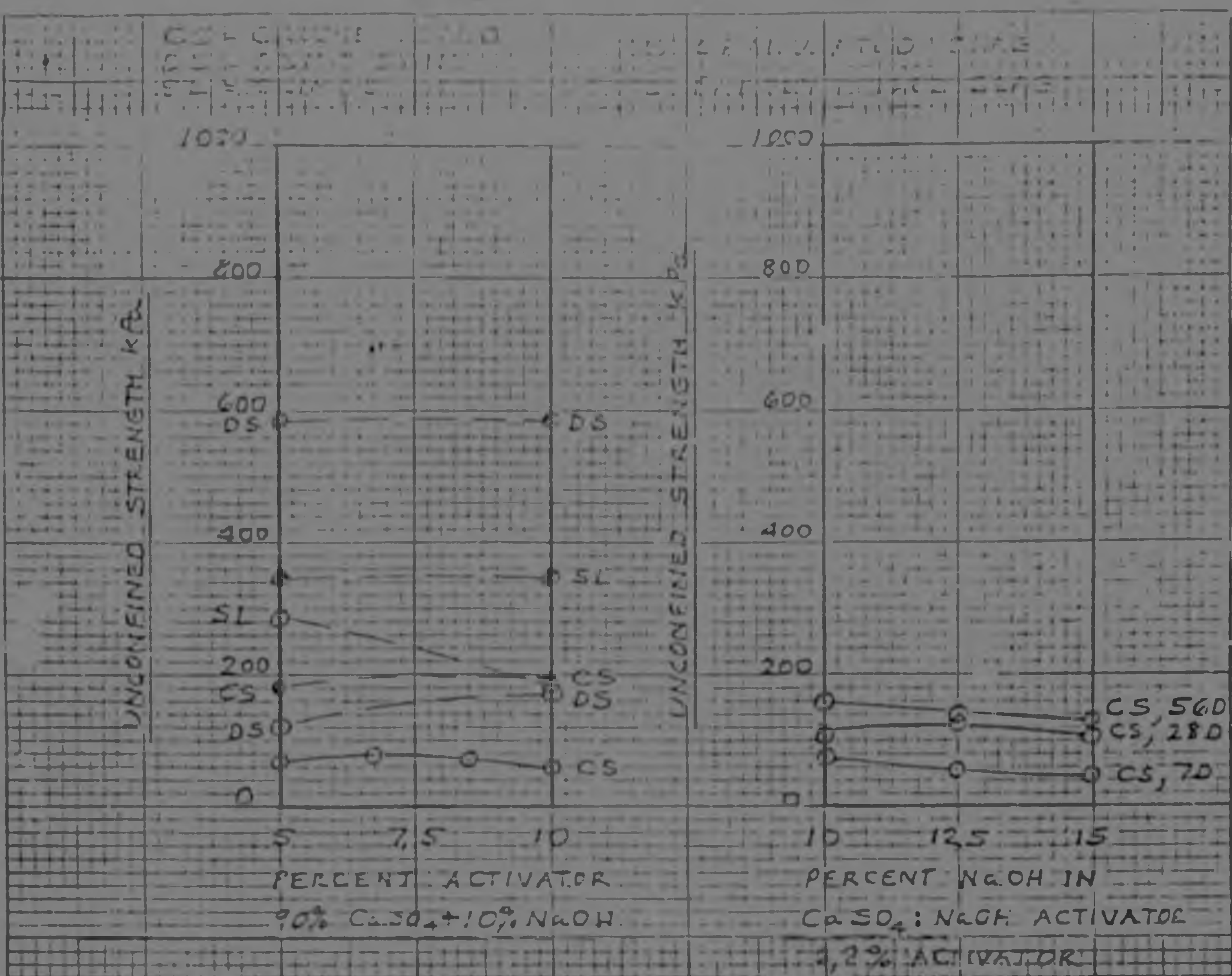

GYPSUM/CAUSTIC SODA ACTIVATORS

FIGURE 50

EFFECT OF ACTIVATOR PERCENTAGE & ACTIVATOR COMPOSITION ON UNCONFINED STRENGTH

EACH EXPERIMENTAL POINT IS THE MEAN OF 3 DETERMINATIONS

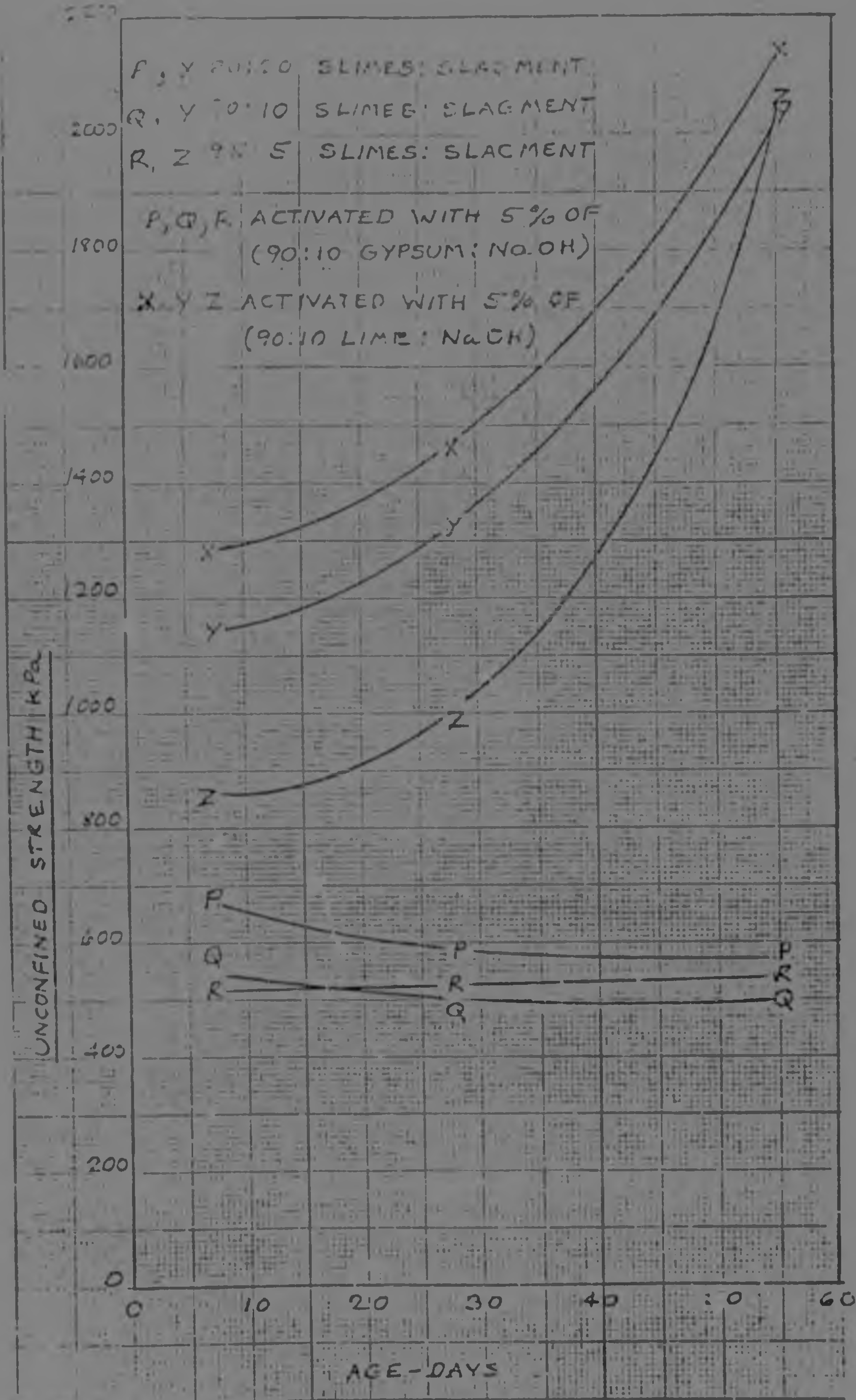

FIGURE 6

STRENGTH-AGE CURVES FOR SLIMES/SLAGMENT MIXES ACTIVATED WITH LIME/NaOH & GYPSUM/NaOH.

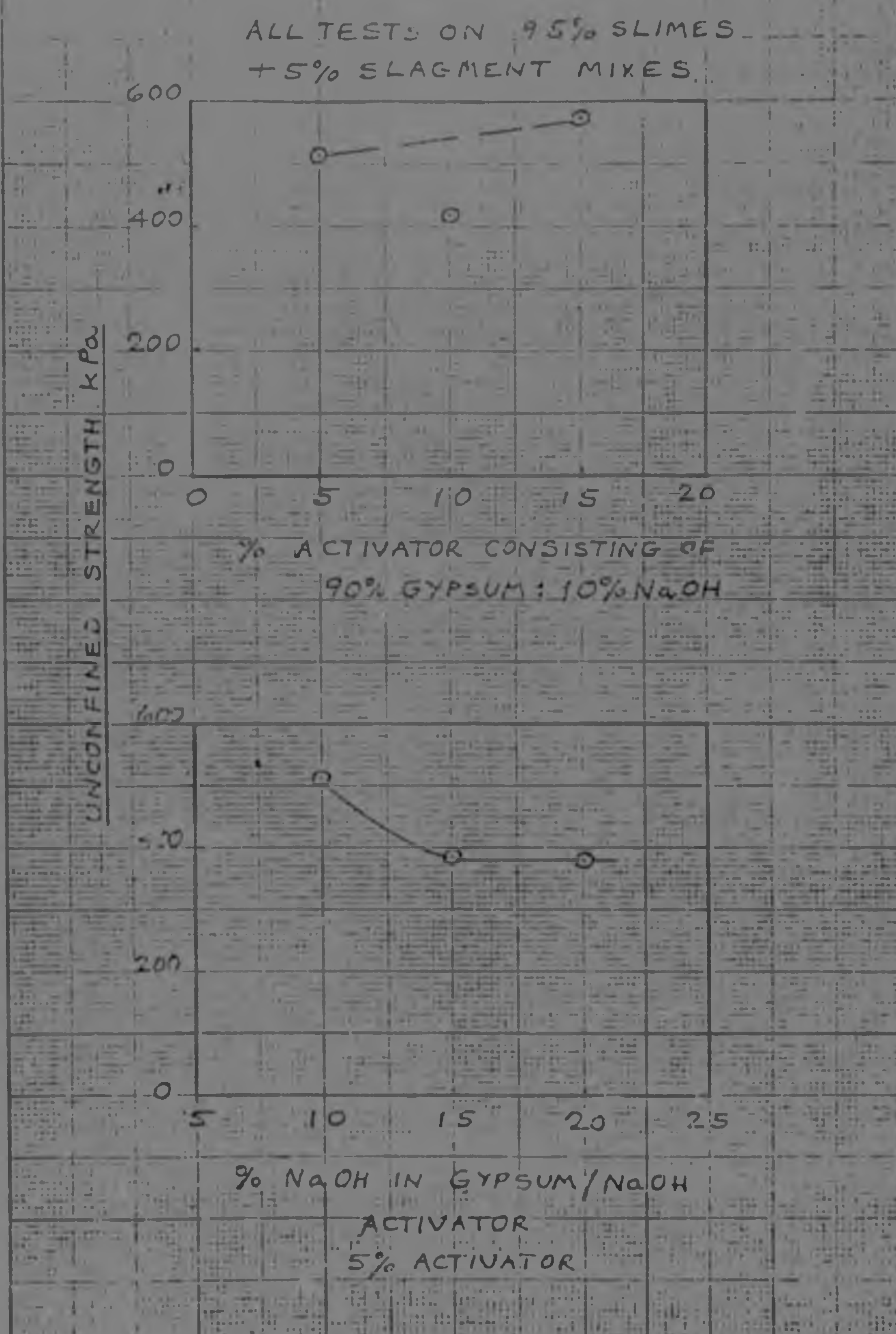

FIGURE 7

EFFECT OF VARYING ACTIVATOR CONTENT & NaOH CONTENT IN ACTIVATOR ON STRENGTH OF 95% SLIME : 5% SLAGMENT MIXES.

APPENDIX 13

Portland Sement Instituut
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1980-03-25

WeE 80/488

Mr. L. Dison,
Reinforced Earth (Pty) Ltd.,
P.O. Box 376,
BERGVLEI.
2012

Dear Sir,

Please find enclosed our report on the tests carried out on the South African and French blastfurnace slags.

Yours faithfully,
PORTLAND CEMENT INSTITUTE

M.B. Welsh
M.B. WELSH

Enc.

UITVOERENDE KOMITEE — P. BYLAND, T.G. COULSON, J.P. CRONJE, G.R. LUYT, J.E. HODGKISS, D.J. ROWE — EXECUTIVE COMMITTEE
PLAASVERVANGERS — H. BYLAND, D.A. GELL, E.F.B. PAAHMAN, A.M. SWARTZ — ALTERNATES

Hoewel alle moontlike sorg gedra word om te verseker dat enige raad of inligting wat verstrek word juis is, word enige raad of inligting hierin vervat verstrek op voorwaarde dat die Instituut of sy werknemers nie aanspreeklik gehou word vir die juistheid daarvan nie.

Whilst every care is taken to ensure the correctness of any advice or information given, any advice or information herein contained is given on the condition that the Institute and its employees incur no liability with regard to the correctness thereof.

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Laboratory report

Nature of test: Strength tests with South African and French blastfurnace slags

Ref: WeE 80/488 1980-03-25

Client: Mr. L. Dixon,
Reinforced Earth (Pty) Ltd.,
P.O. Box 376,
BERGVLEI.

SCOPE OF WORK

A sample of French Blastfurnace slag was received so that tests could be made of its hydraulic properties in comparison with similar types of South African Blastfurnace slags.

METHOD OF TEST

(i) The sieve analysis was performed in accordance with the method specified in SABS METHOD 829.

(ii) The slags as received were used as an aggregate in a 3:1 aggregate/cement mortar, the mixes being made to the same consistency in each case. Due to the large difference in water demand of the French and SA slags the former had a c/w of 1,5 (w/c 0,67) and the latter 1,07 (0,93).

(iii) In this test, the French slag was milled to a fineness similar to that of Slagment. Mortar cubes were made using a binder consisting of a 50/50 blend of normal portland cement and the appropriate fine slag using a total binder to water ratio of 1,67.

(iv) An ISO type mortar was made consisting of one fine slag to three sand by mass. The c/w ratio was 2,0. The activator was 200 g of sodium hydroxide per litre of water.

2/ Results .....

VOORWAARDES VIR TOETSING EN VERSLAGLEWERING Hoewel alle sorg gedra word om te verseker dat alle toetse en verslae juis is, kan die Instituut of sy werknemers hoegenaamd nie aanspreeklik gehou word vir enige foute begaan in die uitvoering en verslaggewing van toetse of enige foutiewe afleidings daarvan gemaak of vir enige gevolge daarvan.

CONDITION OF TEST AND REPORT While every care is taken to ensure the accuracy of all tests and reports, neither the Institute nor its employees shall be liable in any way whatever for any error made in the execution or reporting of tests or any erroneous conclusions drawn therefrom or for any consequences thereof.

RESULTS

(i) The grading analysis is shown on the attached sheet.

(ii) Slag aggregate test.

	7 Days MPa	28 Days MPa	90 Days MPa
SA Slag	4,0	7,0	10,0
French Slag	6,5	15,0	24,0

(iii) Slag-Cement Blend

	7 Days MPa	28 Days MPa	90 Days MPa
SA Slag	8,0	28,5	36,0
French Slag	13,0	34,0	39,5

(iv) Slags activated by sodium hydroxide

	6 hours MPa	24 hours MPa
SA Slag	1,2	9,2
French Slag	8,9	14,2

CONCLUSION

The French slag is undoubtedly superior in strength to the South African Slag, particularly at early ages. In test (ii) the extremely large difference in strength results is due not only to the better hydraulicity of the French material but also to the fact that the French slag required considerably less water to achieve the required consistency of mix.

The reason for the superiority of the French slag is not known but we believe it may be due to its lower magnesia content. Other factors that may be responsible for the strength difference are the mineralogy and/or the methods of processing the slags.

M.B. Welsh

M.B. WELSH

PORTLAND CEMENT INSTITUTE

Laboratory report
Aggregate

ENT Reinforced Earth (Pty) Ltd

Ref WeE 80/488
Date 1980-03-26

RSE AGGREGATE

Sample Description				
Relative Density				
Bulk density loose (air dry) kg/m³				
Consolidated (air dry) kg/m³				
SCREEN ANALYSIS				
S A B S Screens	Per cent Passing (by mass)			
75,00 mm				
53,00 mm				
37,50 mm				
26,50 mm				
19,00 mm				
13,20 mm				
9,50 mm				
6,70 mm				
4,75 mm				

FINE AGGREGATE

Sample Description	SA Slag	French Slag		
Relative Density	2,52	2,64		
Bulk density loose (air dry) kg/m³	920	1 110		
Consolidated (air dry) kg/m³	1 450	1 280		
SCREEN ANALYSIS				
S A B S Screens	Per cent Passing (by mass)			
4 750 µm	100	100		
2 360 µm	98	95		
1 180 µm	74	73		
600 µm	33	30		
300 µm	11	19		
150 µm	8	3		
75 µm	3,0	0,5		
Fineness Modulus	2,76	2,90		

CONDITION OF TEST AND REPORT

While every care is taken to ensure the accuracy of all tests and reports on aggregates, neither the Institute nor its employees shall be liable in any way wha... for any error made in the execution or reporting of tests on aggregates or any erroneous conclusions drawn therefrom or for any consequence thereof

VISIT TO THE EJL LABORATORIES AT DOURDAN ON 19TH SEPTEMBER, 1980.

The purpose of the visit was to see the testing facilities at Dourdan and to continue consideration of the problems associated with the use of South African granulated blast furnace slags and fly ashes.

The first piece of information to come to light via a French reference work is that the temperatures at which bottom ash is formed are usually between 1500 and 1600 degrees C., but with low grade fuels temperatures are only from 900 to 1300 degrees C. Unless firing temperatures are high and the ash is quenched rather than air-cooled, it cannot be expected to have any significant cementitious properties. Firing temperatures and the cooling practice adopted at both Kelvin and Orlando should be investigated prior to undertaking any investigation of the properties of local bottom ashes.

Mr. Deligne reported on progress on tests on Iscor blast furnace slag and Kelvin fly ash. So far specimens have been made for future strength testing, but these are still in the process of curing. Some of the control tests on French materials are available. The programme will use a local sand as the aggregate and will comprise a comparison of Kelvin fly ash and Iscor blast furnace slag with two local French materials, using as activators both South African and French lime and South African and French gypsum. With regard to gypsum, the French gypsum will be a natural product, whereas the South African is phospho-gypsum. French experience indicates that phospho-gypsum is more effective as an activator than natural gypsum. I have arranged for EJL, France, to send us 20 kgs each of French slag and fly ash. They will attempt to match the chemical characteristics of both the slag and the fly ash with the South African products. Although the slag will be from a hamitite ore, it will be quenched with sea water rather than the fresh water used by Iscor, as all hamitite ores are imported into France and are therefore smelted at the ports of entry. The slag they will send us will probably be from Fosse sur Mer. A reciprocal set of comparative tests run in South Africa will give us a feeling for the characteristics of French materials.

DISCUSSION WITH LCPC REGIONAL LABORATORY AT TRAPPES

At this laboratory we discussed the reactivity of slag and fly ash, this time with Mr Astezan. With regard to slag, he made a number of interesting suggestions. Firstly, he mentioned that if slag is quenched at too low a temperature, i.e. below 1400°C, the reactivity is much reduced. This would not noticeably affect the cementitious properties of the slag in the form of slagment, but would have a dramatic effect on its reactivity in coarser form. It is very common in France to partly grind a slag until it contains ten to twenty percent of fines (fines being defined as material passing an eighty micron sieve). He suggested that as we have slagment available, we might attempt to activate the granulated slag by adding a small proportion of slagment. He questioned me closely about the pH and lime content of our mixes, and in discussion we agreed that it was possible that although the pH in our specimen had been adequate at the time of compaction, the slag may be deficient in CaO (which it appears to be in comparison with most reactive French slags) and it may therefore be necessary to add more lime than we have sofar thought necessary, i.e. the pH of the material may drop as hydration of the slag proceeds. However, as was pointed out, even if this procedure is technically successful, it will not be economically viable if the cost of the additional lime together with that the slag exceeds the cost of an equivalent content of ordinary Portland cement.

As far as fly ash is concerned, he stated that the chemical compositio of reactive fly ashes in France varies widely, and he supplied me with a table listing the chemical composition of a number of reactive fly ashes from France. He suggested that we compare the analyses of all available South African fly ashes with this table to settle once and for all the question of whether our ashes are chemically suitable or not. Apart from this, he was unable to add any further suggestions to those earlier made by Dr. Venuat.

DISCUSSION WITH DR. VENUAT - TECHNICAL DIRECTOR OF CERILH

CERILH is the French counterpart of the British C & CA or the South African PCI.

The objective of meeting Dr. Venuat was to discuss our lack of success in South Africa in using granulated slag and fly ash as cementitious materials. Dr. Venuat expressed surprise at our lack of success in both areas, but as the French do have non-reactive slag and fly ashes, he did not regard our experience as beyond the realms of possibility. He suggested that we compare the cementitious properties of fly ash from various precipitators, as it may not prove to be the case that the finest ash is the most reactive. In fact, he thought it possible that the ash which had cooled most quickly, and therefore was in a more glassy state would be the most reactive regardless of particle size. He also suggested that if the particle size was too coarse, we could try milling the ash. He also made the point that bottom ash is usually more reactive chemically than fly ash, because it contains far less unburnt material. If we were looking for a cementitious material, he suggested that we try milling a glassy quenched bottom ash. He also suggested that we attempt to carry out electron microscope studdies of the progress of hydration in slag and fly ash as this may give some clue as to what is going wrong. He made the further suggestion that we compare the properties of French and South African slags and fly ashes in a reverse experiment in which French materials are tested in South Africa, as this would eliminate any doubt as to differences in testing techniques.

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DISCUSSIONS IN PARIS WITH E.J.L. INTERNATIONAL.

These discussions were held between Mr. Tony Dimitri, Mr. Vivier of Salviem and myself on the 22nd June, 1981. They centred about the work that has been, and is to be, done in South Africa on the use of blast furnace slag and PFA as potentially cementitious materials.

On meeting Mr. Dimitri, I referred to the notes of our discussions during September, 1980, and pointed out to him that, whereas he had undertaken to send 20kg samples of French slag and flyash to South Africa, these had not yet arrived. Mr. Dimitri had been under the impression that the said samples had been dispatched to South Africa, and promised to follow up the question and to have the samples dispatched if they had not already been sent.

In discussion, it was agreed that the lack of reactivity of South African materials may be the fault of the lime. It therefore appears essential to match the properties of any South African lime used in future to those of lime used successfully in France. Both the chemistry and the physical characteristics should be comparable. It was agreed that a sample of a suitable French lime should be sent together with the slag and the PFA samples.

With regard to the work planned on Samancor slags, Mr. Vivier stated that a nickel slag produced in the French South Pacific island of Nou Mea has cementitious properties when quenched from a temperature of 1500°C. Two types of slag are available - what is known as a low furnace slag and an electric furnace slag. The chemical analyses of these two slags are as follows:

	SiO_2	FeO	MgO	$Al_2O_3+Co_2O_3$	NiO	CaO
Low Furnace	45-50%	8-12%	25-30%	5%	0.3-0.4%	8-12
Electric Furnace	51%	7%	38%	3.8%	0.2%	Negl gibl

It was suggested that I should put the values of the above analyses into our formula for slag reactivity as a test of its general validity. Apparently there is a similar formula which is commonly used in France, and Mr. Vivier undertook to look up the

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formula and to send it to me.

Mr. Vivier made two further suggestions that in any comparative test, we should compact to the same density to ensure that any strenc differences are not due to differences in density, and also that we should subject samples to scanning electron microscope studies after testing to assess the presence and progress of any cementitiou: reactions. He also mentioned that the use of crushed air-cooled slag together with activated quenched slag, may result in good performance, as one might expect the air-cooled slag to exhibit a low level of reactivity especially if stabilised with an activated quenched slag. He also mentioned that phosphogypsum may reduce the pH of a stabilised mixture because of the traces present of phosphoric acid. It was agreed that in any comparative tests, we should check on the changes in pH by carrying out Edes tests on the material that corresponds to each set of strength tests

Regarding our work on Dunswart steel slag, Mr. Vivier cautioned that slags produced mainly from scrap as at Dunswart, tend to be highly variable in properties and are therefore not promising for use either as cementitious materials or as aggregate. He suggested that our proposed programme with Samancor was much more promising. With regard to this programme, he also suggested that we carry out 180 day tests in addition to the 56 day tests already on the programme. With regard to this programme, he strongly advised that we try to assess the level of glassiness of the experimentally quenched Samancor slags, possibly using X-ray defraction or optical microscope techniques. I feel that I should discuss this problem with Professor John McIver of our Geology Department.

We then discussed the apparent water-reducing effect of PFAs in concrete. The feeling of Mr. Vivier was that the addition of PFA to a normal concrete mix reduces the water demand of the mix purely by improving the grading. He felt that the addition of any other fines of similar grading would probably have the same effect, and in fact, if an inert non-water absorbent fines was used rather than a water absorbent material such as PFA, the effect would probably be even more marked. He did not feel that any "ball-bearing action resulting from the spherical character of the PFA particles would be significant.

...../3

As a result of these discussions and later talks with Mr. Dimitri, I came to the conclusion that it is essential that we carry out carefully planned and documented comparative tests between French slags and PFA and corresponding South African materials in order to decide whether to continue with testing of South African materials or whether to abandon them. Careful tests of the slag and flyash prior to their incorporation into mortars should be carried out. For the slag, electron microscope studies and an assessment of the degree of glassiness, as well as comparative chemical analyses should be performed. Whereas for the flyash, comparative chemical analyses and scanning electron microscope studies should be carried out. During the tests, studies of the variation of pH of the mix with time and also scanning electron microscope studies to follow the progress of these cementitious reactions should be carried out. The resulting tests may prove expensive, but their results will be invaluable in deciding whether or not we are flogging a dead horse in persisting with the idea of using South African PFA and granulated blast furnace slag as cementitious materials.

I also feel at this stage that it is essential to bring in an experienced petrologist to assist us with the determination of the state of glassiness of the slag. I recommend that we involve Professor John McIver, as he is not only a top class petrologist but is extremely interested in engineering applications of petrolog· He has, for example, collaborated extensively with Dr. Derek Davis on the concrete making properties of South African natural aggregate and he has also been collaborating with me on the problem of alkali-aggregate reaction in concrete.

I also asked Mr. Dimitri about progress with the use of LH38. He told me that as far as he is aware, there are no problems with its use, although at the moment it is being produced only at one locality. There is at present an extensive stretch of motorway being built using LH38. He also told me that whereas previously it had been considered that the use of granulated slag activated wit lime in order to stabilise gravel road bases had been thought to reduce shrinkage cracking, this now appeared to be false. Shrinkag cracking experienced when lime activated granulated blast furnace slag is used as a stabiliser appears to be every bit as severe as that experienced using Portland cement. He told me that a typical

,....../4

crack pattern would have a crack spacing of six to nine metres with crack widths of up to 10mm. It is necessary to use a minimum of 140mm of asphalt in order to prevent this cracking from reflecting through to the surface.

Summary – Properties of Dunswart Steel Slag

SABS 1083–1976 Requirement	Slag Properties: New	Slag Properties: 6 months	Slag Properties: 1–2 years
10% FACT – Dry Surface dressing 210 kN Bituminous mixes 160 kN Concrete 70 – 110 kN	Meas: 205 kN Est'd* 237 kN	100 kN	212 kN
ACV – ~~Dry~~ Dry Surface dressings 21% Bituminous mixes 25% Concrete 29%	Dry: 19% Wet: 19%	29% 30%	21% 21%
Total soluble salts – (not specified, but no more than 1% recommended)	2,06%	0,94%	1,04%
Water absorption – (not specified, but up to 1% considered negligible) > 4,75 mm < 4,75 mm	 3% & 6,3% 6,8%	 4,6% & 12,6% 17,3%	 15,9% & 15,9% 18,1%
Dry bulk density All > 4,75 mm < 4,75 mm	2,03 3,20 2,89	1,12 1,79 1.85	0,97 1,54 1,63
Apparent relative density All > 4,75 < 4,75	3,66 3 78 3,59	2,65 2 37 2,72	2,29 2,05 2,35

* From ACV via $10\%\text{FACT} = 12{,}5(38 - \text{ACV})$.

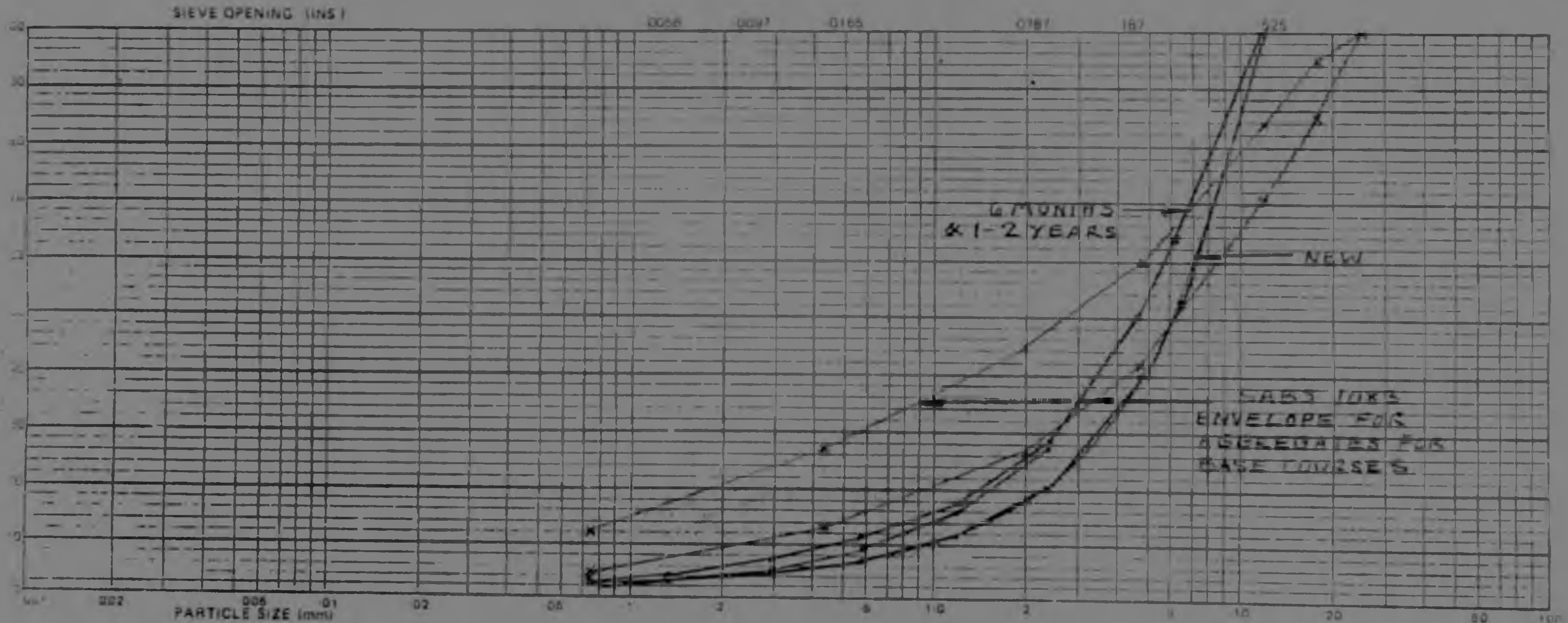
UNIVERSITY OF THE WITWATERSRAND

DEPARTMENT OF CIVIL ENGINEERING – SOILS LABORATORY

PARTICLE SIZE DISTRIBUTION

DATE 6/6/80

PROJECT: Aggregate Tests BOREHOLE No. ______ DEPTH / SAMPLE No. Marked New



	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	
CLAY	SILT FRACTION			SAND FRACTION			GRAVEL FRACTION			

UNIVERSITY OF THE WITWATERSRAND, JOHANNESBURG
DEPARTMENT OF CIVIL ENGINEERING.
Enquiries
Telephone (011) 716-2470
Our ref GEB/sm
Your ref

APPENDIX 17

Date 18th December, 1981.

Mr. R.J. Dismore,
Projects Manager,
Samancor Management Services (Pty) Ltd.,
P.O. Box 8186,
JOHANNESBURG.
2000

Dear Mr. Dismore,

PROGRAMME OF TESTS TO ESTABLISH PROPERTIES OF SLAGS AS AGGREGATES

For a number of reasons, the performance of these tests has been far more protracted than I would have wished. However, I can now report on the results of one series of tests to establish the aggregate crushing value of the various slags. The SABS requirements for aggregate crushing value are maximum values of the following:

1) For surface dressings - 21%
2) In bituminous mixes - 25%
3) In concrete - 29%.

The values recorded for the five slags under test, as well as Witwatersrand quartzite as a standard of comparison, are as follows:

MATERIAL	ACV DRY	ACV WET
Witwatersrand quartzite	13,6%	13,9%
Ferrochrome slag	18,2%	21,2%
Ferromanganese slag	29,7%	31,0%
Silicon manganese slag	27,7%	30,3%
Phosphate slag	18,6%	25,8%

As you will see from the table, it is only the ferrochrome and phosphate slags that appear worthy of further examination. The ferromanganese and silicon manganese slags appear to be too soft to meet normal aggregate requirements. It appears that at this stage, we should pause and assess the situation before carrying out any further tests in the series, although I should mention that the series of 10% FACT tests are already underway.

Yours sincerely,

G.E. Blight

G.E. BLIGHT.

c.c. Mr. L. Dison, E.J.L. (S.A.) (Pty) Ltd., P.O. Box 91231 AUCKLAND PARK

APPENDIX 1B

[illegible]

A REPORT TO

S[illegible] NCL[illegible] MANAGEMENT SERVICES (PTY)LTD,

AND

EJL (SA)(PTY) LTD

BY

G.E. BLIGHT

Department of Civil Engineering,

University of the Witwatersrand,

Johannesburg

February, 1982

1. INTRODUCTION

A survey of the technical literature indicates that the cementitious properties of slags depend on a combination of their chemical composition and glass content. To represent the chemical composition, a modulus M is used, where

$$M = \frac{CaO + MgO + xAl_2O_3}{(1 - x)Al_2O_3 + SiO_2} \qquad (1)$$

in which each term CaO, etc represents the mass percentage of the corresponding component in the slag, and x is usually taken as 1/3. A number of alternative forms of this modulus have been proposed, but all contain the terms CaO, MgO, Al_2O_3 and SiO_2. It is also apparent from the literature that FeO and MnO have adverse effects on strength development and to take account of this, one worker has suggested the modulus

$$M_s = \frac{CaO + 1/2MgO + Al_2O_3}{SiO_2 + FeO + (MnO)^2} \qquad (2)$$

The hydraulic (i.e. cementitious) properties of the slag are measured by means of an index

$$I = 100\frac{(a - c)}{(b - c)} \qquad (3)$$

where a = strength of an ordinary portland cement (OPC)/milled slag mixture;

c = strength of a similar OPC/milled quartz mixture; and

b = strength of OPC by itself.

I has been experimentally related to G, the percentage glass content of the slag by the empirical relationship

$$I = 0,38\ G\ (M - 0,72) + 75 \qquad (4)$$

If one is/...

If one is investigating the suitability of slags for making cement, one can equate I = 100% and then solve for G to see if a realistic percentage of glass is indicated to achieve an ideal result. The transposed equation for I = 100 is:

$$G = \frac{66}{(M - 0,72)} \qquad (5)$$

Alternatively, the modulus M itself can be used to assess the hydraulic properties of the slag according to Table 1 (below)

TABLE 1

Range of Modulus M	Hydraulic Properties
$<1,1$	Poor
1,1 - 1,5	Medium
1,5 - 1,9	Good
1,9	Excellent

Before embarking on the research project, a preliminary assessment of the hydraulic potential of Samancor slags was undertaken. Table 2 summarizes M and G (I = 100%) for 4 slags of known cementitious value as well as 4 Samancor slags and 1 Highveld slag.

2. PREPARATION OF SLAGS FOR TESTING

Because of the importance of obtaining slags with as high a glass content as possible, each slag sample was quenched by pouring from a molten state into a relatively large volume of water. The quenched slag sample was then chemically analysed and examined for glass content by two methods:

(i) A small quantity of powdered quenched slag was immersion-mounted

between microscope slides/...

between microscope slides in bromoform (which has a suitable refractive index). In transmitted light, crystalline particles are transparent while glassy particles are opaque. It is then a simple matter to assess the percentage of glassy material by counting transparent and opaque grains.

(ii) An X-ray diffraction analysis was carried out on the powdered quenched slag. A complete absence of "peaks" on the diffractogram would indicate a completely glassy material, while the presence of peaks indicates the presence of crystalline components.

These two examinations were carried out by Professor J.[illegible]. [illegible]cIver whose two reports are attached. Portions of the X-ray diffractograms appear as Figures 1a, b, c and d.

Table 3 summarizes the chemical and glassy material analyses for the four slags as well as giving values for the moduli M and M_{s}, values for I (calculated from equation (4)) and an assessment of the hydraulic properties of the slag according to Table 1.

When the ferro-manganese slag was quenched, two distinct components arose: the larger particles were "frothy" and a greenish colour, while the smaller particles were denser and brown coloured. Microscopic examination showed that the green material was crystalline and the brown glassy. It proved possible to achieve a reasonable separation of the two components by sieving through a 2mm sieve. However, as the peaks on the X-ray diffractograms in Figure 1a show, the sieved material still contained a small proportion of crystalline material.

Two samples of ferromanganese slag were prepared. As the one had a

considerably/...

darker colour than the other, it was decided to test both in order to give an indication of the extent of variation in slag properties that can be expected from batch to batch from the same furnace.

The quenched ferrochrome, phosphorus and silicon manganese slags all appeared to be homogeneous materials, but as Figures 1b and 1c show, both the ferrochrome and phosphorus slags contain appreciable crystalline proportions. Ironically, the least promising quenched slag from a chemical standpoint, silicon-manganese, is shown by Figure 1d to be completely glassy!

Based on the modulus [illegible], values of I tabulated in Table 3 appear quite promising. Even a slag/OPC blend that will produce 76% of the strength of an equivalent pure OPC is a useful material, especially in low-grade applications such as soil stabilization. The reference to the strength-depressing effects of MnO and FeO was only discovered after the testing programme was well under way, but even if it had been discovered earlier it is doubtful that the testing programme would not have been undertaken because of the discouraging predictions of equations (2) and (4) (see Table 3).

After the glassiness determinations and chemical analyses had been carried out, the slags were milled to a powder in a small ball mill. Milling proved unexpectedly difficult and it did not prove possible to mill the quenched slags to a fineness equal to that of Slagment or ordinary Portland cement (OPC). Table 4 summarizes and compares the fineness characteristics of the milled slag with those of Slagment and OPC.

3. STRENGTH OF CUBES INCORPORATING SLAG

The strength characteristics of OPC/milled slag mixes were investigated by preparing a series/...

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within [illegible]

3. STRENGTH OF CUBES INCORPORATING SLAG

The strength characteristics of OPC/milled slag mixes were investigated by preparing a series/...

by preparing a series of 100 mm cubes of crusher sand and "cement". The mixes consisted of:

(i) a control series consisting of 5 parts sand to 1 part OPC by mass of dry material;

(ii) a second control series consisting of 5 parts sand to 0,5 part OPC and 0,5 part Slagment;

(iii) A series for each slag consisting of 5 parts sand to

(a) 0,83 part OPC and 0,17 part slag

(b) 0,67 part OPC and 0,33 part slag

(c) 0,5 part OPC and 0,5 part slag.

The cubes were cured in water at 20 °C until due for testing and were tested at ages of 7, 28, 56 and 130 days, 3 cubes of each mix being tested at each age.

The results of the control tests are shown in Figure 2a while those for the tests on slag are shown in Figures 2b, 2c, 2d and 2e. Each of Figures 2b to 2e also show the strength-time curve for the sand-OPC cubes.

Figures 3a, b, c, d and e summarize the results of the tests at 130 days. In general, the very disappointing conclusion is reached that if X% of the OPC is replaced by any of the slags tested in this series, the strength will be reduced by X%.

The most promising slag appears to be the phosphorus slag which displayed some small cementitious effect even though the quenched material was only 10% glassy. However, it is understood that apart from the low tonnage of this slag that is produced, it is hazardous to quench.

4. CONCLUSION/...

4. CONCLUSIONS

The reluctant conclusion is that none of the four slags investigated show promise as cementitious materials. Although predictions based on the modulus M were interesting, the modulus Ms was less encouraging. It therefore appears that the MnO levels in the Ferromanganese and Silicomanganese slags are a likely reason for their poor performance.

TABLE 2

Type and Source of Slag	CaO	MgO	Al_2O_3	SiO_2	M	G(I=100)	Cementitious
UK blast furnace	36 43	4 11	22 12	36 28	0,93 1,61	74	yes
Canadian blast furnace	40	11	9	37	1,26	122	yes
French blast furnace	35 45	11 4	15 15	35 31	1,13 1,32	136	yes
Iscor (Pta) blast furnace	30	20	14	33	1,29	116	yes
Highveld Ferrochrome	43	11	16	27	1,57	78	Doubtful
Samancor Ferro-manganese	33	8	4	33	1,17	147	?
Samancor Phosphate	51	-	3	40	1,24	127	?
Samancor Silicon-manganese	24	12	3	48	0,74	3300	?
Samancor Ferro-chrome	6	25	24	27	0,91	347	?

In the light of the values given in the above tabulation, the Samancor phosphate and ferromanganese slags appeared to be the most promising for further investigation, but it was decided, in addition to test the silicon-manganese and ferro-chrome slags.

TABLE 3

SUMMARY OF PROPERTIES OF QUENCHED SLAGS.

SLAG	CaO %	MgO %	Al_2O_3 %	SiO_2 %	FeO %	MnO %	G %	M (eqn 1)	M_s (eqn 2)	I (eqn 3 eqn 4)	Hydraulic Properties (Table 1)
Ferromanganese (Dark)	28,6	5,4	2,9	28,4	0,06	33,1	95	1,15	0,03	M - 91 M_s - 50	M - medium M_s - poor
Ferromanganese (Light)	30,1	6,0	3,2	33,3	0,07	25,9	90	1,05	0,05	M - 86 M_s - 52	M - poor M_s - poor
Ferrochrome	3,5	25,0	24,1	23,0	7,5	-	40	0,94	1,31	M - 78 M_s - 81	M - poor M_s - medium
Phosphorus	50,2	-	2,9	40,4	1,3	-	10	1,21	1,27	M - 77 M_s - 77	M - medium M_s - medium
Siliconmanganese	23,0	13,7	2,9	48,3	0,03	10,6	99	0,75	0,20	M - 76 M_s - 55	M - poor M_s - poor

[illegible]

Material	% Passing sieve No		
	52	100	200
OPC	99,6	98,9	86,8
Slagment	99,6	99,2	94,7
Siliconmanganese	100	97,4	65,8
Ferromanganese	99,9	84,7	54,7
Ferrochrome	95,3	61,8	38,4
Phosphate	92,5	58,8	35,8

FINENESS OF MILLED SLAG

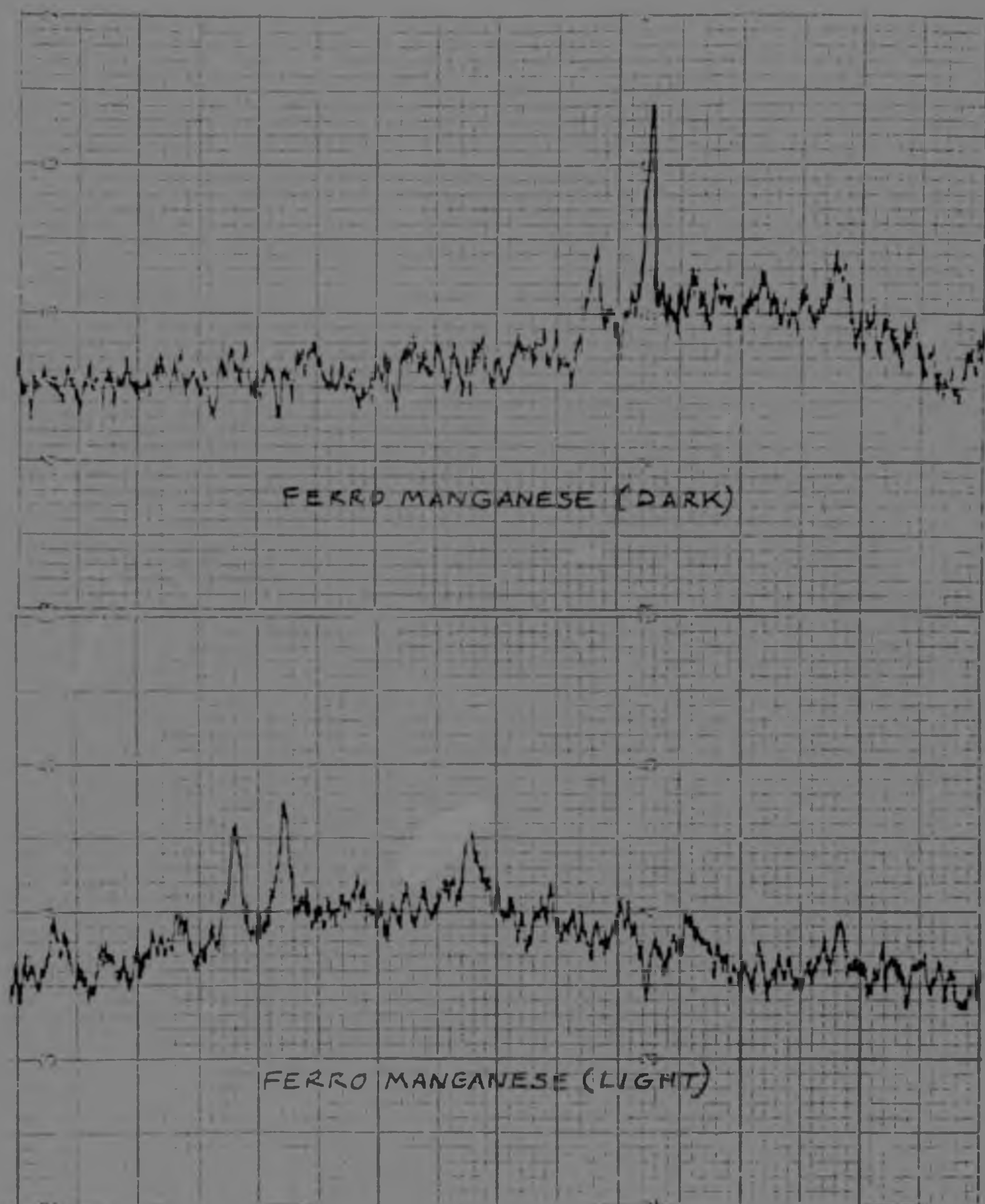


FIGURE 1a

PARTIAL X-RAY DIFFRACTOGRAMS FOR QUENCHED FERROMANGANESE SLAGS.

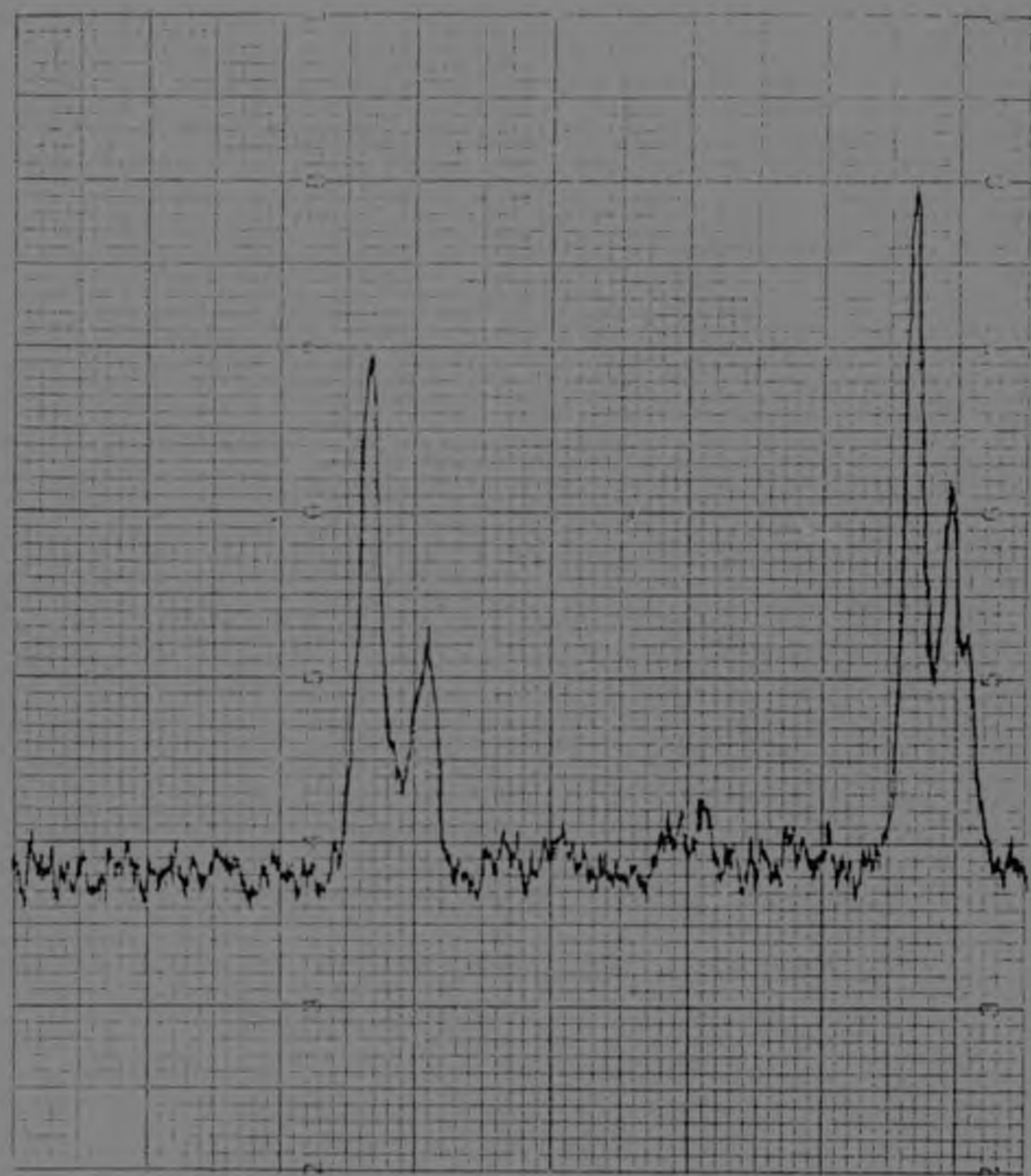


FIGURE 1b

PARTIAL X-RAY DIFFRACTOGRAM FOR QUENCHED FERROCHROME SLAG

FIGURE 1b

PARTIAL X-RAY DIFFRACTOGRAM FOR QUENCHED FERROCHROME SLAG

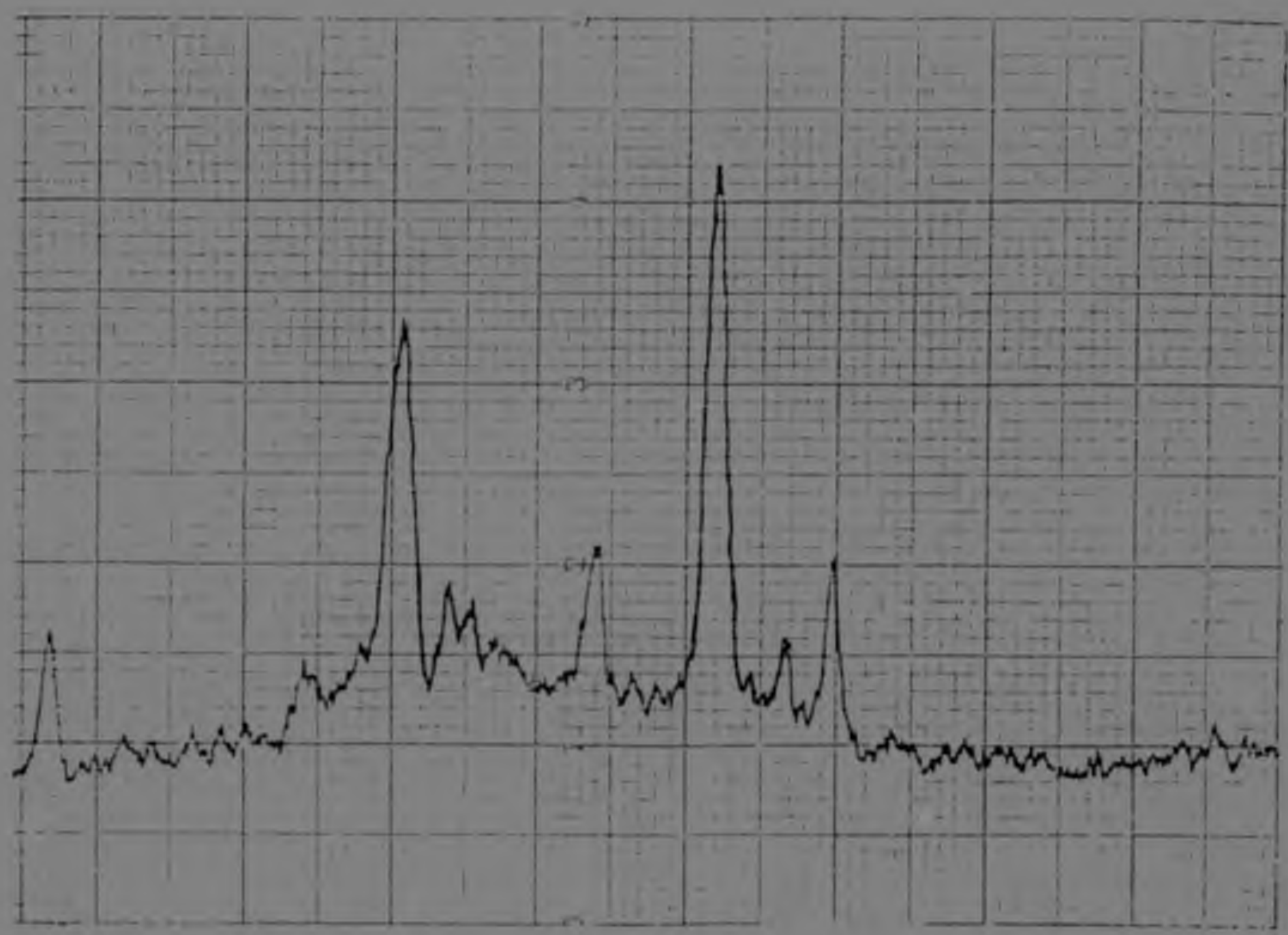


FIGURE 1c

PARTIAL X-RAY DIFFRACTOGRAM FOR QUENCHED PHOSPHORUS SLAG

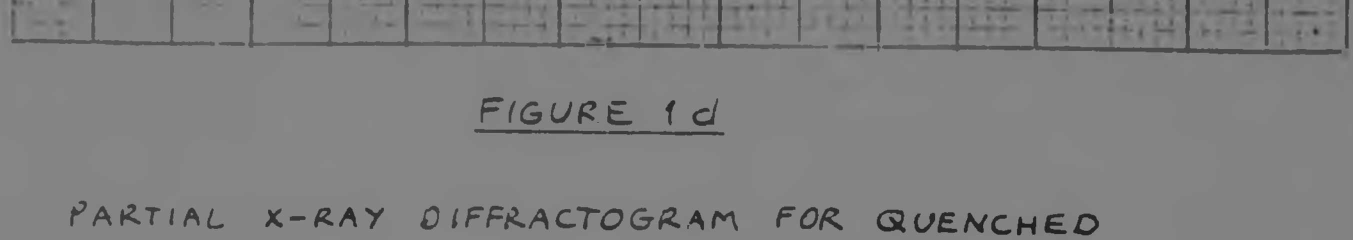


FIGURE 1d

PARTIAL X-RAY DIFFRACTOGRAM FOR QUENCHED SILICONMANGANESE SLAG

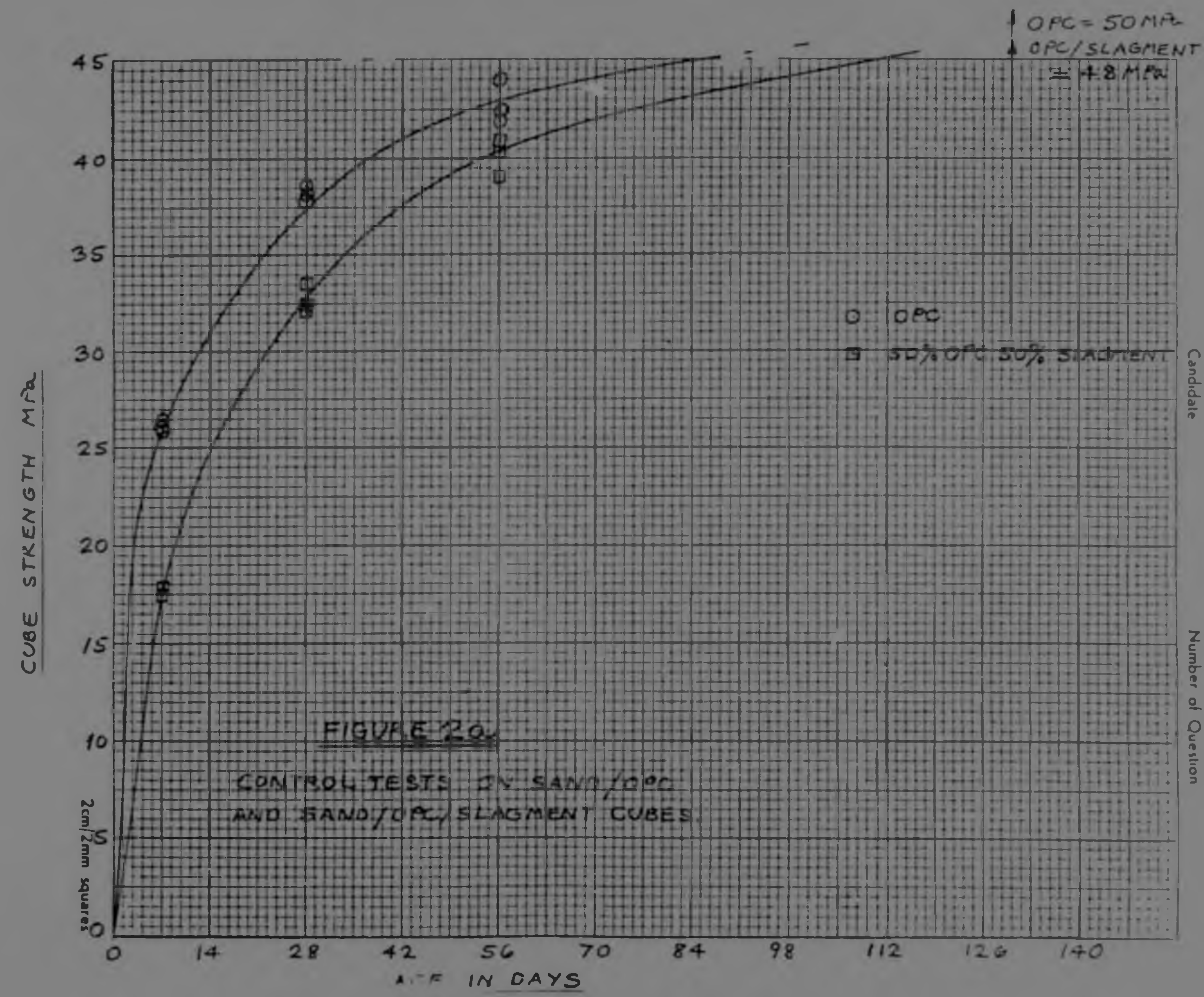
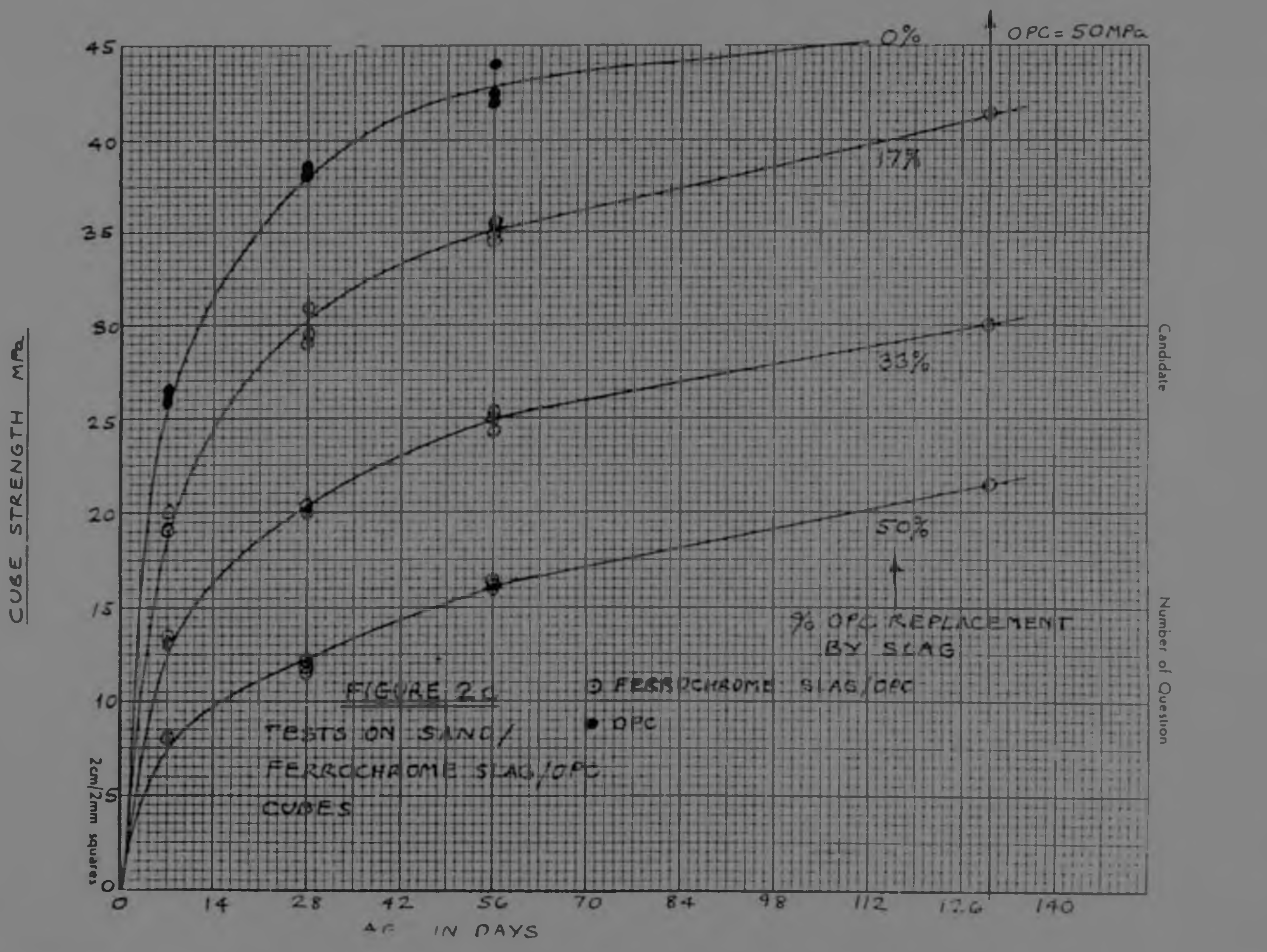
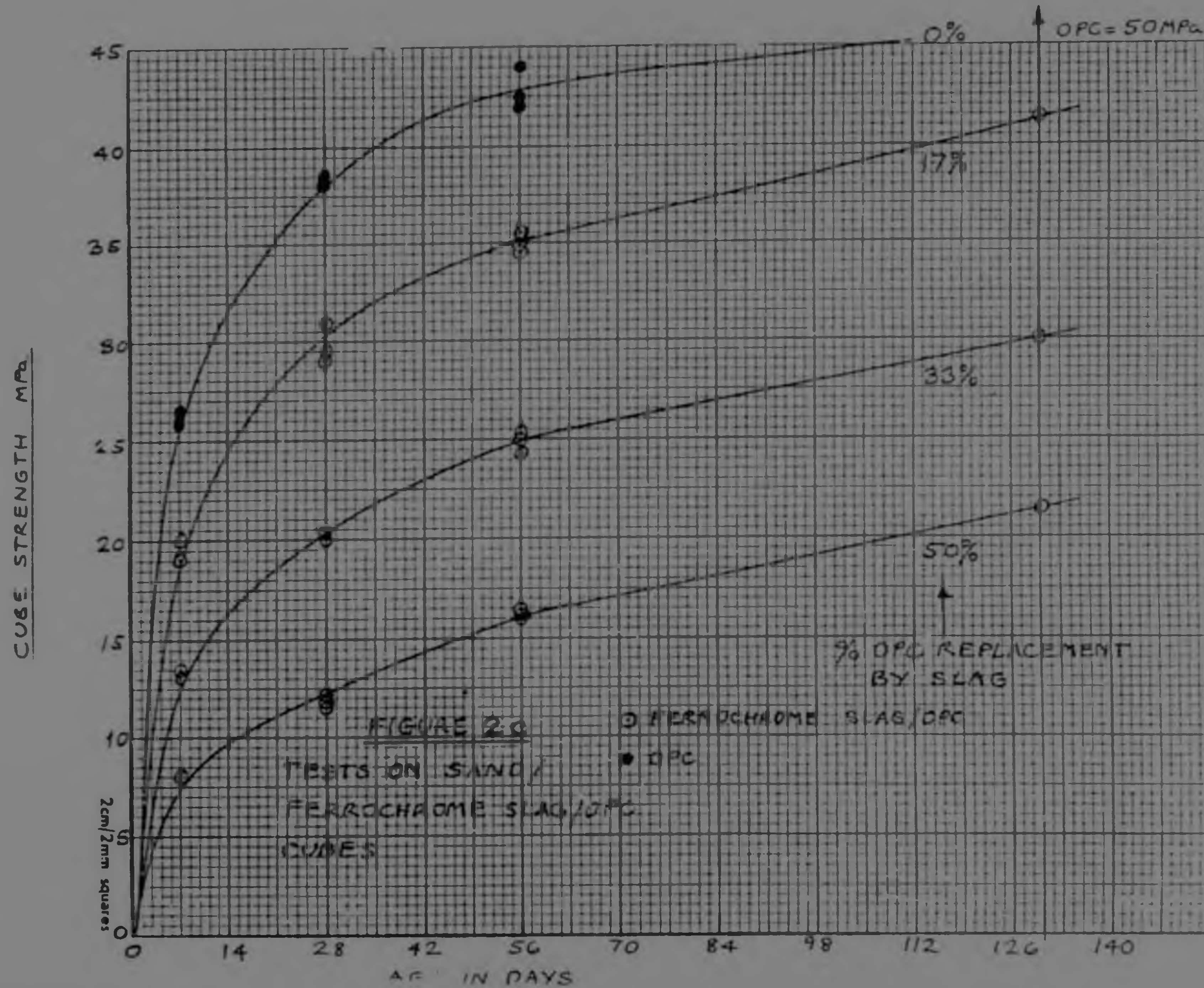


FIGURE 2a.
CONTROL TESTS ON SAND/OPC
AND SAND/OPC/SLAGMENT CUBES
OPC = 50 MPa
OPC/SLAGMENT = 48 MPa
OPC
50% OPC 50% SLAGMENT
CUBE STRENGTH MPa
AGE IN DAYS
0
5
10
15
20
25
30
35
40
45
14
28
42
56
70
84
98
112
126
140
2cm/2mm squares
Candidate
Number of Question



CUBE STRENGTH MPa
AGE IN DAYS
0%
17%
33%
50%
OPC = 50MPa
% OPC REPLACEMENT BY SLAG
FIGURE 2c
TESTS ON SAND/ FERROCHROME SLAG/OPC CUBES
FERROCHROME SLAG/OPC
OPC
Candidate
Number of Question
2cm/2mm squares



Candidate

Number of Question

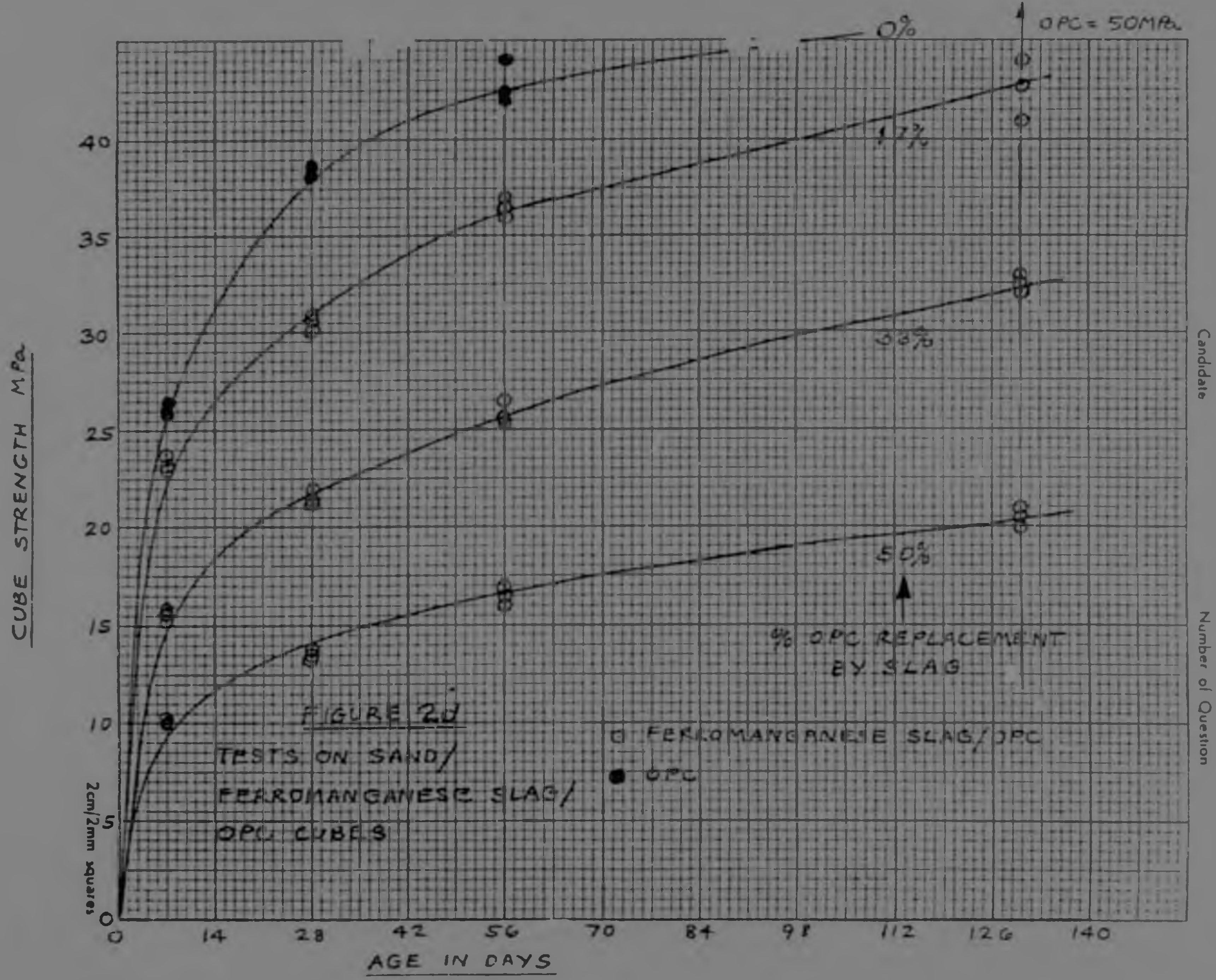
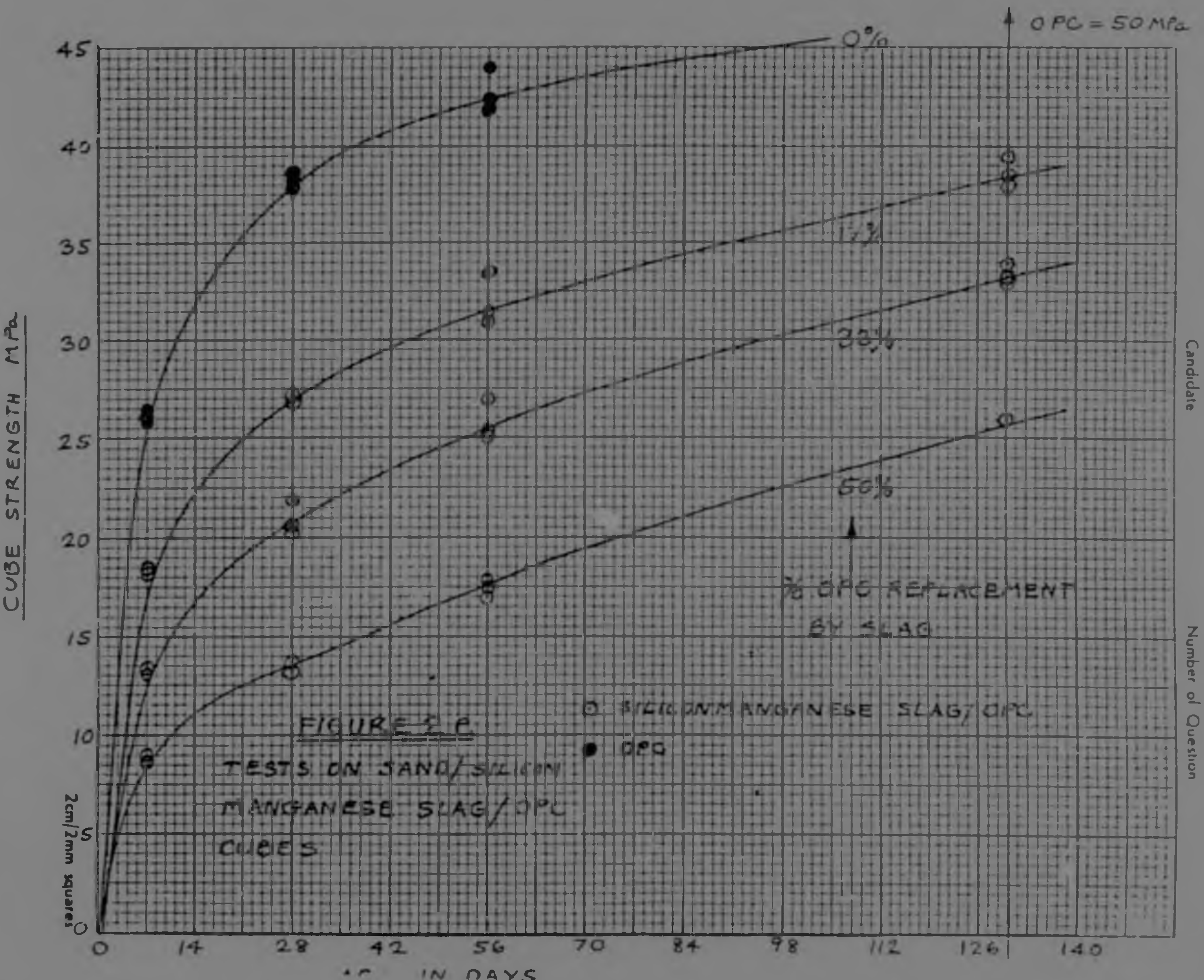
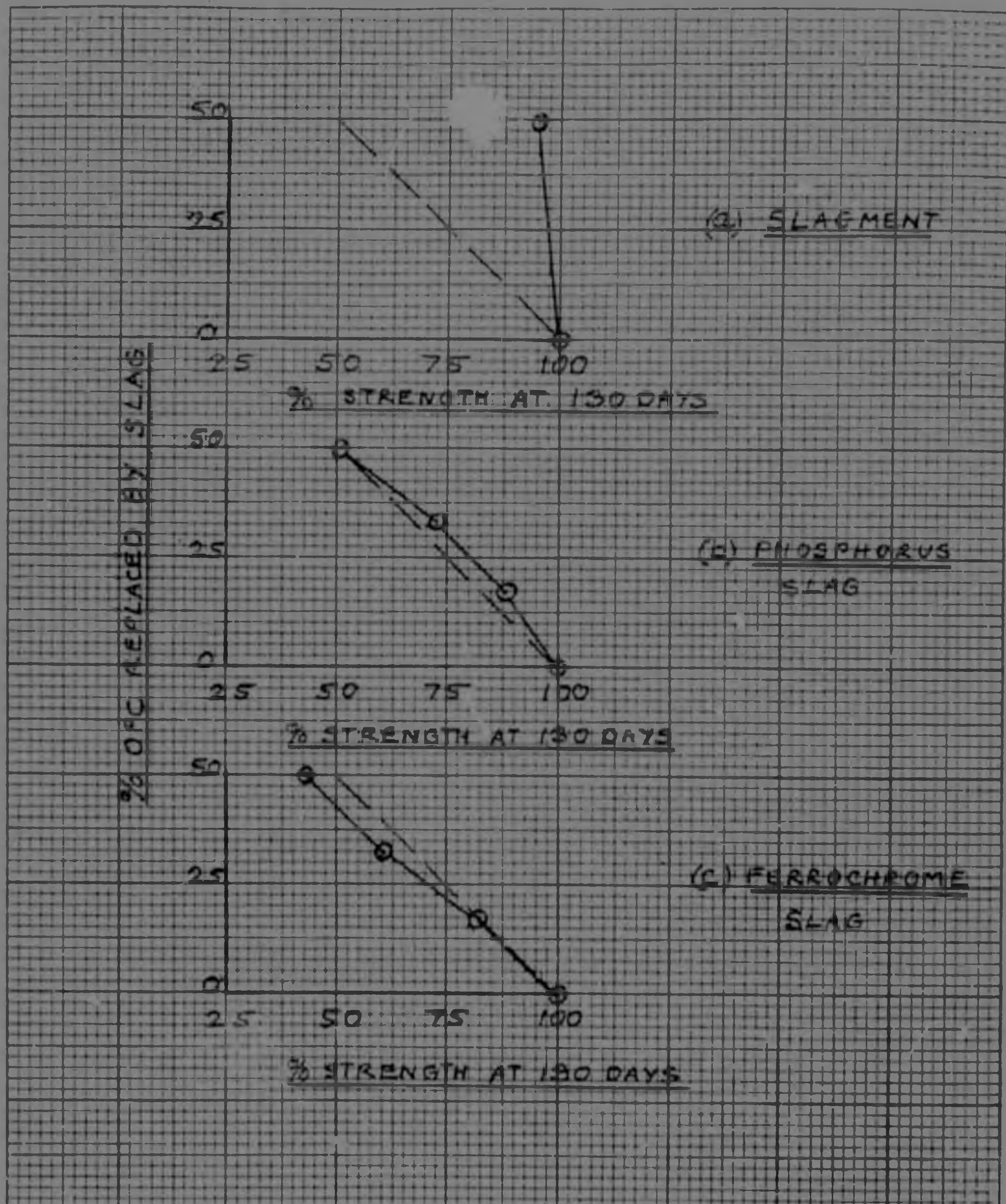


FIGURE 2d
TESTS ON SAND/
FERROMANGANESE SLAG/
OPC CUBES
CUBE STRENGTH MPa
AGE IN DAYS
0
14
28
42
56
70
84
98
112
126
140
5
10
15
20
25
30
35
40
0%
OPC = 50MPa
17%
33%
50%
% OPC REPLACEMENT
BY SLAG
FERROMANGANESE SLAG/OPC
OPC
2cm/2mm squares
Candidate
Number of Question


OPC = 50 MPa
0%
17%
33%
50%
% OPC REPLACEMENT
BY SLAG
FIGURE 2.6
TESTS ON SAND/SILICON
MANGANESE SLAG/OPC
CUBES
SILICONMANGANESE SLAG/OPC
OPC
CUBE STRENGTH MPa
IN DAYS
2cm/2mm squares
Candidate
Number of Question

Candidate Number of Question

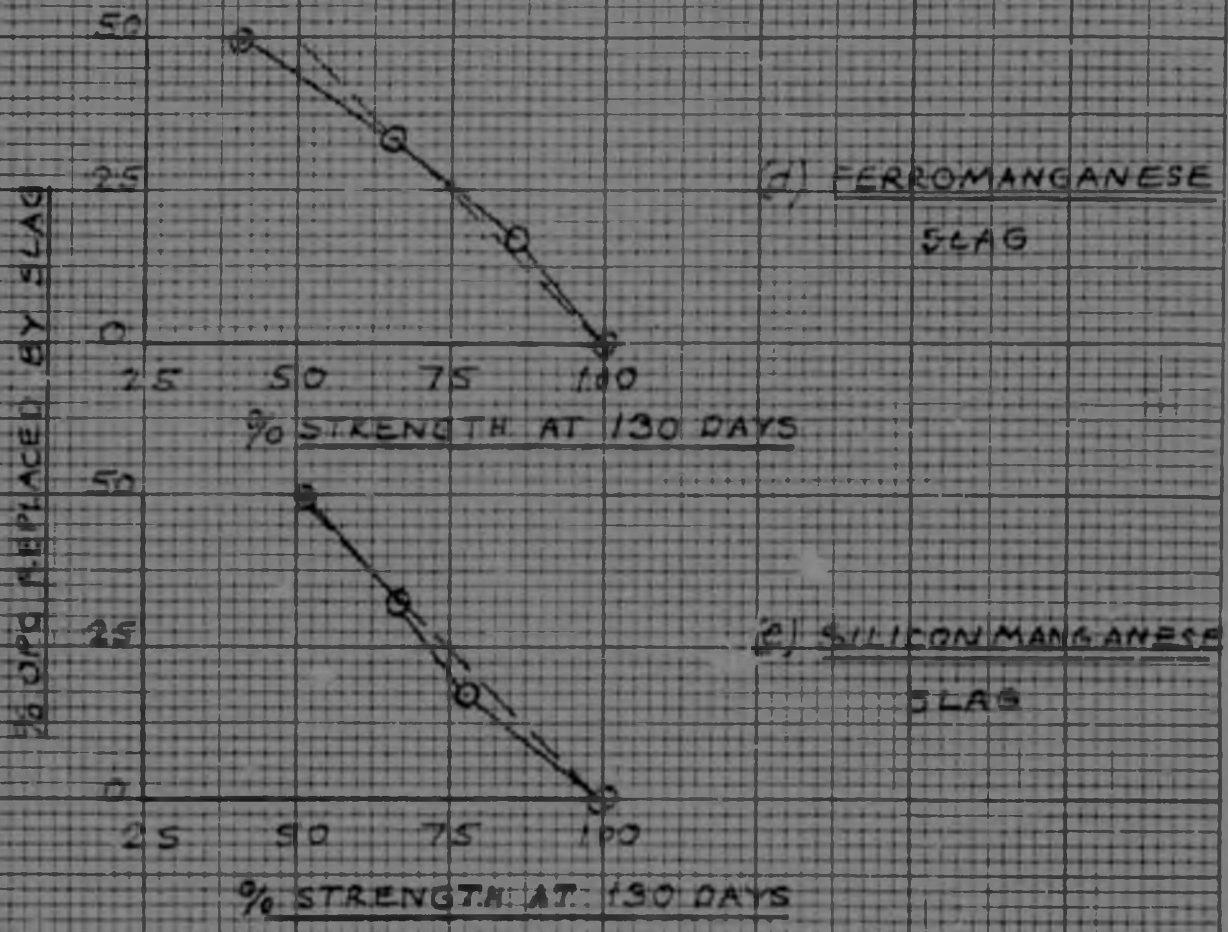


FIGURES 3 a, b, c

2cm/2mm squares

% OPC REPLACEMENT VERSUS % STRENGTH

Candidate Number of Question



FIGURES 3 d, e

% OPC REPLACEMENT VERSUS

% STRENGTH

2cm/2mm squares

Report on Slag Samples

Preamble

Three samples of slag were submitted by Professor G. Blight with the request that the crystallinity of the materials be investigated and compared. To this end, the sample materials were studied in immersion mounts using conventional petrographic techniques. X-ray diffractograms were prepared from the sample constituents and are attached to this report. The results of these investigations are presented below.

Results

Sample 62 Sima slag

This sample comprised white, granular, vesiculated slag which under the microscope was found to consist virtually entirely of glass together with trace amounts (less than 1%) of crystalline material. Reference to the relevant X-ray diffractogram prepared from this sample shows it to be essentially isotropic to X-ray diffraction with the crystalline phases being too sparsely present for their presence to be recorded.

Sample 64 Fema slag

This sample comprises a granular vesicular slag material in which a green and a brown phase are macroscopically recognisable. Microscope study of the green phase showed it to be composed essentially of a felty textured assemblage of crystalline materials together with rare glass. The relevant X-ray diffractogram displays a well defined diffraction pattern indicating the high degree of crystallinity of the green slag phase. Analysis of the pattern shows the crystalline material to be dominated by a phase having a composition between that of monticellite ($CaMgSiO_4$) and kirschsteinite ($CaFeSiO_4$). In addition the presence of minor additional diffraction peaks indicates the presence of another phase or phases which it has not been possible to identify.

In contrast to the green phase, the brown phase of the slag was found to consist of glass accompanied by trace amounts of crystalline material. This observation is reflected in the relevant X-ray diffractogram where the presence of crystalline materials is very weakly indicated.

Sample 9

As in the case of the previous sample, a green and a brown phase are macroscopically recognisable in this sample. The green phase is again dominated by a felted aggregate of crystalline materials with only rare glass. The relevant X-ray diffractogram confirms the high degree of crystallinity of the green phase and shows the crystalline phase to be the same as that present in the green fraction of 64 Fema slag. The brown fraction of Sample 9 is dominated by glass which is accompanied by minor (c 2 - 5%) of crystalline material. This distribution is reflected in the relevant X-ray diffractogram, where the presence of crystalline materials is but weakly indicated.

Comment

On a comparative basis it may be noted that the 62 Sima slag is virtually noncrystalline. The remaining two samples comprise both crystalline and noncrystalline fractions. The crystalline (green) fraction of 64 Fema slag makes up 40 - 50% of this sample while in Sample 9 the crystalline fraction makes up 10 - 15%. Sample 9 and 62 Sima slag thus constitute the least crystalline of the three slag samples.

J.R. McIver
Department of Geology
University of the Witwatersrand 1 August 1981

Second Report on Slag Samples

Preamble

Five samples of slag were submitted by Professor G. Blight with the request that the crystallinity of the materials be investigated and compared. To this end, the sample materials were studied in immersion mounts using conventional petrographic techniques and X-ray diffractograms were prepared from all samples. The results of these investigations are presented below.

Results

Sample Sieved -2mm SiMn slag

This sample comprises granular, white, vesicular slag which under the microscope was found to consist virtually entirely of glass, with crystalline materials being present in trace amounts (c. 1%) only. Reference to the relevant X-ray diffractogram prepared from this sample showed it to be essentially isotropic to X-ray diffraction. Overall the sample is identical to Sample 62 Sima slag reported on in a report dated 1 August 1981.

Sample Sieved -2mm FeMn slag (dark)

This sample consists of brown and green particles of vesicular slag. Grain counts of 100 grains of a random sample of this material indicates that the green phase makes up c. 5% of the total sample. Under the microscope the brown phase was found to consist dominantly of glass accompanied by trace amounts of crystalline material. The green phase in contrast was found to be dominated by a felty textured assemblage of crystalline material with only minor amounts of glass. Overall, therefore, this sample is dominated by glass. The X-ray diffractogram prepared from this sample confirms the foregoing with the presence of crystalline material being only weakly indicated. The diffractogram is to be compared with the diffractogram of Sample 9, brown fraction of the report dated 1 August, 1981. The slight increase of diffraction intensity exhibited by the sieved sample is in accordance with the presence of green crystalline slag in this sample; no such material was present in the X-rayed brown material from Sample 9, brown fraction.

Sample Sieved -2mm FeMn slag (light)

This sample again consists of green and brown vesicular slag particles. Grain counts of 100 grains of a random sample indicates that the green phase makes up c. 10% of the total sample. Under the microscope the brown phase was found to consist dominantly of glass which is accompanied by only minor amounts of crystalline material. The green phase was found to be dominated by a felty assemblage of crystalline materials with only rare glass. Overall, therefore, this sample is again dominated by glass. The X-ray diffractogram prepared from this sample confirms the foregoing with the presence of crystalline material being only weakly indicated. This diffractogram is to be compared with that of the diffractogram of Sample 64 Fema slag, brown fraction of the report dated 1 August 1981. The increase in diffraction intensity exhibited by the sieved sample is in accordance with the presence of green crystalline slag in this sample; no such material was present in the X-rayed brown material from Sample 64 Fema slag, brown fraction.

Sample P slag

This sample comprises fragments of dark grey slag. Under the microscope this material was found to be dominated by an assemblage of crystalline materials with at least two phases being optically recognisable; glass is sparingly present and does not exceed 10% by volume of the material studied. The

X-ray diffractogram prepared from this sample confirms the above with a well defined diffraction pattern (patterns?) being apparent.

Sample FeCr

This sample comprises a black vesicular slag. Under the microscope it is seen to consist of a mixture of finely crystalline material together with a semi-opaque phase which has the optic properties of glass. From optic study the proportions of crystalline material and glass are estimated at c. 60 : 40. The presence of an abundant crystalline phase (phases) in the slag is confirmed by the relevant X-ray diffractogram which displays a sharply defined diffraction pattern.

Comment

On a comparative basis, it may be noted that the sieved -2mm SiMn slag is virtually non crystalline while the sieved -2mm dark and light FeMn slags are characterised by the presence of slightly more crystalline materials. Both P and FeCr slags are dominated by crystalline phases.

J.R. McIver
Department of Geology
University of the Witwatersrand
Johannesburg
8 August 1981

United States Patent [19]

Hahn

[11] **4,425,057**

[45] **Jan. 10, 1984**

[54] **METHOD OF MINING**

[75] Inventor: **John A. Hahn**, Johannesburg, South Africa

[73] Assignee: **IPI Contractors AG**, Switzerland

[21] Appl. No.: **200,186**

[22] Filed: **Oct. 24, 1980**

[30] **Foreign Application Priority Data**

Oct. 26, 1979	[ZA]	South Africa	79/5752
May 14, 1980	[ZA]	South Africa	80/2884
Aug. 12, 1980	[ZA]	South Africa	80/4920
Oct. 26, 1980	[ZA]	South Africa	80/5591

[51] **Int. Cl.**³ **E02D 15/00**

[52] **U.S. Cl.** **405/258**; 405/288; 405/290

[58] **Field of Search** 405/258, 262, 229, 230, 405/288, 290, 138-145; 299/11-13

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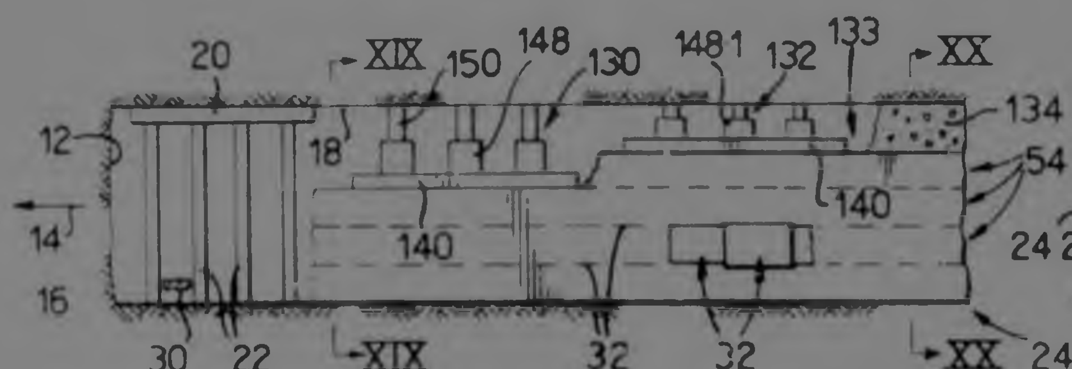
Primary Examiner—Dennis L. Taylor
Attorney, Agent, or Firm—Karl W. Flocks; A. Fred Starobin

[57] **ABSTRACT**

In mining the hanging wall is supported by pillars comprising a particulate backfill material which has been consolidated so that it has at the most one quarter voids by volume. Layers of reinforcing material are provided in the backfill to take horizontal loads associated with vertical loads being taken by the pillars in supporting the hanging wall. The pillars extend back from a work space behind the work face transversely to the work face. They are spaced in a direction along the workface and their loading ends advance as the work face advances.

15 Claims, 31 Drawing Figures



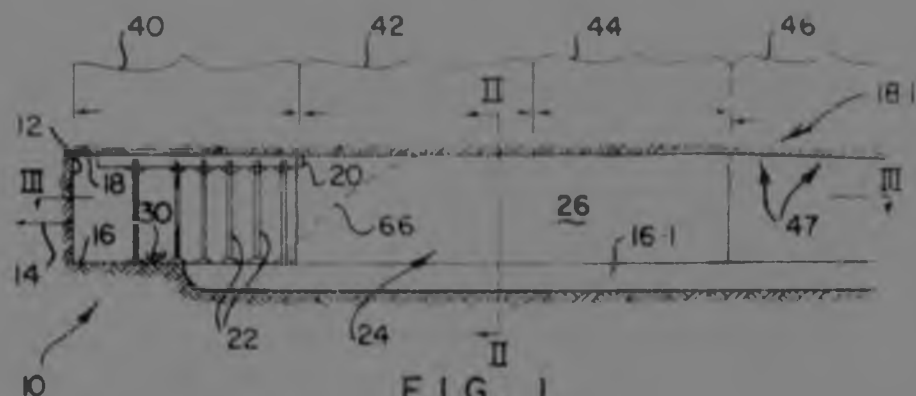


FIG. 1

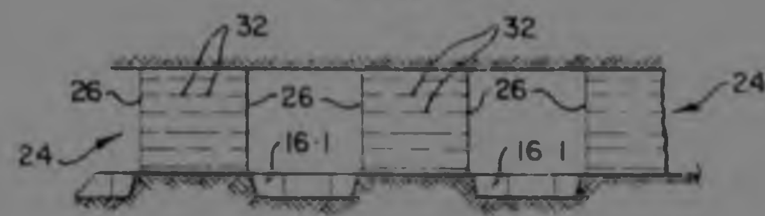


FIG. 2

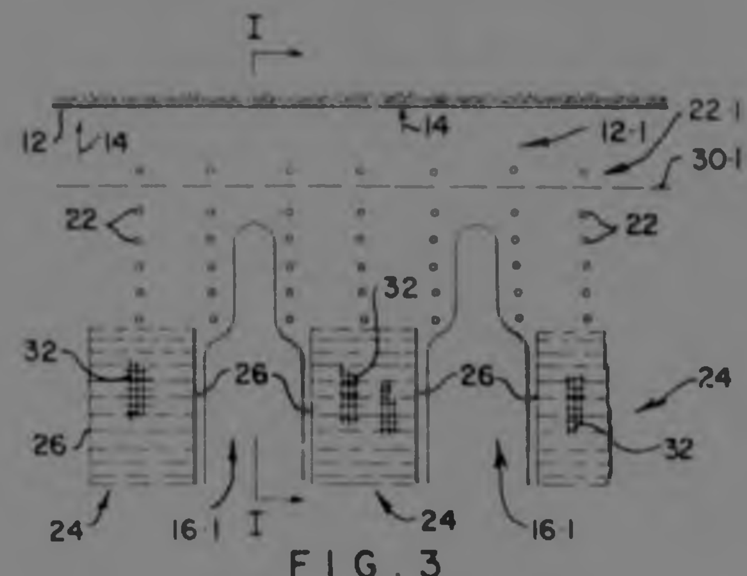


FIG. 3

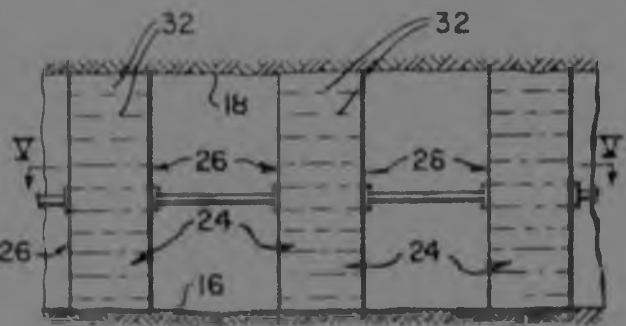


FIG. 4

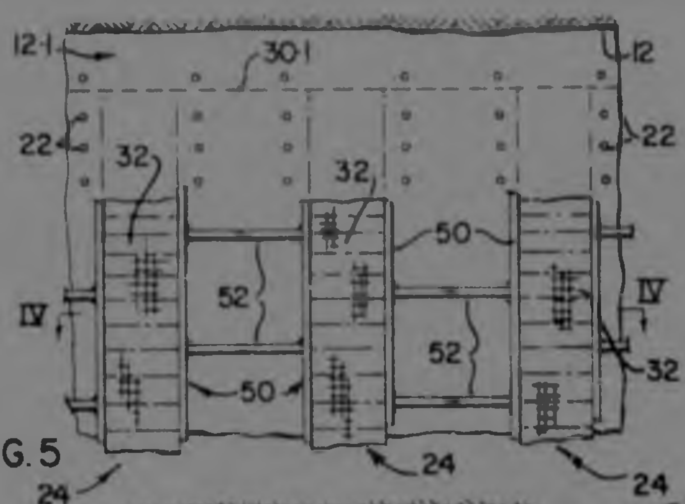


FIG.5

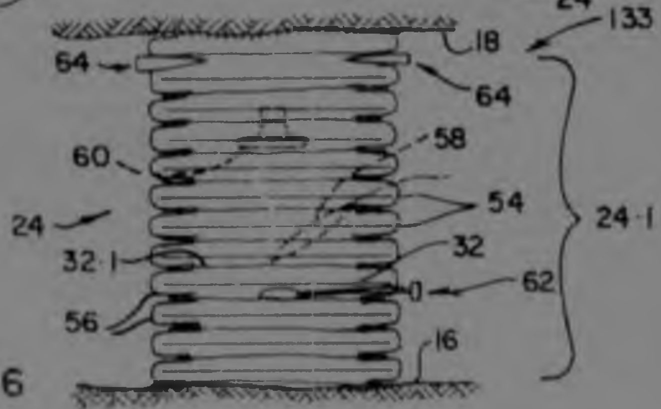


FIG. 6

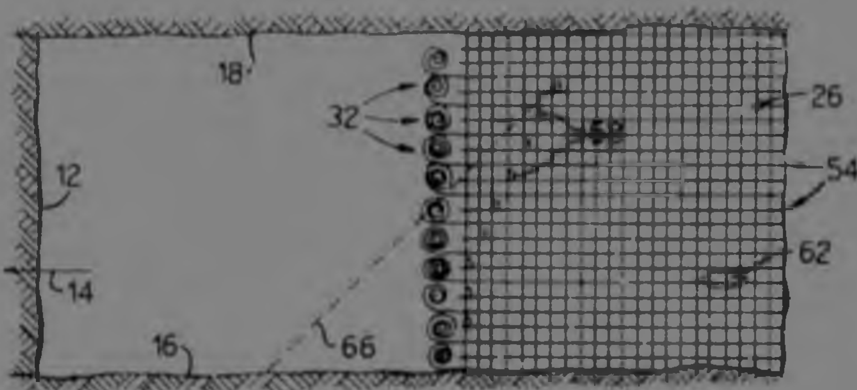


FIG. 7

FIG. 10

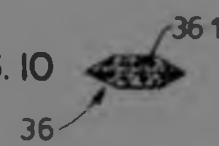


FIG. 8

FIG. 9

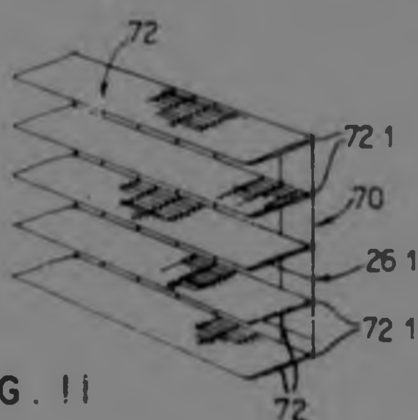


FIG. 11

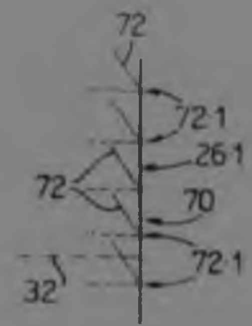
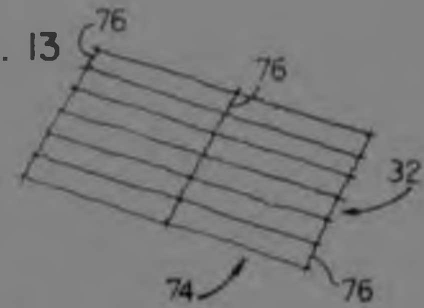


FIG. 12

FIG. 13



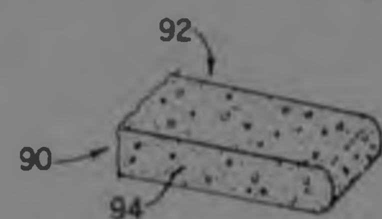


FIG. 14

FIG. 15

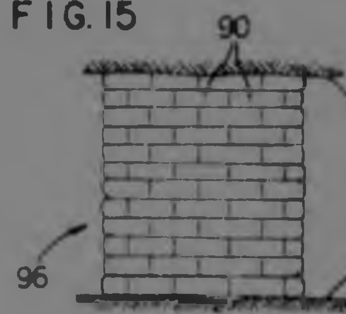


FIG. 16

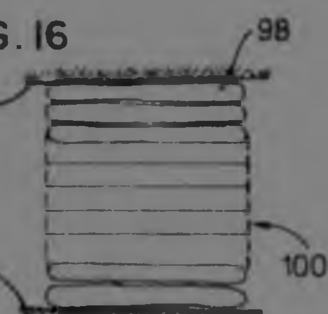
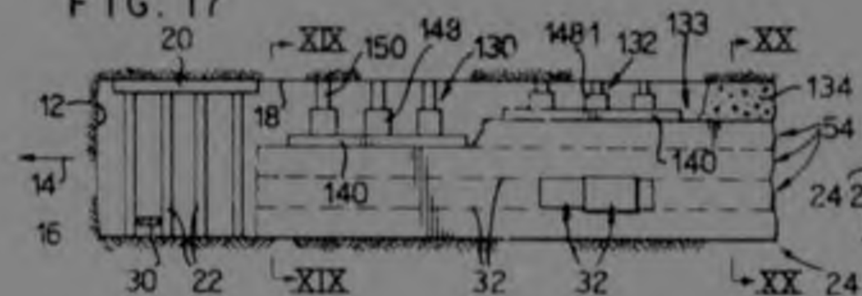


FIG. 17



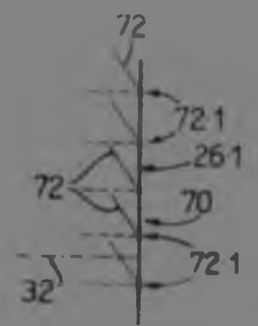


FIG. 13

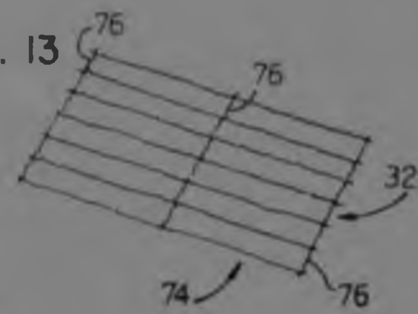


FIG. 12

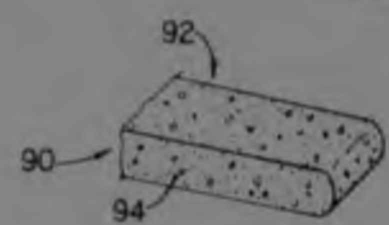


FIG. 14

FIG. 15

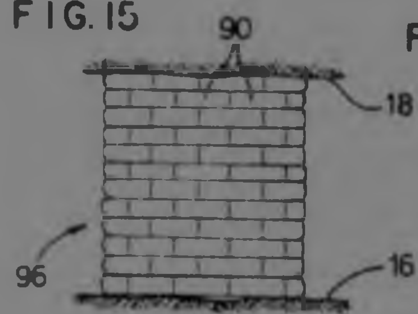


FIG. 16

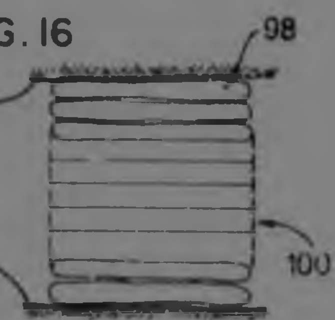
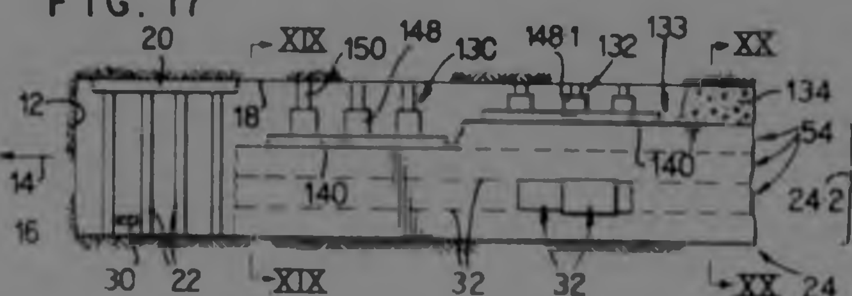


FIG. 17



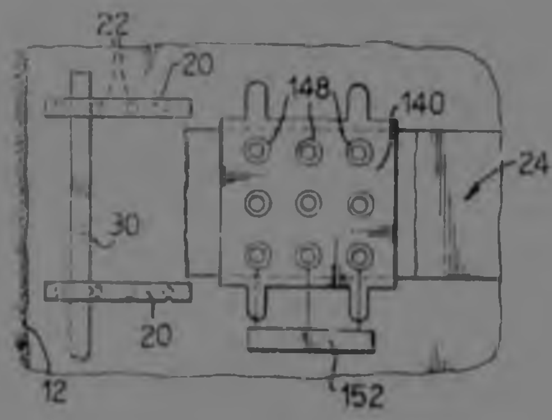
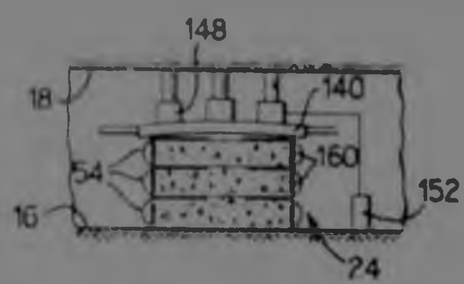


FIG. 18

FIG. 19



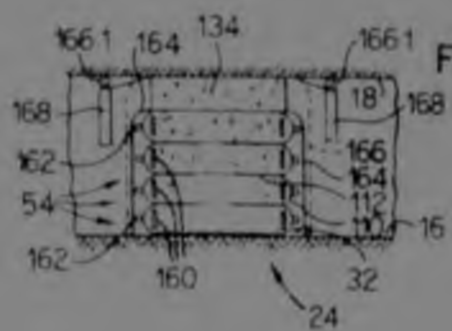


FIG. 20

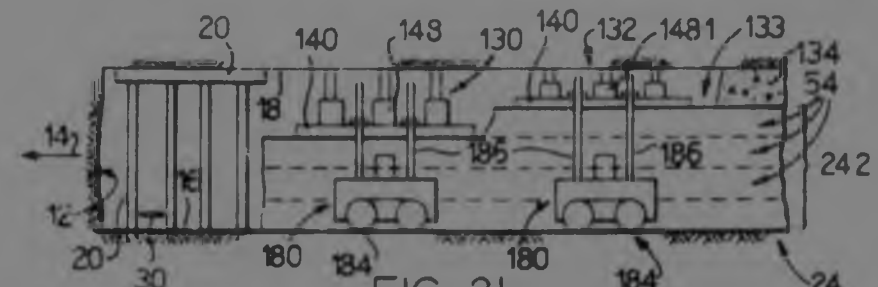

20
140
148
130
140
132
1481
133
134
54
242
14
18
186
186
16
12
20
30
180
184
180
184
24

FIG 21

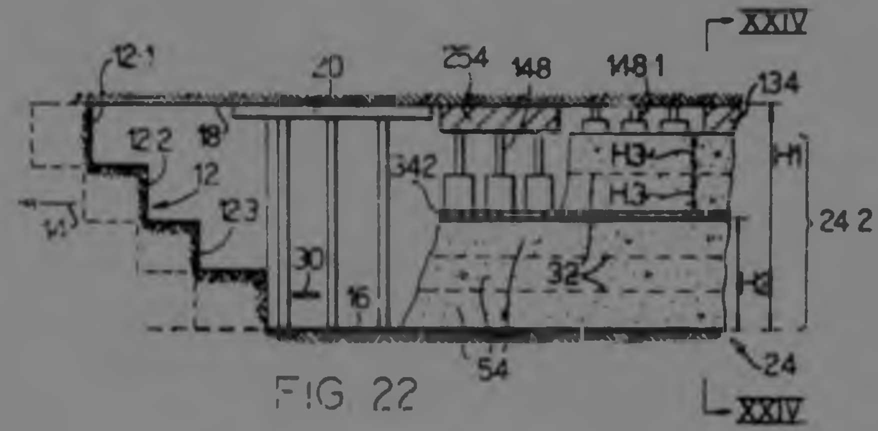
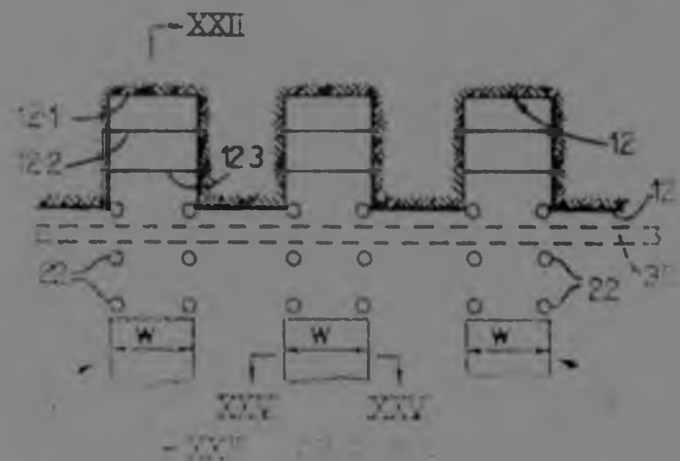

XXIV
12.1
20
254
148
1481
134
18
12.2
12
H3
H1
342
H3
242
14
12.3
30
32
H2
16
54
24
XXIV

FIG 22


XXII
12.1
12.2
12.3
12
12
22
22
W
W
W
XXV
XXV

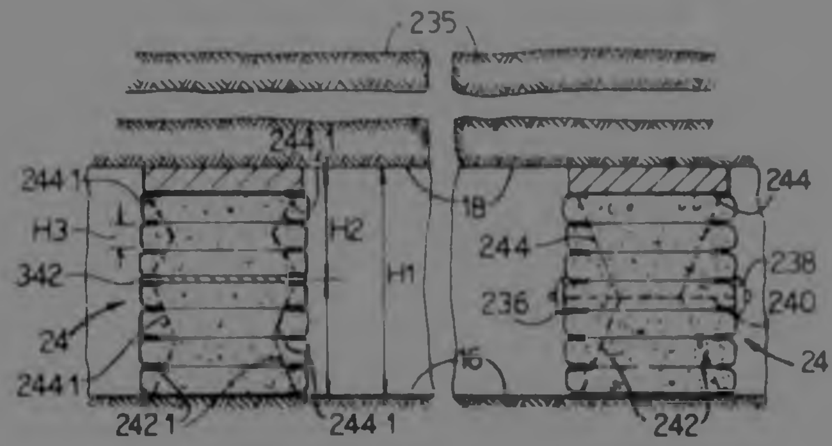


FIG 24 FIG 25

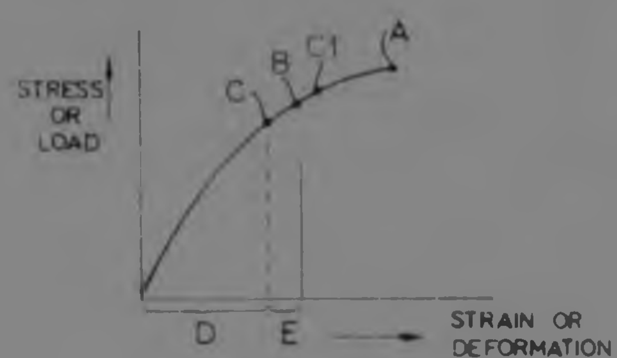


FIG 26

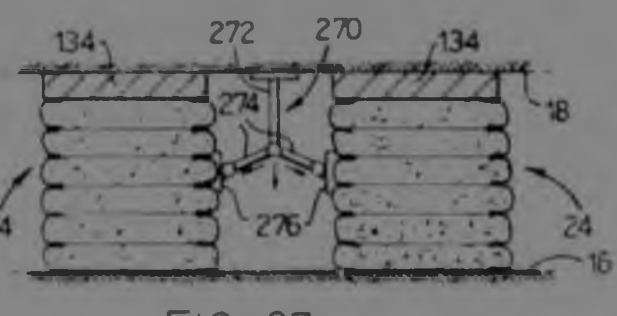


FIG 27

FIG. 28

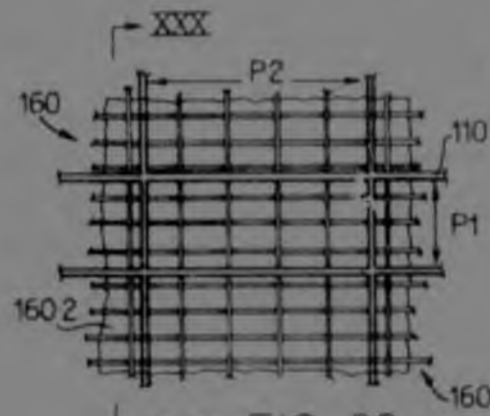


FIG 29

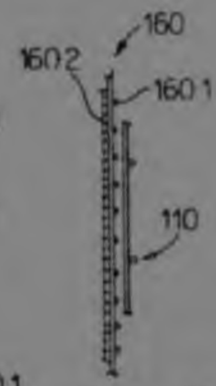


FIG 30

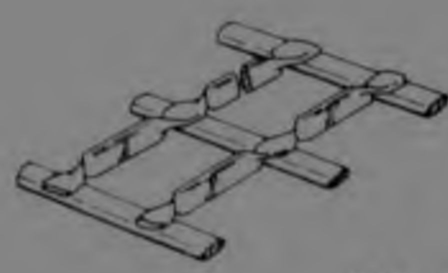


FIG 31

METHOD OF MINING

This invention relates to a method of mining. It relates in particular to a method of supporting the hanging wall in a mine, or in other undergound workings, particularly a coal mine, or in any mine in which the volume of ore or valuable fraction extracted is large in relation to the ore left behind for support.

In the past, coal has been mined by a method known to the Applicant as the 'bord and pillar' method. It is also sometimes referred to as the 'pillar and stall' method. This method results in large volumes of coal being left as pillars for support of the hanging wall underground.

The stress imposed upon pillars left for support underground, depends upon the depth at which mining takes place below the surface. The nature of the overburden will also be considered in determining the load on the pillars and hence the stress.

According to experts in this field, the earth's crust, prior to mining, is in equilibrium under the action of compressive stressses, also referred to as primitive stresses. The vertical component of the primitive stress in geologically undisturbed ground, is given by the equation

$$q = 24.88\ H \approx 25\ H \text{ kPa}$$

(Rock Mechanics in Coal Mining, Salamon & Oravecz, page 16, published by the Chamber of Mines, South Africa, 1976)

In this equation the density of the rock is assumed to be 2744 kg per cubic meter. At depth H (in meters), the vertical component (q in kPa) of the primitive stress is equal to the pressure exerted by the mass of a rock prism of cross-section one square meter and height H meters.

The horizontal component of the primitive stress is regarded by them as being equal in all directions and as being a constant proportion K of the vertical stress. In South African collieries, the value of K is given by them (S&O) as ranging from 0.1 to 0.4.

On this basis the following Table gives the vertical components of the primitive stress underground, before mining.

Depth below Surface Meters	Vertical Component of Primitive Stress (kPa)
30	750
100	2 500
150	3 750
200	5 000
300	7 500

Removal of rock by mining underground raises the average stress in pillars left for support. It is believed that the increase in the average stress will be approximately in the ratio:

Total area mined : area of pillars

A method of calculating the load on pillars is given by S&O, page 22. They also recommend (page 41) a factor of safety of 1.6 in the design of supports. In other words, the calculated load on a pillar will be increased by an amount corresponding to the factor of safety, and

the pillar will then be designed to take such increased load.

It is an object of this invention to provide a method of support for the hanging wall underground in mines, which will permit the extraction of greater volumes of coal and the leaving of less coal underground for support, than with other mining methods known to the Applicant.

Accordingly, in mining, there is provided a method of supporting the hanging wall which includes providing support pillars extending between the hanging wall and the foot wall, the pillars having lower portions incorporating particulate backfill material which has been consolidated so that it has, at the most, one-quarter voids by volume.

If desired, the lower portion of a pillar may have only one fifth voids by volume.

The backfill material may be fine such that about half by mass would pass a sieve of 0.85 mm, and such that only about 1% by mass would be retained on a sieve size of 6.75 mm.

The particulate material used as a backfill in the pillar may be in the form of soil, sand, tailings from previous mining operations, or quarried material, ash, burnt dolomite, or limestone, or the like. Thus, in coal mines which have low grade coal which may not be suitable for other uses, the coal may be burnt for example in a fluid bed or other system with dolomite and limestone. The ash and burnt dolomite and limestone may then be crushed, and may then be used with or without other materials as a backfill. Such burnt ash and dolomite and limestone may be very advantageous in providing a cementitious binding material in the backfill.

Different particulate materials have different natural angles of repose or angles of internal friction. When a particulate material is confined to form a pillar, and a vertical load is then applied to the pillar, then interparticulate friction causes a horizontal component of the vertical load to be imparted to the particles in the pillar. This horizontal component causes a stress which tends to cause lateral displacement of the material. If the vertical load is increased until failure occurs, then failure takes place in diagonal tension along planes, also referred to as shear planes.

For a particulate material having an angle of internal friction A, it is believed that the relationship between the horizontal component and the vertical load is given by the following expression:

$$L_H = C L_v$$

where

L_v is the vertical load imposed on the pillar;

L_H = the horizontal component of the load; and

C =

$$\frac{1 - \sin A}{1 + \sin A},$$

where angle A depends on the particulate material being used.

When the angle A = 30° then C = ⅓.

The magnitude of L_H therefore depends upon the angle A. The angle which the shear planes make with the horizontal is

$$\left(45^\circ + \frac{\phi^\circ}{2}\right)$$

The lower portion of a pillar may be provided by the erection of retaining walls and by the hydraulic placement of the particulate backfill material in a hydraulic carrier behind the retaining walls, the consolidation of the particulate backfill material taking place by settlement of the particulate material under gravity in the hydraulic carrier. Thereafter, the hydraulic carrier may be allowed to drain away.

Alternatively, the particulate backfill material may be pneumatically placed behind the retaining walls, the consolidation of the particulate backfill material taking place by the discharge of the material at speed under pneumatic pressure.

The consolidation of the backfill material in the lower portion of a pillar may take place by the application of mechanical force to the backfill material. Alternatively, or in addition, consolidation of the backfill may include exerting pressure on the lower portion in a clearance space between such lower portion and the hanging wall, and by abutment against the hanging wall before filling up the clearance space with load taking fill. The pressure exerted on a lower portion before filling of the clearance space may be at least 9/10ths of the load which the pillar is expected to take. Preferably, the pressure exerted is in excess of the load which the pillar is expected to take.

Generally, if desired, the step of consolidation of a layer by pressure may be preceded by compaction of this layer by impact, vibration, or the like. Such compaction may take place by stampers, vibrators, or the like. Vibration and compaction methods may be used throughout the construction process to assist in achieving effective consolidation, including the use of external removable supporting formwork, shapes, or structures. The pressure may be exerted by a shoe or platen which may also be subjected to vibration while pressure is being applied. Pressure may be applied by abutment of a press against the hanging wall. The platen may have a slight camber transversely to the direction of the pillar.

The making of the pillar may include the final step of providing a load taking fill which may be in the form of a layer of grout in between the hanging wall and an upper layer of backfill. The grout may be tamped or rammed in under pressure to ensure that there is active support between the upper layer of the pillar and the hanging wall.

Alternatively, the provision of the load-taking fill on top of the lower portion may include the driving in of wedges into the clearance space between the lower portion and the hanging wall, thereby exerting a downward pressure on the lower portion. As a further alternative, the load-taking fill may be provided on top of the lower portion by pumping particulate material carried in suspension by a hydraulic carrier, into the clearance space, the pumping taking place under hydraulic pressure to ensure that a downward pressure is exerted on the lower portion by abutting against the hanging wall.

The lower portion of a pillar may be built up in layers of backfill with layers of reinforcing material at different elevations between layers of backfill in the lower portion. The vertical spacing between layers of reinforcing material may, at the most, be equal to one third

the minimum cross-sectional dimension of the pillar. The pillar may include opposing retaining walls on its opposite sides, and the reinforcing material may engage with the retaining walls to constrain them against outward bulging.

The reinforcing material may comprise uninterrupted sheet material, expanded sheet material, or mesh, such as wire mesh or netting, steel grid or plate, metal strips or sheets. It may also comprise materials such as synthetic polymers, e.g. polyurethane, polystyrene, polyethylene, and so on, cloth like synthetic fibres, wooden lath structures, metal strips or sheets, and so on. The reinforcing material may comprise the combination of two or more of the above-mentioned items. Thus, for example, a sheet of polymer could be reinforced by steel and wire mesh or by synthetic or natural fibre mesh or cloth. The reinforcing material may, if desired, also be in the form of a slab, metal plate, fibreglass moulding, or a geotextile. It is designed to accept, with a sufficient factor of safety, the horizontal component of load L_H associated with the vertical load on the pillar. The reinforcing layer may have indentations or projections providing an uneven surface so as to improve the frictional grip between the layer and the particulate material in contact with it. If desired, fibreglass or other material may be reinforced by high tensile wire. Alternatively, if desired, the reinforcing layer may be in the form of a relatively thin concrete slab which has been prestressed by arrays of high tensile wires at right angles to each other.

The particulate backfill material may be made up into the form of gabions which comprise the backfill material contained in envelopes of reinforcing material, and in which the lower portion of a pillar is built by laying the gabions in layers. At least some of the gabions may be pre-compressed before being installed.

The invention extends to a pillar when made according to the method as described.

The invention extends also to a method of mining, which includes providing a plurality of pillars behind a work place adjacent a work face, the pillars being provided in accordance with the method as described, and being spaced in a direction along the work face and extending back in the direction transversely to the work face. The lengths of pillars in a direction transverse to the work face may be at least twice the width of the pillar in a direction along the work face.

The adjacent pillars may be strengthened against outward bulging under load by having beams extending along their lengths in a direction transverse to the working face, the beams being supported by struts spaced in series generally in a direction away from the work face, and the struts themselves extending generally in a direction along the work face. The spacing between struts may, at the most, be equal to the minimum cross-sectional dimension of the pillars.

The invention extends also to a method of building pillars underground for supporting the hanging wall in underground mining operations, which method includes calculating the vertical load which a pillar is to take, calculating the horizontal component of load associated with such vertical load, laying particulate materials in layers between hanging wall and foot wall, consolidating such layers, and providing reinforcing material within or between the layers of particulate material to accept, with the desired degree of safety, the horizontal components of load associated with the vertical load which the pillar is expected to take.

The invention will now be described by way of example with reference to the accompanying diagrammatic drawings.

In the drawings,

FIG. 1 shows a sectional side elevation of a working place in a coal mine, taken at I—I in FIG. 3 of the drawings,

FIG. 2 shows a sectional elevation taken at II—II in FIG. 1 of the drawings,

FIG. 3 shows a sectional plan taken at III—III in FIG. 1 of the drawings,

FIG. 4 shows a cross-sectional elevation at IV—IV in FIG. 5, through a plurality of pillars spaced along the work face in another arrangement,

FIG. 5 shows a sectional plan view at V—V in FIG. 4, of the pillars of FIG. 4,

FIG. 6 shows a cross-sectional elevation through a pillar during the final stages of building,

FIG. 7 shows a side elevation of the front end of a pillar during building,

FIG. 8 shows a plan view of a wedge suitable for use in building pillars,

FIG. 9 shows a side elevation of the wedge, corresponding to FIG. 8,

FIG. 10 shows a cross section taken at X—X in FIG. 8,

FIG. 11 shows, an oblique side view of a template element for defining the side of a pillar,

FIG. 12 shows diagrammatically an end view of the template element of FIG. 11,

FIG. 13 shows a detailed view of the mesh used as reinforcing material between opposing template elements in use,

FIG. 14 shows one end of a gabion suitable for use in the making of a pillar according to another aspect of the invention,

FIG. 15 shows a cross sectional end elevation of a pillar when made with gabions of FIG. 14,

FIG. 16 shows a cross-sectional end elevation of a pillar when made with gabions having a length equivalent to the width of the pillar,

FIG. 17 shows a sectional side elevation of the underground working of a mine, in which a pillar according to another aspect of the invention, is being built,

FIG. 18 shows a part plan view corresponding to FIG. 17,

FIG. 19 shows a sectional elevation at XIX—XIX in FIG. 17,

FIG. 20 shows a sectional end elevation taken at XX—XX in FIG. 17,

FIG. 21 shows a sectional side elevation of a pillar according to the invention, during the process of making it underground by means of a mobile press,

FIG. 22 shows a sectional side elevation at XXII—XXII in FIG. 23,

FIG. 24 shows a sectional front elevation of a pillar in accordance with the invention, taken at XXIV—XXIV in FIG. 22,

FIG. 25 shows a view similar to FIG. 24, but with a variation in construction, taken at XXV—XXV in FIG. 23,

FIG. 26 shows a stress strain (load deformation) diagram of a pillar according to the invention,

FIG. 27 shows a sectional front elevation of a further development of the invention,

FIG. 28 shows a three-dimensional view of reinforcing material suitable for use in carrying out the invention,

FIG. 29 shows a detail plan view of typical reinforcing material in the form of wire mesh,

FIG. 30 shows a section at XXX—XXX in FIG. 29 and,

FIG. 31 shows a wire mesh arrangement having wires of elliptical section secured in criss-cross fashion, and suitable for use as reinforcing in the pillars according to the invention.

Referring to the drawings, reference numeral 10 refers generally to a work place underground in a coal mine. It has a work face 12 which advances in the direction of arrow 14. The coal face extends between the foot wall 16 and the hanging wall 18. Immediately behind the work face, the hanging wall 18 is supported by a head plate 20 which is itself supported by a plurality of props 22. These props and head plate extend rearwardly, more or less in line with the forward end of pillars 24 which are arranged to advance at more or less the same rate as the working face.

The pillars 24 are made by the use of retaining walls 26 on either side, the space between such walls and the hanging wall and foot wall is then filled with a particulate backfill material. This may be in the form of ash, external make-up in the form of sand or soil, gas beton, waste, in any proportions. When sufficient waste material is not available, then the foot wall 16 may be undercut as at 16.1 in the spaces between pillars 24.

Just behind the work face 12, a work space 12.1 is left free for mining activity. Immediately behind the first line 22.1 of props 22, there is provided a conveyor belt 30 whose centre line is indicated in FIG. 3 by reference numeral 30.1.

In operation, pairs of retaining walls 26 will be arranged in spaced relationship extending rearwardly from near the working face. Several pairs of retaining walls 26 are so provided and are located in spaced relationship along the width of the coal face relative to other retaining walls for other pillars.

The retaining walls 26 as described, may be in the form of cladding which is capable of advance to follow at more or less the same rate as the rate of advance of the work face. However, such cladding 26 may instead be permanent and may be of cementitious material and mesh. The opposing retaining walls 26 of a pillar may be tied together by means of reinforcing material 32. Where the cladding 26 is movable, the attachment between the material 32 and the cladding 26 will be of a temporary or disconnectable nature. However, when the cladding 26 is in the form of a permanent or semi-permanent structure, such as a cementitious coat with reinforcing, then the connections between the reinforcing material 32 at different elevations, and the cladding 26, may be permanent.

Referring now to FIG. 1 of the drawings, it will be noted that immediately behind the work face the hanging wall 18 is supported by the head plate 20 together with its supporting props 22. This zone 40 will be a non-subsidence zone.

Immediately behind the props, there follows what is regarded as a safe zone 42 which contains also the forward end of the pillar 24. The broken material between the retaining walls 26, rests at an angle of repose, indicated by reference numeral 66. The immediately adjacent zone rearwardly is a zone 44 in which settling has not yet taken place. It is in the next zone 46 that the hanging wall 18 first bears up against the upper end of the pillar 24, as shown at 47.

Immediately behind the zone 44, there is the settled zone 46 where the hanging wall 18 has already settled onto the upper end of the pillar 24. It will be noted that the hanging wall 18.1 in zone 46 is at a somewhat lower level than the hanging wall 18 immediately behind the work face 12.

The fill, when introduced into the space between the retaining walls may be tamped to ensure good contact with the hanging wall. Alternatively, or in addition, the space above the lower portion of the pillar and between the fill and the hanging wall may be grouted. The purpose is to obtain early acceptance of the load preferably before roof fracture takes place from subsidence.

The retaining walls can be very thin and could in fact be in the form of a skin merely, and when in the form of cladding may be movable for later re-use. Alternatively, depending upon economics, the retaining walls may be left in situ. The prime purpose of the retaining walls is to ensure that the particles of fill do not fall out at the sides of the running pillars.

For ease of advance of the work face, the conveyor 30 may be mounted upon transverse skids to permit easy displacement towards the work face in a direction transverse to its longitudinal axis 30.1.

The members 32 act as reinforcing members and may be in the form of metal plate or metal sheet or wire mesh. They need not be connected to the retaining walls 26 as long as the retaining walls 26 ensure that the particles of the backfill do not fall out.

Referring to FIGS. 4 and 5 of the drawings, the arrangement of the pillars at the work face is similar to that described with reference to FIGS. 1 to 3 of the drawings. Like reference numerals refer to like parts. The pillars 24 shown in FIGS. 4 and 5 are however, taller. In order to provide strength in the middle against outward bulging under load, beams 50 are provided on opposite sides of the columns 24. These beams are supported by struts 52 spaced in series, generally in a direction away from the work face. The struts themselves extend generally in a direction along the work face, and are provided across spaces which are not required for access to the work face.

Referring to FIG. 6 of the drawings, reinforcing material in the form of wire mesh 32 and 32.1 are shown inside backfill arranged in layers 54. The thickness of a layer 54 is defined by U-shaped wire mesh side members 56. The layers of backfill are consolidated by vibration or compaction by making use of a vibrator 58 or compactor 60.

In order to monitor the behaviour of the pillar 24 under load, a pressure-sensitive device 62 is embedded within the backfill. Readings or recordings can then be taken periodically of the pressure in the backfill.

After the lower portion 24.1 of the pillar 24 has been built up then the clearance space between the top of the lower portion 24.1 and the hanging wall 18 may be taken up by wedges 36.

Referring to FIGS. 8, 9, and 10 of the drawings, there is shown a wedge 36 which is made of concrete and which has reinforcing 36.1. The wedge is tapered from a parallel portion 36.2 at its rear end to a thin, pointed end 36.3 at its front end. The wedges may be from half a meter to one and half a meters in length. A wedge may be tapered over most of its length, i.e. from a pointed end rearwards. But at least one-quarter of its length, at the rear end, will be parallel to ensure that it is not ejected under load. The wedges may be of concrete, and may be driven by wooden mallets. If desired, the wedges may be reinforced with steel rod or wire mesh or netting embedded within it. The wedges may also be of hardwood or plastic.

Referring to FIG. 7 of the drawings, the retaining walls 26 are shown made of wire mesh having a coat of cementitious material. The natural angle of repose of the particulate material being charged into the space between the opposed retaining walls 26 is indicated by reference numeral 66. The front end of the particulate material may, however, be confined by providing a plurality of bars 68.

The reinforcing means 32 separating the backfill into courses ensures that instead of a single tall pillar there is provided a plurality of squat pillars on top of one another. By merely containing the sides of the courses to prevent falling away at the sides, a robust pillar having a high load bearing capacity is provided.

The retaining walls 26 and reinforcing means 32 may be provided in roll form, and may be unrolled in the direction of advance of the working face, as indicated by arrow 14, as the working face advances and as the pillar advances.

Referring now to FIGS. 11 and 12 of the drawings, there is shown a template element generally indicated by reference numeral 70 and comprising a retaining wall member 26.1 and leaf elements 72 flexibly or hingedly connected to the retaining wall member 26.1. In order to facilitate transport, the leaf elements 72 may be provided with hinges 72.1 extending along their lengths. This will permit folding of the template element into a narrower item which can be more easily transported than when it is wide. The retaining wall member 26.1 may be made up of wire of 2 to 3 mm diameter at spacings of 15 to 20 cm square. It may, however, have openings which may be larger or smaller or which may be rectangular in shape, depending upon what is required to contain the backfill.

The leaf elements 72 may be made up of a mesh such as is shown in FIG. 13. It will be noted that there are many more wires 74 in the one direction than wires 76 in the other direction. In use, the mesh will be laid in such a manner that the wires 74 extend transversely across the width of a pillar, and the wires 76 extend longitudinally. The stressable area of the reinforcing means 32 in a direction across the width of the pillar 24 will be about 10 to 40 times as much as the stressable area of the reinforcing means, in a longitudinal direction relative to the length of the pillar. If desired, the wires 74 and 76 may be of different diameters to meet this condition. The stressable area or the strength may, of course, be equal for the two directions.

In use, the template elements 70 will be erected to define the sides of a pillar, the leaf elements 72 being raised as shown in FIG. 12. As the courses fill up, so the leaf elements 72 will be lowered onto the top of a course of back-fill material which has been laid. Thereupon, reinforcing means in the form of mesh 32, similar to the mesh shown in FIG. 13, will be secured to the member 26.1. Alternatively, the mesh 32 may merely be laid on top of the leaf element 72 and the reinforcing means 32.

Referring now to FIGS. 14, 15 and 16 of the drawings, there is shown one end of a gabion generally indicated by reference numeral 90, comprising an envelope 92 and particulate material 94 within the envelope. The permeability of the envelope 92 will be matched to the fineness of the particulate material 94 used within it.

Thus, the envelope must be able to contain the particulate material within it.

In use, gabions 90 are used and stacked on top of one another to form the lower portion of a pillar 96. Such a pillar may also be rendered to provide active support to the hanging wall 26, by making use of wedges, hydraulic fill material, or grout in the clearance space against the hanging wall.

Referring now to FIG. 16 of the drawings, there are shown gabions 98, similar to those shown in FIG. 14 but having a length corresponding to the width of a pillar 100 which is to be built. The pillar 100 may also be rendered to provide active support to the hanging wall 18, by means of the use of wedges as previously described, or of providing hydraulically placed filler material or grout in the clearance space between the top of the lower portion of the pillar and the hanging wall 18. The degree of support provided can be determined by the use of pressure-sensing devices 62, as previously described.

When making the gabions, it is important to ensure that the particulate material within the gabions is properly consolidated, such as by vibration, compaction, or compression.

It is an important feature of this invention that the particulate material in the back-fill, in the various courses, be vibrated and compacted as fully as possible, when laid. The gabions should, in turn, also be compacted as fully as possible, whether before or after laying.

Referring now to FIG. 17 of the drawings, a further variation of the pillars 24 previously described is shown. Like reference numerals refer to like parts.

The pillar 24 is made of backfill material arranged in layers 54 and comprising vertically spaced layers of reinforcing material 32 embedded within back-fill particulate material. As the successive layers 54 are laid, so they are compacted and later consolidated by means of mobile presses, generally indicated by reference numerals 130 and 132, until the whole of the lower portion 24.2 of the pillar has been pre-loaded. If desired, each layer 54 may be consolidated by pressure after it has been laid. Alternatively, two or more layers 54 may be consolidated together. The press 132, is specially adapted to provide consolidation for the uppermost layer with minimum clearance between the upper surface of such layer, (i.e. the top of the lower portion 24.2), and the hanging wall 18. The rounded shape of the layers 54 at the opposing sides of the pillar may be obtained by formwork which is removable after consolidation of the layers. The clearance space 133 between the upper surface of the lower portion 24.2 of the pillar and the hanging wall, 18, is filled with grout 134 after consolidation by the mobile press 132 has taken place. The grout 134 is tamped in solidly under pressure to ensure that the pillar is suitably prestressed or preloaded to support the hanging wall 18.

The degree of consolidation by the presses 130 and 132, is preferably such, that the load applied to consolidate the layers, will approximate and even exceed the load which it is estimated that the pillar will ultimately have to take, in supporting the hanging wall 18. This is to ensure that the amount of deflection (if any) of the pillar under the load which it is to take ultimately will be as small as possible. It is also for this reason, that the grout layer 134, is firmly tamped in, by mechanical or hydraulic rams if necessary to ensure that the load will be taken with minimum and preferably no deflection.

The presses 130 and 132 are generally of the same construction excepting that the press 132 is made to operate within a smaller clearance space 133.

The press 130 comprises a platen 140 movable by means of forklift-type trucks or mobile cranes from one layer or zone requiring consolidation to another. On top of the platens, there are provided hydraulic jacks 148 having head members 150 adapted in operation to abut against the hanging wall 18 and to press the platen 140 firmly onto the layers 54 thereby consolidating them.

Referring to the mobile press 132, the construction is similar, excepting that the jacks 148.1 on the platen 140 are shorter than the jacks 148 because they have to operate in a smaller space, namely the clearance space 133. Thus the press 132 may be arranged to operate in a space of say, 30 to 50 cm. More jacks 148 and 148.1 may be used than are shown in the drawings.

Each mobile press 130 and 132 is conveniently provided with its own hydraulic pump and reservoir arrangement 152 together with appropriate valve gear, to supply hydraulic fluid under pressure, to the hydraulic jacks 148 and 148.1. This will enable the hydraulic jacks to be placed under load, so as to consolidate the layers 54, as and when required.

Referring to FIGS. 19 and 20 of the drawings, the reinforcing material 32 is in the form of a wire mesh which has been suitably protected against corrosion and which has its side panel 110 and end panel 112 (see FIG. 28) standing upright while the particulate backfill material is being charged to form the layer 54. Once the particulate material has reached a pre-determined depth, corresponding to the height of the side panel 110, then the end panel 112 is folded over onto the backfill. If desired, prior to charging with backfill, panels 160 impervious to the particulate backfill material may be provided on the inside of the side panels 110 to ensure that the backfill particles does not pass through them. To this extent, the panels 160, also form part of the reinforcing material 32. The panels 160 may be made up of smaller mesh 160.1 (say a mesh also referred to as bird or canary mesh) and a lining 160.2 of cloth, cardboard, sheet material or plastic film (see FIGS. 29 and 30).

The length of the end panels 112 of the reinforcing material 32, will conveniently be at least half a meter but may be a meter or more if desired, so as to ensure a good purchase and frictional restraint between consecutive layers of material 32. Where desired, the end panels 112 may be bound or otherwise secured to the underside of the next succeeding layer 32 before charging of particulate material starts.

The panels 160, may be of plastic sheet material, timber panels, metal sheeting, or the like. If desired, there may be provided in addition, longitudinal beam elements in the form of rods 162 secured to the panels 110, and adapted to span the joints between adjacent reinforcing material 32. If desired, successive reinforcing layers 32 may be arranged to overlap in a longitudinal direction, along the length of the pillar 24.

In order to exclude moisture and water borne corrosive materials, which may corrode the reinforcing material 32, a plastic sheet of film 164 (see FIG. 20) may be draped over the uppermost layer before the grouting layer 134 is introduced. Instead or in addition, a plastic film or sheet 166 may be provided between the hanging wall 18 and the grout layer 134. The rolls 166.1 of plastic film will then be temporarily supported by temporary support posts 168, which can be removed and the film **166** can then be allowed to fall and drape down over the sides of the pillar **24**, when grout **134** has been placed. If desired plastic sheet material or other suitable material may be used as a damp course between the lower most layer of the pillar and the foot wall **16**.

Referring to FIGS. **29** and **30** of the drawings, there is shown reinforcing material in the form of wire mesh. The wire may be of round, rectangular or elliptical cross-section. Depending upon the load which is to taken, the cross sectional area of the wire used for the wire mesh, may be equivalent to the area of wire having a diameter of between, say, 2 mm and, say, 6 mm. The pitch P_1 between wires in a longitudinal direction, may be between 10 and 30 times the diameter or transverse dimension of the wire. The pitch P_2 may lie between once and six times the pitch P_1 but is preferably of the order of four times P_1. Reinforcing material **32** can be made of this mesh.

If desired, the reinforcing means **32** may be made up of flat metal strips, extending transversely across the width of the pillar **24**, and by wires extending along the length of the wall. The cross-sectional area of metal adapted to take tensile loads in a direction transverse to the wall, that is in the direction of arrow **170**, may conveniently be twice to ten times the cross sectional area of metal, adapted to take tensile load in the direction of arrow **172**. Alternatively, the wires taking load in the direction of arrow **170** may be of high tensile steel so as to be able to take a greater load.

Referring now to FIG. **21** of the drawings, the pillar **24** built is of similar construction to that already described, and like reference numerals refer to like parts. The difference is in the type of mobile press used. The mobile press used in the building of the running pillar as shown in these drawings, is in the form of a forklift-type of vehicle generally indicated by reference numeral **180**. It has wheels or tracks **184**, a pair of spaced posts **186**, a platen **140**. The platen **140** carries a number of jacks **148** (or **148.1** where the jacks are to operate in spaces with little clearance).

The use of forklift-type of vehicles **180** makes it possible for the vehicle to move around, and for the width of the pillar, to be varied by making the platens **140** laterally movable relative to the pillar **24**. For strength and lightness the platens **140**, the jacks **148**, and **148.1**, and indeed many articles used in carrying out the method may be made of manganese aluminium alloy, for example, Duralumin.

If desired, instead of laying backfill and reinforcing material separately, prefabricated units (gabions) can be laid in courses brick-fashion to form the pillar. The courses laid correspond to the layers **54**. Thereafter the courses of units may be consolidated under pressure as described with reference to FIGS. **17** to **21** of the drawings. The units (gabions) may be precompressed before laying.

Referring now to FIGS. **22** to **25**, there is shown a pillar **24** for use in mining a thick seam, say, in excess of three meters. A plurality of pillars extend back from the work face **12**, continuously from their front ends near the work face for a length equal to at least twice their widths **W**.

Where the height between the foot wall **16** and the hanging wall **18** is not excessive, say, up to a maximum of three meters, then the pillar previously described may be used, comprising a plurality of layers **54** of particulate material reinforced with reinforcing material **32**. Such reinforcing material may be in the form of wire mesh, steel strips, steel plate, fibreglass mouldings, pre-stressed concrete slabs, or the like. The amount of reinforcing which is inserted will depend upon the ultimate load which the pillar is expected or designed to take. This will depend upon the depth D at which mining is taking place below the surface **235**, and upon the density of the rock. As previously mentioned, at 100 meters depth the loading could be of the order of 200–300 tons per square meter. At 200 meters depth the loading could be 400–600 tons per square meter, and at 300 meters it could be, say, 700–800 tons per square meter. (1 Ton per square meter = 10 kPa).

If the load on the pillar is increased, then it will ultimately fail in diagonal tension along planes **242** and **244** (see FIGS. **24** and **25**). The pillar be strengthened against such failure by means of beams **236** and **238**, extending longitudinally along the sides of the pillar **24**, and at about the middle, more or less in line with the intersection of the planes **242** and **244**. These beams **236** are then tied across to each other by means of transverse tensile elements **240** passing through the pillar and which may be in the form of bolts or steel wire ropes. The bolts or steel wire ropes may be sheathed in a steel or plastic tube or plastic film sheath for protection against corrosion and for easy withdrawal. The beams **236** and **238** may be in the form of cold-rolled metal plate to provide a stiff section for a beam. The longitudinal spacing between the tensile elements **240** may vary from one meter to two or three meters, depending upon the strength required and upon the load which is to be taken. The spacing will also depend upon the strength of the beams **236** and **238**. The spacing between elements **240** will generally not be greater than the width or thickness of the pillar.

Referring to FIG. **24** of the drawings, the arrangement there is similar to that shown for FIG. **25**, except that a strong reinforcing layer **342** is provided, which is of adequate strength to turn the high pillar **24** having a height H_1 into two shorter and stiffer superimposed squat pillars having heights H_2, and have the effect that the points of intersection of the planes **242.1** and **244.1** in FIG. **24** are more widely spaced than the points of intersection of the planes **242** and **244** in FIG. **25**.

In practice, the height H_1 will depend upon the thickness of the seam of coal which is being mined. This may vary from 2–3 meters to 5–10 meters. However, when the height H_1 is very large then, depending upon the loads which are to be taken, the heights H_2 of the squat pillars will be reduced to a value preferably not exceeding the minimum cross-sectional dimension of the pillar, i.e. the width or thickness of the pillar.

When the seam of coal which is being mined is shallow, there may be some relaxation with regard to the height of the pillar relative to its width. However, when seams deep down are being mined, and where loads of the order of 1000 tons per square meter are contemplated, then the overall height H_2 of a squat pillar, as shown in the lower half of the pillar shown in FIG. **24**, will be much less than the tall pillar of FIG. **25**, shown for an application where the contemplated loading is much less.

The various layers of particulate material **54**, together with their reinforcing mesh layers **32**, and the reinforced cap or foot plate **342**, are consolidated by vibration, compaction, or the like, and ultimately by being compressed by means of jacks **148** and **148.1**, pressing directly against the hanging wall **18**, as shown by jacks

148.1, or indirectly via a spacer 254, as shown by jacks 148 (see FIG. 22).

When the seam being mined is very thick, then the working face 12 may be worked in steps 12.1, 12.2, and 12.3.

The spacing between layers of reinforcing mesh 32 is given by H. Here again, the strength of the mesh will be determined by the horizontal component of the vertical loading which the mesh is expected to take when the pillar is subjected to its vertical load. The strength of the mesh will be chosen with a suitable factor of safety being taken into account, say, 1.6.

Referring now to FIG. 26 of the drawings, reference numeral 260 indicates a Stress-Strain or Load Deformation curve which the layers of particulate material 54, with reinforcing material 32, are expected to take as they are loaded. The pillar is designed to take an ultimate stress indicated by point A which is appreciably higher than the stress indicated by point B and which represents the stress or load which it is expected (from the depth of working below surface) that the pillar will ultimately have to take. In practice, the various layers 54 and reinforcing material 32 will be stressed by pre-loading by abutment against the hanging wall 18, to an extent indicated by point C. Thereafter the upper layer 134 in the form of grout is tamped or rammed in under pressure in an endeavour to take the stress of the grout also, up to a value indicated by point C. All the layers will then have been pre-compressed, and upon the load being taken subsequently, the initial strain D will already have been taken up and the minor amount of strain E is all that will take place in the pillar. The spacing between points C and B has been exaggerated in the diagram, for clarity.

It is, of course, possible for the pre-loading, to take place to the same value as the stress B, or even slightly beyond it, say, to a point C1. This will then ensure that the particulate material in the pillar has been fully consolidated by being pre-loaded so as to ensure that the load from the hanging wall will be taken with minimum or no deflection or deformation of the pillar. The degree of deflection or deformation of the pillar under pre-loading or precompression is expected to be about ⅛ or 1/16 of the original height of the pillar.

Referring to FIG. 27 of the drawings, a further possibility suggests itself of preventing bursting of adjacent pillars. The arrangement shown in FIG. 27 is believed to be particularly useful in very thick seams which are being mined, say, from about 6 meters upwards. If the descent of the hanging wall 18 is accurately predeterminable, i.e. it is known almost exactly how far it will descend to the final settled position, then, by making use of the toggle mechanism 270, the descent of the roof 18 can be transmitted to the post 272 which will then urge the laterally extending posts 274 to abut against beams 276 to prevent outward bulging of the pillars 24.

Referring to FIG. 31 of the drawings, there is shown a wire mesh material of elliptical section, but which, between adjacent wires, are twisted to present the maximum width as a greater depth. It is believed that such wires of elliptical section, when twisted in this fashion, will provide increased grip and greater frictional resistance to movement within the particulate backfill material.

The invention accordingly extends also to a method of mining coal in coal mines, which includes the step of having the work face more advanced in some places than in others, there being provided pillars extending backwardly from a work space immediately behind the work face, the leading ends of the pillars being aligned with those parts of the work face which are more advanced than the other parts of the work face.

It will be realised that the length of various pillars may vary depending upon working conditions, access to workings for men, and movement of materials. The length of a pillar in a particular location may accordingly be as small as twice its width. In another, more remote, location it may extend continuously. Where possible, continuous pillars will be preferred because of cost savings in not having to make off ends in a manner similar to the sides.

The step of consolidation of a layer of particulate backfill, may include the use of cementing materials or synthetic chemical materials to promote cohesion in the backfill.

When formwork or temporary structures are used while backfilling and consolidation of the particulate backfill material is in progress, then such formwork and structures will be capable, where necessary, of resisting all the pressures resulting from the construction of the pillar.

The slight camber which the platen 140 may have (see FIG. 19), will assist in providing good access for effective grouting. The grout layer 134 is intended to have the same width as the pillar and may be constrained between removable forms, while setting. Such removable forms will exert pressure on the grout and will prevent grout breaking out.

The width of the pillar, while depending upon the thickness of the coal seam, the depth below the surface, and the condition of the hanging wall, will also depend upon the availability, quality, and nature of the backfill. The use of burnt ash and dolomite and limestone, besides providing a cementitious binding material in the backfill, will also mitigate against corrosive attack of reinforcing material in the pillar structure. It will also provide savings in the cost of the backfill.

The vertical spacing between successive reinforcing layers 32, will be the subject of design by considering the load which the pillar is expected to take, the nature of the backfill, the cost of the reinforcing layers 32, and the economic gain which is to be achieved by making use of the pillar in winning material otherwise lost economically when left in situ for support. The vertical spacing between successive layers of lateral constraint means will vary depending upon its position in the pillar.

The pillars will be designed to take loads which will be less than those which will cause them to collapse or fail due to diagonal stress. In other words, the maximum resistance of the pillar to diagonal tensile stress will be above that imposed by the load which the pillar carries.

The use of the mobile press reduces the degree of deflection of the pillar under load to a minimum when the pillar receives its full loading of the roof. Such minimum deflection also reduces to a minimum the subsidence of strata and other movement.

It is believed that even if the loading on the pillar increases so that it fails under diagonal tension along the planes 242 and 244, then the pillar will yield gradually and will not fail by shattering as when brittle material shatters under excessive compressive loads.

This method of supporting a hanging wall according to the invention, may be used advantageously by building pillars or running pillars according to the invention between in situ pillars left for support in mined-out

148.1, or indirectly via a spacer 254, as shown by jacks 148 (see FIG. 22).

When the seam being mined is very thick, then the working face 12 may be worked in steps 12.1, 12.2 and 12.3.

The spacing between layers of reinforcing mesh 32 is given by H1. Here again, the strength of the mesh will be determined by the horizontal component of the vertical loading which the mesh is expected to take when the pillar is subjected to its vertical load. The strength of the mesh will be chosen with a suitable factor of safety being taken into account, say, 1,6.

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It is, of course, possible for the pre-loading, to take place to the same value as the stress B, or even slightly beyond it, say, to a point C1. This will then ensure that the particulate material in the pillar has been fully consolidated by being pre-loaded so as to ensure that the load from the hanging wall will be taken with minimum or no deflection or deformation of the pillar. The degree of deflection or deformation of the pillar under pre-loading or precompression is expected to be about ⅛ or 1/16 of the original height of the pillar.

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Referring to FIG. 31 of the drawings, there is shown a wire mesh material of elliptical section, but which, between adjacent wires, are twisted to present the maximum width as a greater depth. It is believed that such wires of elliptical section, when twisted in this fashion, will provide increased grip and greater frictional resistance to movement within the particulate backfill material.

The invention accordingly extends also to a method of mining coal in coal mines, which includes the step of having the work face more advanced in some places than in others, there being provided pillars extending backwardly from a work space immediately behind the work face, the leading ends of the pillars being aligned with those parts of the work face which are more advanced than the other parts of the work face.

It will be realised that the length of various pillars may vary depending upon working conditions, access to workings for men, and movement of materials. The length of a pillar in a particular location may accordingly be as small as twice its width. In another, more remote, location it may extend continuously. Where possible, continuous pillars will be preferred because of cost savings in not having to make off ends in a manner similar to the sides.

The step of consolidation of a layer of particulate backfill, may include the use of cementing materials or synthetic chemical materials to promote cohesion in the backfill.

When formwork or temporary structures are used while backfilling and consolidation of the particulate backfill material is in progress, then such formwork and structures will be capable, where necessary, of resisting all the pressures resulting from the construction of the pillar.

The slight camber which the platen 140 may have (see FIG. 19), will assist in providing good access for effective grouting. The grout layer 134 is intended to have the same width as the pillar and may be constrained between removable forms, while setting. Such removable forms will exert pressure on the grout and will prevent grout breaking out.

The width of the pillar, while depending upon the thickness of the coal seam, the depth below the surface, and the condition of the hanging wall, will also depend upon the availability, quality, and nature of the backfill. The use of burnt ash and dolomite and limestone, besides providing a cementitious binding material in the backfill, will also mitigate against corrosive attack of reinforcing material in the pillar structure. It will also provide savings in the cost of the backfill.

The vertical spacing between successive reinforcing layers 32, will be the subject of design by considering the load which the pillar is expected to take, the nature of the backfill, the cost of the reinforcing layers 32, and the economic gain which is to be achieved by making use of the pillar in winning material otherwise lost economically when left in situ for support. The vertical spacing between successive layers of lateral constraint means will vary depending upon its position in the pillar.

The pillars will be designed to take loads which will be less than those which will cause them to collapse or fail due to diagonal stress. In other words the maximum resistance of the pillar to diagonal tensile stress will be above that imposed by the load which the pillar carries.

The use of the mobile press reduces the degree of deflection of the pillar under load to a minimum when the pillar receives its full loading of the roof. Such minimum deflection also reduces to a minimum the subsidence of strata and other movement.

It is believed that even if the loading on the pillar increases so that it fails under diagonal tension along the planes 242 and 244, then the pillar will yield gradually and will not fail by shattering as when brittle material shatters under excessive compressive loads.

This method of supporting a hanging wall according to the invention, may be used advantageously by building pillars or running pillars according to the invention between in situ pillars left for support in mined-out areas. This makes possible the recovery of coal from such in situ pillars without increasing the danger of the hanging wall coming down. The value of the coal to be extracted will, of course, have to be balanced against the cost of making such pillars, to ensure that it will be economically possible to extract such coal.

The particulate material used as a backfill in carrying out this invention should preferably be strong and have high inter-particulate friction.

I claim:

1. In mining, a method of supporting the hanging wall which includes providing support pillars between the hanging wall and the foot wall, by

building lower portions of the pillar to include particulate backfill material, and

using jacking means in clearance spaces above the backfill and below the hanging wall to bear against the hanging wall to exert a downward pressure on the particulate backfill material, thereby obtaining consolidation of the backfill material in the lower portions of the pillars.

2. A method as claimed in claim 1, in which the backfill material is fine such that about half by mass would pass a sieve of 0.85 mm, and such that only 1% by mass would be retained on a sieve size of 6.76 mm.

3. A method as claimed in claim 1 in which, after consolidation of the backfill in the lower portion of a pillar the clearance space is filled under pressure with load-taking fill which thereby exerts a downward pressure on this lower portion by bearing against the hanging wall.

4. A method as claimed in claim 1, in which the downward pressure exerted on a lower portion before filling of the clearance space, is at least 9/10 of the load which the pillar is expected to take.

5. A method as claimed in claim 4, in which the downward pressure exerted on a lower portion before filling of the clearance space takes place, is in excess of the load which the pillar is expected to take.

6. A method as claimed in claim 1, in which the lower portion of a pillar is built up in layers of backfill with layers of reinforcing material at different elevations between layers of backfill in the lower portion.

7. A method as claimed in claim 6, in which the vertical spacing between layers of reinforcing material is at the most equal to one third the minimum cross-sectional dimension of the pillar.

8. A method as claimed in claim 6, which includes providing retaining walls on opposite sides of a pillar, and in which the reinforcing material engages with the retaining walls to constrain them against outward bulging.

9. A method as claimed in claim 7, in which consolidation of backfill material in a lower portion takes place on one or more layers of backfill at a time.

10. A method as claimed in claim 7, in which the particulate backfill material is made up into the form of gabions which comprise the backfill material contained in envelopes of reinforcing material, and in which the lower portion of a pillar is built by laying the gabions in layers.

11. A method as claimed in claim 10, in which at least some of the gabions are precompressed before being laid.

12. A method as claimed in claim 1, in which reinforcing material is provided in the backfill material in the form of a slab, metal plate, fibreglass or geotextile cap, or the like, at a height which is at the most equal to the minimum cross-sectional dimension of the pillar.

13. A method as claimed in claim 1, in which a pillar is reinforced against outward bulging under load, by having beams on opposite sides extending generally parallel to one another, and by having tie members passing transversely through the pillar and through the beams, and placing the tie members under stress, thereby tying the beams on opposite sides of the pillar together.

14. A method as claimed in claim 13, in which the tie members are in the form of bolts or steel wire ropes which have been sheathed in outer protective sheaths.

15. A method as claimed in claim 13, in which the tie members are spaced along the length of the pillar at intervals at the most equal to the minimum cross-sectional dimension of the pillar.

* * * * *

PPENDIX 9

Support in shallow mines using horizontally reinforced systems

by J. A. HAHN* (Pr. Eng., B.Sc. (Eng.), F.I.C.E., F.S.A.I.C.E.), G. E. BLIGHT† (D.F.C., B.Sc. (Eng.)) and L. DISON* (Pr. Eng., D.Sc. (Eng.), M.A.S.C.E., F.S.A.I.C.E.)

SYNOPSIS

A system of artificial support for shallow mines is described, analysed, and evaluated. The system can be designed rationally to support a predetermined overburden load, and tests have shown that the design theory gives accurate predictions of strength. The support system, consisting as it does of two basic materials, reinforced granular material and reinforced cemented material, is flexible and can be varied to suit changing requirements of height and load.

The system of preloading to the design load by jacking off the roof ensures that the walls are both stiff and proof-tested.

SAMEVATTING

Daar word 'n stelsel van kunsdaksteune vir vlak myne beskryf, ontleed en geevalueer. Die stelsel kan rasioneel ontwerp word om 'n voorafbepaalde bolaaglas te dra en toetse het getoon dat die ontwerpteorie akkurate voorspellings van die sterkte gee. Die steunstelsel wat uit twee basiese stowwe, gewapende korrelrige materiaal en gewapende gesementeerde materiaal, bestaan, is buigsaam en kan verander word om wisselende hoogte- of lasvereistes te pas.

Die stelsel van voorbelasting tot by die ontwerplas deur die dak op te domkrag verseker dat die wande stewig en geproeftoets is.

Introduction

In shallow mines, the replacement of pillars by artificial supports could permit complete extraction of the seam being mined. Almost complete extraction is now achieved by stoping or by long walling, but these methods can cause severe settlement of the surface and severe surface distortion above the boundaries between areas where extraction has been complete and areas that remain supported. Such practices permanently affect the land surface; damage surface roads, railways, powerlines, and other installations, affect surface drainage patterns; destroy the surface ground water table; and generally have a permanently adverse effect on the surface environment.

Pressure will mount in future for mining to optimize the utilization of resources by increasing the percentage of extraction while at the same time protecting and preserving the surface environment and, if possible, reducing accumulations of mining waste on the surface. The authors believe that permanent artificial support systems can be designed to achieve these objectives rationally and economically. In addition, artificial roof support would be valuable where a sequence of several seams is available for mining. Without such support, the lower seams in a sequence cannot be extracted totally until mining of the upper seams has been completed.

This paper proposes a system of support using horizontally reinforced materials in which the support system can be accurately designed to accept a load predetermined from the mining situation with a predetermined load factor. Two sub-systems are proposed: one using placed *in situ*, horizontally reinforced granular materials, and the other using reinforced precast cemented elements.

A list of the symbols and terms used is given at the end of the paper.

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Support Requirements in Shallow Mines

For the purposes of this paper, a shallow mine is defined as one in which the lateral extent of the workings is large in comparison with their depth below the surface. For such a mine, the support must be competent to carry the full overburden load if settlement of the surface is to be avoided. The geometry of the situation is illustrated in Fig. 1.

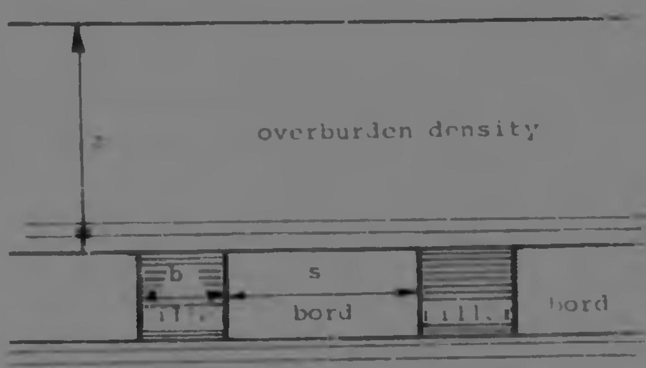

Fig. 1—The geometry of bords and pillars

If σ is the stress required to be carried by the walls or pillars, then, for parallel walls,

$$\sigma = \frac{\gamma z(b+s)}{b} = \gamma z(1+s/b)$$

or $\sigma/\gamma z = 1 + s/b$ . . . (1a)

For square pillars on a square grid,

$$\sigma = \frac{\gamma z(b+s)^2}{b^2} = \gamma z(1+s/b)^2$$

or $\sigma/\gamma z = (1+s/b)^2$. . . . . (1b)

Relationships (1a) and (1b) are illustrated graphically in Fig. 2.

In addition to pillars or walls that are strong enough in compression to carry the overburden load, the following requirements must also be met.

(*a*) The stress σ applied by the supports to the roof or floor must not be so high as to cause a danger of

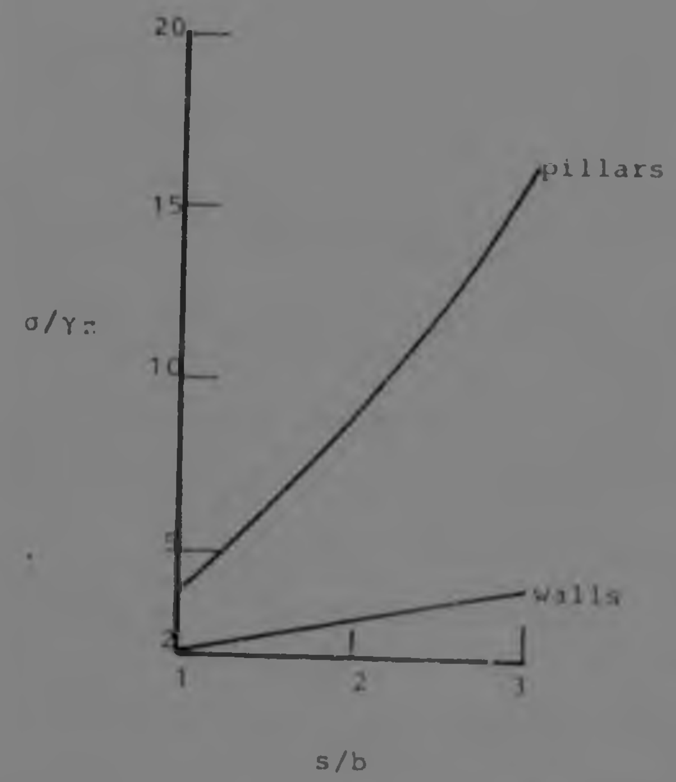

Fig. 2—The relationship between pillar stress/overburden, $\sigma/\gamma z$, and bord span/pillar width, s/b, for pillars and walls

punching failure of the roof or floor.

(*b*) The roof must be capable of spanning between the supports without any danger of bending tensile failure.

(*c*) The supports will compress as load is transferred onto them. This compression will affect bending moments in the roof and the distribution of load in supports. The compression must be small enough for requirements (*a*) and (*b*) above to continue to be met.

(*d*) The supports must be durable and must retain their integrity indefinitely.

Requirements (*a*) and (*b*) are largely site specific since they depend on the properties of the roof and floor material. Requirement (*c*) is partly a property of the pillars and will be examined in more detail.

Fig. 3(a) shows a typical cross section through a shallow mine adjacent to a working face. The roof can be regarded as the equivalent beam illustrated in Fig. 3(b), which has an equivalent fixed support at some distance behind the face. In this analysis, the distance is assumed to be $b/2$. The roof carries a distributed load in bending that will not be uniform, but will be concentrated over the supports and less at midspan.

Suppose that, as shown in Fig. 4, the reinforced wall supporting the roof at B compresses by an amount $\triangle$ relative to the coal support at C. If the tangents to the roof beam at B and C remain horizontal (which would represent the worst conditions), moments M will be induced in the beam as shown. These two clockwise moments will cause additional reactions R to be generated at the supports, the total reaction at C being increased by R while that at B is reduced by an equal amount.

In terms of the dimensions of Fig. 4,

$$M = \frac{6EI\triangle}{(s+b)^2} \quad \ldots \ldots \ldots \ldots (2a)$$

$$\text{and } R = \frac{2M}{(s+b)} = \frac{12\triangle EI}{(s+b)^3} \quad \ldots \ldots \ldots (2b)$$

in which E is the elastic modulus of the roof beam and

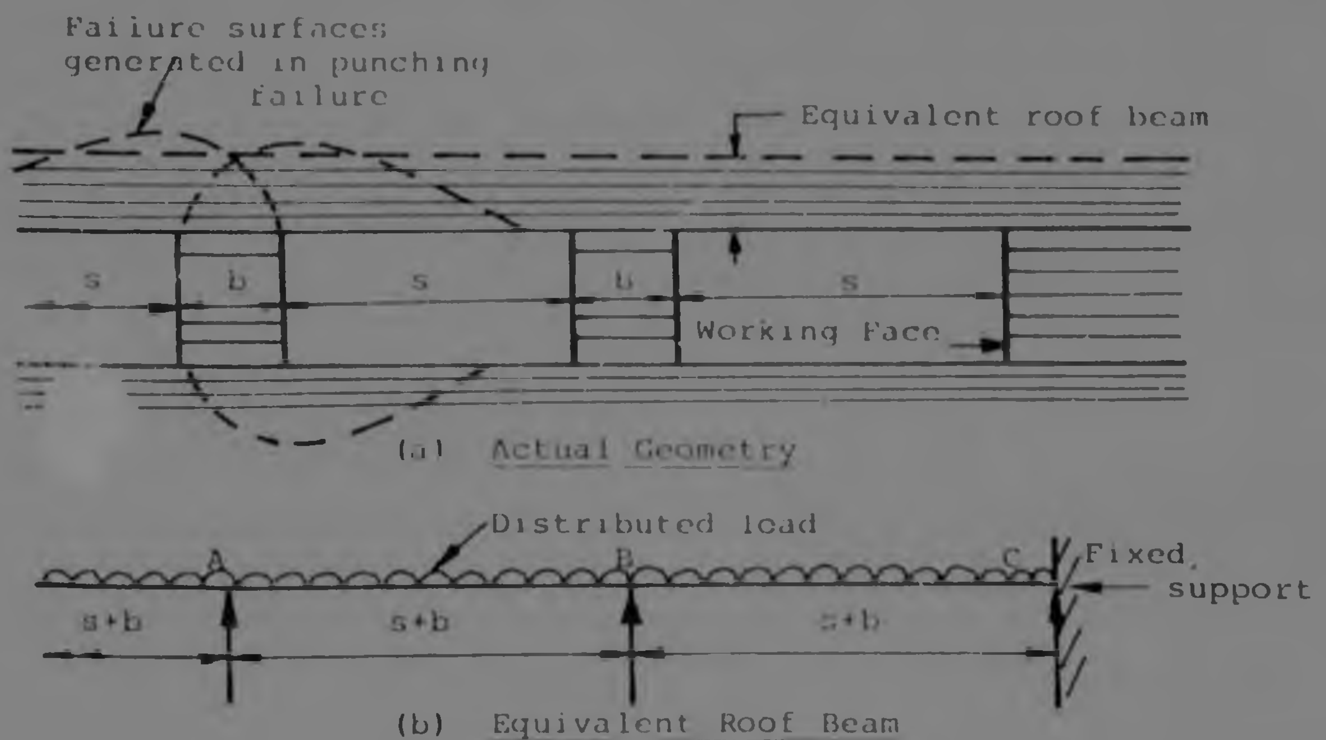

Fig. 3—Support requirements in a shallow mine

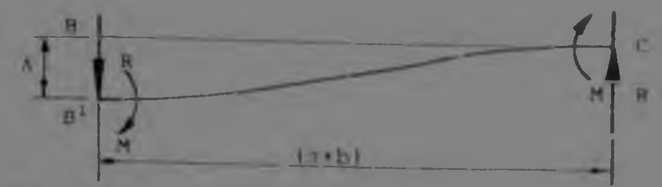

Fig. 4—Deflected shape resulting from relative compression of wall and abutment

I is the second moment of area of its cross-section.

If the depth of the roof beam is δ,

$$I = \frac{\delta^3}{12} \text{ per unit width of roof}$$

$$\text{and} \quad R = \triangle E\left(\frac{\delta}{s+b}\right)^3$$

$$\text{or} \quad \triangle = \frac{R}{E}\left(\frac{s+b}{\delta}\right)^3 \qquad (2c)$$

The increase in tensile bending stress at B will be

$$\sigma_b = \frac{M\,\delta/2}{I} = \frac{3\triangle E\,\delta}{(s+b)^2} \qquad (3)$$

Suppose that initially load W per unit length is carried at both B and C. After the differential compression has occurred,

B will carry $(W-R)$
and C will carry $(W+R)$

If E_w is the compression modulus of the wall and E_c that of the coal, the differential compressive strain between B and C will be

$$\frac{(W-R)}{bE_w} - \frac{R}{bE_c} = \frac{W}{bE_w} - \frac{R}{b}\left(\frac{1}{E_w} + \frac{1}{E_c}\right) = \frac{\triangle}{H}. \qquad (4a)$$

where the height of the coal seam is H.

The corresponding expression for an equivalent bord-and-pillar operation would be

$$\frac{(W-R)}{bE_c}\left(\frac{s+b}{b}\right) - \frac{R}{bE_c} =$$

$$\frac{1}{E_c}\left\{\frac{W}{b}\left(\frac{s+b}{b}\right) - \frac{R}{b}\left[\frac{s+b}{b} + 1\right]\right\} = \frac{\triangle}{H} \qquad (4b)$$

Here, the differential compression would result from subdividing a continuous wall of coal of width b into a series of pillars measuring b by b and s apart.

Some numerical examples of the above expressions are given in the Addendum. It is not claimed that the simplified approach set out above will give an entirely accurate representation of the stresses set up in the roof and coal abutments. However, the expressions enable the effect to be assessed of varying the span and width of the reinforced walls, the wall stiffness, etc.

Horizontal Reinforcing for Granular Material

The effect of horizontal reinforcing on granular materials is to develop a horizontal confining stress. The horizontally reinforced granular material will tend to go into a state of failure as soon as a vertical stress is applied to it. However, actual failure will not occur because of the development of tension in the horizontal reinforcing. As Fig. 5 indicates, on the Mohr diagram or the p-q diagram, the stress path oa of the granular material will follow the failure line for the material. On the stress path, a represents the point at which the horizontal reinforcement reaches its yield stress. At this stage the vertical stress σ will be related to the horizontal stress σ_h by the relationship

$$\sigma = K_p \sigma_h, \qquad (5a)$$

where

$$K_p = \frac{1+\sin\phi}{1-\sin\phi} \qquad (5b)$$

and ϕ is the angle of shearing resistance of the granular material (in Fig. 5, $\sin\psi = \tan\phi$).

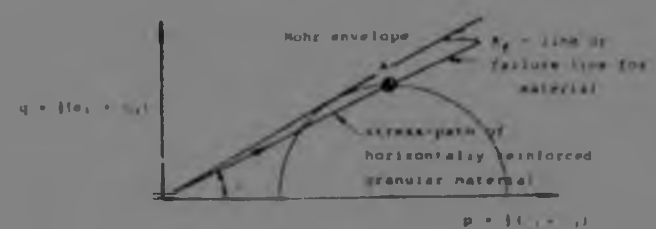

Fig. 5—Stress path for reinforced granular material

Equating the tension in the horizontal reinforcing to the compression in the granular material,

$$\sigma_{yR} A_R = \sigma_h (A_m - A_R)$$

$$\text{or} \quad \sigma_h = \frac{\sigma_{yR} A_R}{(A_m - A_R)}. \qquad (6a)$$

Hence

$$\sigma = \sigma_{max} = \frac{K_p \sigma_{yR} A_R}{(A_m - A_R)}. \qquad (6b)$$

If A_R is small compared with A_m,

$$\sigma = K_p \cdot \sigma_{yR}\, p \qquad (6c)$$

The strength of a horizontally reinforced granular material is thus directly proportional to the reinforcing ratio p and the yield stress of the reinforcing σ_{yR}.

Bond of Reinforcing to Reinforced Material

When a material is reinforced, the tension developed in the reinforcing depends on stress transfer by bond between the reinforcing and the reinforced material. The mechanism of bond development is more conveniently demonstrated in the case of a granular material, although a similar mechanism applies in the case of cemented materials. The mechanism of bond development is illustrated in Fig. 6.

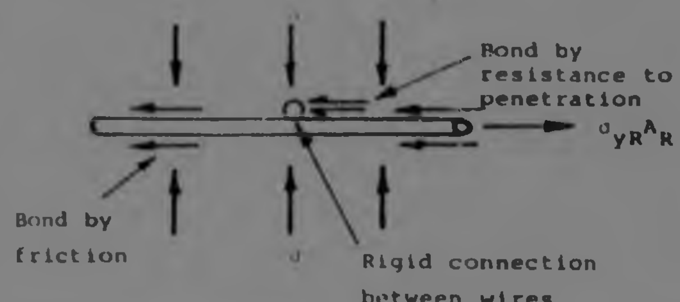

Fig. 6—Mechanism of bond development in reinforced granular material

The average bond stress transmitted to a reinforcing wire over a length x by friction is

$$\frac{\sigma}{2}\left(1 + \frac{1}{K_p}\right)\cdot\tan\phi\pi D\frac{x}{2},$$

in which the diameter of the wire is D.

The best form of horizontal reinforcing consists of a system of orthogonal wires rigidly bonded to each other at points where they cross. Readily available systems consist of either square or rectangular welded mesh or twisted diamond mesh. The wires that run at an angle to the direction of tension under consideration contribute considerably to the bond because they must penetrate the reinforced material before the wire can slip relative to the material. If the bond wires in an orthogonal system are spaced l apart, the load transferred through these wires in length x will be a minimum of

$$\frac{x}{l}\left(\frac{\sigma}{K_p}d + 2d\sigma\tan\phi\right)h,$$

in which d is the diameter of the bond wires and h the spacing of the main tension wires. Full bond resistance of the wire is therefore developed in a length x given by:

$$\frac{\pi D^2}{4}\sigma_{yR} =$$

$$\frac{\sigma}{4}\left(1 + \frac{1}{K_p}\right)\tan\phi\pi Dx + \frac{\sigma}{4}\frac{d}{l}\left(\frac{4}{K_p} + 8\tan\phi\right)h\,x,$$

$$\text{i.e. } x = \frac{\pi D^2\,\sigma_{yR}/\sigma}{\pi D\left(1 + \frac{1}{K_p}\right)\tan\phi + \frac{4dh}{l}\left(\frac{1}{K_p} + 2\tan\phi\right)} \qquad (7)$$

The value of x is astonishingly small. For example,

if $\sigma = 5$ MPa, $\sigma_{yR} = 600$ MPa
$D = d = 2{,}5$ mm, $K_p = 3{,}5$
$h = l = 25$ mm, $\tan\phi = 0{,}7$,

then $x = 98$ mm.

Fig. 7 illustrates the results of tests on reinforced walls that were designed to investigate the validity of equation (7). Fig. 7(a) shows the stress at failure in a series of tests on walls in which the number of bond wires at each side of the wall was increased progressively from zero. The spacing of the bond wires was 12,5 mm and the theoretical bond length was 60 mm; hence, full bond could theoretically be achieved with 5 bond wires. The results in Fig. 7(a) confirm this. Fig. 7(b) shows the compression of this series of walls at failure and illustrates that the compression at failure was reasonably constant at 15 to 16 per cent provided the requirements for full bond were satisfied. Where insufficient bond length was provided, the compression at failure increased enormously. Here again, the validity of equation (7) is established.

Horizontal Reinforcing for Cemented Material

It is often observed that brittle materials such as rock, unreinforced concrete and brickwork fail in uniaxial compression by splitting parallel to the direction of stress. A simple explanation of this phenomenon can be given in the following terms.

Assume that the brittle material has a stress-strain curve in uniaxial compression that is linear up to failure. Before failure, the vertical and lateral strains in the

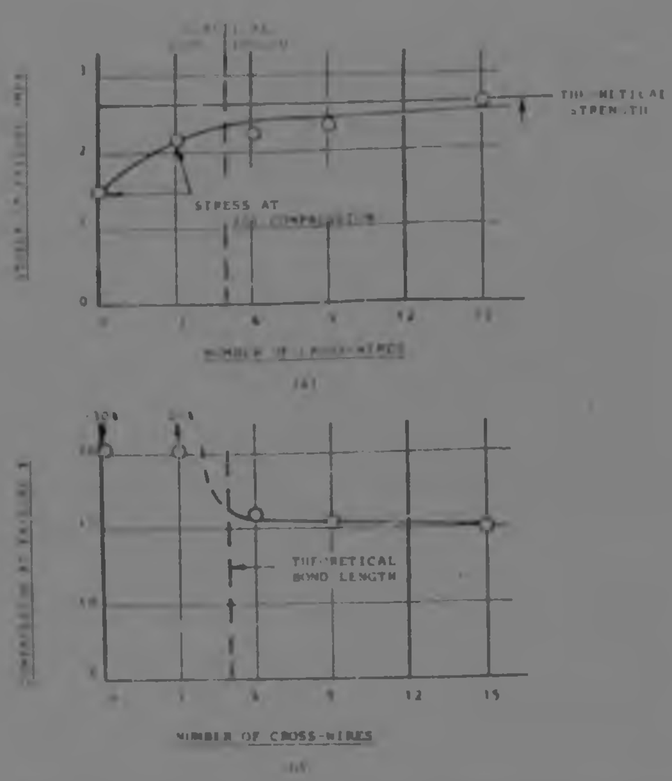

Fig. 7—Investigation of bond length requirements for reinforced granular material

material under a stress σ are given by

$$\epsilon_v = -\frac{\sigma}{E_m}, \quad \epsilon_h = \frac{\nu\sigma}{E_m} \quad \ldots\ldots\ldots\ldots (8)$$

(a negative sign denotes compression), where E_m is the elastic modulus of the material and ν is Poisson's ratio. Most brittle or cemented materials have a Poisson's ratio of about 0,1. Hence,

$$\epsilon_h = -\frac{\epsilon_v}{10}.$$

The failure strain in tension, however, is also about one-tenth of that in compression for most brittle or cemented materials. Hence, failure by horizontal tension either coincides with or precedes failure by vertical compression. It is fairly obvious that it should be possible to increase the compressive strength of these materials by restraining the lateral tensile strain using horizontal reinforcing.

Suppose that the introduction of area A_R of horizontal reinforcing having a Young's modulus E_R reduces the lateral strain under σ to a value ϵ_h^1. The stress in the reinforcing will then be $E_R\epsilon_h^1$.

For horizontal equilibrium, the tensile force induced in the reinforcing must be balanced by a compressive force in the cemented material, i.e.

$$\epsilon_h^1 E_R A_R = (\epsilon_h - \epsilon_h^1)E_m(A_m - A_R).$$

$$\text{Hence, } \epsilon_h^1 = \frac{\nu\sigma(A_m - A_R)}{E_R A_R + E_m(A_m - A_R)} \quad \ldots\ldots (9)$$

As σ is increased, ϵ_h^1 will increase until it reaches a limiting value, at which the cemented material will fail by vertical splitting. For brittle materials such as concrete and brick, the limiting value of ϵ_b^1 is about $200 \cdot 10^{-6}$. At that stage, σ will reach a maximum value, the compressive failure stress for the reinforced material.

In the limit,

$$\epsilon_b^1 = 200 \cdot 10^{-6} = \frac{\sigma_{max} \nu (A_m - A_R)}{E_R A_R + E_m (A_m - A_R)} \quad (10)$$

or

$$\sigma_{max} = \frac{200 \cdot 10^{-6} \{E_R A_R + E_m (A_m - A_R)\}}{\nu (A_m - A_R)}$$

If A_R is small in comparison with A_m (typically about 1 per cent of A_m),

$$\sigma_{max} = \frac{200 \cdot 10^{-6} (E_R A_R + E_m A_m)}{\nu A_m} \quad . \ (10a)$$

When the ratios $p = A_R / A_m$ and $a = E_R / E_m$ are introduced and $\nu = 0,1$,

$$\sigma_{max} = 2000 \cdot 10^{-6} \mathrm{Em} \, (a \, p + 1) \quad \ldots\ldots\ldots (11)$$

or

$$\sigma_{max} / E_m = 2000 \cdot 10^{-6} (a \, p + 1) \quad \ldots\ldots (11a)$$

Note that, if p tends to zero, σ_{max} tends to $2000 \cdot 10^{-6} E_m$, which approximates the crushing strength of the unreinforced cemented material. Equation (11) represents a family of straight lines relating the ratio σ_{max}/E_m to p for given values of a. Fig 8 shows typical relationships between σ_{max}/E_m and p for a range of values of a.

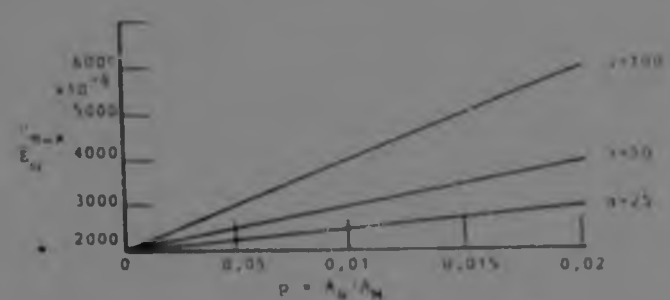

Fig. 8—Graphical representation of the equation for the strength of reinforced brittle material

Design of Walls or Pillars of Reinforced Granular Material

Granular materials are reinforced with layers of steel mesh placed at a vertical spacing v. If h is the horizontal spacing of the wires in the mesh and A_R is the cross-sectional area of each wire, it follows that

$$p = \frac{A_R}{vh},$$

and equation (6c) becomes

$$\sigma_{max} = K_p \sigma_{y R} \frac{A_R}{vh} = F\sigma, \quad \ldots\ldots (12)$$

where F is the factor of safety on the design stress σ.

For selected steel quality and mesh dimensions, v can be calculated; alternatively, for a selected v, a suitable mesh can be selected.

The properties of horizontally reinforced granular materials have been extensively investigated in both model scale and full-scale tests. Fig 9 shows a comparison between σ calculated from equation (12) and corresponding measured values. It will be seen that there is generally an excellent correlation between the calculated and the measured values. Some tests show a measured σ that is considerably less than the calculated value. However, there is a valid explanation for each of these understrength failures. Test 7, for instance, featured a wall reinforced with woven steel mesh in which the contribution to bond from horizontal resistance on the bond wires was small. Failure resulted from bond failure, and the bond wires that should have provided the bond slid across the main tensile reinforcing wires. In contrast, Test 8 featured a wall reinforced with identical woven mesh (although at a different vertical spacing) in which the nodes were tied with wire. This specimen developed the necessary bond and failed in accordance with the calculated stress.

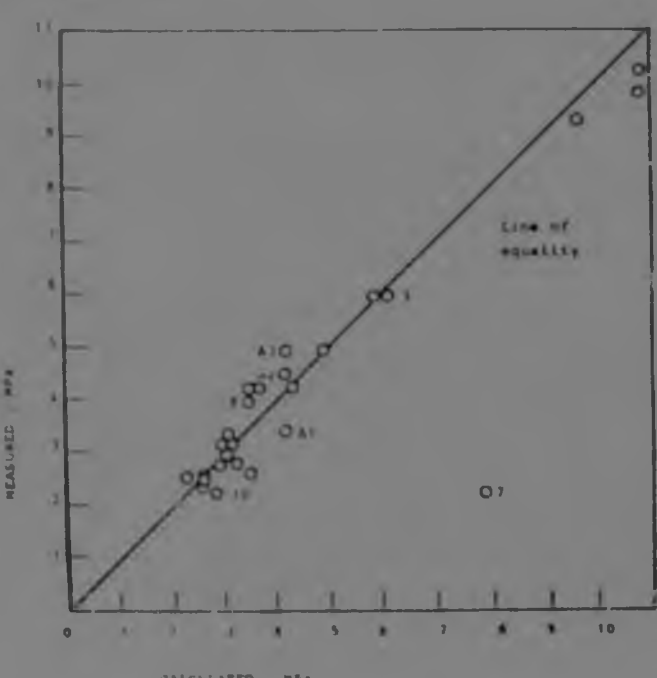

Fig. 9—Relationship between calculated and measured strengths of walls built of horizontally reinforced granular material

Tests A1, A2, and A3 (Figs. 10 to 12) represent the results of three tests in which all three walls were designed to have the same strength, but different combinations of A_R, $\sigma_{y R}$, v, and V were used.

Figs 10 to 12 illustrate an important feature of walls or pillars of horizontally reinforced granular material. Each of these walls was designed to have a factor of safety of 1,6 on a design vertical stress of 2,39 MPa. The walls were loaded to the design load, and were then unloaded and reloaded to failure. The stress-compression curve for the first loading indicates a low compression modulus in each case. However, the reloading curve is considerably steeper, and the reloading modulus is much higher. These diagrams illustrate how it is intended to use the walls of reinforced granular material in practice. After building the walls *in situ*, they will be

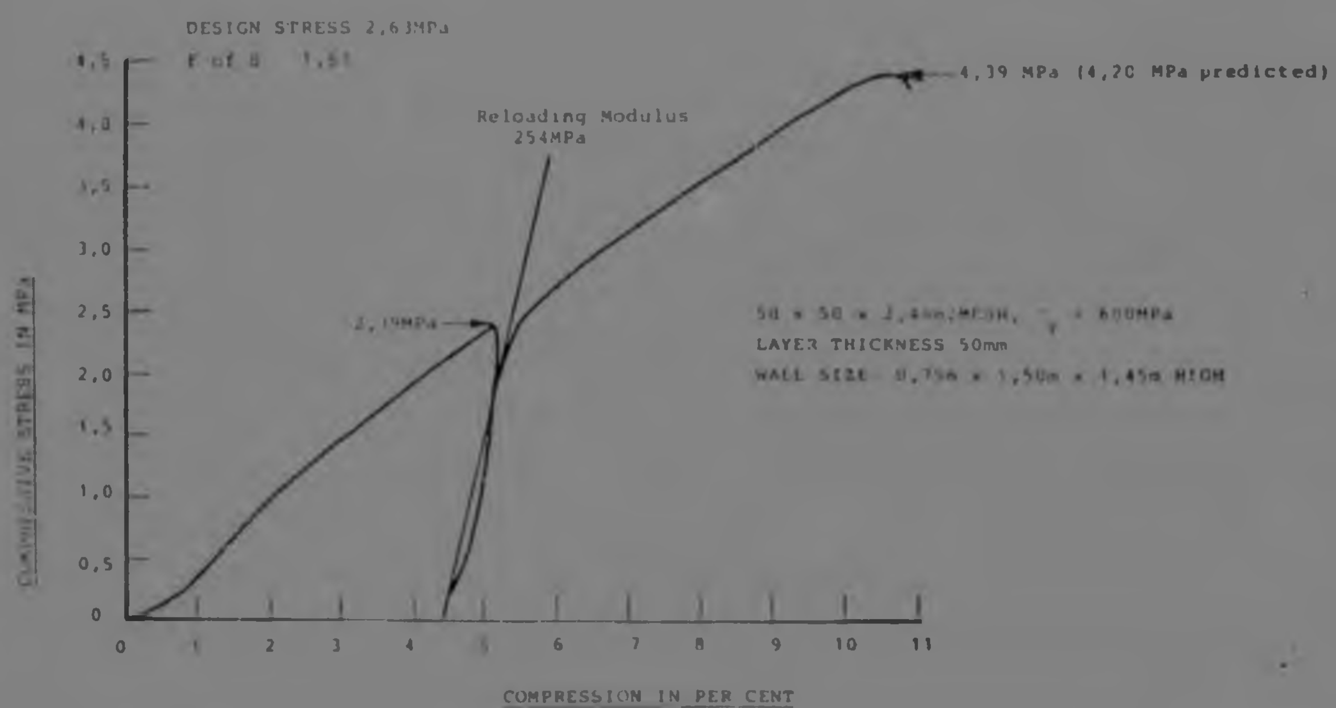

Fig. 10—Stress–compression curve for a wall of horizontally reinforced ash (Wall A1 in Fig. 9)

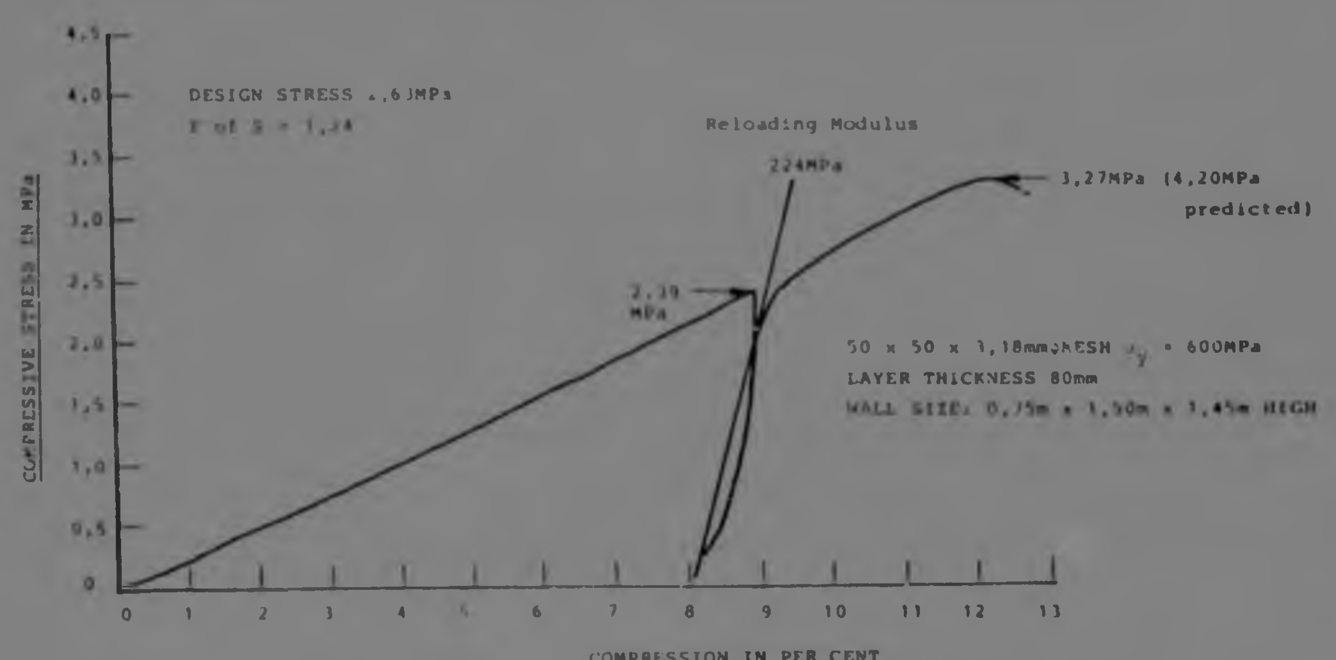

Fig. 11—Stress–compression curve for a wall of horizontally reinforced ash (Wall A2 in Fig. 9)

precompressed and test-loaded to the design load by jacking off the roof. The space created between the top of the wall and the roof by the precompression will then be filled by horizontally reinforced cemented slabs that will be bedded against the roof by grout or wedges. As the load is subsequently transferred to the wall when the roof tends to sag, the wall will be recompressed along the recompression curve back to the design load. The support system thus created will be both relatively stiff and pretested to the design load.

The recompression modulus of horizontally reinforced walls is related to their strength as shown by Fig. 13. The scatter of results is caused mainly by variations in the characteristics of the granular materials used. The upper band of results relates to walls built of a highly frictional weathered quartzite sand. Test 3 featured a wall built of the same sand, but compacted at a water content dry of the optimum for compaction and therefore in a more compressible state. However, as shown in Fig. 9, the strength of the wall was not affected. The lower band of results relates to walls built of power station bottom ash, a highly frictional material that is, however, considerably more compressible than the residual quartzite sand.

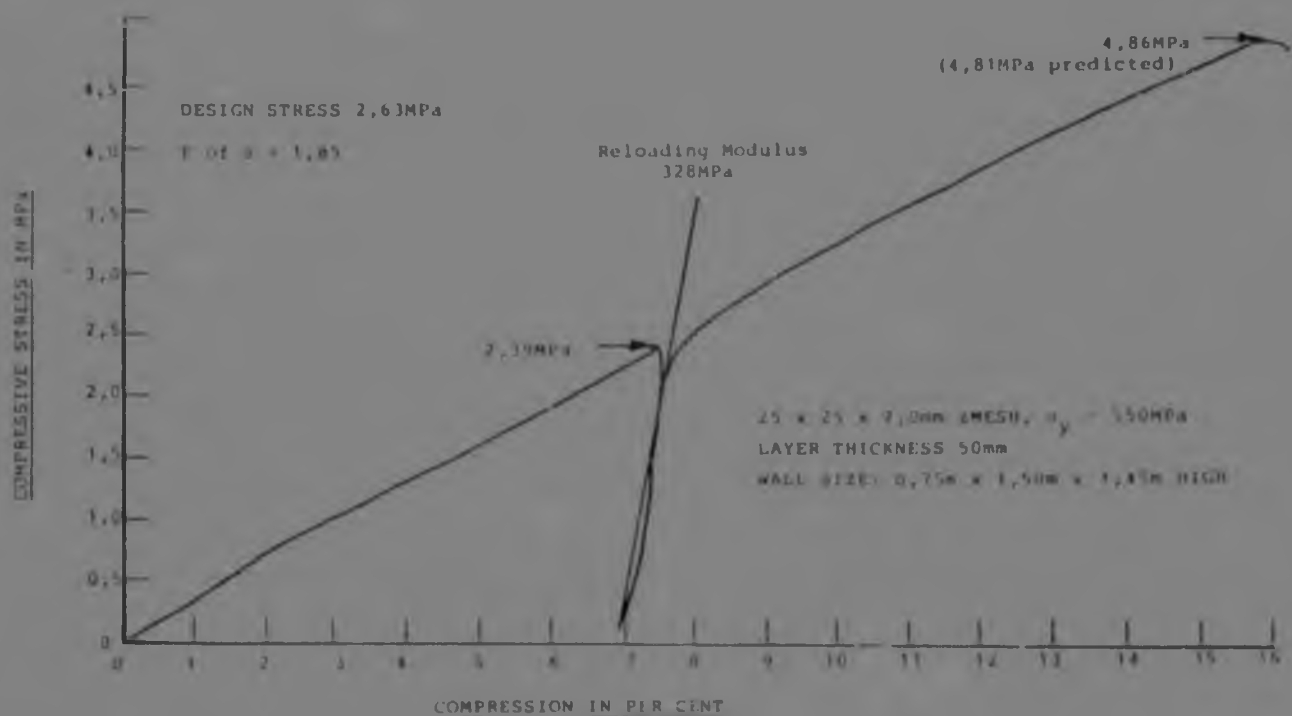

Fig. 12—Stress-compression curve for a wall of horizontally reinforced ash (Wall A3 in Fig. 9)

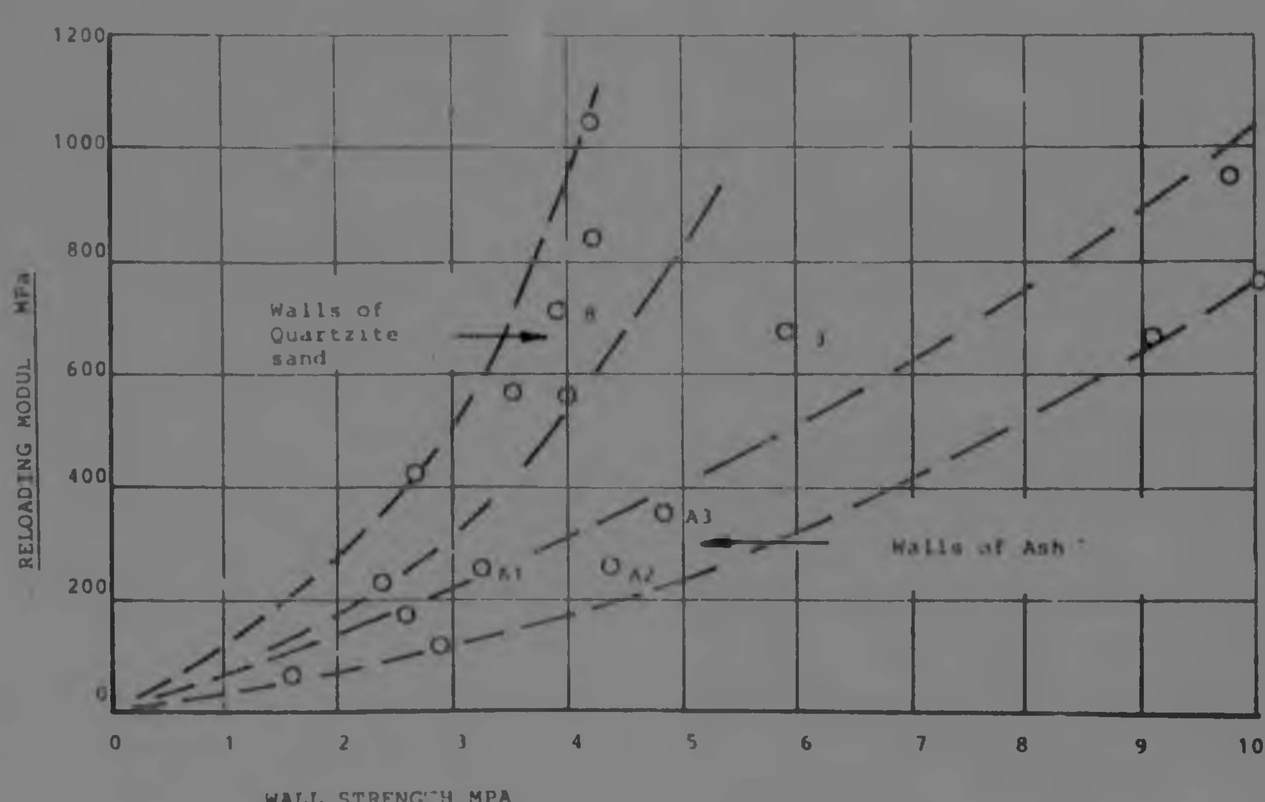

Fig. 13—Relationship between wall strength and compression modulus on reloading for walls of horizontally reinforced granular material

The results shown in Figs. 9 and 13 were obtained in tests on walls ranging from 300 to 1000 mm in width and from 850 to 1500 mm in length. There appears to be no scale effect in this size range, and it is expected that the results will be valid in the prototype size range of widths from 1500 to 2500 mm

Failure Mechanism of Reinforced Granular Walls

As the design theory indicates, reinforced granular walls fail when the tensile strength of the horizontal reinforcing is reached. When this occurs, the reinforcing wires fracture and the granular material shears along diagonal planes inclined at approximately $(45° + \phi/2)$ to the direction of the major principal stress (which is the vertical stress)

Figs. 14 and 15 illustrate the observed positions at which the reinforcing wires fractured in two test walls. Fig. 14 illustrates a case in which multiple fractures of the reinforcing occurred, defining a system of multiple shear planes inclined at a mean angle of 25° to the direction of the major principal stress. This mean angle corresponds exactly to the theoretical angle $(45° - \phi/2)$ for the fill material. Fig. 15 shows the observed positions of the fractures in the wires for a wall in which only a single shear plane developed. Here also, the observed angle of the shear plane corresponds to the theoretical inclination of $(45° - \phi/2)$ to the vertical

Theoretically, it is possible for an infinite number of shear planes to develop in the granular material. In practice, however, the development of shear planes is affected by the frictional restraint exerted on the top and bottom of the wall. The combination of end restraint and the limited height to width ratio of the walls prevents shear planes from developing fully except in the diagonal 'corner to corner' position illustrated in Figs. 14 and 15.

Following on this observation, it appeared that it might be possible to increase the strength of a wall by inhibiting the development of the shear planes. Accordingly, a wall was built that was stiffened by horizontal diaphragms of heavy reinforcing mesh at the third-points of its height, which subdivided it into three short sub walls. However, the strength of this wall (10 in Fig. 9) was no greater than that of corresponding unstiffened walls.

Figs. 16 and 17 show, respectively, a reinforced wall after being tested to maximum load and a sheet of mesh reinforcement from the wall after it had been dismantled. It should be noted that the only sign of distress shown by the failed wall is a slight bulging of the sides, and that the double tensile failure of the transverse reinforcing wire is consistent with the observations summarized in Fig. 14

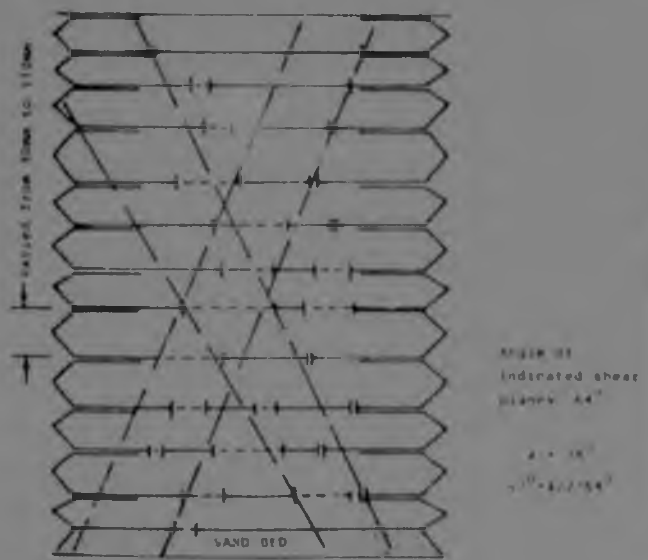

Fig. 14—Positions of tensile failures in the reinforcing of a horizontally reinforced wall of granular material — multiple shear planes

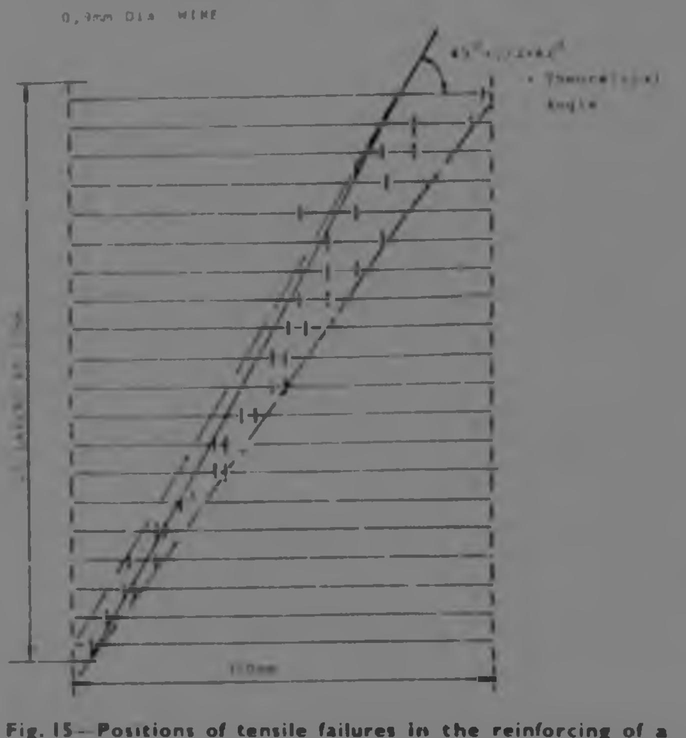

Fig. 15—Positions of tensile failures in the reinforcing of a horizontally reinforced wall of granular material — a single shear plane

Fig. 16—Appearance of a large-scale model wall after being tested to maximum load

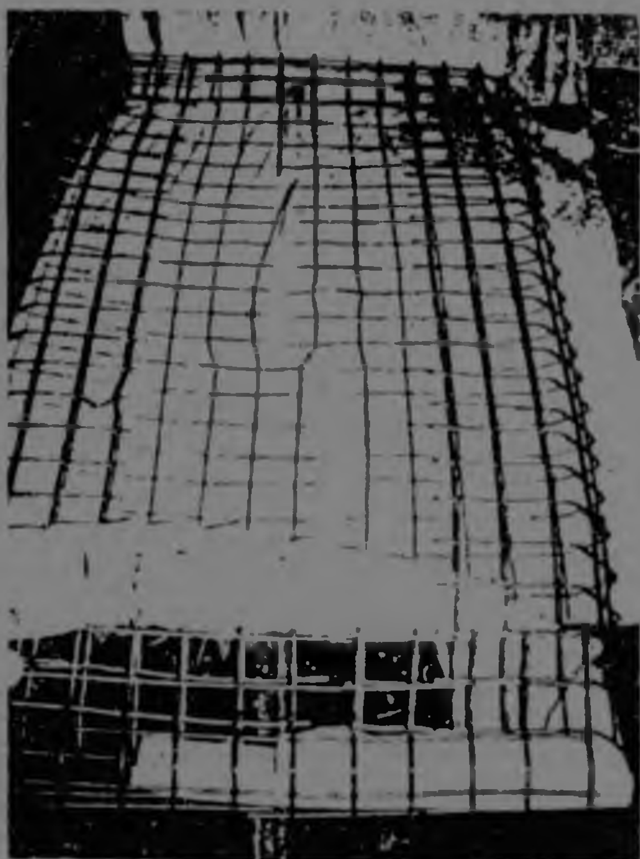

Fig. 17—Appearance of a sheet of mesh reinforcing after test. Note the double tensile failure of the transverse reinforcing wires

Design of Walls or Pillars of Reinforced Cemented Material

The most convenient way of using cemented materials is in the form of precast slabs or wedges laid horizontally with the reinforcing in the joints. The wedges or slabs may be of burnt clay, precast concrete, cemented mine waste, cemented bottom ash, etc. If the vertical spacing of the joints is v and the horizontal spacing of the reinforcing is h,

$$\sigma_{max} = 2000\ 10^{-6} E_m \left[\frac{aA_s}{vh} + 1 \right] \quad \ldots \ldots \quad (11b)$$

If a vertical stress σ is required to be carried with a factor of safety F, then the corresponding σ_{max} is given by

$\sigma_{max} = F\sigma$.

The most efficient form of reinforcing is welded steel square mesh. For a selected steel mesh and the cemented material that is to be used a, A_s, and h are known and hence the value of v that corresponds to the required σ_{max} can be calculated. Alternatively, a convenient value of v can be selected based on handling considerations, and a suitable mesh can be selected. The mesh can either be placed in the joints or it can be cast into the slabs or bricks.

So far, only preliminary confirmatory tests have been carried out on reinforced cemented materials. Specimens have consisted of burnt clay building bricks with steel mesh reinforcing located in the horizontal joints. A number of tests have been performed on small piers measuring 220 mm by 103 mm (i.e. one brick) in plan, and consisting of single bricks laid one above the other and bedded in sand–cement mortar. The bulk of the testing, however, has been on brick 'sandwiches' consisting of two bricks with reinforcing in the mortar bed between them. These 'sandwiches' were compressed between soft board end pieces to eliminate stress concentrations on the ends and to reduce frictional restraint from the platens of the testing machine. All the specimens were tested after being cured in air for 28 days.

Fig. 18 shows the experimental results, which are very similar in form to the predictions of Fig. 8.

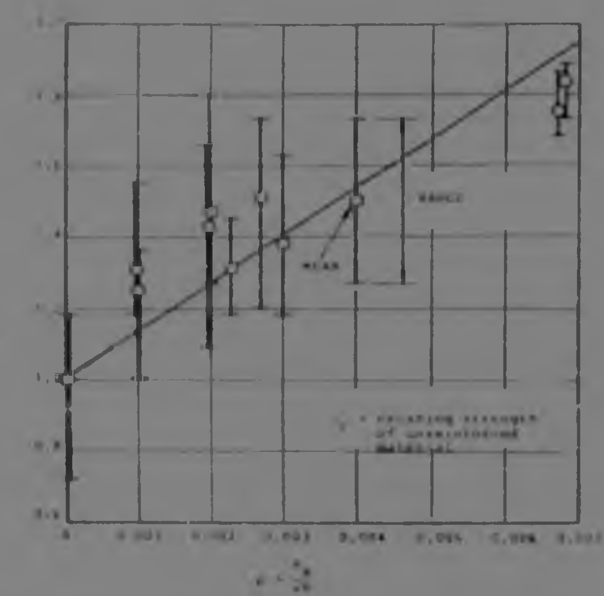

Fig. 18—Observed strength characteristics of horizontally reinforced brittle materials

Walls of Cemented Horizontally Reinforced Granular Material

If the granular material is mixed with a small percentage of Portland cement, it is possible to produce a hybrid wall of cemented granular material.

Fig. 19 compares the stress–compression curve for a wall built of horizontally reinforced granular material with that of a similar wall built of horizontally reinforced cemented granular material. The walls were identical except that the fill of one incorporated 5 per cent ordinary Portland cement. Both walls were precompressed. The uncemented wall was then tested to failure. The cemented wall was wrapped in plastic sheeting to seal in its moisture, and was cured for 25 days before being tested to failure.

The precompression modulus of the cemented wall was about twice that of the uncemented wall, while the failure stress was 1,6 times as large. After the cemented wall had failed, its strength dropped to approximately that of the uncemented wall, indicating that, once the cementation bonds between the granular particles had failed, the wall behaved like a wall of uncemented granular material.

Fig. 19 illustrates another feature of horizontally

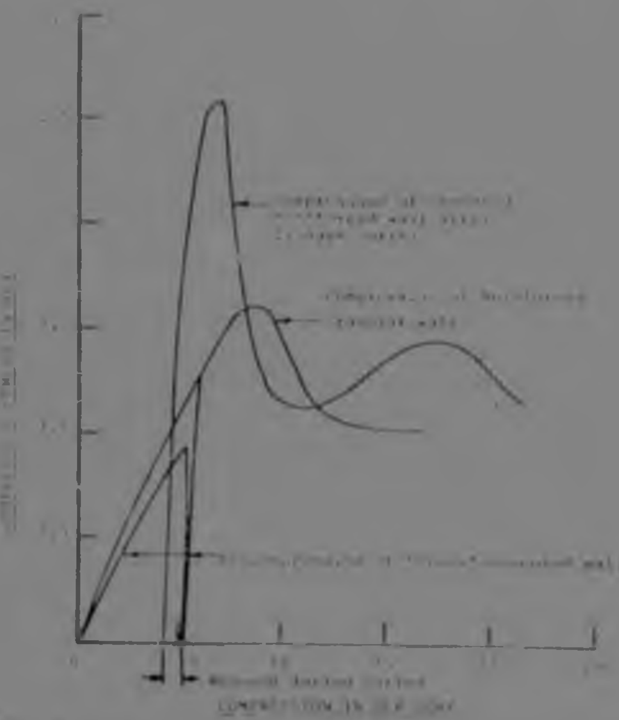

Fig. 19—Comparison of stress–compression curves for horizontally reinforced walls of cemented and uncemented granular material

reinforced walls the fact that, once the failure load has been passed, the wall continues to carry load. (The termination of the stress–compression curves for the walls shown in Figs 10 to 12 represents a deliberate unloading.) Hence, walls of this type are inherently safe.

Relationship between Load and Mass of Reinforcing

The load per unit length carried by a horizontally reinforced granular wall of breadth b supporting spans s (see Fig. 1) is given by

$$W = \frac{K_p \cdot \sigma_{ys} \cdot A_s \cdot b}{F \cdot vh} \quad \ldots\ldots\ldots \quad (13a)$$

$\frac{A_s \cdot b}{vh}$ represents the volume (or the mass) of the reinforcing per unit area of the side area of the wall, per unit length of wall. As the factor $K_p\sigma_{ys}$ is constant for given qualities of granular materials, the mass of reinforcing incorporated in unit length of wall of given height for a given reinforcing quality is directly proportional to the load.

A given load can be supported by one of the following:

(a) a wide wall with a small reinforcing ratio A_s/vh,
(b) a narrow wall with a large reinforcing ratio; or
(c) a narrower wall with a smaller reinforcing ratio, but in combination with reinforcing having a higher yield stress.

Hence, the economics of wall building can be adjusted to suit the quality (K_p) and cost of placing the granular fill, and the quality (σ_{ys}) and cost of the reinforcing.

For a wall of horizontally reinforced cemented material, the load carried by a wall of breadth b supporting spans s is given by

$$W = 2000 \cdot 10^{-6} \frac{bE_m}{vh}\left(\frac{\alpha A_s}{vh} + 1\right) \quad \ldots\ldots \quad (13b)$$

Once again,

$\frac{A_s \cdot b}{vh}$ represents the volume (or the mass) of the reinforcing per unit of side area of the wall. The load carried is now a linear function of the reinforcing ratio and the breadth of the wall.

Durability of Wall Materials

The durability of walls built with slabs of reinforced cemented material should not be inferior to the well-tried durability of reinforced concrete.

The durability of walls of reinforced granular material depends largely on the durability of the steel reinforcement. For this reason, the properties of the granular fill should be such that no corrosion will be induced in the steel. It may be necessary, for instance, to reject fill containing chlorides or sulphates. Alternatively, lime or cement may be added to the fill (at additional cost) to create a high pH environment for the steel and thus inhibit corrosion.

Proposed Underground Trial

As a sequel to the laboratory programme and the concurrent development of a design theory, preparations are being made for tests on the system underground. It is planned to build a series of walls in a section of No. 5 Seam in an eastern Transvaal colliery, and to subject the walls to the full load of the overburden. The design, layout, and performance of the walls will be officially monitored.

Fig. 20 shows a cross-section of the proposed walls, which will comprise the following materials.

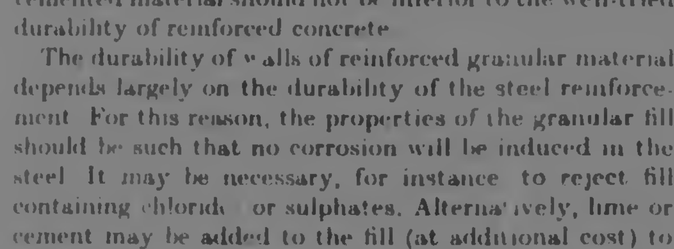

Fig. 20—A typical cross-section of a reinforced wall

Granular Material

The following potential fill materials were used in the laboratory trials: washed concrete sand, weathered quartzite sand, and power station ash. For economic and environmental reasons, it was decided that power-station bottom ash is the most appropriate material for the trial section since it is produced in vast quantities at a nearby power station. Weathered quartzite sand may also be used in certain selected lengths in the trial.

Tests indicate that the ash will not create short-term problems in respect to corrosion of the reinforcement. A programme of research into long-term corrosion will continue.

Side Retaining Elements

Laboratory trials have shown that the prime function of these elements is to retain the fill at the edges of the wall, and that they do not need to resist lateral forces of any magnitude. The elements require to be compressible so that they are able to deflect under precompression.

Woven polypropylene tubes filled with the same granular material as will be used in the main fill adequately fulfil the function of side retainment.

Slabs

These units will comprise a mixture of ash and cement with reinforcing of mild steel mesh. Sand or stone aggregate will be added to the mix if required.

In Situ *Gunited Concrete*

As an alternative to precast slabs, a gunited mixture of ash, sand, and cement, probably lightly reinforced, may be used.

Grout

A grout of ash, sand, and cement will prove suitable.

Steel Reinforcement

This will comprise a welded mild steel mesh fabric as

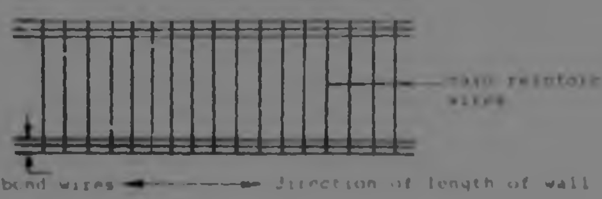

Fig. 21—The proposed welded mild-steel mesh

shown in Fig. 21. Steel reinforcement is the major cost item, and research is therefore continuing into possible new patterns and designs of reinforcement using both mild and high tensile steel.

Construction Methods and Equipment

The principal construction operations envisaged in this system are

(i) procurement, placing, and compaction of backfill,
(ii) fixing of reinforcement,
(iii) fixing of side retaining elements,
(iv) precompression of the reinforced granular material to its design load by jacking,
(v) placing of precast slabs or wedges, and
(vi) final grouting (if wedges are not used).

The trial section underground will occupy a total area of about 1 ha, and the trial will be used to develop construction methods. The experience thus gained will give a lead to the methods to be employed for future large-scale construction.

A self propelled jacking unit that has been designed and built for the trial is shown in Fig. 22. The possibility of adopting pneumatic or hydraulic placing methods involving special equipment is also being investigated. For the rest, standard mining and construction equipment will be used to carry out the various operations.

Power station bottom ash will be transported by road to the mine, conveyed underground through a wide-diameter borehole, and then placed and compacted in

Fig. 22—The self-propelled jacking unit designed and built for the precompression of reinforced wall

the fill. Large scale operations will eventually require a cheaper method of transport for the fill. A hydraulic pipeline may well be used.

Analysis of Costs

At this stage it is not possible to predict the total costs of wall building very accurately. Nevertheless, an attempt has been made in the analysis to arrive at an order of magnitude cost and to relate this to the cost and profit per ton of coal extracted.

Complete Extraction in One Operation

Where coal is completely extracted in one operation using reinforced walls for roof supports, the costs will be as follows, if M is the cost of mining in rands per ton of coal, S the selling price in the same units, and C the cost of wall building in rands per cubic metre of wall. Then, the cost of wall building per ton of coal mined, i.e. the cost of replacing coal pillars by walls, will be

$$\frac{CV_w}{\rho V_c} \text{ R/t,}$$

in which V_w is the volume of walls supporting the space created by extracting an *in situ* volume V_c of coal and ρ is the mass per unit volume of the *in situ* coal.

If $\alpha = \frac{V_c}{V_w}$, then, for a system of parallel walls of width w supporting spans s of roof,

$$\alpha = \frac{w+s}{w} = 1 + \frac{s}{w}. \quad \ldots \quad (14)$$

Hence the profit per ton mined will be

$$P_1 = S - \left(M + \frac{C}{\alpha\rho}\right) \quad \ldots \quad (15)$$

and the total profit from mining V_c of coal will be $V_c\rho P_1$.

Now consider a conventional bord and pillar operation in which the extraction ratio is e_x:

$$e_x = \frac{(s+b)^2 - b^2}{(s+b)^2} = 1 - \frac{(b)^2}{(s+b)^2} \quad \ldots \quad (16)$$

The profit per ton milled will be

$$P_2 = S - M, \quad (15a)$$

but the total profit from the same original volume V_c of coal will be

$$e_x V_c \rho P_2$$

Hence, the additional profit from complete extraction using reinforced walls will be

$$V_c\rho\,(P_1 - e_xP_2) = V_c\rho\left[(1 - e_x)(S - M) - \frac{C}{\alpha\rho}\right],$$

and the additional profit per ton mined will be

$$\triangle P = (1 - e_x)(S - M) - \frac{C}{\alpha\rho}$$

or $100\,\frac{\triangle P}{(S-M)}$ = percentage additional profit resulting from complete extraction.

$$\frac{\triangle P}{(S-M)}\% = \left[(1-e_x) - \frac{C}{(S-M)\alpha\rho}\right]100 \quad (17)$$

The relationship defined by equation (17) is presented graphically in Fig. 23(a) for values of $\alpha = 4$ and $\rho = 1{,}55$ t/m³ and various extraction ratios, e_x.

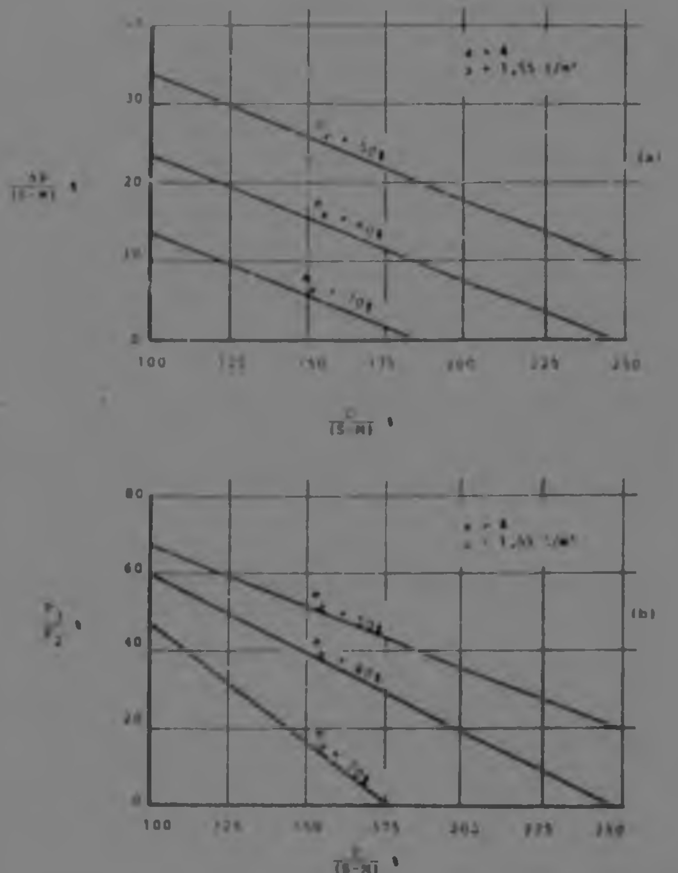

Fig. 23—Cost and profit relationships for mining with reinforced wall support

The additional profit is clearly heavily dependent on the cost of wall building in relation to the price difference $(S-M)$. Current costs are such that $C/(S-M)$ is in the region of 175 to 200 per cent. The major component of C is the cost of steel reinforcing, which can be expected to be less for large-scale operations, lowering $C/(S-M)$ to less than 175 per cent. Hence, the proposed method would become attractive only in situations where the extraction ratio by conventional bord and pillar methods is less than 60 per cent. The proposed method would also become more attractive for higher-priced products, namely export coal.

Secondary Pillar Extraction

The cost of wall building per ton of mineable coal, i.e. per ton of coal in pillars, is

$$\frac{C}{(1-e_x)\alpha\rho}$$

Hence, the profit per ton mined will be

$$P_3 = S - \left(M + \frac{C}{(1-e_x)\alpha\rho}\right), \quad \ldots \quad (18)$$

and the total profit from extracting the pillars will be $(1-e_x)V_c\rho P_3$.

The profit from pillar extraction as a proportion of the profit from a conventional bord and pillar operation is given by

$$\frac{P_3}{P_2}\% = \left(1 - \frac{C}{(1-e_x)(S-M)\alpha\rho}\right).\,100 \quad \ldots \quad (19)$$

This relationship is shown graphically in Fig. 23(b) for the same values of α, ρ, and e_x as used in Fig. 23(a). Equation (19) is valid provided the cost of secondary mining using walls can be equated to that of mining using bord-and-pillar methods.

Fig. 23(b) shows that extraction using wall building becomes attractive only when the primary extraction ratio is less than 60 per cent and when $C/(S-M)$ is in the range 175 to 200 per cent. It becomes progressively more viable as the ratio $C/(S-M)$ or the primary extraction ratio becomes less.

Conclusion

The tests showed that a system can be designed rationally to support a predetermined overburden load, and that the design theory gives accurate predictions of strength. The support system of reinforced granular material and reinforced cemented material is flexible and can be varied to suit different heights and loads.

The system of preloading to the design load by jacking off the roof ensured that the walls were both stiff and proof-tested.

A study of the economics of the system shows that it can be used profitably provided the cost of wall building lies within definable limits. Economies of scale would probably bring about significant reductions in the expected cost, specifically in regard to the steel reinforcement and the bulk movement and processing of backfill material.

Further aspects of the system, not quantifiable economically, are that it allows mining to proceed without adverse effect on the surface environment and that it effectively increases the coal reserves of a region by freeing coal for exploitation that would otherwise be sterilized.

Finally, the utilization as backfill of very large quantities of waste material could reduce adverse environmental impacts on coal mining areas.

References

The technical contents of this paper are believed to be completely original and there are no references to related work.

Addendum: Examples of Roof Stresses

In these examples of roof stresses generated by differential compression between parallel reinforced walls and coal, the following typical values are taken.

$$\frac{W}{b} = 5\ \text{MPa} \qquad \delta = 2\ \text{m} \qquad H = 2\ \text{m}$$

$$E_c = 3\ \text{GPa} \qquad E = 5\ \text{GPa}.$$

Calculated values of R, $\triangle$, and σ_b corresponding to the above for a small range of values of s, δ, and E_w are given in Table I. The last line of the table represents the effect of an equivalent bord and pillar operation in which $b = s = 5$ m.

Table I shows that bending stresses in the roof are most sensitive to the stiffness of the reinforced walls and the flexibility of the roof beam. Wall stiffness can be optimized by the use of good quality granular material (see Fig. 13), while it appears that, under certain conditions, it may be possible to control roof flexibility by rock bolting to a carefully predetermined height above the roof. Both these factors are being considered in further research.

There are obviously major advantages to be derived from permanently stressing the reinforced walls against the roof, thus transforming the walls into active supports. Research is currently in progress to determine how this can be done effectively, permanently, and at low cost.

TABLE I

CALCULATED VALUES

s	δ	E_w	R	$\triangle$	σ_b
m	m	GPa	MN per m width	m · 10^{-3}	MPa
5	2	1	1,01	9,65	5,30
5	2,5	1	1,75	7,67	5,87
4	2	1	1,48	8,02	6,68
5	1,5	1	0,46	9,39	4,31
5	2	1,5	0,70	5,97	3,66
5	1,5	1,5	0,31	6,35	2,91
5	2	3	0,36	3,09	1,89
5	2	3	0,065	1,63	0,40

List of Symbols and Definition of Terms

Symbol (with or without subscript)	*Unit*	*Explanation*
σ	megapascal (MPa)	Direct stress
γ	kilonewton/(metre)^3(kN/m^3)	Unit weight
z	metre (m)	Overburden depth
b	metre (m)	Pillar breadth
s	metre (s)	Span between pillars
β	dimensionless fraction	
E	gigapascal (GPa)	Compression or extension modulus
I	(metre)^4 (m^4)	Second moment of area
K	dimensionless	Ratio of stresses
ϕ	angle (°)	Angle of shearing resistance
ψ	angle (°)	

Symbol (with or without subscript)	*Unit*	*Explanation*
A	$(\text{metre})^2$ (m^2)	Area
D	metre (m)	Diameter of reinforcing
x	metre (m)	Distance
l	metre (m)	Spacing of reinforcing
d	metre (m)	Diamet. r of reinforcing
h	metre (m)	Spacing of reinforcing
ϵ	dimensionless	Linear strain
ν	dimensionless	Poisson's ratio
p	dimensionless	Reinforcement ratio
α	dimensionless	Ratio of elastic moduli
W	kilonewton/metre (kN/m)	Load carried by wall per unit length
F	dimensionless	Factor of safety
M	rand/ton (R/T)	Cost of mining
S	rand/ton (R/T)	Selling price of coal
C	rand/(metre)³ (R/m³)	Cost of building reinforced wall
V_w	(metre)³ (m³)	Volume of reinforced wall
V_c	(metre)³ (m³)	Volume of coal *in situ*
ρ	ton/(metre)³ (T/m³)	Unit mass
a	dimensionless	$a = V_c/V_w$
w	metre (m)	Wall width
P_1	rand/ton (R/T)	Profit from total extraction using walls
e_x	dimensionless	Extraction ratio
P_2	rand/ton (R/T)	Profit from conventional bord and pillar mining
ΔP	rand/ton (R/T)	Additional profit
P_3	rand/ton (R/T)	Profit from extracting pillars using walls
H	metre (m)	Thickness of coal seam

Refining processes

The Gesellschaft Deutscher Metallhutten- und Bergleute and the Institution of Mining and Metallurgy, London, will hold a joint symposium Refining Processes in Metallurgy, 1983, in Hamburg, Federal Republic of Germany, from 20th to 22nd October, 1983.

Members of the organizing bodies and non members are equally welcome to attend the symposium and to present papers.

In the production of metals, refining is performed in various steps of the process. The papers should cover fundamentals and industrial applications of metallurgical refining processes for main, accessory, and special metals.

The Organizing Committee requests the submission of titles and abstracts (up to 300 words) of papers, and will select about 20 papers for presentation at the symposium and for publication as preprints, which will be distributed to registrants at the symposium. Abstracts in English or German should be sent before 1st December, 1982, to the Secretary, Gesellschaft Deutscher Metallhutten- und Bergleute, P.O.B. 210, D-3392 Clausthal-Zellerfeld, Federal Republic of Germany. Complete manuscripts of the papers selected must be submitted before 1st June, 1983.

In addition to the technical sessions, which will be conducted in English and German, plant visits will be arranged for registrants from outside the Federal Republic of Germany. Full details of these and of the symposium programme will be given in the Second Circular, which will include a registration form. Copies will be available in April 1983.

Another honour for Professor Krige

We congratulate Professor Daniel Gerhardus Krige, who has been awarded the gold medal of the Suid-Afrikaanse Akademie vir Wetenskap en Kuns. The award was made at the Rand Afrikaans University on the 24th of June, 1982.

He has already received several awards of this kind, as detailed in 'Spotlight on geostatistics – Professor D. G. Krige' by A. N. Brown, which was published in the November 1981 issue of this *Journal*. At present Professor Krige holds the Chair of Mineral Economics at the University of the Witwatersrand.

Discussion: Support in shallow mines using horizontally reinforced systems*

Contribution by M. D. G. SALAMON†

The first sight of a paper by members of another branch of engineering in the *Journal*, especially when the contribution is in my field of interest, fills me with anticipation. Also, the notion of using horizontally reinforced artificial pillars to support tabular excavations undoubtedly justifies attention. It is disappointing in view of these factors that the arguments and data presented in the paper do not vindicate the conclusions offered.

The authors assert, if I interpret the implications of their statements correctly, that the proposed support system is safe and ready for a full-scale underground trial. However, in my opinion, so much of the mechanism of the system has been left unexplored that no firm conclusion can be drawn at this stage. Furthermore, I do not believe that a field trial can be initiated safely without first obtaining and analysing the results of additional theoretical and laboratory studies.

To substantiate these remarks, I touch upon a number of aspects of the paper.

Rock and Support Deformation

Little attention is devoted in the paper to the deformation of the supports and of the surrounding rock mass. On p. 278 an elementary analysis of the 'equivalent roof beam' is introduced. It is apparent from the Addendum (p. 289) that the authors visualize the depth of the roof beam as approximately 1,5 to 2,0 m. This view of the beam and the relevant analysis are unacceptable oversimplifications. In practice, the support and the strata above and below the seam all participate in the deformation mechanism[1-3]. The simple beam theory is incapable of describing this complex process.

The cost analysis presented by the authors suggests that the use of artificial pillars in the case of complete extraction in one operation would be economic only if the extraction ratio achieved by the conventional bord and pillar methods, r_e, is less than 60 per cent (p. 288). According to the procedure currently employed in pillar design[4,5], this requirement is satisfied if the working height *exceeds* the height defined by the curve in Fig. 1. This curve was calculated to exemplify representative mining conditions. It would appear from this illustration that, at shallow depths, say under 60 m, artificial pillars are economic only if the working height is substantial.

Assume that a thick seam of coal exists at a depth of 100 m. According to Fig. 1, the use of artificial pillars at this depth would break even at a working height of about 3 m. To give the artificial pillars a chance of economic success, a working height of 4 m is postulated. If the bord width and safety factor are left unaltered, the procedure of pillar design for this geometry yields a width of coal pillar of 11,9 m, which is equivalent to an extraction of 55,7 per cent. It is estimated on the basis of some detailed theoretical work[1,3] and field observations[1,3] that the surface subsidence over these workings would be in the region of 7 to 8 mm.

Postulate that reinforced ash pillars (rib pillars or 'walls') of adequate strength can be designed with a cross-sectional area to ensure that about 25 per cent of the roof and floor is in contact with the new pillars. The use of ash is proposed because this is the material favoured by the authors for the field trial (p. 287). The ultimate load on these artificial pillars would be four times that of the virgin vertical stress, which is about 2,5 MPa at a depth of 100 m. Thus, the average pillar load would be about 10 MPa. As the strength should not be less than the load, it follows from the data in Fig. 13 (p. 283) that the reloading modulus of these ash pillars would be about 900 MPa. In practice, the modulus would be less than this value because of the compaction of the top section of the pillars and imperfections in the underground construction. A realistic estimate of the modulus appears to be about 600 MPa. In these circumstances, the ultimate convergence of the artificial pillars would be about 70 mm and, as the surrounding strata also deform, the surface subsidence, which is a useful measure of the deformation caused by mining, would approach 100 mm. This value is an order of magnitude greater than the subsidence that would occur in conventional pillar mining.

original text submitted to authors gave this figure as 700 mm

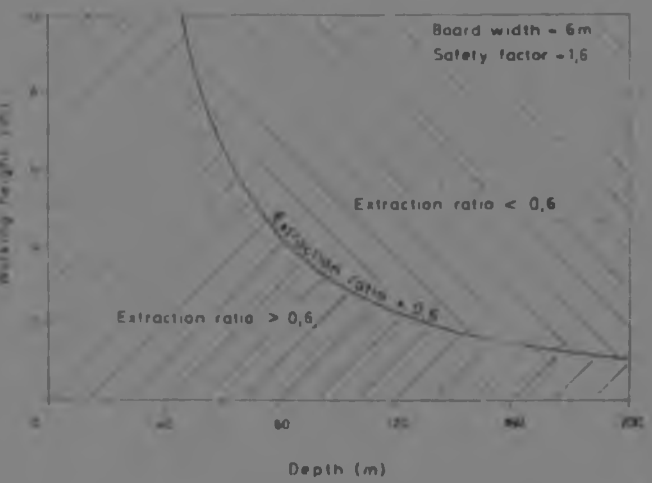

Fig. 1 Relationship between working height and depth for 60 per cent extraction

* Paper by J. A. Hahn, G. E. Blight, and L. Dison published in *J. S. Afr. Inst. Min. Metall.*, vol. 82, Oct. 1982, pp. 277–290.

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The effects of a large increase in displacement may be serious, regardless of whether complete extraction in one operation or secondary pillar extraction is practised. The much softer pillars would provide support to the roof only after considerable pillar convergence and rock deformation. Thus, maximum pillar support would develop only at a considerable distance away from the position where the artificial pillars are first called upon to act as supports. Would the roof strata fracture and fail as a result of a long, perhaps inadequately supported, span and large deflection? We do not know the answer to that question.

Strength of Reinforced Cemented Material

On pp. 280 and 281 an attempt is made to derive a strength formula for pillars consisting of horizontally reinforced cemented material. The resulting expression (10a) is fundamentally flawed.

The derivation of the formula is based on Poissons' effect, i.e. on the observation that most materials tend to expand laterally owing to vertical pressure and that the magnitude of this expansion for elastic materials is determined by the Poisson's ratio. Of course, the reinforcement will resist this expansion and in the process will provide some confinement to the cemented material. This thought process leads to (9) on p. 280, which is the expression for the lateral tensile strain in both the cemented material and in the reinforcement. At this point the authors suggest that 'as the vertical load on the pillar increases, the tensile strain will increase until it reaches a limiting value, at which the cemented material will fail by vertical splitting. For brittle materials such as concrete and brick, the limiting value of [the tensile strain] is about 200 10⁻⁶'. No attempt is made to substantiate these statements.

The substitution of the so-called limiting value of tensile strain into (9) leads to the erroneous formula in (10a), where the pillar strength, σ_{max}, is *inversely* proportional to the Poisson's ratio. If this formula were correct, a pillar built from a material that has a Poisson's ratio of zero would have infinite strength. This is, of course, impossible: a zero Poisson's ratio would imply the absence of lateral confinement, and hence no strength enhancement would occur. In such a situation, the strength of the pillar would be equal to the uniaxial compressive strength of the cemented material.

It is simple to derive a strength formula that avoids this inconsistency and that is still based on Poisson's effect and on the assumption of elastic behaviour by both the cemented material and the reinforcement. This model would have to satisfy the following criteria.

(i) The tensile force exerted on the reinforcement must be in equilibrium with the compressive forces of constraint to which the cemented material is subjected.

(ii) The horizontal strain in the reinforcement must be equal to that in the cemented material.

(iii) The strength of the pillar must satisfy the Mohr-Coulomb criterion for failure of brittle materials[6]:

$$\sigma_{max} = k\sigma_h + \sigma_c, \quad (1)$$

where σ_h and σ_c are the horizontal constraining stress and uniaxial compressive strength of the brittle material respectively, and k is a dimensionless parameter in the region of 6 for a variety of brittle rocks.

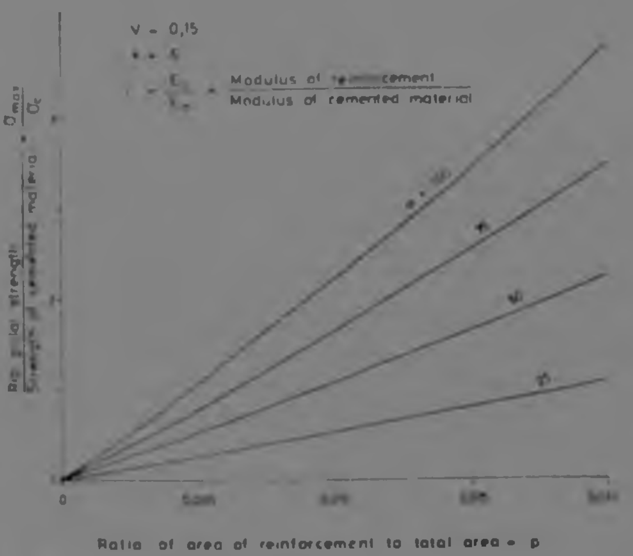

Fig. 2 Strength enhancement due to lateral reinforcement

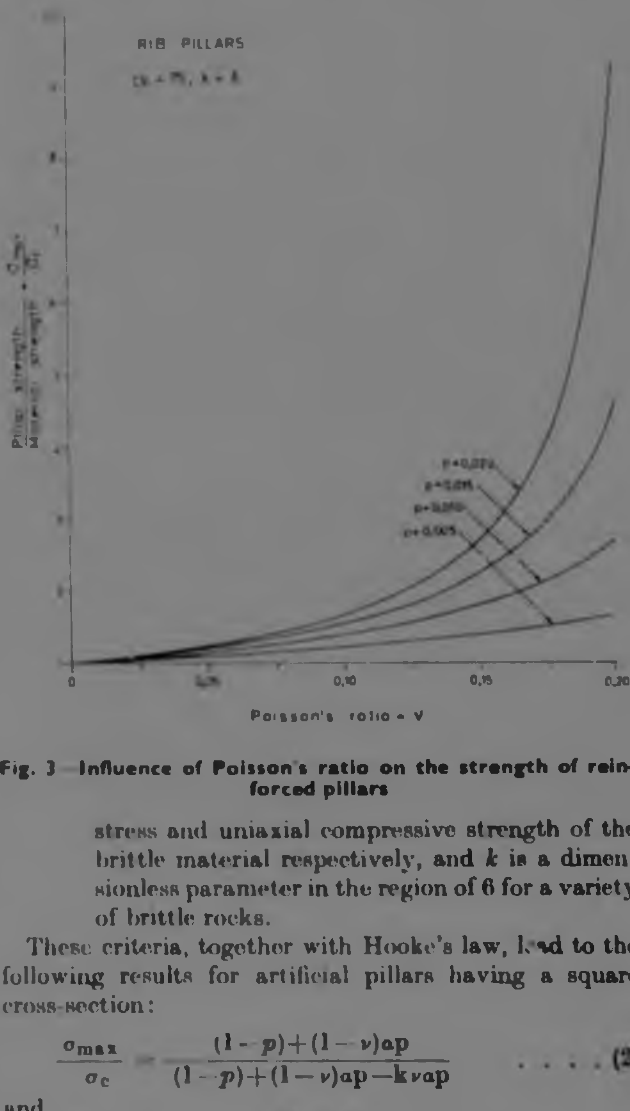

Fig. 3 Influence of Poisson's ratio on the strength of reinforced pillars

These criteria, together with Hooke's law, lead to the following results for artificial pillars having a square cross-section:

$$\frac{\sigma_{max}}{\sigma_c} = \frac{(1-p)+(1-\nu)ap}{(1-p)+(1-\nu)ap - k\nu ap} \quad \ldots \ldots (2)$$

and

$$\frac{\sigma_r}{\sigma_c} = \frac{\nu(1-\nu)a}{(1-p)+(1-\nu)ap - k\nu ap}, \quad \ldots (3)$$

where σ_r is the tensile stress in the reinforcement. The other notations are those used in the paper. The expressions in (2) and (3) are valid as long as

$$k < k_0 = \frac{(1-p)+(1-\nu)\alpha p}{\nu \alpha p} \quad \ldots\ldots\ldots (4)$$

The results corresponding to long rib pillars can be obtained from those in (2) to (4) by the multiplication of factors $(1-\nu)$ and ν wherever they occur by $(1+\nu)$. To derive these latter results, plane strain conditions were postulated. In Fig. 2 the strength of a rib pillar is plotted as a function of the reinforcement area for various values of the ratio of moduli. These results do not appear to differ greatly from those depicted in Fig. 8 of the paper (p. 281). However, it is important to note that the strength is strongly dependent on the value of the Poisson's ratio and that it *increases* as the Poisson's ratio is *increased* (Fig. 3).

There are two criteria that must be satisfied if the integrity of the artificial pillars is to be assured: it is necessary for the pillar strength in (2) and the yield strength of the reinforcement material to exceed the pillar load and the tensile stress in (3), respectively. The latter criterion is not mentioned in the paper in respect of pillars constructed from cemented material. This is a serious omission. It is clear from Fig. 4 that the tensile stress in the reinforcement may well become a critical condition for pillar soundness.

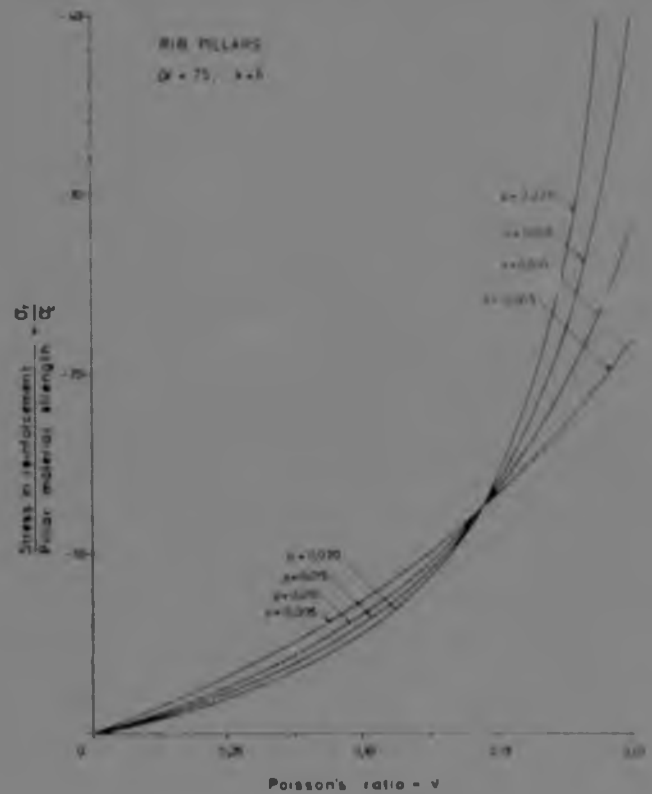

Fig. 4 Tensile stress in the reinforcement

Economic Feasibility

The authors concede on p. 288 that 'it is not possible to predict the total costs of wall building very accurately'. That is understandable at this stage. It is of greater concern to me that no attempt is made in the paper to describe how the construction of the artificial supports is to be integrated with the mining process. Is it possible to achieve the integration without the slowing down of mining operations? If not, which seems to me a strong possibility, the cost analysis as presented is invalid. Increased mining costs, in addition to the cost of the new supports, will have to be covered by the extra revenue earned.

General Comment

It is to be regretted that the authors ventured beyond the presentation of their ideas concerning the theoretical strength of horizontally reinforced artificial supports. In such a paper, the statement that the 'technical contents of this paper are believed to be completely original, and there are no references to related work' (p. 289) would perhaps be acceptable.

As it is, the paper goes far beyond this point. It claims that (i) artificial support systems can be designed rationally; (ii) the theory gives accurate predictions of support strength; (iii) the proposed support systems are flexible and can be varied to suit different working heights and loads; (iv) preloading by jacking ensures that the supports are stiff (relative to what?) and proof-tested; and (v) it can be used profitably provided the cost of construction lies within definable limits, etc.

I submit that these claims are not justified by the material presented, and that the paper would have benefited from a close study of the related literature.

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Contribution by H. WAGNER* and B. MADDEN*

The use of artificial pillars to extract mineral deposits as completely as is feasible is not new, and has taxed the minds of many generations of mining engineers. Three factors have prevented the application of this attractive concept on any significant scale: it was found to be extremely difficult, if not impossible, to design a system of artificial pillars of sufficient *strength* and *stiffness* that would (i) carry the weight of the overburden, (ii) ensure *stable* mining layouts, and (iii) be *economic*. It was, therefore, with considerable interest that we followed the development outlined in the paper on support in shallow mines using horizontally reinforced systems.

Close scrutiny of the concepts proposed in the paper and the data presented casts considerable doubt on some of the conclusions reached by the authors. In particular, the conclusions that the system can be designed rationally to support a predetermined overburden load, and that the system can be used profitably provided the cost of wall building lies within definable limits, are misleading since they are based on simplistic assumptions. Because of the far-reaching consequences of these statements, we consider it necessary to elaborate on some of the aspects of horizontally reinforced systems.

Load Transfer

In deriving the conclusion that the system can be designed rationally to support a predetermined overburden load, the authors consider the simplified but unrealistic case that the behaviour of the strata surrounding bord-and-pillar workings can be resembled by the modelling of the behaviour of a thin roof beam. In the examples given in the Addendum of p. 289, the thickness of the roof beam varied between 1,5 and 2,5 m. A detailed analysis of strata movements in bord and pillar workings shows that the support, the strata above the seam, and the strata below the seam participate in the deformation mechanism[1]. Furthermore, because of the compressibility of the supports, the critical dimension from the point of view of overall strata control is the minimum lateral dimension of the mining panel, and not the width as determined by the distance between the support pillars, s, and the pillar width, b, as suggested by the authors in equation (2) on p. 278. The main shortcoming of the simplified approach is that it ignores the load transfer from the mined-out area to the panel abutments. It will be shown that it is this load transfer that can lead to dangerous conditions in the working areas and regional instabilities.

The load transfer from the mined-out area to the panel abutments and the working face is a function of the amount of elastic strata deflection that is allowed to take place in the mined-out area. The strata deflection is a function of the panel dimensions and geometry, the thickness and elastic properties of the overlying strata, and the load-deformation characteristics of the support in the mined-out area. In the case of total extraction panels, that is panels with no internal support, the load acting on the panel abutments is limited by the strength properties of the roof strata, which, in turn, govern the amount of strata deflection that can take place before failure of the roof strata occurs. Once the roof strata have caved, only the weight of the overhanging strata has to be supported by the panel abutments and working face. Early caving of the roof strata is therefore a desirable feature of total extraction systems. In the case of mining systems employing internal support in the mined-out area, either in the form of natural pillars or artificial support, the magnitude of the abutment stresses is controlled by the compressibility of the pillars or supports. As a rule, the stiffer the support elements, the lower will be the abutment stresses.

To illustrate the above concepts, two situations envisaged by the authors will be considered in detail. First, the effects of artificial supports of different stiffness on the stress distribution around a 200 m-wide extraction panel situated at a depth of 100 m are examined. The second example concerns the stress distribution in a conventional bord-and-pillar panel in which coal pillars are being extracted under the protection of artificial support pillars.

Artificial Supports of Different Stiffness

Fig. 1 shows the distribution of vertical stresses acting on the abutments and internal support pillars. For convenience, the stress values have been normalized with respect to the value of the vertical component of primitive stress. In the example, continuous artificial rib pillars 1,2 m wide and 1,8 m high, which are separated by 6,3 m wide bords, have been modelled. The compressive modulus of the pillar material in Fig. 1 is 1 GPa.

In Fig. 2 the maximum stress concentration on the artificial support pillars and the panel abutments is plotted as a function of the modulus of compression of the pillar material. The upper value of 3 GPa corresponds to the *in situ* modulus of elasticity of coal. Two important observations can be made from the trends shown in Fig. 2. First, the artificial support pillars in the centre of the panel carry a significant proportion of the overburden load only if the modulus of compression of the pillar material exceeds a value of 1 GPa. In terms of the tributary-area concept, the vertical stress concentration, C, in the artificial support ribs of width, b, which are separated by bords of width s, is

$$C = 1 + \frac{s}{b}.$$

With the appropriate values for $s = 6,3$ m and $b = 1,2$ m, a stress concentration of 6,25 results. Second, in the case of support pillars having a modulus of compression of less than 1 GPa, most of the weight of the overburden in the mined-out area has to be carried by the panel abutments and the working face. The rapid increase in

* Chamber of Mines Research Organization, Carlow Road, Melville, Johannesburg 2092.
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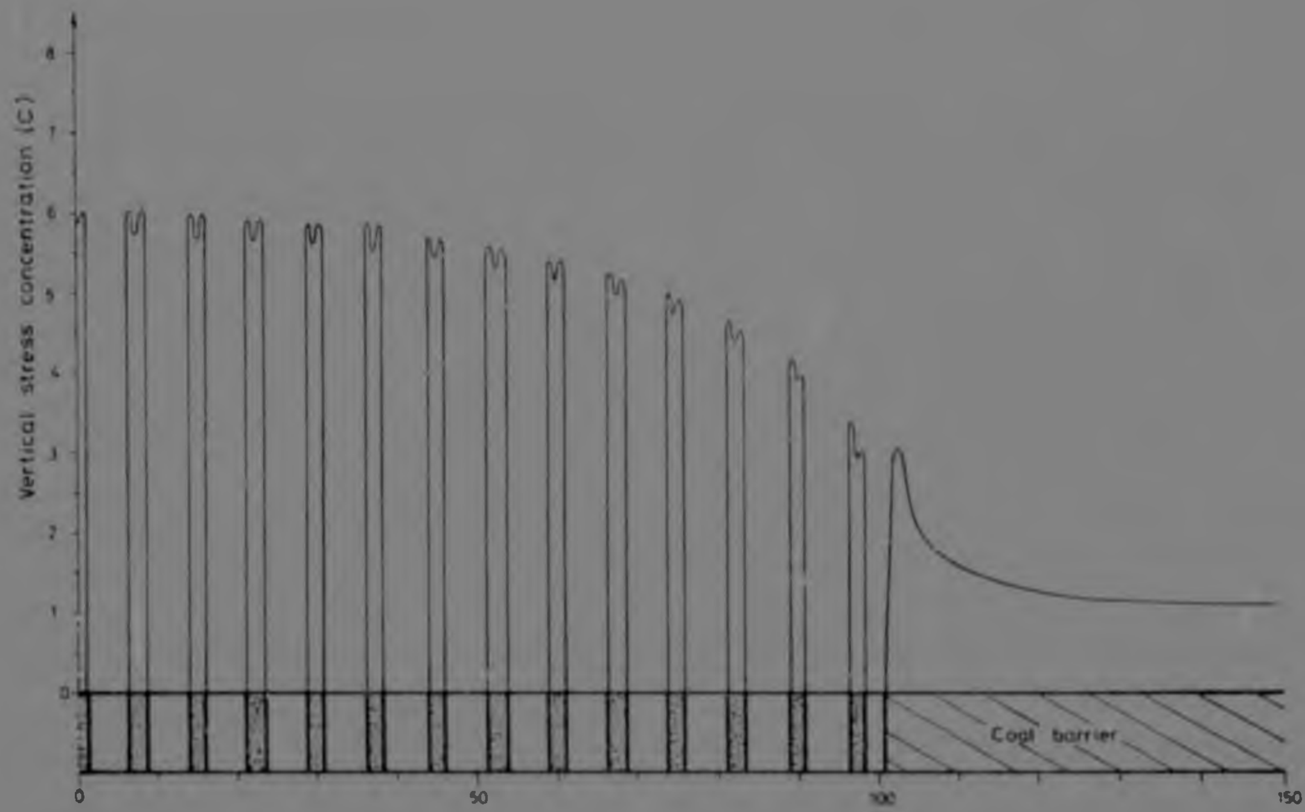

Fig. 1—Stress distribution around a 200 m-wide total extraction panel situated at a depth of 100 m. The panel is supported by artificial pillars that are constructed from a material with a modulus of compression of 1 GPa. Because the panel is symmetrical, only the right-hand side is shown.

the stress concentration at the panel abutments and working face for low values of pillar modulus of compression is evident from Fig. 2. High stress concentrations at the working faces can give rise to serious strata control problems, and often require special support measures. In this connection, mention must be made of the difficult ground conditions encountered in total extraction panels when mining occurs under a thick sandstone roof or massive dolerite sills. An important feature of mining systems employing relatively soft support ribs is that the condition of high stress concentrations at the working face is maintained throughout most of the life of the panel, since it is the function of these ribs to prevent failure of the roof strata in the mined-out area.

It should be noted that the authors propose to compress the lower portion of the support rib to the design load so that advantage can be taken of the very much higher reloading modulus. To do this effectively, a space of at least 300 mm is required for the insertion of the jacking system. This space will have to be filled after the lower half of the pillar has been compressed. The authors envisage the use of precast slabs or wedges, and final grouting if wedges are not used. Because of the unavoidable roof irregularities and the variations in the width of the jacking space, it appears unlikely that efficient use can be made of precast elements and that extensive use will have to be made of either guniting or grouting. Because of the time required for the grout material to develop its strength properties, it will not be

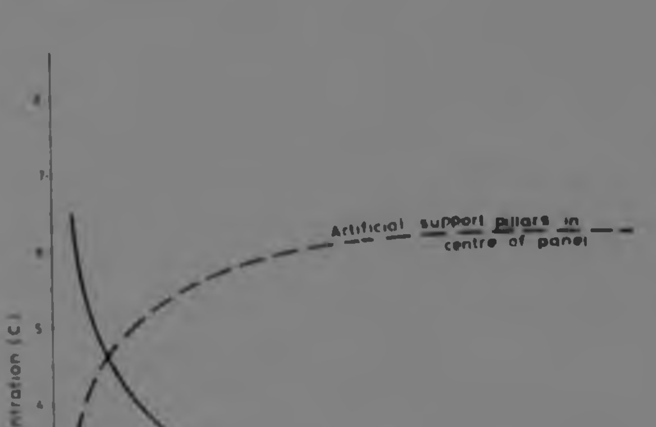

Fig. 2 Effect of the reloading modulus of the pillar material on the stress concentration at the internal support pillars and the panel abutments.

able to resist any initial strata movement effectively, nor will it be able to prevent any strata separation that may have taken place in the roof close to the working face

Fig 3 gives an estimate of the effects of unsupported gaps on the reloading modulus of the artificial support pilars. The trends shown in the diagram are for a pillar material that has a reloading modulus of 1 GPa and are valid for stress levels of 10 MPa The effective reloading modulus of the pillar will be lower than shown in the diagram for stress levels below 10 MPa but higher for pillar stress levels above 10 MPa The influence of even very small gaps on the effective reloading modulus of pillars is very marked, and highlights the sensitivity of the artificial pillar system to operator abuse. If normal mining conditions and the operational environment are taken into account, it is difficult to visualize that values of pillar reloading modulus in excess of 0.5 GPa can be achieved consistently and reliably It follows therefore that, under most conditions, the artificial pillars will carry only a relatively small portion of the weight of the overburden, while the panel abutments and working faces will be highly stressed. The need for prestressed support pillars has been recognized by the authors, but its importance on the working conditions in total extraction panels has not been fully appreciated

So far only the effect of the pillar material on the stress conditions around total extraction panels has been discussed. However, this is only one of the design parameters Other parameters are the extracted seam height and the cross sectional area of the pillars. These two factors, together with the pillar material, determine the stiffness, ', of the support system The latter is directly proportional to the cross sectional area of the pillar and the compression modulus of the pillar material, but inversely proportional to the pillar height

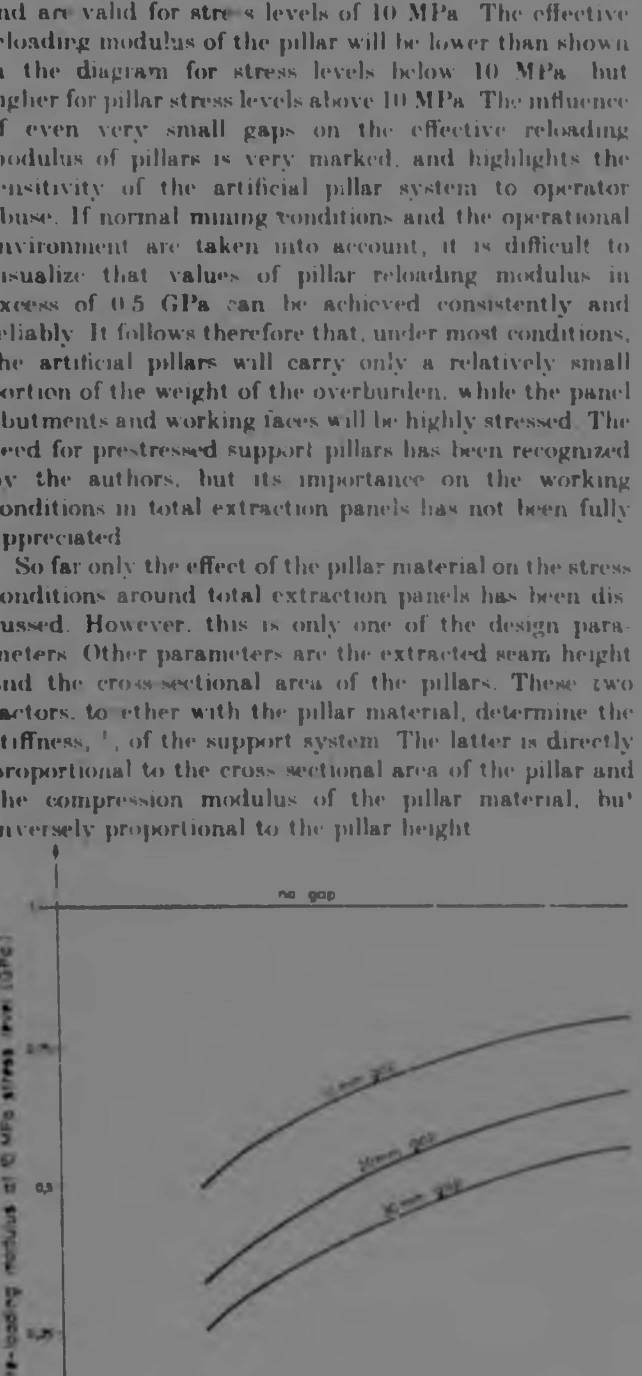

Fig. 3—Effects of unsupported gaps between tops of support pillars and immediate roof contact or strata separation on the effective reloading modulus of the pillar material.

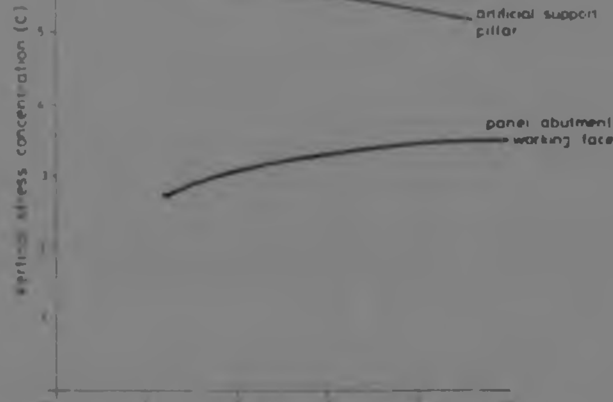

Fig. 4—Effect of pillar height, *h*, on the stress concentration at the internal support pillars and the panel abutment. The depth of mining is 100 m, the panel width 200 m, and the reloading modulus of the material 1 GPa.

The influence of the latter on the stress concentration around a total extraction panel is examined in Fig. 4 The reduced efficiency of the artificial support pillars at higher seam thicknesses is evident. As was pointed out above, the stiffness of the support system and consequently its effectiveness, can be improved by an increase in the pillar area As attractive as this approach is from the point of view of strata control it results in unfavourable economics since the cost of the support system and the time required to install it increase more or less proportionally with pillar size

Stress Distribution in Bord-and-pillar Workings

In their paper the authors mention a second application of the artificial pillar support system, namely that in connection with the extraction of pillars from existing bord and pillar workings In this instance it is envisaged that the coal pillars are extracted under the protection of the artificial support pillars. The authors give no details as to how this could be done in practice

A possible approach is to first install the artificial support pillars in the bords that are parallel to the line of pillar extraction, and then to split the coal pillars and install artificial pillars in the splits before the remainder of the pillar is extracted A close examination of this, as well as of several other possibilities of extracting the coal pillars, reveals a number of logistic problems as far as the basic mining operations are concerned By far the most serious of these are the transportation of the coal from the working place, and of the support material to the working place Another problem area is the interaction between the extraction of the coal on the one hand and the installation of the supports on the other. For reasons of ground control, safety, and section productivity, support installation has to be concurrent with the extraction of coal. Since the building of support pillars is time consuming and interferes with the mining of coal, the two activities have to be separated

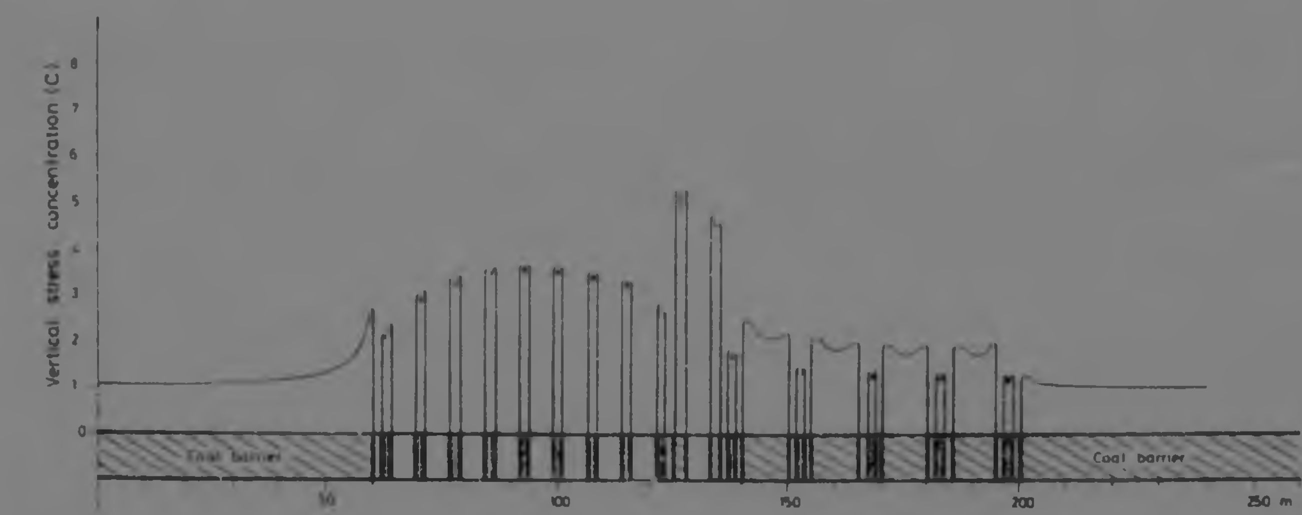

Fig. 5—Vertical stress concentration around an existing bord-and-pillar panel in which pillars are extracted according to the concept of artificial support pillars. The depth of mining is 100 m, the seam thickness 1,8 m, and the modulus of compression of the pillar material 1 GPa.

geographically. To reconcile these conflicting requirements, the mining operations have to be split into support operations in one half of the pillar extraction line, and pillar removal in the other half. Although this is organizationally feasible, it means that the number of available working places is greatly reduced. This has serious implications as far as the productivity and section output are concerned when conventional mechanized equipment is used, but is likely to be of less importance in the case of continuous miner operations. From this discussion it follows that the economics of the artificial pillar support depend not only on the cost of the support but, even more so, on the effects of the concurrent support and mining activities on productivity, equipment utilization, and section output.

More serious than the economic aspects of the use of artificial support pillars in existing bord-and-pillar workings are some of the strata control aspects. As an illustration of these, the extraction of coal pillars from existing workings situated at a depth of 100 m was examined. Fig. 5 shows the stress concentrations on the solid abutments, the intact coal pillars, the pillars under extraction, and the artificial support pillars. Despite the relatively small mining span of 60 m, the vertical stress concentration on the pillars that are being extracted is extremely high. Considering that the original pillars are designed with a safety factor that ranges typically from 1,6 to 2,5, it is immediately apparent that the pillars in the extraction line and its vicinity cannot withstand the very high stress concentrations without severe fracturing. The possibility of sudden failure of coal ribs in the extraction line cannot be denied. The implications of this become apparent if it is considered that most mining operations, including the installation of artificial support pillars, take place in this area. In conventional pillar-extraction operations, this danger is overcome largely by encouraging the roof strata to cave as soon after pillar removal as possible. To achieve this, a minimum amount of support is left in the mined-out area, and one of the golden rules of successful pillar

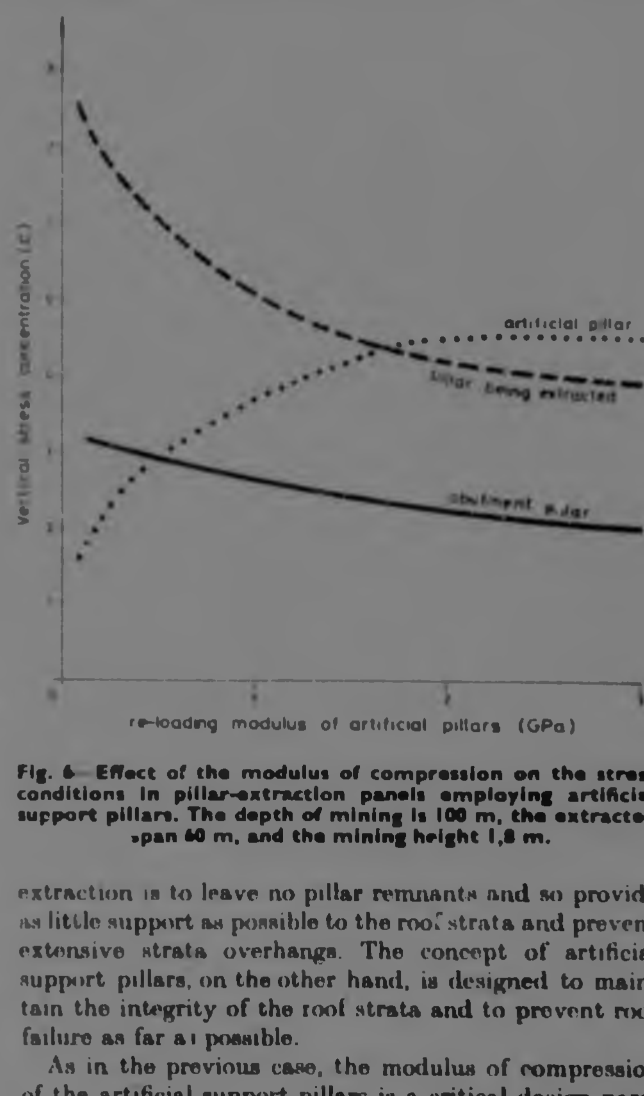

Fig. 6—Effect of the modulus of compression on the stress conditions in pillar-extraction panels employing artificial support pillars. The depth of mining is 100 m, the extracted span 60 m, and the mining height 1,8 m.

extraction is to leave no pillar remnants and so provide as little support as possible to the roof strata and prevent extensive strata overhangs. The concept of artificial support pillars, on the other hand, is designed to maintain the integrity of the roof strata and to prevent roof failure as far as possible.

As in the previous case, the modulus of compression of the artificial support pillars is a critical design parameter. Fig. 6 shows how the stress conditions in the extraction panel depend on the modulus of compression of the artificial pillar material. Unlike the previous case, a significant deterioration of the situation is observed at moduli of compression of as high as 2 GPa

Conclusion

We hope that our contribution has helped to clarify some of the inherent problems of artificial support pillar systems.

Reference

1. SALAMON, M. D. G., and ORAVECZ, K. I. Displacements and strains induced by bord and pillar mining in South African collieries. *Proc. 1st Congr. Int. Soc. Rock Mech., Lisbon, 1966*, vol. 2, pp. 227-231.

Authors' reply to Dr Salamon

We thank Dr Salamon for the time and trouble he has put into his discussion. We shall answer his points *seriatim*.

System of Horizontally Reinforced Support

Firstly, we accept that, when one attempts to apply expertise and knowledge developed in one field to another, the attempt will draw criticism and will, perhaps because of differences in terminology and experience, not even be fully understood.

We certainly do not regard the system of horizontally reinforced support to be completely developed. Indeed, we believe that development is only starting. However, we also believe that no engineering development can be finalized on the basis of theory and laboratory experiment alone. It is essential to proceed with underground trials on a limited basis in order to probe some of the queries concerning cost, the possibility of integrating wall building with mining operations, etc., that have rightly been raised by Dr Salamon (as well as by Drs Wagner and Madden).

We regard the system as described in the paper to represent a presentation of fundamental principles and ideas relating to reinforced supports. Any field trial would have to be designed specifically for the chosen site, and the entire system including supports and supported rock mass would have to be considered. We do not regard ourselves as capable of performing the necessary rock mechanics study, and the design of a field trial would necessarily be undertaken on a multi-disciplinary basis. The beam analysis contained in the paper demonstrates our awareness and concern for the interaction between the support system and the supported rock mass. It gives results that we recognize as being not entirely favourable to the proposed system. We also state quite plainly in the paper that the analysis is simplified and not claimed to represent the deformation process accurately.

Compression of Artificial Pillar

We do not agree with Dr Salamon's analysis of the compression of a 4 m high artificial pillar under a stress of 10 MPa. If the pillar has a modulus of compression of 600 MPa, the strain in the pillar will be 1/60 and the pillar convergence 4000/60 = 67 mm. Hence, we presume from Dr Salamon's figures that the surface subsidence would be about 10 mm, and not 100 mm as he states.

Dr Salamon has completely missed the point of our analysis of the strength of a reinforced cemented material. Equation (9) is perfectly general but, as soon as we insert a numerical value for the failure strain of the cemented material, it becomes material-specific. $200 \cdot 10^{-6}$ is so well accepted a value for the failure strain in tension of concrete, brickwork, and similar materials that we did not think it necessary to substantiate the value. For substantiation, Dr Salamon is referred to any textbook on concrete technology. The analysis leads to equation (10a), which, true enough, has Poisson's ratio in the denominator. However, as Poisson's ratio for the type of material under consideration lies in the range 0,1 to 0,15, we are saved the embarrassment of a predicted infinite strength.

We also point out that the tensile failure strain of a material is related to its Poisson's ratio. Thus, rubber has a Poisson's ratio of 0,5 and a tensile failure strain of several hundred per cent. Mild steel has a Poisson's ratio of 0,35 and a failure strain of 20 per cent. The corresponding values for glass are 0,2 and 0,05 per cent (depending on the state of the glass surface). We know of no real materials having a Poisson's ratio of zero, but predict that such a material would have a tensile failure strain of zero. Hence, equation (10a) would predict an indeterminate strength for this hypothetical reinforced material. This is just as sensible as the conclusion from Dr Salamon's equation (2) that the reinforcing would not affect the strength of the material.

Another point missed by Dr Salamon is that the reinforced material is deemed by equation (10a) to have failed when the strain in the reinforcing is $200 \cdot 10^{-6}$. As the yield strain of mild steel is $1000 \cdot 10^{-6}$ (again no substantiation is deemed necessary) and its failure strain 20 per cent, there is absolutely no chance that the tensile stress in the reinforcing will become a critical condition.

Fig. 19 in our paper shows that a reinforced cemented material reverts, after failure, to a granular reinforced material, and its resistance to compression is greatly reduced. This, together with the fact that the reinforcing is very inefficiently utilized, has led us to abandon the idea of reinforcing cemented material.

Reiteration of Conclusions

Finally, we reiterate our conclusions, which we have not been persuaded to change.

(i) Figs. 9 and 11 of our paper show that artificial support systems can be designed rationally.

(ii) The theory predicts support strength remarkably accurately.

(iii) Supports can be designed to suit various loads. Provided the support is not too slender (i.e. has a height to width ratio of less than 3 to 4), the strength is independent of the working height.

(iv) Preloading by jacking ensures that the modulus of compression is of the same order of magnitude as that of a coal pillar, although the compression modulus of coal cannot be matched.

Our final conclusion, concerning the economics of horizontally reinforced systems, will be tested as development proceeds.

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Authors' reply to Drs Wagner and Madden

We find Drs Wagner and Madden's discussion very interesting and useful. It answers many of the questions that we, as civil engineers not versed in the technology of mining and rock mechanics, have been unable to answer.

Rational Design of Supports

We have already answered the criticism of our statement that, for a predetermined overburden load, suitable supports can be desigaed rationally. Drs Wagner and Madden's contribution goes a long way to demonstrating how the loads can be predetermined. Our statement was not, however, based on any assumptions, simplistic or otherwise, but on the solid evidence of the agreement between test results and predictive theory.

It is interesting to follow through the example illustrated by their Figs 1 and 2 by looking at the actual stresses represented by the stress concentration factor C. Taking the unit weight of the overburden strata as 25 kN/m^3, the overburden stress is 2,5 MPa and the stress on the abutment 7,5 MPa. According to reference 5 of Dr Salamon's discussion, the abutment, if treated as a wide pillar, would fail at a stress considerably in excess of this. Hence, the factor of safety against failure of the abutment appears to be well above 2. The design strength for the artificial pillars would be $1,6 \times 2,5 \times 6,25 = 25$ MPa. Reference to our Fig. 13 shows that, even if ash were used as backfill, a reloading modulus of 2 GPa would be possible and, with sand backfill, easily attainable. This would reduce the abutment stress to $2,3 \times 2,5 = 5,75$ MPa, which gives a factor of safety of more than 3 against failure according to reference 5. This encourages us to think that, with some further design and tailoring of the properties of the rib pillars, the hypothetical layout illustrated in Fig. 1 of the discussion would be feasible.

The discussors rightly point out the difficulty of providing artificial supports that are installed hard against the roof and are therefore as stiff as they are designed to be. We fully appreciate this difficulty and are giving it a lot of attention in our current research. We appreciate their treatment of the effect of pillar height on abutment stresses, and are relieved to find the effect so small. In our numerical example above, it appeats possible to go to a pillar height of 5 m without causing the abutment stress to rise above 6,5 MPa if the pillar reloading modulus is 2 GPa. This, once again, indicates that a feasible situation should be possible with careful design of artificial pillars. We would point out, in passing, that the stiffness of a horizontally reinforced pillar depends only on the height and compression modulus of the pillar, and not on its cross sectional area.

Figs 5 and 6

The situation represented by Figs 5 and 6 certainly appears alarming. However, let us follow it through in terms of the dimensions used elsewhere in the discussion. The width of the unsplit coal ribs is not given, but let us assume that it is 6,3 m with 6,3 m bords. These sizes would fit in with the vertical stress concentration C of 2 shown in Fig. 5 for the unsplit coal ribs, and are the same as those used in Fig. 1. It would not be possible to split a pillar of this width and leave remnants of a sensible size in place. Therefore, we would suggest the building of two artificial ribs in each bord, each artificial rib being hard against the adjacent coal rib. For a vertical stress concentration of 4,5, these ribs could each be 1,5 m wide, leaving a 3,3 m bord between them. The coal rib could then be removed completely, leaving no remnants and resulting in a new bord between the artificial ribs of 6,3 m. Hence, again, we consider that a feasible system is possible by careful design.

Conclusion

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PAPER PRESENTED TO "MINING WITH BACKFILL" SYMPOSIU[illegible]
AT LULEA, SWEDEN, JUNE, 1983

APPENDIX 21

Supports of reinforced granular fill

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ABSTRACT: A system of support is described which utilizes waste material reinforced with either horizontal steel mesh or spirally-wound high tensile steel wire. Supports can be accurately designed to carry a pre-determined load with a given factor of safety. Design theory for both types of support is developed and the load-compression characteristics are described.

INTRODUCTION

Most of the coal mines in South Africa are relatively shallow (30m to 100m) below surface) and may contain a sequence of superimposed coal seams.

The top or shallowest seam of a sequence can be almost completely extracted by long wall mining or stooping (following bord and pillar mining) if the severe surface subsidence that ensues can be tolerated. Deeper seams, however, cannot be totally extracted unless they are extracted in sequence from the shallowest to the deepest. The coal contained by the different seams is usually of different qualities and types, and commercial demand for coal of specific qualities dictates the sequence in which the seams are extracted. As a result, most multi-seam mines are mined using the bord and pillar method. This practice results in percentages of extraction varying from 75 per cent in very shallow seams to 40 per cent or less in deeper seams.

Although coal provides a cheap form of roof support, coal pillars represent a natural energy resource that has been sterilized and much research is currently in progress to find ways and means of winning this sterilized coal.

This paper describes one of the techniques that is currently under development. The method consists of replacing coal pillars by artificial pillars or walls of horizontally reinforced granular material. In principle, any granular material can be used for this purpose. However, from environmental and mining or other waste such as power station bottom ash. Ash is a particularly appropriate choice if the colliery is "captive" to a power station, i.e. if its main purpose is to supply a nearby power station.

STRENGTHENING A GRANULAR MATERIAL BY HORIZONTAL REINFORCING

The effect of horizontal reinforcing on a granular material is to develop a horizontal confining stress when the material is subjected to vertical loading. The horizontally reinforced granular material will tend to go into a state of failure as soon as a vertical stress is applied to it. Actual failure will, however, not occur because of the development of tension in the horizontal reinforcing. As Figure 1 indicates, on a p-q diagram, the stress path oa of the granular

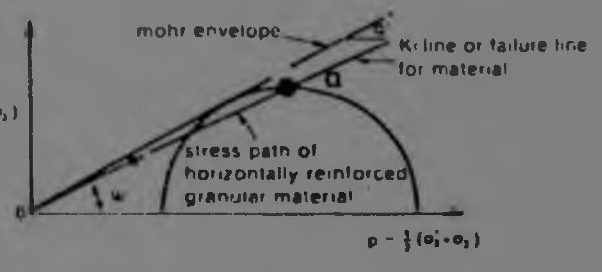


Figure 1. Stress path for reinforced granular material

material will follow the failure line for the material. On the stress path: a represents the point at which the horizontal

The stress-compression curve for the first loading indicates a low compression modulus. However, the reloading curve is considerably steeper, hence the reloading modulus is much higher. Figure 3 illustrates how it is intended to use the walls of reinforced granular material in practice. After building the walls in-situ, they will be precompressed and test-loaded to the design load by jacking off the roof. The space created between the top of the wall and the roof by the precompression will then be filled by horizontally reinforced cemented slabs and wedges which will be bedded against the roof by grout. As the load is subsequently transferred to the wall when the roof tends to sag, the wall will be recompressed along the recompression curve, back to the design load. The support system thus created will be both relatively stiff and pre-tested to the design load. The stiffness is particularly important in limiting bending deflexions and therefore bending stresses in the roof.

The recompression modulus of horizontally reinforced walls is related to their strength as shown by Figure 4. The scatter of results illustrated in this figure is caused mainly by variation in the characteristics of the granular materials being used. The upper band of results corresponds to walls built of highly

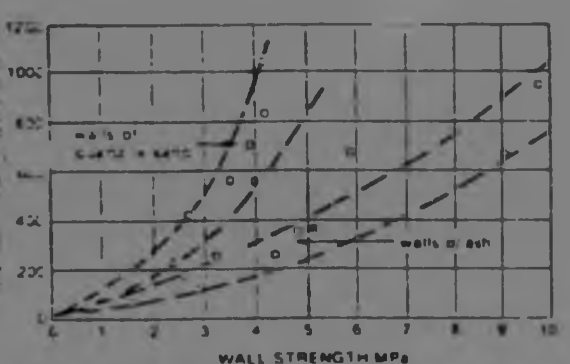


Figure 4.

frictional weathered quartzite sand. The lower band of results corresponds to walls built of power station bottom ash which although highly frictional, is considerably more compressible than the sand.

Alternatively, in situations where a yielding support is required, the stiffness and yield stress of the support can be designed to give the required qualities.

BOND OF REINFORCING TO REINFORCED MATERIAL

The tension developed in the reinforcing of a horizontally reinforced granular material is transmitted by the bond between the reinforcing and the reinforced material. The mechanism of bond development is illustrated in Figure 5a.

The average bond stress transmitted to a reinforcing wire over a length x by friction is

$$\frac{\sigma}{2}.\left(1+\frac{1}{K_p}\right)\tan\phi\,\pi D.\frac{x}{2}$$

in which the diameter of the wire is D.

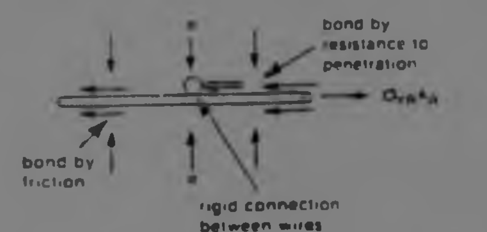


Figure 5a Mechanism of bond development in reinforced granular material

The most convenient form of horizontal reinforcing for a wall consists of a system of orthogonal wires rigidly bonded to each other at points where they cross. Readily available systems consist of either square or rectangular welded mesh or twisted diamond mesh. The wires that run at an angle to the direction of tension under consideration contribute considerably to the bond because they must penetrate the reinforced material before the reinforcing can slip relative to the material. If the bond wires in an orthogonal system are spaced ℓ apart, the load transferred through these wires in length x will be a minimum of:

$$\frac{x}{\ell}\left(\frac{\sigma}{K_p}.d + 2d\sigma\tan\phi\right)h$$

in which d is the diameter of the bond wires that are spaced h apart. Full bond resistance of the wire is therefore developed in a length x given by

$$\frac{\pi D^2}{4}\sigma_{yR} = \frac{\sigma}{4}\left(1+\frac{1}{K_p}\right)\tan\phi\,\pi Dx + \frac{\sigma}{4}\frac{d}{\ell}\left(\frac{4}{K_p}+8\tan\phi\right)h.x$$

$$\text{i.e. } x = \frac{\pi D^2\,\sigma_{yR}/\sigma}{\pi D\left(1+\frac{1}{K_p}\right)\tan\phi + \frac{4dh}{\ell}\left(\frac{1}{K_p}+2\tan\phi\right)} \quad ....\ (4)$$

The value of x is astonishingly small. For example

if σ = 5MPa $\quad\sigma_{yR}$ = 600MPa

D = d = 2,5 mm $\quad K_p$ = 3,5

At this stage the vertical stress σ will be related to the horizontal stress σ_h by the relationship

$$\sigma = K_p \sigma_h \ldots\ldots\ldots\ldots \quad (1)$$

where K_p is the passive pressure coefficient of the granular material, i.e. the ratio σ_1/σ_3 at failure in a triaxial compression test on the material.

Equating the tension in the horizontal reinforcing to the compression in the granular material:

$$\sigma_{yR} A_R = \sigma_h (A_m - A_R)$$

where A_R is the cross-sectional area of a reinforcing wire and A_m is the area of reinforced material corresponding to A_R

$$\text{or } \sigma_h = \frac{\sigma_{yR} A_R}{(A_m - A_R)} \ldots\ldots\ldots\ldots \quad (2a)$$

Hence

$$\sigma = \sigma_{max} = \frac{K_p \sigma_{yR} A_R}{(A_m - A_R)} \ldots\ldots\ldots\ldots \quad (2b)$$

If A_R is small as compared with A_m (A_R is usually a few tenths of a percent of A_m)

$$\sigma = K_p . \sigma_{yR} . \frac{A_R}{A_m} \ldots\ldots\ldots\ldots \quad (2c)$$

The strength of a horizontally reinforced granular material is thus directly proportional to the reinforcing ratio A_R/A_m and the yield stress of the reinforcing σ_{yR}.

DESIGN OF WALLS OR PILLARS OF HORIZONTALLY REINFORCED GRANULAR MATERIAL

Artificial pillars and walls can be constructed of granular materials reinforced with layers of steel mesh placed at a vertical spacing v. If h is the horizontal spacing of the wires in the mesh and A_R is the cross-sectional area of each wire, it follows that

$$\frac{A_R}{A_m} = \frac{A_R}{vh}$$

and equation (2c) becomes

$$\sigma_{max} = K_p \sigma_{yR} . \frac{A_R}{} = F\sigma \ldots\ldots\ldots\ldots \quad (3)$$

where F is the factor of safety on the design stress σ. For a selected steel quality and mesh dimensions, v can be calculated or alternatively, for a selected v, a suitable mesh may be selected.

The properties of horizontally reinforced granular materials have been extensively investigated by means of both model scale and full scale tests. Figure 2 shows a comparison of σ calculated from equation (3) with corresponding measured values.

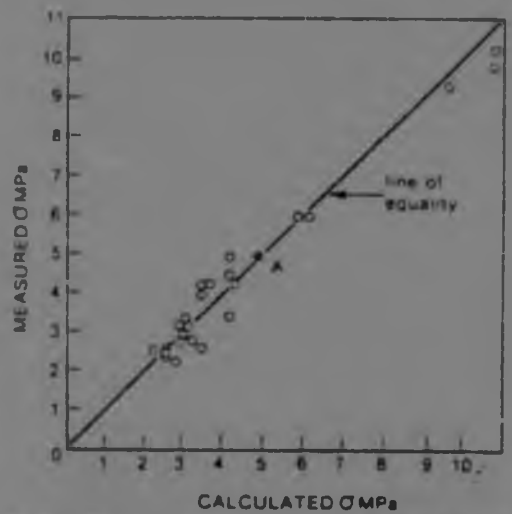


Figure 2 Relationship between calculated and measured strengths of walls built of horizontally reinforced granular material

It will be seen from Figure 2 that there is an excellent correlation between calculated and measured values over a wide range of stresses.

Figure 3 (test A on Figure 2) represents the results of a typical compression test on a reinforced wall. This diagram illustrates an important feature of walls or pillars of horizontally reinforced granular material. The wall was designed to have a factor of safety of 1,6 on a design vertical stress of 2,39 MPa. The

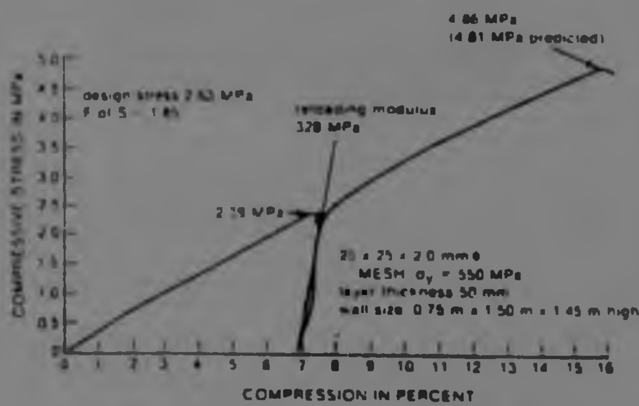
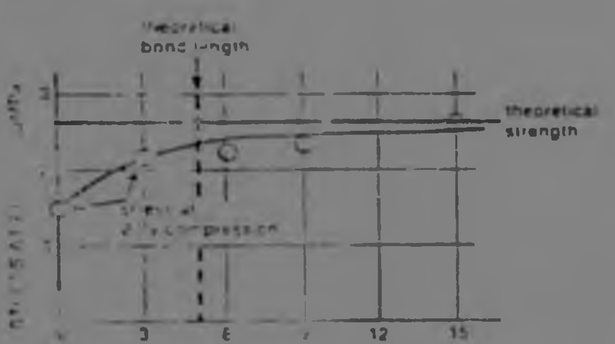


Figure 3 Stress compression curve for wall of horizontally reinforced gran

wall was loaded to the design load and was [illegible] = 96 mm.

[illegible]re 5b illustrates the results of a [illegible] of tests on reinforced walls that [illegible] to [illegible] the validity [illegible] (4). The [illegible] shows the [illegible] at failure in a series of tests on [illegible]lls in which the number of bond wires [illegible] side of the wall was increased [illegible]ressively from zero. The spacing of [illegible] bond wires was 12,5 mm and the [illegible]tical bond length was 60 mm, hence [illegible] bond could theoretically be achieved [illegible] 5 bond wires. The results in Figure [illegible] confirm this.



[illegible] requirements for reinforced granular material

[illegible] OF [illegible]FORCED GRANULAR

[illegible] theory [illegible]ates, reinforced [illegible]lls fail when the tensile [illegible] the horizontal reinforcing is [illegible] this [illegible]rs, the reinforc- [illegible] fracture and the granular [illegible] along diagonal planes [illegible] at (45°-ϕ/2) to the [illegible] of the major principal or [illegible]cal stre[illegible]

Figure 6a illustrates the observed [illegible]tions at which the reinforcing wires [illegible] in a typical test wall. The [illegible] fractures of the reinforcing [illegible] a [illegible] of multiple shear planes [illegible]ined at a mean angle of 26° to the [illegible]tion of the major principal stress. This mean angle corresponds exactly to the [illegible] angle (45°-ϕ/2) for the fill [illegible] for which ϕ = 38°. Figure 6b [illegible] of one of the mats of mesh [illegible] analysed in Figure 6a. [illegible] of the steel are [illegible]ly visible.

Theoretically, it is possible for an [illegible] number of [illegible] planes to develop [illegible] granular [illegible]. In practice, [illegible] of shear planes [illegible]d by [illegible]ctional restraint [illegible] on the [illegible] bottom of the wall.

limited height to width ratio of the walls

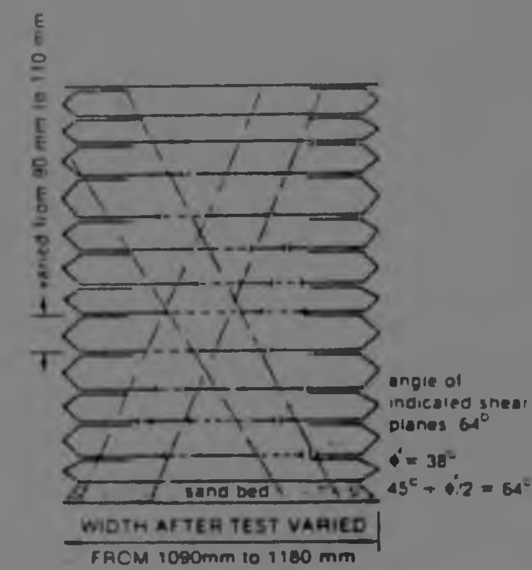


Figure 6a Positions of tensile failures in reinforcing of horizontally reinforced wall of granular material

prevent shear planes from developing fully except in the diagonal corner-to-corner configuration illustrated in Figure 6a.

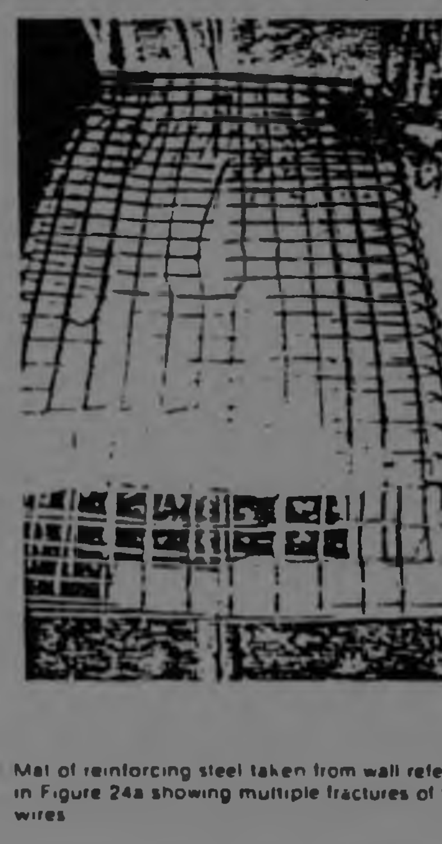


Figure 6b Mat of reinforcing steel taken from wall referred to in Figure 24a showing multiple fractures of transverse wires

SPIRALLY OR HOOP REINFORCED COLUMNS

As an alternative to embedding reinforcing mesh in the fill, a cylindrical column of fill can be reinforced by surrounding it with a steel spiral or a series of steel hoops. A radial confining stress is then developed as shown in Figure 7.

$$2Rp\sigma_h = 2T = \frac{2A_R\sigma_{yR}}{F}$$

i.e. $$\sigma_h = \frac{A_R\sigma_{yR}}{FRp}$$

and $$\sigma = \frac{K_p A_R \sigma_{yR}}{FRp} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

in which p is the pitch or spacing of the reinforcing hoops or coils.

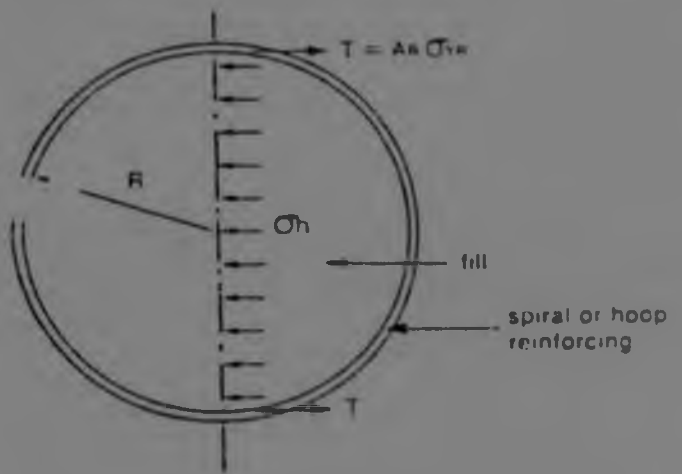


Figure 7. Principle of calculating the strength of a spirally reinforced column

Figure 6 shows a comparison of measured strengths of hoop and spirally reinforced columns with values predicted from equation (5). The agreement between theory and measurement is excellent and the comparison also shows that equation (5) applies equally well to both spiral and hoop

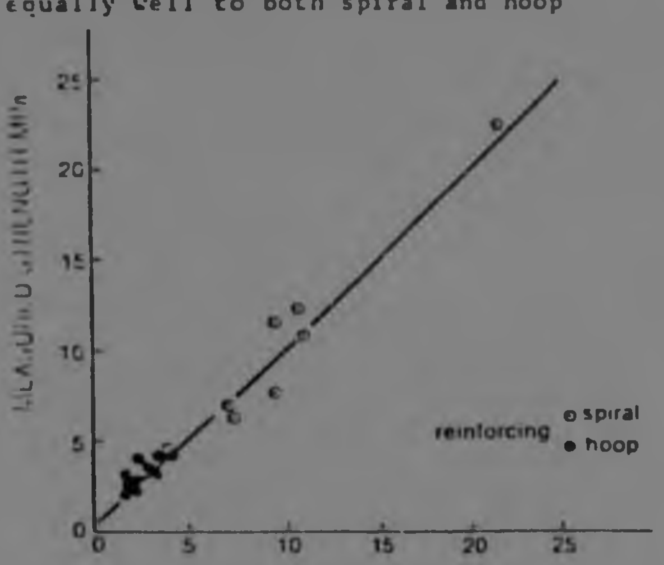


Figure 8. Comparison of calculated and measured strengths

reinforcement.

Bond between the reinforcement and the fill plays no part in a spirally reinforced column. However, to transfer the confining stress into the fill, it is necessary to have a retaining membrane between the fill and the reinforcing. In the laboratory tests carried out so far, a woven polypropylene hessian has been used for this purpose.

The reloading moduli of spirally or hoop reinforced columns are comparable with those of mesh reinforced walls. This is shown in Figure 9 which compares reloading moduli measured on reinforced ash walls with corresponding measurements on spirally reinforced ash columns.

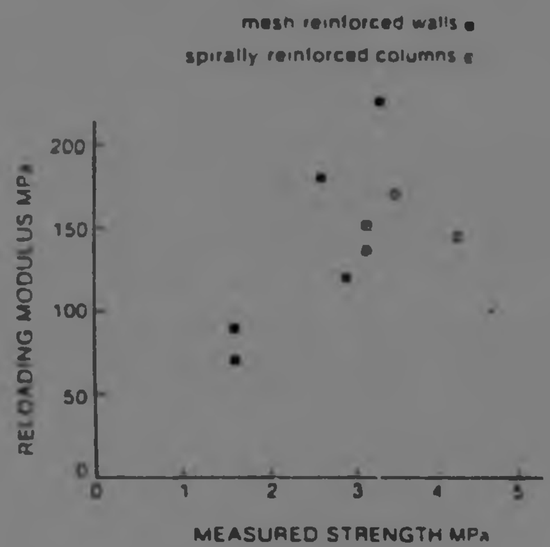


Figure 9. Comparison of reloading moduli for mesh reinforced walls and spirally reinforced columns

COMPARISON OF WALL AND COLUMN SYSTEMS

According to equation (3) the load carried by unit plan area of horizontally reinforced wall is

$$\sigma = \frac{K_p\sigma_{yR}}{F} \cdot \frac{A_R}{vh} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3a)$$

A_R/vh is then the volume of reinforcement per unit volume of wall.

Similarly, from equation (5) the load carried per unit area of spirally reinforced column is

$$\sigma = \frac{K_p\sigma_{yR}}{F} \cdot \frac{A_R}{Rp} \quad \ldots\ldots \quad (5a)$$

In this case, $2A_R/Rp$ is the volume of reinforcement per unit volume of column.

The major cost item in a reinforced

steel, hence the ratios A_R/vh or $2A_R/Rp$ are, to a large extent, the key to the economic viability of the system. For the same quality of fill and reinforcing and the same factor of safety $K_p\sigma_{yR}/F$ will be the same. Hence comparing horizontally reinforced walls with spirally reinforced columns, the two systems will be equally viable economically if

(1) $$\frac{2A_{R(col)}}{Rp} = \frac{A_{R(wall)}}{vh} \quad \text{.............} \quad (6)$$

and (2) equally strong if

$$\frac{A_{R(col)}}{Rp} = \frac{A_{R(wall)}}{vh} \quad \text{..................} \quad (7)$$

. two requirements clearly cannot be met simultaneously. For equal strength, if $A_{R(col)} = A_{R(wall)}$, then $Rp = vh$. It then follows from equation (6) that a spirally reinforced column will require twice the steel required by the equivalent horizontally reinforced wall.

In practical terms p will be of the same order of magnitude as v (25 to 100 mm). Hence R must of necessity be of the same order of magnitude as h if $A_{R(col)}$ is to be kept within realistic bounds. The situation is best illustrated by an example.

Consider a wall for which

σ = 5 MPa $\quad K_p$ = 3,5 $\quad \sigma_{yR}$ = 1000 MPa

F = 1,75 $\quad$ h = 50 mm and $A_{R(wall)}$ =

($\quad$ 20 mm² (equivalent to a wire diameter of 5 mm)

Then $$v = \frac{K_p\sigma_{yR}A_R}{Fh\sigma} = 160 \text{ mm}.$$

Now consider an equivalent spirally reinforced column having p = 160 mm.

$$R = \frac{K_p\sigma_{yR}A_R}{\sigma Fp} = 50 \text{ mm}.$$

which is clearly impractical.

If p is reduced to 30 mm, R = 267 mm, a more reasonable value. The example shows that whereas a horizontally reinforced wall may be constructed to any plan dimensions, the radius of a spirally reinforced column is limited to a maximum of about a metre by practical considerations.

Calculating the ratio of volume of reinforcement to volume of fill for the above example, we find that for the wall:

$A_R/vh = 0,25\%$

whereas for the column

$2A_R/Rp = 0,5\%$

which agrees with the conclusion arrived at above.

A square or rectangular horizontally reinforced pillar will need to have the same reinforcement normal to each of its pairs of sides. Hence the ratio of volume of reinforcement to volume of fill will be $2A_R/vh$ in this case. (Because of the necessity of providing bond steel in a wall, the ratio of reinforcement volume to fill volume is actually slightly higher than A_R/vh).

It is seen, therefore, that for isolated supports both systems require the same amount of reinforcing. The spirally reinforced column does, however, have potential advantages in that

(1) ultra-high tensile steel can be used in spiral form whereas this is not possible in mesh form because welding of the nodes locally reduces the strength.

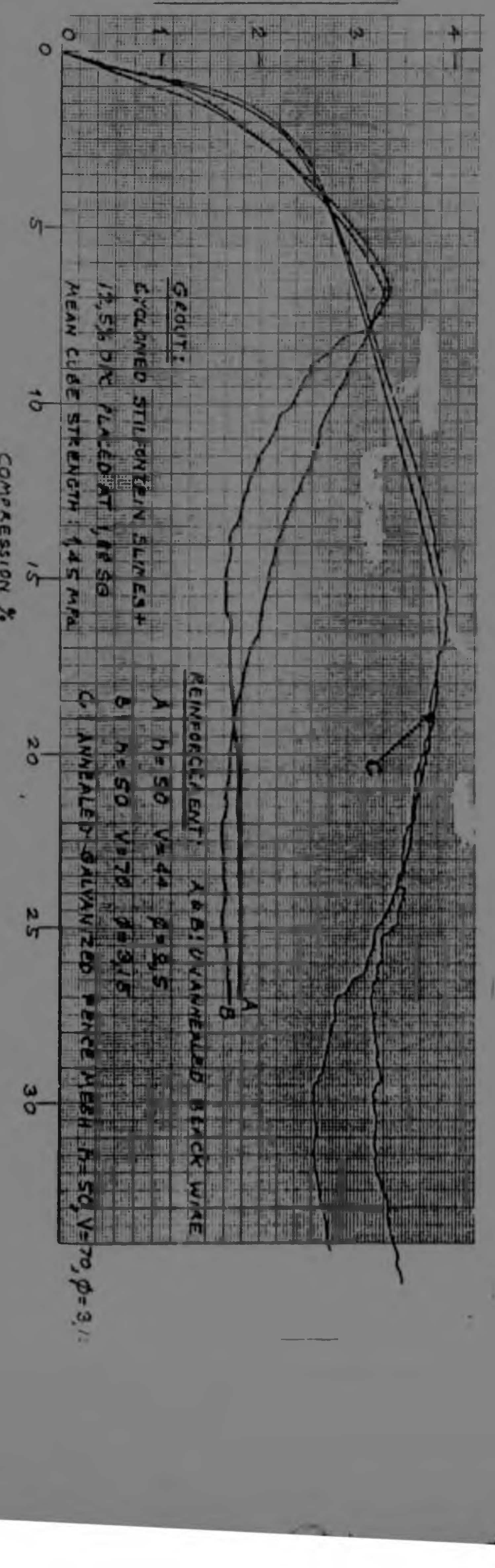
(2) the cylindrical mass of fill can be placed and compacted either before or after placing the reinforcing; and

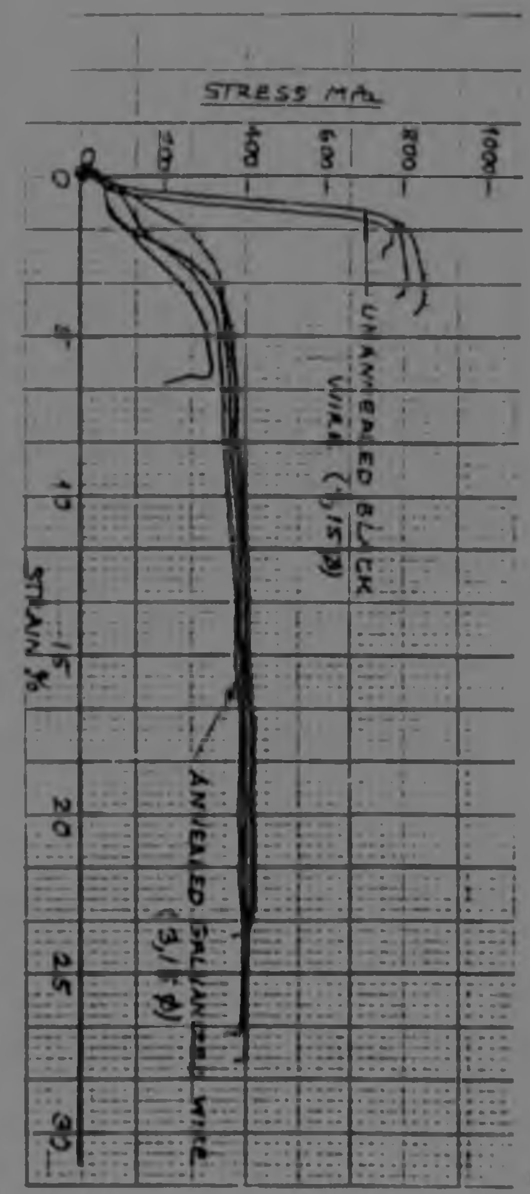
(3) the steel is open for inspection.

CONCLUSIONS

It is possible to build very effective artificial supports for shallow mining by using horizontally or spirally reinforced granular fill. The economic viability of the method will depend on local cost and circumstances and the selling price of the product. It is believed that at present the method has most promise for specialist applications. Walls, or rows of columns, could provide artificial barrier pillars for stoping operations in bord-and-pillar mines or for travelways in longwall mines. Spirally- or mesh-reinforced pillars could provide permanent or temporary point supports, or yielding supports. However, when the system is linked to the extraction of a higher-priced product (e.g. in South Africa, export coal) the method may well have wider applicability in primary mining operations.

APPENDIX 22


APPLIED STRESS MPa
GROUT:
CYCLONED STILFONTEIN SLIMES+
12,5% OPC PLACED AT 1,88 SG
MEAN CUBE STRENGTH: 1,45 MPa
REINFORCEMENT: A & B: UNANNEALED BLACK WIRE
A h=50 V=44 ø=2,5
B h=50 V=70 ø=3,15
C ANNEALED GALVANIZED FENCE MESH h=50, V=70, ø=3,15
COMPRESSION %


STRESS MPa
UNANNEALED BLACK WIRE (3,15 ø)
ANNEALED GALVANIZED WIRE (3,15 ø)
STRAIN %

Author Dison L
Name of thesis Management of invention and innovation in Civil Engineering 1985

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