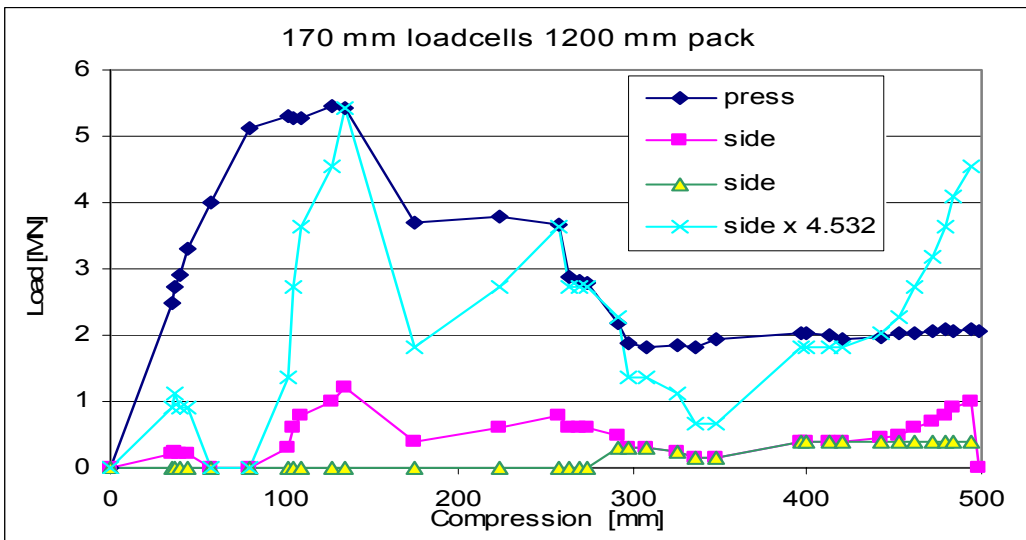


Figure 5.4 Press and two 170 mm diameter load cells readings with a scaled curve

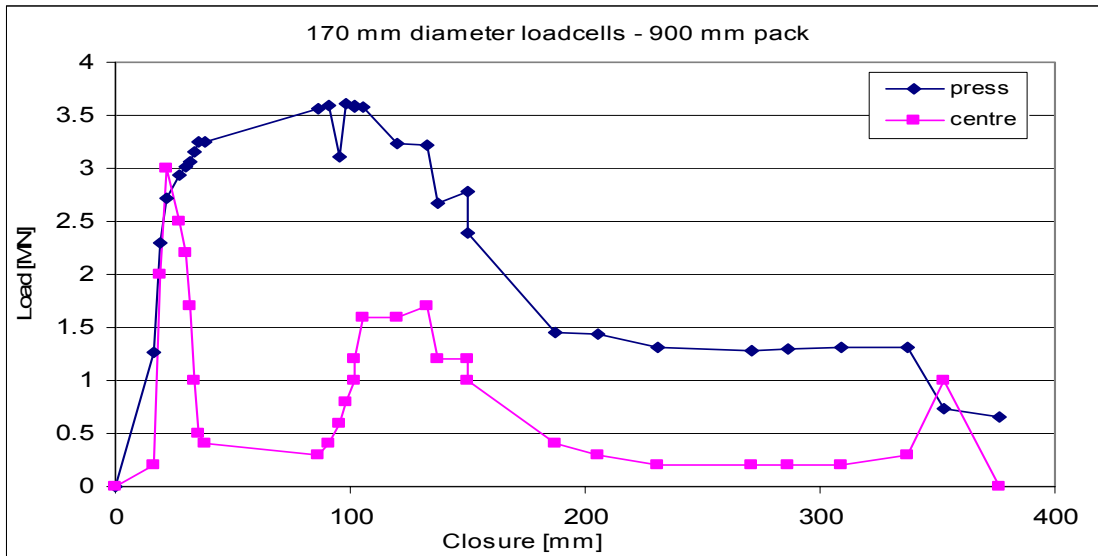


Figure 5.5 Press and a 170 mm diameter load cell reading

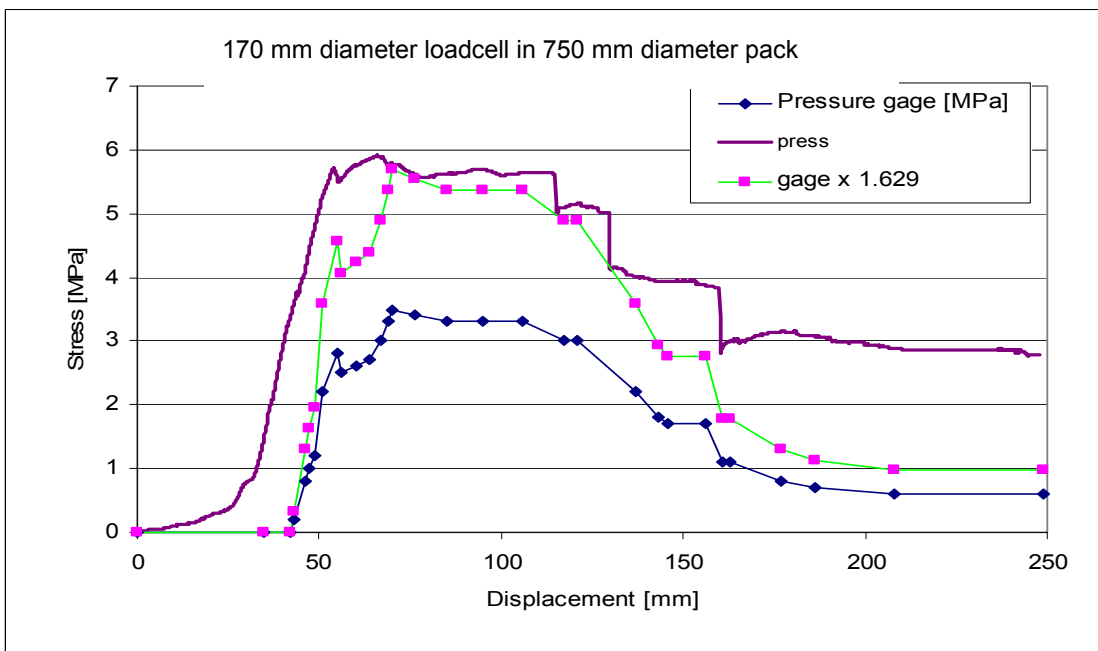


Figure 5.6 Press and a 170 mm diameter load cell readings with a scaled curve

The results from loadcells placed within the Cempack are presented in Figure 5.7. The press test conducted on the instrumented Cempack was temporarily interrupted due to mechanical interference, adversely affecting the readings taken from the loadcells (evident in Figure 5.7).

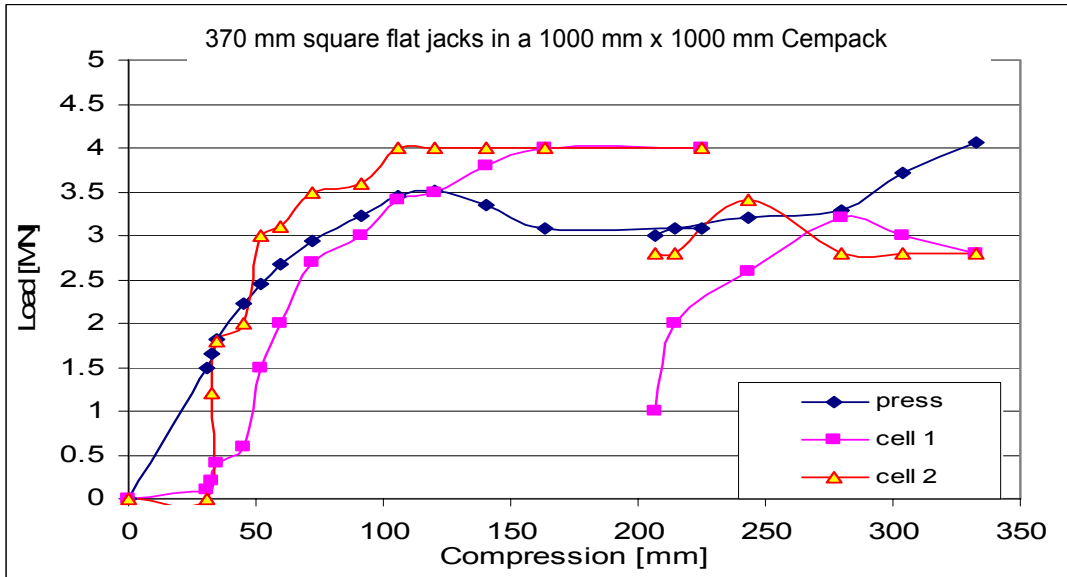


Figure 5.7 Press and 370 mm square load cell readings from a square Cempack

Figures 5.8 and 5.9 show the press test results of a 750 mm RSS pack with a 370 mm square flatjack installed beneath them. These results, when multiplied by a scaling factor correlated better with the loads recorded by the press than did the smaller load cells.

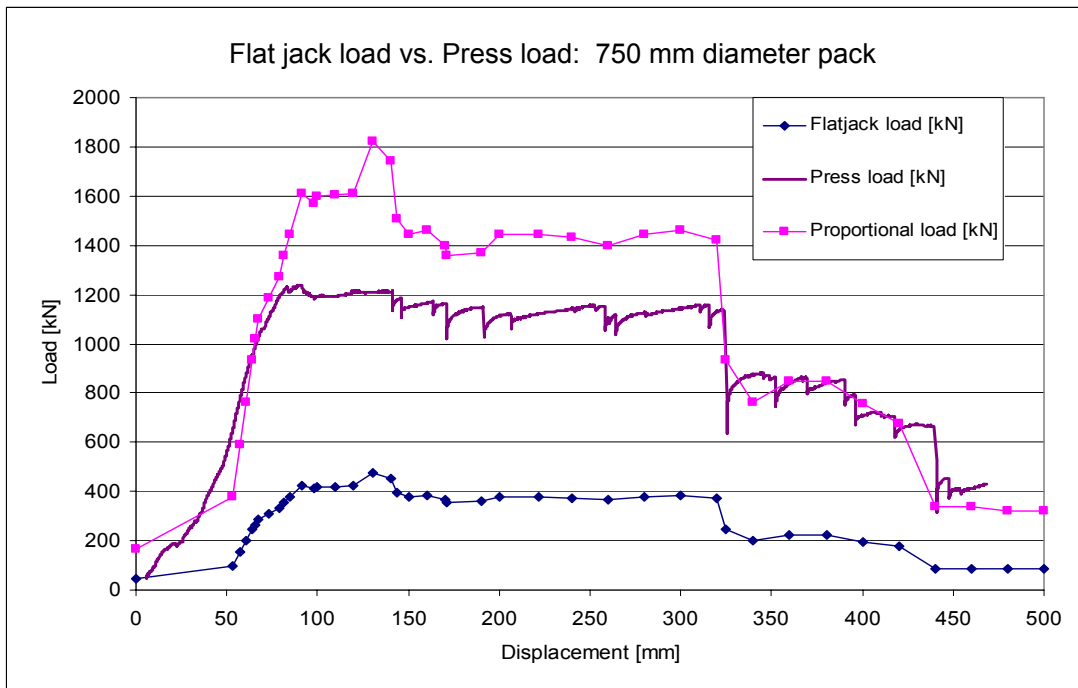


Figure 5.8 Press and 370 mm loadcell readings with a proportional load (scaled) curve

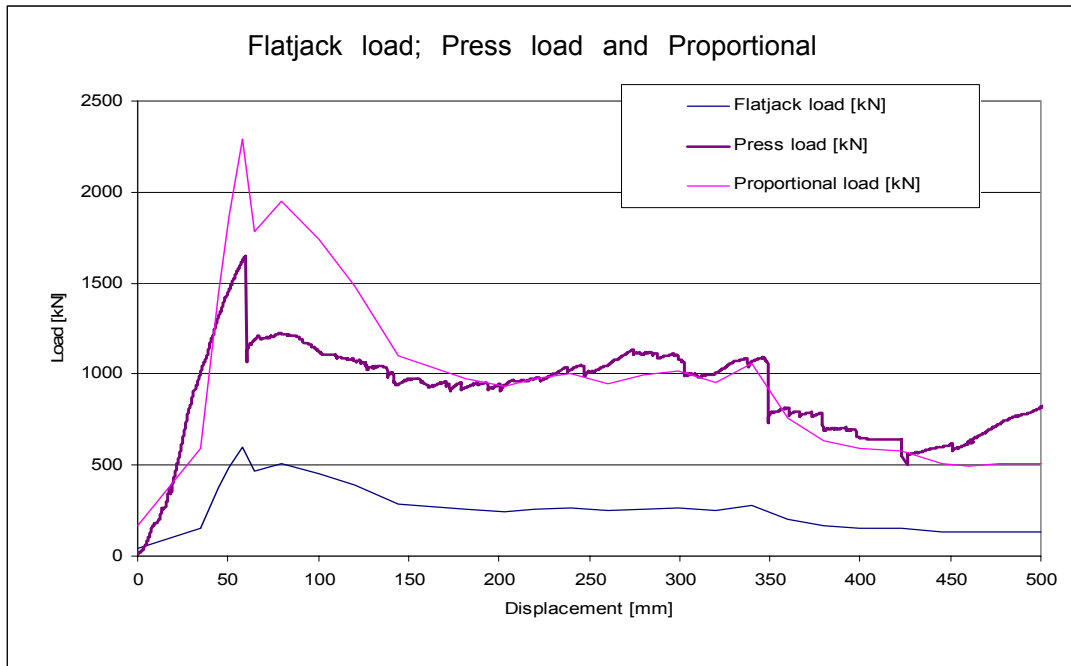


Figure 5.9 370mm square flatjack load, Press load, and a proportional load (scaled) curve

The poor correlation between the press loads and those recorded by the small loadcells brings into question the reliability of these loadcells and the even distribution of load across the packs. In order to verify the accuracy of hydraulic stress meters, Piper in 1993 buried four small hydraulic stress meters in backfill inside a steel tube (i.e. oedometer) as shown in Figure 5.10. The backfill inside the oedometer was loaded by the press, which had the same piston diameter as the oedometer. Since the area of the piston and load applied on the backfill were known, the amount of stress applied to the backfill could be determined. The test results are given in Figure 5.11. The agreement between the applied stress and the stress measured by the stress meters inside the backfill was very good (Piper et al 1993). However, despite Pipers endorsement of flatjack loadcells readings, a difference exists between the manner in which backfill and grout packs yield under load, making the use of backfill instrumentation inappropriate for measuring stress in grout packs. The early peripheral slabbing and later pulverisation of the inner core of a pack under load result in uneven load distribution. This makes small load cell readings unrepresentative of the load across the entire pack. Larger load cells, which monitored a larger portion of the pack area, gave a better representation of the average pack load as presented in Figures 5.8 and 5.9.

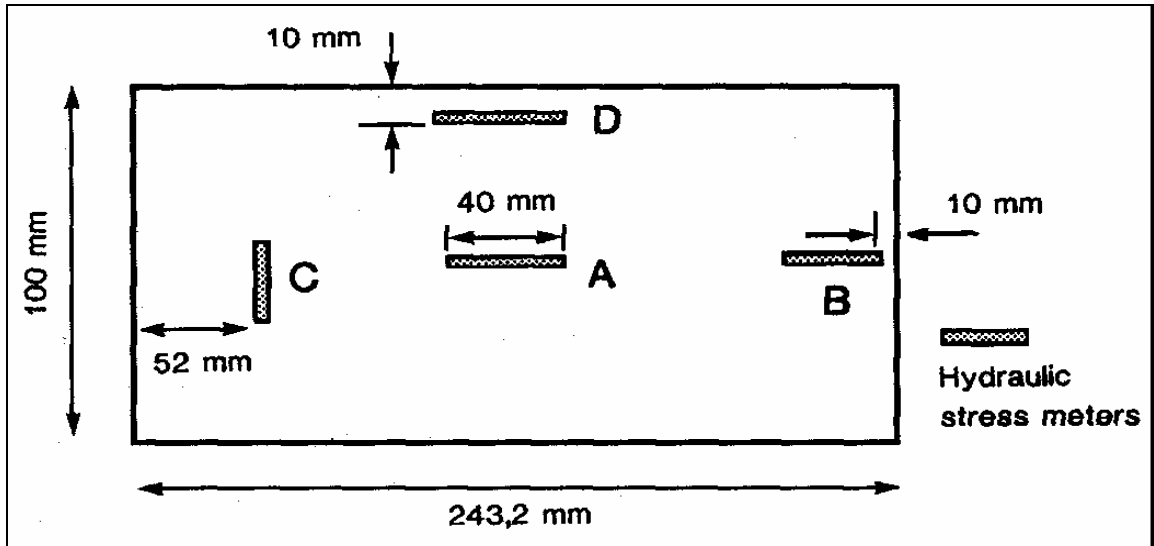


Figure 5.10 Position of small hydraulic stress meters in an oedometer (Piper 1993)

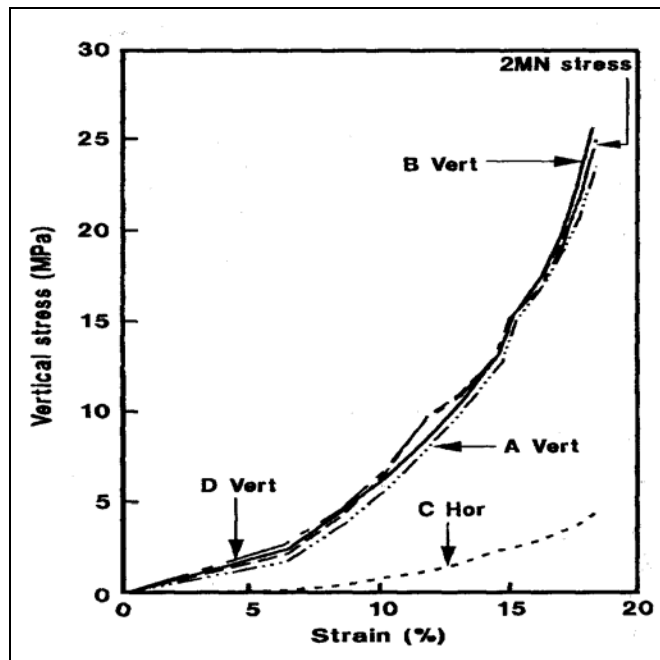


Figure 5.11 Results of oedometer stress readings (Piper, 1993)

In order to measure the load across the entire cross sectional area of the pack during testing, quadrant type load cells were made up to the same cross sectional area as the pack and were tested as shown in Figures 5.12 and 5.13. The results are presented in Figure 5.14.



Figure 5.12 Quadrant loadcells consisting of four separate units      Figure 5.13 Undergoing calibration testing

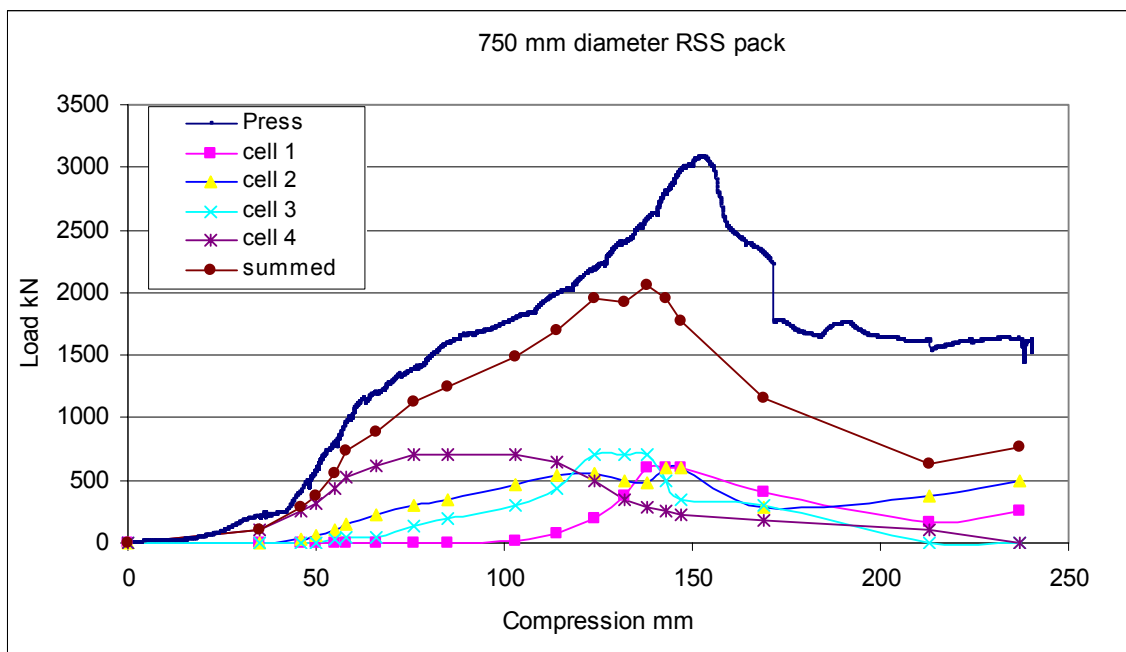


Figure 5.14 Load compression curve for quadrant type load cells

Two of the loadcells failed during the compression test as shown in Figure 5.15 (a). This resulted in a poor correlation of the summed curve, compared to the press load beyond 120 mm compression. A second problem associated with flat jack type loadcells is illustrated in Figure 5.15 (b) where evidence of a “flat spot” on the loadcell was noticed during post testing evaluation. Once formed, a flat spot would allow load to be commuted through the loadcell without further influencing the reading on the pressure gauge.

In underground testing it would be impossible to establish whether or not flat spots had formed on loadcells installed in or beneath a pack, thereby making their readings unreliable. It is recommended therefore that an investigation be undertaken into the causes of failure of the type of loadcells used underground and that a more reliable load measuring system be developed prior to load testing in the future.



Figure 5.15 (a) Damaged loadcells after the press test (b) Side view of a damaged cell

The packs, which were cast on grid metal took up the grid pattern on their lower surface during curing and subsequently stamped that pattern on the loadcells surfaces, as shown in Figure 5.15(a) Also evident are the imprints of the mine poles that were incorporated in the pack.

Prior to the underground testing program, the issue of reliability was addressed by the manufacturer in areas identified as weak spots, however, during the investigation, some loadcells did fail.

The loads borne by the pack were determined as follows: The dial gauges (evident in Figure 5.12) measured the pressure in the load cells and gave a reading in kilo-Pascals (kPa), this reading was converted to kilo-Newtons (kN) using a conversion table supplied by the manufacturer.

## 5.2 Selecting load cells for underground use

The quadrant loadcells were selected for the underground monitoring of pack loads, despite the limitations highlighted previously in this chapter, as they best monitored the entire cross sectional area of the pack and their readings were closest to those of the press prior to their failure.

The results of the calibration test conducted on the quadrant loadcells showed uneven load distribution across the four quadrants of the packs. Among other factors this is the result of incorporating poles within the rings of the pack and point loading, resulting in uneven loading surface between the platens. The summed load recorded by the loadcells underestimated the press reading by less than 20% before they began to rupture. The underground readings were accordingly multiplied by a factor of 1.2, to bring them in line with the calibrated press readings.

### **5.3 Summary**

The results of three designs of flatjack type loadcells, incorporated into or placed beneath the packs during testing, have been presented in this chapter. All three designs gave pressure readings that, when converted to load, were found to underestimate the load on the packs. The reliability of the loadcells was also found to be an issue that should be addressed before additional load monitoring of packs is considered.

The underground testing of grout packs was conducted using the quadrant type loadcells. The results of the underground investigation are presented in the next chapter.

## 6 Underground tests

Subsequent to completing the press tests and the selection of a suitable design of loadcell, a few grout pack designs were chosen for underground tests. This chapter briefly discusses the geological conditions present at sites where the *in situ* packs were tested and explains how the packs were instrumented. The results are presented graphically and are compared with results from a typical press test. A summary table is presented with each set of results.

### 6.1 Underground sites

Suitable sites for *in situ* testing were found at Union Section's Spud shaft and at Lonmin's Rowland shaft. The panels in which the packs were installed were between 1.2 m and 1.6 m high at the time of placing the packs. The packs chosen for testing were the 550 mm and 750 mm diameter RSS packs and the 750 mm diameter Yield pack. All packs were installed as front row support in production panels and all were monitored for several weeks as the face advanced away from them. Due to difficulties associated with mining in faulted ground, the test panel at Spud shaft was stopped before closure had exceeded 300 mm thus limiting the extent of post failure behaviour information obtained from this site.

The two stopes mentioned above were chosen because they were considered to have potentially high closure rates. This was considered desirable for the purpose of pack monitoring as it would result in greater pack deformation and would potentially reduce the monitoring time required. 13-24-4S panel on Union Section's Spud shaft UG2 reef was about to begin mining beneath a Merensky remnant, 18 m above it. The resulting high abutment stress combined with a laminated and highly jointed hangingwall were likely to produce high closure rates and, upon monitoring, the stope was found to close at 10 mm to 15 mm per blast in the face area. A plan view of the panel is presented in Figure 6.1 with photographs of the hangingwall presented in Figures 6.2 and 6.3. Packs were installed as front row support within three metres of the face and were monitored at least once a week as the face advanced away from them. Four RSS packs of 550 mm diameter and eight of 750 mm diameter were installed at this site.

The position of the pack installations on the UG2 horizon, (beneath the remnant) are shown in plan view in Figure 6.1.

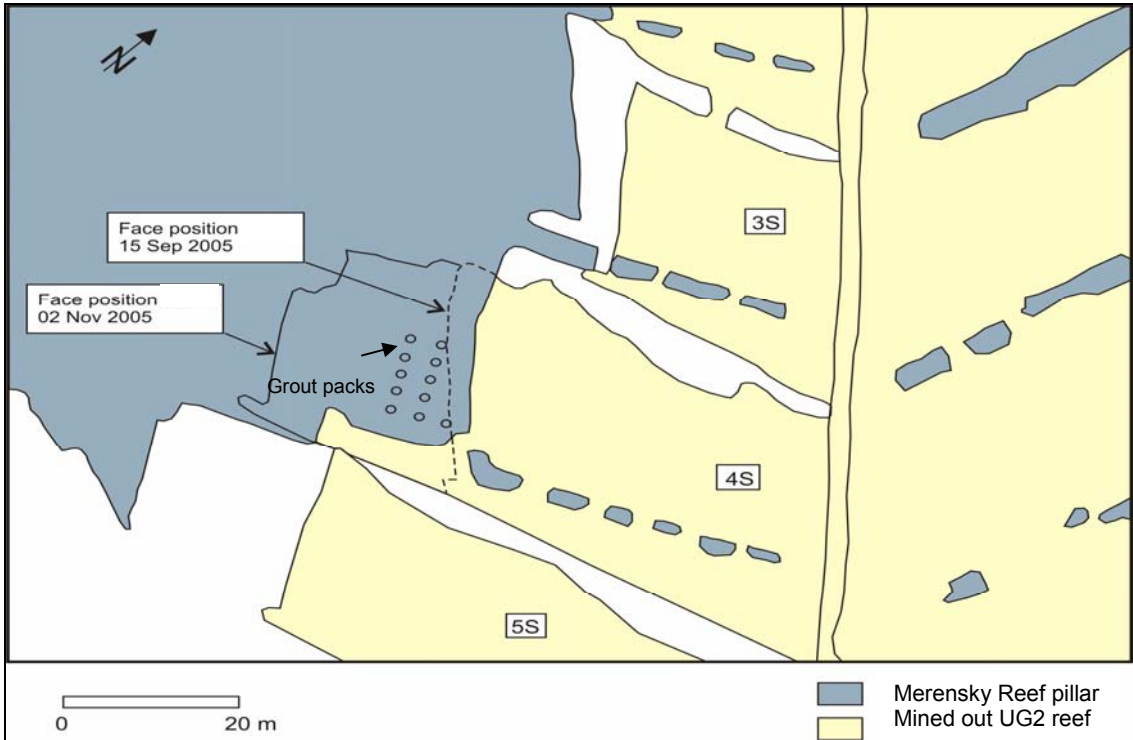


Figure 6.1 Union Section's 13-24 4S panel advancing beneath a Merensky remnant.



Figure 6.2 Low angle fractures throughout the panel



Figure 6.3 A face parallel shear zone in the hangingwall

Due to the early closure of the site at Spud shaft, a second site was required to complete the range of tests. At this time a site became available at Lonmin's Rowland shaft, which was to form part of the mechanised mining area on the mine, with associated wider mining spans. High closure rates were anticipated, however during the course of the test program, low closure rates were prevalent, typically ranging between two and five mm per blast. 750 mm diameter Yield packs were installed at this site. However, due to the low closure rates in this area, the packs did not yield laterally sufficiently for the larger diameter rings to take up any significant load. The results from these packs were therefore recorded as standard RSS packs. A plan view of the panel can be found in Figure 6.4 and a photograph depicting the stable hanging wall conditions is presented in Figure 6.5.

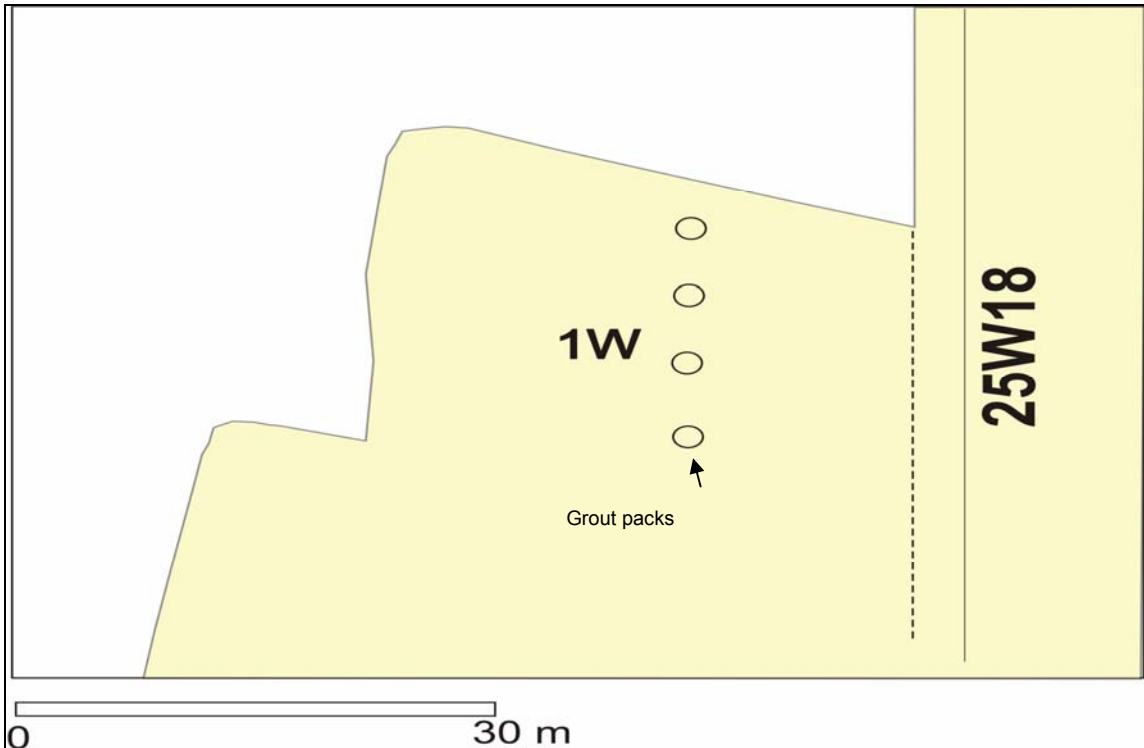


Figure 6.4 Rowland shaft's 25W18-1W UG2 panel



Figure 6.5 Stable hangingwall conditions prevalent in Rowland shaft 25W18-1W panel

For each installation, a base was made up using a layer of building sand at least 100 mm thick. The loadcells were embedded in this sand layer, thus protecting them from the unevenness of the footwall and possible puncturing under load. The packs were then constructed on top of the load cells as illustrated in Figure 6.6. Pegs were installed in the hangingwall and footwall near the packs so that closure could be monitored.

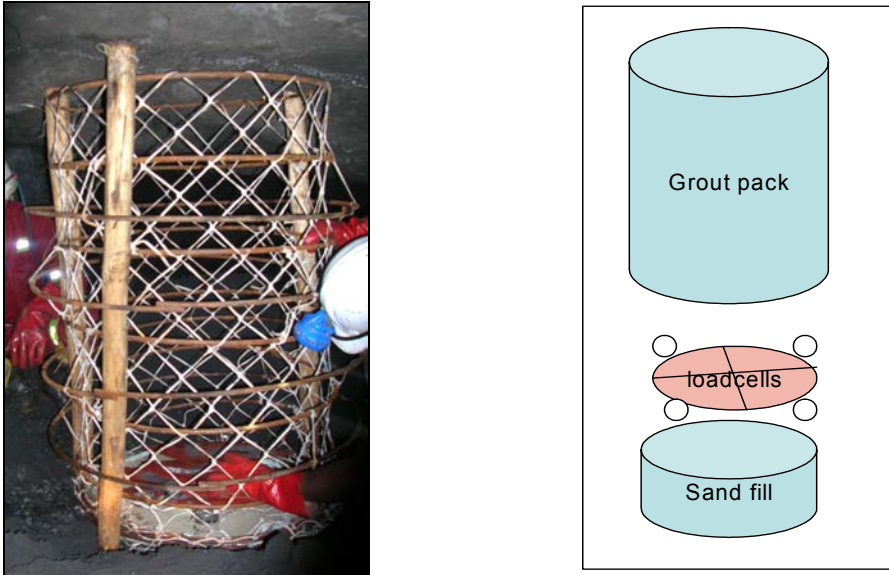


Figure 6.6 Quadrant loadcells on a layer of building sand beneath a future pack.

Before the initial blast on the newly installed front row of packs occurred, the loadcell pressure gauges, which protruded from beneath the packs, were buried in sand to protect them from fly rock. Despite these precautions, only eight of the eleven 750 mm diameter packs and three of the four 550 mm diameter packs installed at the two underground sites produced useable results, the remaining instrumented packs had one or more damaged loadcells soon after installation as a result of mining operations, and their results were unusable.

Loadcell and closure readings were taken at least once a week and observations were made on the behaviour of the packs, for the entire 13 week duration of each test program.

### 6.1.1 Underground test results

The results from the two underground sites are presented together in this section. The higher closure rates and lower grout strength used at Spud shaft resulted in the packs from the two sites having different stiffnesses and load bearing capacities.

#### 750 mm diameter RSS packs

The 750 mm diameter RSS pack tests results are presented in Figure 6.7. A typical press test result is included for comparative purposes and a summary is presented in Table 6-1. After 90 mm closure and 91 days of monitoring, the packs at the Rowland shaft site had not yet passed their peak load. At this stage the face had advanced 45 m away of the packs and the closure rate in the panel had reduced to less than 2 mm / week. The test packs did however show evidence of cracking on their outer surfaces which, from experience, suggested that they were near their peak load.

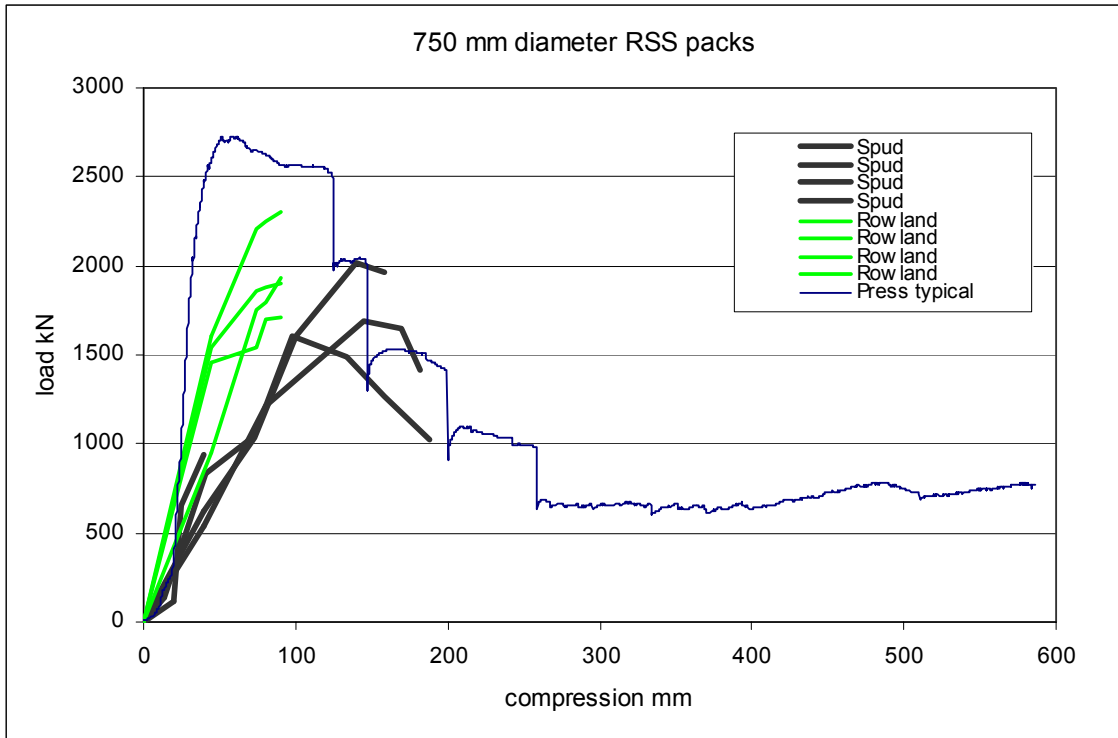


Figure 6.7 Results from 750 mm diameter RSS packs, comparing readings from Rowland shaft Spud shaft and a typical press test.

Table 6-1 Results from 750 mm RSS packs tested underground

	Average. initial stiffness kN/mm	Average maximum		Average. at 200 mm compression		Average. at 300 mm compression	
		Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>
Press	62	2700	364	1314	156	1000 (estimated)	119 (estimated)
Rowland	31	2000	264				
Spud	21	1792	236	1400 (estimated)	185 (estimated)		

Results showed that the *in-situ* packs have neither the load bearing capacity nor the stiffness of those tested in a press. The limited amount of closure at the Rowland shaft site, and the premature closing of the Spud shaft site, limited the post peak strength data obtained from these two sites. Sudden drops in load that are noticeable in the press test results are absent in the Spud shaft data. The absence of these sudden drops is possibly the result of the lower closure rates at the underground sites compared to the press tests, where the rapid loading would not allow the cement time to adjust under strain.

## 550 mm RSS packs

The results of the 550 mm diameter RSS packs tested underground are presented with a typical curve from the press tests in Figure 6.8 and are summarised in Table 6-2. The press tests gave maximum pack load bearing capacity 83% higher than those recorded underground. Only one of the the three packs (shown in Figure 6.9) tested underground experienced ring failure despite closures of between 160 mm and 220 mm. The associated stress drop was significantly lower than that recorded for the press tested packs. This lower stress drop is consistent with the behaviour of packs made from a weaker grout, but may also be because the grout was curing under closure at strain rates of the order of 1% per day. The post peak performances of the *in situ* packs at 100 mm and 200 mm were however, only slightly less than those of the press tests.

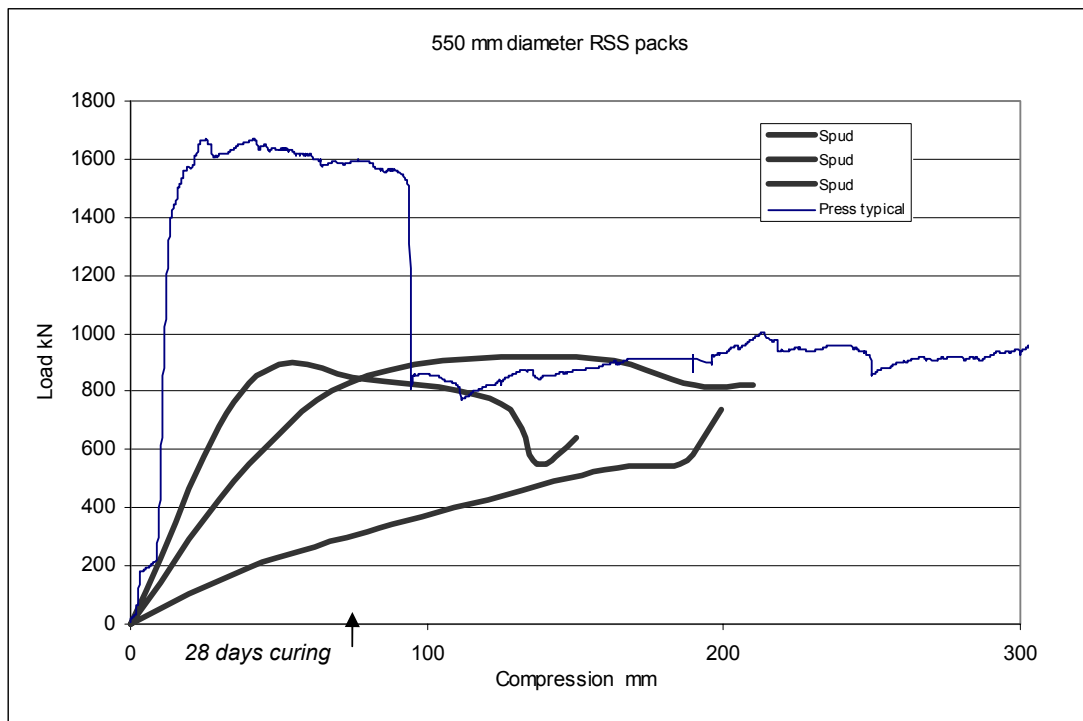


Figure 6.8 Results of 550 mm diameter RSS packs and a typical press test result

The arrow in the above diagram indicates the amount of compression on the underground packs after 28 days of curing. At this point all grouts tested would have cured for the same period of time. The difference in load bearing capacity suggests that other factors, apart from the maturity of the grout were responsible for the reduced strength of the underground packs.

Table 6-2 Results of 550 mm RSS packs

Test site	Initial stiffness kN/mm	Average maximum		Average at 200 mm compression		Average at 300 mm compression	
		Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>	Load kN	Support resistance kN/m <sup>2</sup>
Press	92	1650	253	960	147	950	146
Spud shaft	13	900	138				



Figure 6.9 550 mm RSS pack with 150 mm closure at the Spud shaft site

## 6.2 Summary

Two sizes of RSS packs were tested *in situ* at two underground sites, with different closure rates. The load bearing capacity and stiffness of the packs was found to be less than that indicated by the press tests. The following chapter analyses factors that influence the performance of grout packs while Chapter 8 examines factors likely to cause a difference between the laboratory and *in situ* performances.

## 7 Analysis of factors influencing pack performance

The press test results presented in Chapter 4 indicated differences in load bearing capacity and stiffnesses of different grout pack designs. The variables relating to these differences have been examined during the course of this test program and are analysed in the following subsections.

### 7.1 Strength of grout

Results from 750 mm diameter packs made up of 3 MPa and 10 MPa grout presented in Figure 7.1 show that the grout strength determined by, among other factors, the water / cement ratio and the use of pulverised fly ash, significantly affects the peak strength of the pack. The strength of the 10 MPa pack is approximately 70% greater than that of the 3 MPa pack. Following failure of the steel rings (from around 300 mm compression or 25% strain), both packs show a similar residual strength. This is to be expected as the internal friction angle, which is not affected by grout strength, dominates the response beyond this point. Note that the load drop-off associated with ring failure is lower for the weaker pack (no more than 19 % of peak) than for the stronger pack (as much as 29 % of peak).

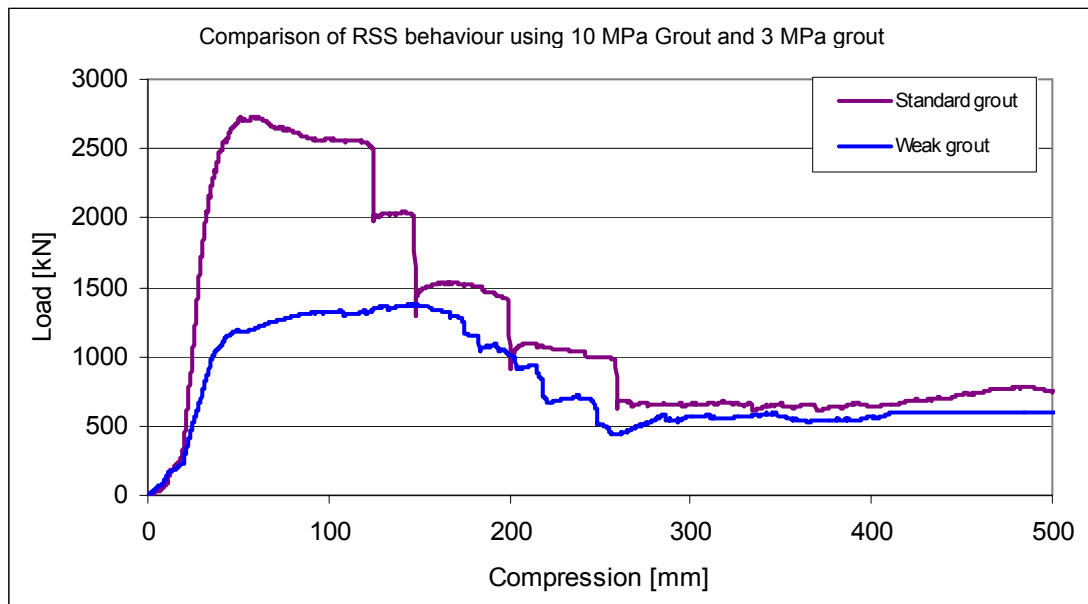


Figure 7.1 The effect of grout strength on standard RSS packs

## 7.2 The effect of pack diameter and height

This analysis is focused on the RSS grout pack tests. The effect of pack diameter on load bearing capacity for a constant height and reinforcement spacing is presented in Figure 7.2. The pack load bearing capacity shows a linear increase with diameter from a diameter-to-height ratio of 0.44 to 1.0. The rate of strength increase is notably greater between the diameter-to-height ratio of 1.0 to 1.2 (corresponding to the 1400 mm diameter pack).

The peak strength as a function of the pack diameter is illustrated in Figure 7.3. The pack strength decreases as the diameter increases up to the 1200 mm diameter pack. The strength then increases between the 1200 mm and 1400 mm pack. The reason for the counter-intuitive decrease in strength with diameter is that the volume proportion of steel reinforcement to grout decreases with pack size. The equal spacing of steel reinforcement rings for the packs tested is such that the ratio of the steel volume to grout volume shows a non-linear decrease with increasing pack diameter. This is reflected in the trend shown in Figure 7.3. The 1400 mm diameter pack is of sufficient size for the w:h ratio strengthening effect to mobilise sub-horizontal confining stresses within the grout material, increasing the peak strength of the pack.

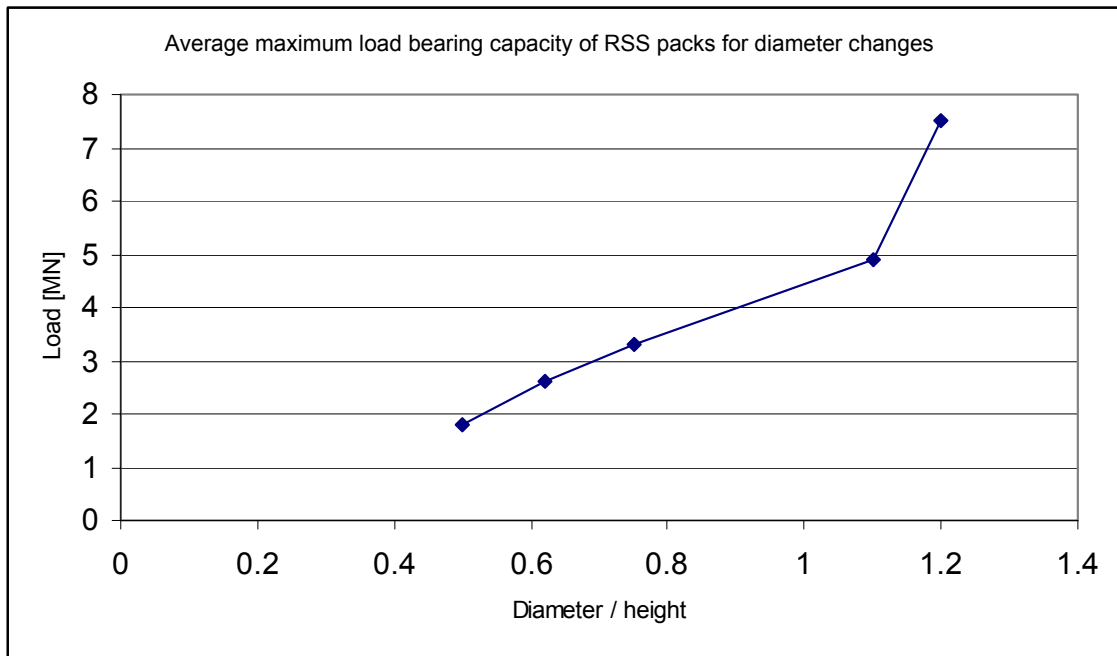


Figure 7.2 Average maximum load bearing capacity of constant height RSS packs with varying diameters

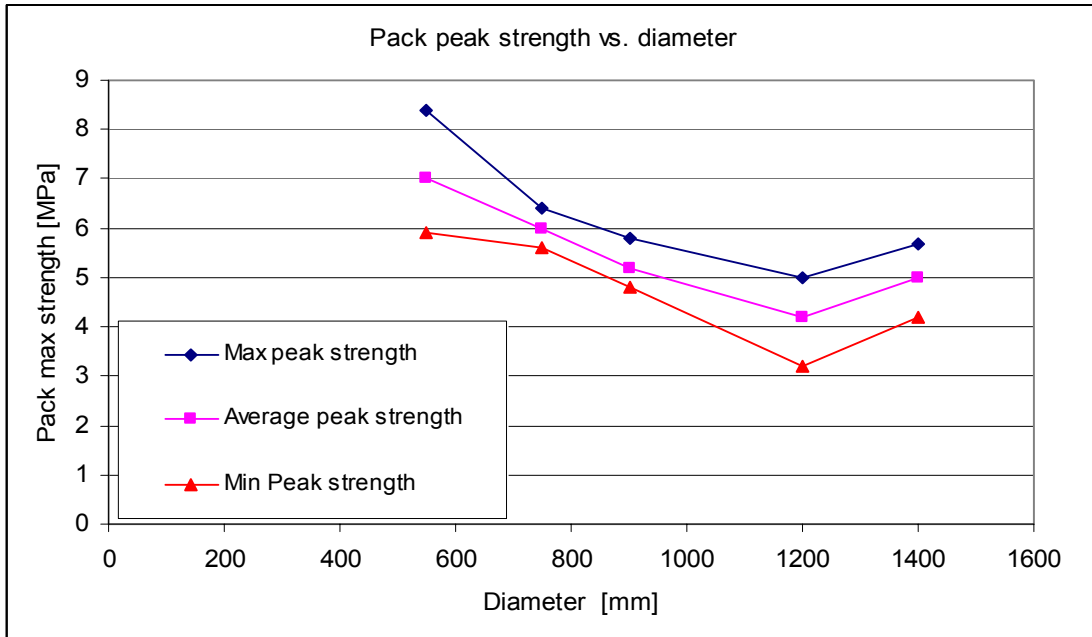


Figure 7.3 Peak, average and minimum strengths of constant height RSS packs with varying diameters

The effect of pack height on strength, for a constant 550 mm diameter and reinforcement spacing is presented in Figure 7.4. An approximately linear increase in pack strength is noted for packs with diameter-to-height ratios of between 0.48 and 1.2. This indicates that, for packs taller than a critical height, the pack strength will decrease in a linear manner with respect to the height. The reason for this is that the packs effectively became a series of shorter packs, which fail by the same mechanism in sequence. This appears to be due to the variation in grout strength from the top to the bottom of the pack as evidenced by the tendency of the rings to always fail from the top. In one case, the pack was tested upside-down and failed from the bottom upwards, confirming this finding. The very short packs (with diameter-to-height ratios of greater than 1.2) indicated increasing strength because of the w:h ratio strengthening effect.

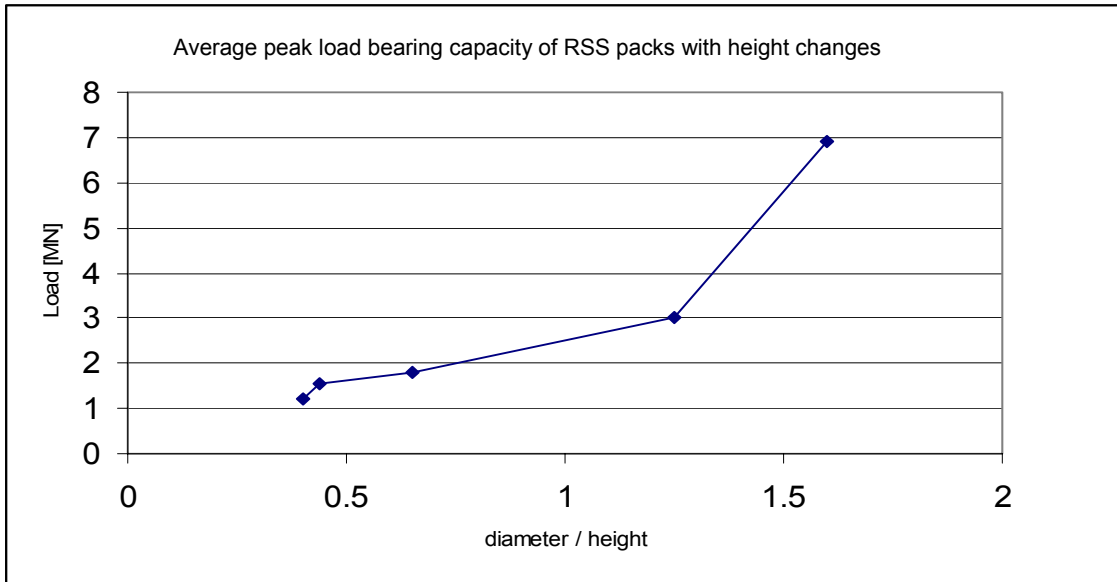


Figure 7.4 Average maximum strength of same diameter RSS packs with varying height

### 7.3 Post-failure characteristics

Most packs showed a decrease in strength following failure to a residual strength of less than 50 % of peak. The Cempack and the RSS Yield pack showed good post-failure characteristics and are discussed further in this section. The effect of including annealed steel reinforcement on the yieldability of the Cempack is illustrated in Figure 7.5.

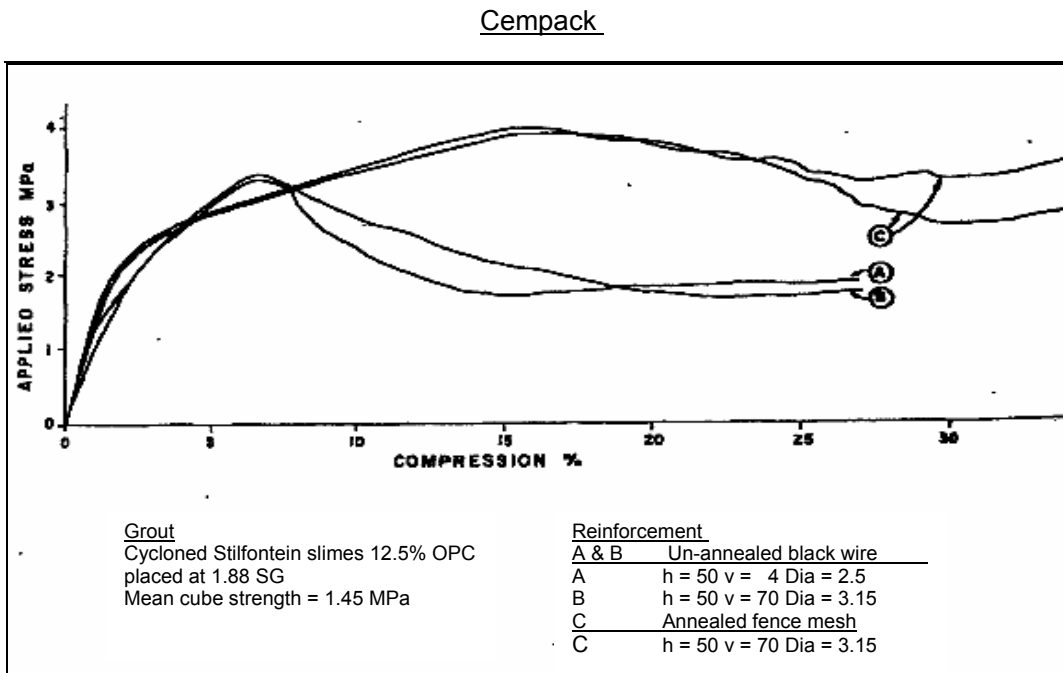


Figure 7.5 Stress – compression curves for Cempacks reinforced with Weldmesh (A and B) and annealed Weldmesh (C) (Dison and Blight, 1988)

Figure 7.6 compares the load-compression characteristics of a 1000 mm x 1000 mm Cempack with those of a 750 mm diameter RSS Yield pack, both 1.2 m high. Both tests were conducted on packs comprised of the weaker 3 MPa grout. The Yield pack includes 2.2 % steel by mass compared to 1 % and 1.3 % respectively used in the two designs of Cempack. The higher steel to grout ratio results in the Yield pack being initially stiffer, but both show yielding at 3000 kPa to 4000 kPa over the 500 mm displacement range. Recompaction is evident in the Cempack from approximately 400 mm compression. The height to width ratio of the Yield pack was greater and therefore no compaction was observed. Both of these pack configurations show impressive yield characteristics, maintaining, at worst, 64 % of the peak strength over a 500 mm displacement range.

The Yield pack showed significant and sudden drops in load as the inner rings failed, (shown in Figure 7.6). However, the effect of these drops will be greatly ameliorated in a stope, where individual packs in a support system will fail at different levels of closure. This results in a response that averages out these sudden drops in load.

Beyond 250 mm deformation the Cempacks and Yield packs provide better support resistance than do RSS packs despite using a weaker grout (Table 4-13).

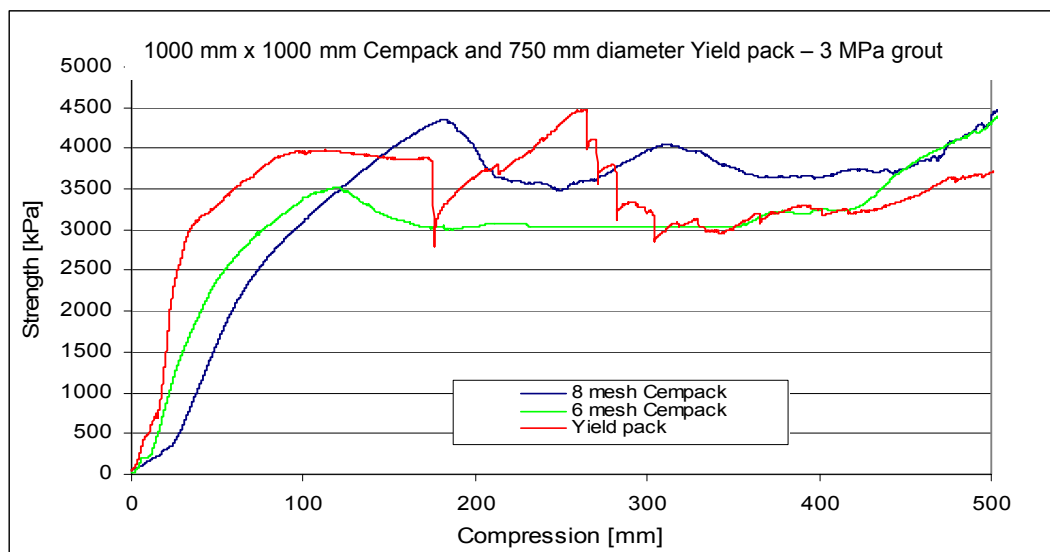


Figure 7.6 Strength levels of the two types of yielding packs tested

## 7.4 Summary

Factors that have a direct bearing on the performance of all Grout packs have been examined in this chapter. In addition to these factors it has been shown that, by engineering the reinforcement, the yieldability of packs can be enhanced. In the next chapter, the factors that may result in a difference between laboratory performances of packs and those tested underground, are examined.

## **8 The difference in performance between underground tests and press tests**

Chapter 7 illustrated how, by engineering the size, height, strength of grout and the strength and ductility of the reinforcement, the useful properties of Grout packs can be enhanced. This chapter examines the behavioural differences that were found to exist between grout packs tested in a press and those placed underground. The main differences noted were:

- Underground packs were less stiff than those tested in a press.
- Underground packs have lower load bearing capacity.
- The packs strained more underground before the reinforcing rings yielded.

In order to formulate a relationship between the performance of packs tested in a press and those placed underground it is necessary to examine the environmental factors responsible for the differences in performance, and attempt to quantify them. These factors are identified and discussed in the following sub sections.

### **8.1 Grout**

Underground tests were carried out at Rowland shaft and Spud shaft, while the press tested packs were cast on surface at Karee 3 shaft. The three shafts each used different grout recipes and, in addition, both 10 MPa and 3 MPa grouts were made up at Karee 3 shaft for the pack tests. Four different grout recipes (listed in Table 2.1) each with different grout strengths were therefore used in the course of this test programme. Variations in total binder content, ratio of OPC to flyash, composition of tailings and water content are noted. The effect of adjusting the water-cement ratio when using mine tailings has been examined by Lamos [1993] and is illustrated in Figure 8.1.

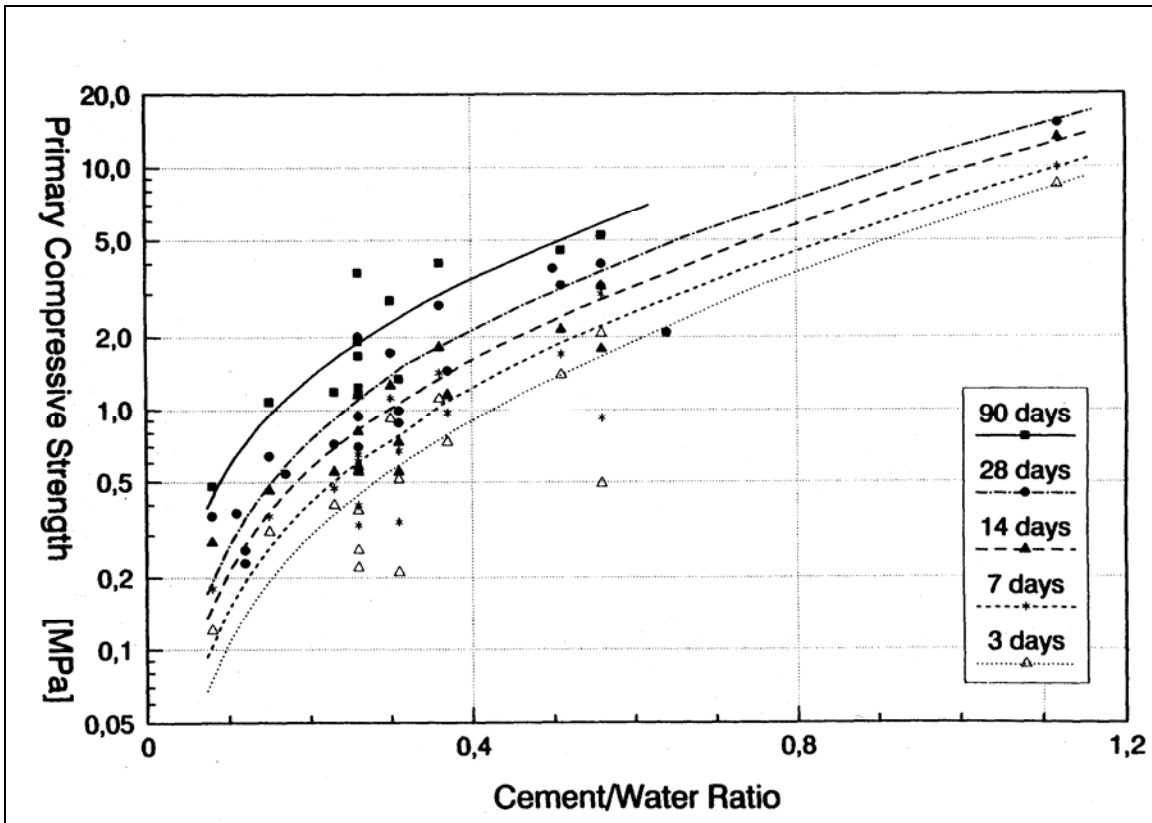


Figure 8.1 The relationship between cement/water ratio and grout strength (Lamos 1993)

Note that the strength is presented on a log scale. Clearly small variations within the cement / water ratio, will significantly affect the strength of the grout. These variations may occur as a result of losses during the hydraulic transportation and placement of slurry and will affect the strength of packs placed underground, however no detailed investigation into this aspect of grout strength was carried out in this project.

## 8.2 Temperature

Figure 8.2 shows the effect of curing temperature on the strengths of cemented mine tailings. The tailings used for this test series included 10% (in dry mass) of an OPC/FA 1:2 binder, at a water/solids ratio of 0.28. The samples were cured and tested as 100 mm cubes. This, combined with their low water/solids ratio, accounts for their relatively high strengths.

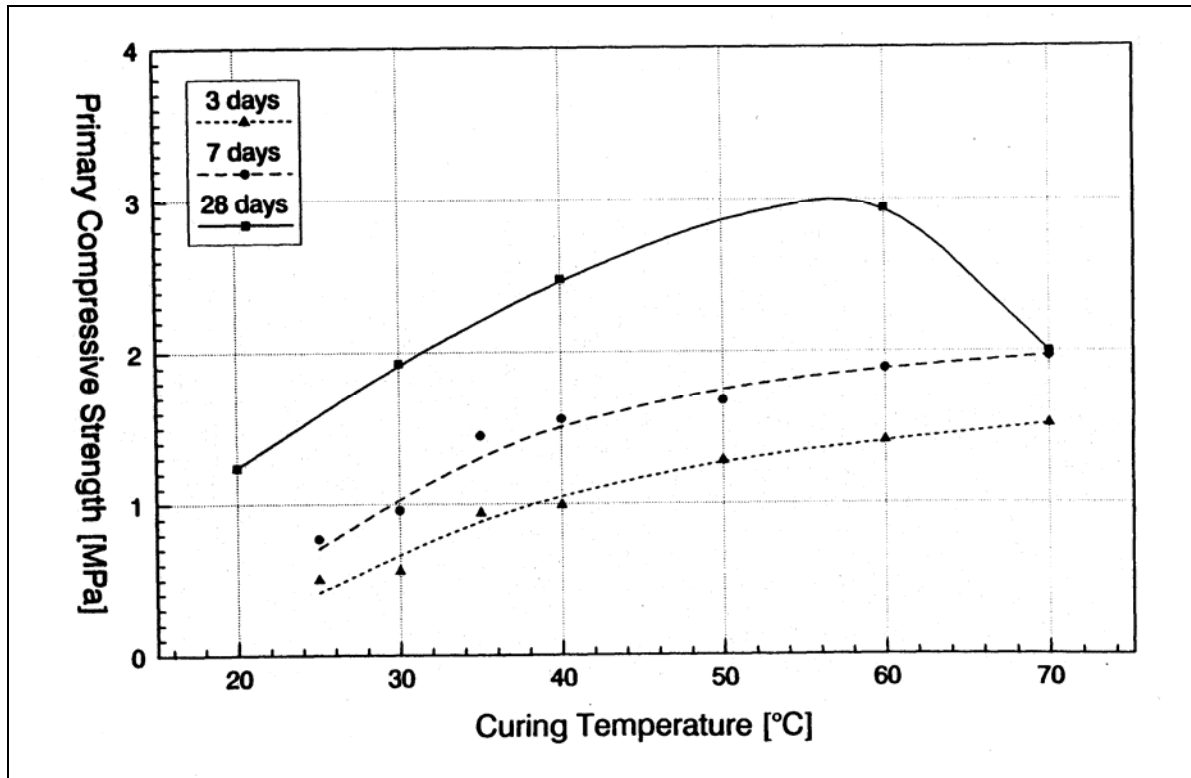


Figure 8.2 Effect of curing temperature on cemented classified tailings (Lamos 1993)

As expected, elevated curing temperatures accelerate the gain in strength of cemented tailings provided that full saturation is maintained and the temperature does not exceed a level at which excessive evaporation would result in a drying out of the tailings. This may explain the loss in strength evident after 28 days above 60°C (Figure 8.2).

Temperature had a significant effect on the pack test programme because packs cast for the press test were cured in the open with variations in day-night temperatures whereas the packs monitored in the underground trials cured under reasonably consistent underground temperatures.

### 8.3 Curing under closure

During curing the hydrated binder components in cemented backfill gradually forms a rigid crystalline structure which bonds the slurry into a solid mass. Provided that the slurry is well mixed, a measure of rigidity will be present within the slurry within one to two hours. If this cemented slurry is strained below a critical rate during the curing process, the cementitious bonds undergo elastic deformation and, provided the yield point of the crystals is not exceeded, normal curing should continue unabated, protected by the existing rigid structure.

In a set of experiments (Lamos 1993), six cemented backfill specimens were loaded at different, but constant strain rate during the curing process, to determine whether the straining of the material had any deleterious effect on their primary compressive strength. Compared to the unstressed control samples, a 20% deterioration of the compressive strength and the elastic modulus was measured, which was directly attributable to the straining of the backfill. In a subsequent test series, the constantly strained samples again showed much reduced strengths (30% lower), but a parallel suite of samples, which were stressed with a static, constant load during curing (a creep test), that never exceeded the yield point, showed no deterioration at all.

A further set of Laboratory tests (Squelch 1993) indicated that cementitious bonds are damaged if cemented slurry is loaded at a high strain rate (i.e. greater than 1.5% per day during curing). Figure 8.3 shows that samples loaded during curing are initially weaker than those loaded after curing, but will surpass their strength after 4% strain. This effect is not explained by Squelch but may be a result of the test specimens having being cast at different heights or containing variations in the water / cement ratio.

During the underground tests, the strain rate at Spud shaft was initially over 1% per day which could have had a negative effect on the grout strength. However the effect of strain rate during curing on grout strength with a binder content of approximately 25%, as opposed to cemented backfill with 10% binder has not been determined. It is recommended that such work should be carried out as the effect of binder content and composition (OPC to flyash ratio) is unknown.

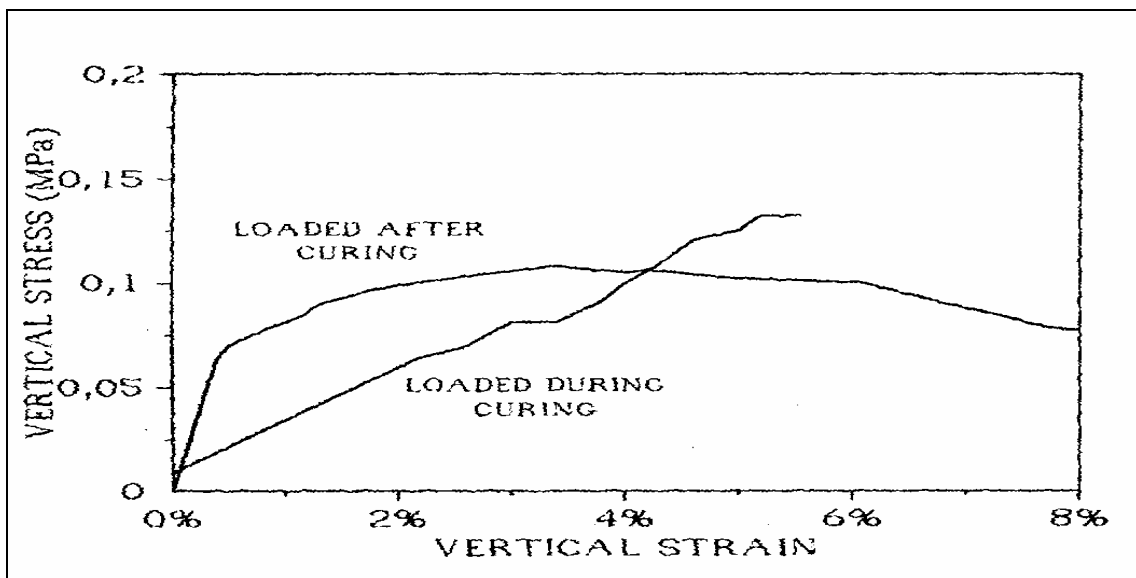


Figure 8.3 Cemented backfill curing under strain (Squelch 1993)

## 8.4 Loading rate

Press tests are generally conducted at a rate of about 15 mm / minute. A 1.2 m high pack will therefore have achieved 40% strain within 32 minutes and the test would usually be regarded as complete. A pack installed in a panel underground is usually expected to fulfil its support role for several weeks before failure occurs and will very seldom be strained to this extent.

Various studies [Daehnke et al 1998] have been conducted to quantify the force-deformation characteristics of support units installed in working stopes underground. It was shown that force-deformation behaviour can be significantly downgraded compared to the laboratory derived curves as a result of the loading rate. Following a series of systematic laboratory tests carried out to assess the influence of loading rates on support units, the following equation was derived for adjusting the support resistance provided by cementitious packs (Lightfoot 1997):

$$F_{u/g} = F_{lab} \left( 1 + \frac{10}{100} \right)^{\log \frac{V_{u/g}}{V_{lab}}} \dots\dots\dots(1)$$

Where:  $F_{u/g}$  is the adjusted underground force

$F_{lab}$  = force measured during laboratory tests

$V_{u/g}$  = velocity of underground closure (typically 5 – 30 mm/day)

$V_{lab}$  = laboratory test velocity (typically 10 – 30 mm/min)

The above equation can be rewritten as:

$$F_{u/g} = kF_{lab}, \text{ Where } k \text{ is the force correction factor.}$$

A graphical representation of the force correction factor for cementitious packs can be found in Figure 8.4.

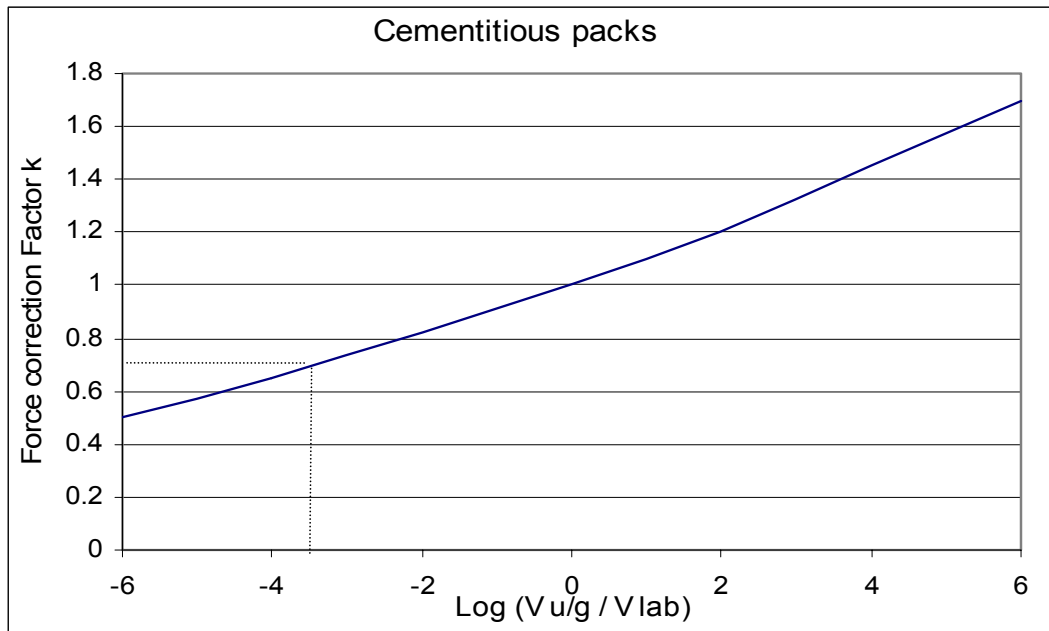


Figure 8.4 Graph for adjusting the load-deformation curves for cementitious packs

(Lightfoot 1997) The dotted line applies to the following calculation

Using Eq. 1, the following calculation would apply when determining the peak load of a pack installed in a stope (with a closure rate of 15 mm / day), from a press test result (conducted at 15 mm / min) with a peak load of 3000 kN.

$$F_{ug} = 3000(1.1)^{\log \frac{15}{15 \times 60 \times 24}} \text{ kN}$$

$$= 2220 \text{ kN}$$

Therefore  $F_{u/g} = 74\% F_{lab}$

## 8.5 Point loading

Packs that are not tightly packed against the hangingwall, or those whose ends are not cast parallel for a press test, will be subjected to asymmetric loading, adversely affecting the strength of the pack. In addition irregular, rough hangingwall or footwall contacts can cause point or line loading or wedging effects that will weaken the pack. The quadrant type load cells used for underground tests often showed a wide variation in loads on the four different segments as the packs were compressed. The effect is illustrated in Figure 5.14, where readings varied widely on the four segments of a pack, during a calibration test. For example, during the test and at 100 mm compression, readings of 0 kN, 370 kN, 480 kN and 720 kN were recorded simultaneously on the four loadcells.

## 8.6 Summary

In this chapter five factors, that may result in differences between the loads recorded from press tested and those that apply to *in situ* packs have been identified. In the next chapter these factors will be taken into account in an attempt to formulate a relationship between the packs in the two environments.

## 9 Relating press test results to the performance of packs placed underground

For the reasons outlined in Section 8 and illustrated by the underground test program, press tested packs generally have greater load bearing capacities and stiffnesses than packs placed underground. It is therefore necessary to adjust the press test results, from the limited testing underground, to establish a preliminary relationship between press tests and underground performance of grout packs. The following procedure can be used to estimate underground pack performance from press tests.

- 1. Strength of grout:** Cube or UCS tests should be conducted simultaneously with press tests as an indication of pack grout strength with respect to time. The strength of underground packs should be de-rated (particularly during the early stages) accordingly. It is also good practice to conduct UCS tests from grout specimens taken from underground packs and compare them with those taken from packs poured on surface. This provides a simple audit of the efficiency of the hydraulic transportation and placement processes. Samples should be taken near the top and bottom of the packs.
- 2. Curing temperature:** Using Figure 8.2 in conjunction with known differences between surface and underground curing temperatures, the grout strength should be adjusted accordingly. Generally underground temperatures are higher than surface temperatures, normally resulting in better grout strengths for the underground packs.
- 3. Curing under closure:** Strain rates in excess of 1.5% per day during curing have been shown to reduce cemented backfill strength by 20% to 30%, based on 10% binder tests. Closure rates for ground control districts need to be determined to establish whether the critical strain rate is exceeded or not. Preferably a controlled test programme should be carried out on various composition grouts (binder content approximately 25%) where specimens are strained in a press at the expected underground strain rate, to determine whether the backfill results apply equally to stronger grouts.
- 4. Rate of closure:** Strain rates on press tested packs are generally of the order of 80% per hour. Except in the event of a seismic event or in some cases, immediately following the blast, packs placed in a panel underground will strain less than 1% per day. From Figure 8.4 the load borne by a pack placed underground will be 20% and 50% less than that borne in the press.

## 9.1 Example

From the results presented in Figures 6.7 and 6.8, two worked examples are presented in Tables 9-1 and 9-2 comparing the RSS packs placed at Spud shaft and those tested in the press, at different points of compression. The purpose of this example is to illustrate a method that can be used to estimate the load bearing capacity of an underground pack at different points of its compression by making use of a result obtained from a press test. In order to make these estimations, a practitioner will require the following information:

- Press test results.
- UCS test results from a mature underground pack and from grout used in the press test.
- Curing temperatures from the stope and from the site where the press tested packs matured.
- Strain rates in the stope.
- Strain rates from the press.

In the following table the four factors, identified as being different between the two test methods that are likely to affect the strength of the grout, are compared at different points of compression. The table is structured as follows:

1. The last column shows the percentage by which the press tested pack must be adjusted to equate its result with those of the pack placed underground.
2. The percentage adjustments are summed in the “Nett” row.
3. The “Average load recorded” row compares the load recorded from the two different methods and once again shows by what percentage the press tested pack result would have to be adjusted to equate it with that of the underground pack.
4. The percentage difference between the two results is given in the following row.

Table 9-1 A worked example showing the required strength adjustments to estimate pack strength from press test results. - 750 mm diameter packs

<b>50 mm Compression</b>	<b>Press pack</b>	<b>Average u/g packs</b>	<b>Strength adjustment</b>
<i>UCS of grout</i>	4.5 MPa {1}	2.5 MPa {2}	-44 %
<i>Curing temperature</i>	25° C {3}	30° C	+25 % {4}
<i>Curing under closure</i>	0 %	1 % - 1.5 % per day	-20 % {5}
<i>Strain rate</i>	80 % per hour	1 % – 1.5 % per day	-26 % {6}
<i>Nett</i>			-65 %
<i>Average load recorded</i>	2650 kN	950 kN {7}	-65 %
<i>Difference</i>			0 %
<b>100 mm Compression</b>	<b>Press pack</b>	<b>Average u/g packs</b>	<b>Strength adjustment</b>
<i>UCS of grout</i>	4.5 MPa {1}	3.5 MPa {8}	-22 %
<i>Curing temperature</i>	25° C	30° C	+20 % {9}
<i>Curing under closure</i>	0 %	0.5 % -1 % per day	-10 % {10}
<i>Strain rate</i>	80% per hour	0.5 % – 1% per day	-26 %
<i>Nett</i>			-38 %
<i>Average load recorded</i>	2550 kN	1600 kN	-37 %
<i>Difference</i>			-1 %
<b>150 mm Compression</b>			
<i>UCS of grout</i>	4.5 MPa {1}	3.7 MPa {11}	-17 %
<i>Curing temperature</i>	25° C	30° C	+16 % {12}
<i>Curing under closure</i>	0 %	0.5 % per day	-5 % {10}
<i>Strain rate</i>	80 % per hour	0.5 % per day	-26 %
<i>Nett</i>			-32 %
<i>Average load recorded</i>	1750 kN	1600 kN	-9 %
<i>Difference</i>			-23 % {13}

Table 9-2 A second worked example using 550 mm diameter pack data

<b>50 mm Compression</b>	<b>Average press packs</b>	<b>Average u/g packs</b>	<b>Strength adjustment</b>
<i>UCS of grout</i>	4.5 MPa {1}	2.5 MPa {2}	-44 %
<i>Curing temperature</i>	25° C {3}	30° C	+25 %
<i>Curing under closure</i>	0 %	1 % -1.5 % per day	-20 %
<i>Strain rate</i>	80% per hour	1 % – 1.5 % per day	-26 %
<i>Nett</i>			-65 %
<i>Average load recorded</i>	1500 kN	600 kN	- 60 %
<i>Difference</i>			-5 %
<b>100 mm Compression</b>	<b>Average press packs</b>	<b>Average u/g packs</b>	<b>Strength adjustment</b>
<i>UCS of grout</i>	4.5 MPa	3.5 MPa	-22 %
<i>Curing temperature</i>	25° C	30° C	+20 %
<i>Curing under closure</i>	0 %	0.5 % -1 % per day	-10 %
<i>Strain rate</i>	80% per hour	0.5 % – 1% per day	-26 %
<i>Nett</i>			-38 %
<i>Average load recorded</i>	1400 kN	650 kN	-54 %
<i>Difference</i>			+16 %
<b>150 mm Compression</b>			
<i>UCS of grout</i>	4.7 MPa	3.7 MPa {11}	-17 %
<i>Curing temperature</i>	25° C	30° C	+16 % {12}
<i>Curing under closure</i>	0 %	0.5 % per day	-5 % {10}
<i>Strain rate</i>	80 % per hour	0.5 % per day	-20 %
<i>Nett</i>			-26 %
<i>Average load recorded</i>	1250 kN	750 kN	- 40 %
<i>Difference</i>			14 % {13}

{1} Figure 2.6 – UCS strength of Karee cylinders at 28 days

{2} Figure 2.6 – UCS strength of Spud cylinders at 14 days

{3} Cured outside in early summer

- {4} Figure 8.2 – Difference between outside temperature and stope temperature (7 day curve)
- {5} Section 8.3 (ref A.W. Lamos)
- {6} Calculation 8.5 and Figure 8.4
- {7} Figure 6.7- At 50 mm closure
- {8} Figure 2.6 - UCS strength of Spud cylinders at 24 days
- {9} Figure 8.2 –Difference between outside temperature and stope temperature (14 day curve)
- {10} Closure rate has slowed but the grout strength is already degraded
- {11} Figure 2.6 – UCS strength of Spud cylinders 28 days
- {12} Figure 8.2 – Difference between outside temperature and stope temperature (28 day curve)
- {13} Ring failure on press tested packs, but not underground, has resulted in reduced press pack strength.

## 9.2 Summary

Using the above technique, the strength of *in situ* grout packs can be estimated from press test results at various degrees of compression. The percentage error ranges from -23% to +16% with an average of -4%. The results are therefore encouraging but should be enhanced by further data.

## 10 Conclusions

In this research report tests have shown that grout packs are suitable for inhibiting inelastic closure and can be designed to provide stiff support or support over a wide deformation range. The factors that most affect the initial strength and post failure characteristics of a grout pack are: the grout strength, the amount and type of steel reinforcement, the inclusion of ancillary columnar support, the height and diameter of the pack and the temperature and rate of closure in the stope.

It was found that failure of the RSS packs was associated with failure of the confining rings. The top ring was nearly always the first to fail, indicating that the dilation at the top of the pack was greater than at the bottom. Since dilation is associated with failure of the grout material, it was inferred that the pouring process resulted in a weakening of the grout from the bottom to the top. A pack was tested upside-down and the rings failed progressively from the bottom to the top, providing further evidence for this finding.

The load bearing capacity of packs increased with diameter; however, it was found that the nominal peak strengths decreased with increasing diameter. It was found that the relative decrease in load bearing capacity was directly associated with the decreasing ratio of steel reinforcement to grout volume. Using smaller packs with a reduced skin-to-skin spacing might therefore prove more efficient in terms of grout use than using large packs. An additional advantage when using smaller packs is that the shuttering required is more compact and easier to transport into the stope panels.

At a 2 m skin-to-skin spacing only the 1200 mm diameter RSS pack, and the two versions of the 1000 mm x 1000 mm Cempacks (using 3 MPa grout) met the 340 kN/m<sup>2</sup> support resistance criteria set for this project. However, it was considered that the installation of 1200 mm diameter packs underground would be impractical. It would appear that 900 mm diameter Yield packs using 10 MPa grout may also meet this criterion, but this needs to be confirmed by further tests.

Maintaining a constant pack diameter and changing the pack height showed that the influence of pack height was linear with respect to strength for diameter-to-height ratios of less than 0.7. The results indicated that these taller packs behave as a stack of shorter packs, failing in succession. This finding should not be taken to infer that packs of any height would have a linear height-to-strength relationship. Very tall packs may be subject to other failure mechanisms (toppling or buckling, for example), which will decrease the pack stability and load bearing capacity.

Packs with diameter-to-height ratios of greater than 1.2 showed strain-hardening behaviour, so much so that the pack with a diameter-to-height ratio of 1.7 proved to be indestructible (up to 10 MN).

In terms of post-failure behaviour, the RSS Yield pack and the Cempack performed best, maintaining at least 64 % of the peak load over 400 mm of compression. These supports were similar in every aspect of their behaviour. The Yield pack, with its higher percentage steel reinforcement, was slightly stiffer, with better load bearing capacity, but showed greater load drops as the rings failed. It is believed that this effect will be greatly ameliorated in a situation where many packs are present; however a statistical analysis of many test results is necessary to confirm this.

Different types of load cells were included in the tests to establish a correlation between the press loads and the loads measured by the load cells. It was found that the smaller load cells placed in the grout did not provide a realistic estimate of the total load on the pack. Post-failure examination of the packs showed that these cells had rotated in some cases. The results from the larger load cells placed under the packs were far more consistent and should provide a reliable estimate of the actual loads within packs installed underground. The limited number of tests does not provide a reliable relationship between the load indicated by the loadcells and that borne by the packs. However, it seems that this relationship may be possible with more testing. The reason for the poor correlation between the loads calculated from the load cell's pressures and the load applied by the press needs to be determined.

When determining the support resistance of a system of packs placed underground it is necessary to adjust the strength and stiffness of the press test results by taking into account the following:

- A hot working environment will improve the strength of the packs initially.
- Closure strain rates in excess of about 1.5 % per day will adversely affect pack strengths.
- Variations in the water/cement ratio and binder content will affect pack strengths and performance.

From the worked examples in Section 9 it is evident that adjustments made using the appropriate strength adjustment factors will give an estimate only. The margin of error noted when making these adjustments ranged from 0% to 16% where neither packs had experienced ring failure. Once ring failure has occurred on a pack, comparisons with an un-failed pack are no longer valid. A 23% difference was noted when comparing an underground 750 mm pack at 150 mm compression, with all rings intact, with a similar pack with a broken ring, tested in the press.

Evidence from underground tests conducted during the course of this investigation suggests that the maximum pack load bearing capacity may be as much as 60% less than in press tests and that underground packs are less stiff. Load bearing capacity and stiffness adjustments are therefore necessary and the technique demonstrated in this project generally presented a fair estimate of in situ performance.

# 11 Recommendations

The results from the standard RSS grout packs indicated that the steel to grout volume ratio is crucial to pack performance. It is suggested that for support systems that require a high support resistance while yielding, packs with greater diameters should be reinforced with greater volumes of steel. This will at least, result in a higher pack strength, and may cause the w:h strengthening effect to be invoked for packs with relatively large diameters.

Further optimisation is possible for the design of standard RSS grout packs. Given that the units fail from the top, the bar diameter of the upper steel rings could be increased to ensure that the dilation throughout the pack is uniform. While this may increase the strength of the pack, the failure may be more violent. More research is recommended to explore this option.

The design of these packs can be further optimised to improve yieldability. The “Yield pack” design could potentially be improved by using multiple rings of various bar diameters and circumferences. Testing is recommended on a number of different configurations to realise the optimisation potential.

The material characteristics of the reinforcement rings may also contribute to the yieldability of the packs. Given the success of the Cempack, it is suggested that annealing or otherwise increasing the ductility of the steel rings may improve post-failure performance.

A preliminary correlation has been found to exist between press test results and the in-situ behaviour of packs installed in stopes. To make use of this correlation it is necessary to conduct UCS tests on grout simultaneously with press tests and to record the likely closure rates and temperatures in both environments. In addition a further more comprehensive program of underground testing is required to establish a more reliable relationship based on better statistics between the performance of grout packs in the laboratory and underground. This is particularly important for the post peak performance range and for Yield pack designs where no data were collected in this project. This information is essential for the design of grout pack based support systems required to replace in-stope pillars or where high closure rates are expected.

Further UCS testing is required on grouts with binder compositions the same as the grout used on the mines. It is noted that the grout UCS was still increasing after 60 days as a result of including PFA binder. It is also necessary to establish the critical strain rate above which these grout strengths are significantly diminished.

An understanding of the reasons why the quadrant load cells underestimate the pack load is required. This could lead to an improved design of loadcell.

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