

BeFo

Shotcrete

**Punch-loaded shotcrete linings
on hard rock**

Jonas Holmgren

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**Swedish Rock Mechanics Research
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ABSTRACT

Large scale punch load tests on plane and arch-shaped shotcrete layers interacting with granite blocks are presented.

Tests were made on plane unreinforced layers of different thickness. Tests were also made on plane layers with different amounts of reinforcement interacting with rock bolts in different ways.

Arch shaped shotcrete layers were tested both with and without end supports.

The load-deformation curve for reinforcement bars of different kinds embedded in concrete was investigated. The same kind of tests were made on strips of reinforcement nets embedded in shotcrete.

Calculations were made for a rough estimate of the influence on the carrying capacity of a two-dimensional extension of the shotcrete layers.

Calculations were also made for rough estimates of the action of reinforced shotcrete both at a service stage and at local cave in.

Finite element calculations were made of the state of stress in the rock-shotcrete interface near a joint at different loadings.

A literature survey is presented together with a discussion of shotcrete design.

Key words: Tests, shotcrete, reinforcement, rock bolts, adhesion, carrying capacity, load-deformation curve, finite element calculations, design.

PREFACE

In 1972 discussions about the interaction of shotcrete and rock were held between the Swedish Rock Mechanics Research Foundation (BeFo) represented by S Bjurström and the Royal Swedish Fortifications Administration (FortF) represented by E Abrahamsson, W v Essen, T Hahn and H Sundquist.

At the time being BeFo already had put shotcrete on their research program. The theme rock reinforcement also interested FortF, which builds a lot of military plants in rock caverns. Therefore it was natural to start a cooperative project on the behaviour and carrying capacity of shotcrete linings between BeFo and FortF through its Research Department.

Thanks to the absence of bureaucracy in these two organizations the project was started very quickly - the Author became employed part-time by BeFo as a research officer in January 1973, a reference group was set up at the same time and the first tests were made in the spring of 1973.

From the beginning the reference group consisted of E Abrahamsson, T Hahn, H Sundquist, all FortF, S G A Bergman, Consultant, S Bjurström, BeFo and C Alberts, Stabilator AB. Later on B Broms, Royal Inst of Technology (KTH), and S Sällström, Swedish State Power Board were associated to the reference group. When leaving the Board S Sällström was replaced by S Jansson, also at the Board.

Large-scale shotcrete tests were performed at the Test Station of FortF, where an excellent work was done by the staff. The most involved in this project were F Adolfsson and P Nilsson.

The tests on concrete-embedded reinforcement bars were performed at the Research Inst of National Defence (FOA) by J Eriksson and E Wernqvist. The test specimens were manufactured by B Karlström and T Wall, both KTH, who also assisted at the tests and their evaluation.

The input data for the finite element calculations were prepared by P-O Sundquist, FortF. M Viklund, FortF, contributed with remarks on the linguistic usage and the reference list.

Through the years a lot of valuable help with typing and drawing was received from P Alm, C Ericson, I Jäderberg, G Lennestedt, all FortF and E Ask, B Ahlgren, both KTH.

I wish to express my deep gratitude to all these persons mentioned above for their support and participation in the project.

I also wish to express my thanks to K Lindgren, KTH. Without being associated with the project he has always been willing to discuss equipment problems and related questions.

This project has given me a lot more friends than those mentioned above, and I wish to thank them all for what they have taught me about shotcrete and tunnelling works.

Finally I wish to thank my teacher, Professor H Nylander, KTH, for his kind support and for what he has taught me during the years within the field of building statics.

Stockholm, March 1979

Jonas Holmgren

DISSERTATION

This dissertation consists of a survey and the following three parts:

- (A) Holmgren J "Plane shotcrete layers subjected to punch loads". Royal Swedish Fortifications Adm, Research Dept, Report 121:4, Stockholm 1975, special edition 79-02-01.
- (B) Holmgren J "Deformationer hos ingjutna armeringsstänger". (Deformations of concrete-embedded reinforcement bars). Royal Inst of Technology, Div of Building Statics, Bull no 114, Stockholm 1975.
- (C) Holmgren J "Curved shotcrete layers subjected to punch loads". Royal Swedish Fortifications Adm, Research Dept, Report 121:5, Stockholm 1979. (Published only in the present thesis).

Shotcrete linings on hard rock - a survey.

By

Jonas Holmgren

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1. INTRODUCTION

This thesis deals with some aspects on the use of shotcrete as a means of supporting surfaces of hard rock masses. In this survey a short literature review will be given, where the investigations reported in the thesis will be looked upon in a wider context.

In the survey different approaches to the design of shotcrete linings will be distinguished:

- a) One way is to make use of others experiences. In the literature one finds case histories, which report data from shotcrete works. The different systems for rock classification are also assigned to this group since they also contain collected experiences through that support measures are correlated to the rock classes.
- b) Another way is trial and error approach where measurements are used to check the behaviour of the rock mass and the lining.
- c) A third way is rough calculations for basic failure mechanisms, either pure theoretical block models or large scale tests combined with theoretical considerations. In this connection the expression large scale tests means test where the loading block is of a realistic size and the surrounding rock-shotcrete structure is sufficiently large for an examination of a local failure.

2. DESIGN BASED ON EXPERIENCE

2.1 CASE HISTORIES

A typical case history contains a description of the geology, the selected support system and the execution of the construction work.

Examples of case histories are /1/, /2/, /3/, /4/ and /5/.

These histories of course give valuable information to experienced engineers. If the results are generalized and referred to in other papers there is a great risk, however, that we suddenly have got quantitative rules of thumb for which the prerequisites have been forgotten. In /6/ and /7/ an example of this process can be found.

In /6/ it is stated that certain design rules for the use of shotcrete in the blocky rock of Washington were found valid during construction. These rules were related to the diameter of the opening, the height of vertical sidewalls, the block size and joint characteristics. In /7/ then the height of the sidewalls is said to be generally important in very blocky ground with reference to /6/.

The other three parameters are said to be limiting the use of shotcrete in loosening ground in general without any reference to /6/.

The authors of /7/ are of course aware of the connection with /6/ but the readers of /7/ may not be and thus a new rule of thumb has been created.

This is unfortunate because it is not told what influence different adhesion conditions, a different ratio of σ_h / σ_v or even a different direction of the tunnel would have had.

It is the Author's opinion that case histories as the basis for design rules should be used with the utmost care in discussions and written papers. Of course, the experiences of others shall be compared with ones own but they have to be analyzed carefully before they are generalized.

2.2 DESIGN BASED ON ROCK CLASSIFICATION

There are various systems for rock classification which serve as a basis for the design of tunnel support systems. The most common systems are the RQD - index by Deere, the Rock Mass Quality by Barton et al and Rock Classes by Lauffer. They are described in several papers, see for instance /8/, /9/, /10/ and /11/.

These systems are not scrutinized here but some remarks concerning the use of shotcrete will be made.

In /9/ a correlation is found between the degree of support and the rock class for Scandinavian tunnels. According to the Author's opinion, this result is not astonishing. The rock-classes, independent of system, describe in some way or other how stable the rock is, and the selected degree of support then is a mirror for the expectations of this degree of stability. There is nothing to be said about this - the systems of rock classification account for collected experiences from a lot of tunneling works.

There are, however, some shortcomings which have to be pointed out. At least two of these remarks are earlier known, others have become evident as a consequence of the Swedish shotcrete investigation presented in chapter 4.2.

- a) The rock class does not tell what influence on the stability a variation of the tunnel direction may have.
- b) The state of stress in the rock mass is not considered, except for Barton's system.
- c) The mineral composition of the rock mass, which is of the utmost importance for the adhesion and thus the carrying capacity of a shotcrete lining in several loading cases, is not told by the rock class.
- d) Normally a thicker lining and/or decreased bolt distances are recommended when the rock class indicate worse conditions. In many cases, however, as shown by the investigation presented in chapter 4.2, a thin lining may have the same carrying capacity as a thick one.

PLANE SHOTCRETE LAYERS SUBJECTED TO PUNCH LOADS

by

Jonas Holmgren

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NOTATIONS

f_{ad}	adhesion strength	(MPa)
f_{bt}	flexural strength	(MPa)
f_c	compressive strength	(MPa)
$\bar{p}$	line load along the crack that separates two rock blocks	(kN/m)
$\bar{p}_f$	line load at failure	(kN/m)
P	total load at each section of the test equipment	(kN)
P_f	failure load	(kN)
δ	nominal width where adhesion stresses act	(mm)
σ	with different indices represents stresses	(MPa)

1. INTRODUCTION

The investigation presented in this report is the first step in a joint project between the Royal Swedish Fortifications Administration (FortF) and the Swedish Rock Mechanics Research Foundation (BeFo).

Its aim is better knowledge of shotcrete as a means of rock strengthening. If this aim is reached considerable economic savings could be made since the thicknesses of shotcrete used in practice probably are overestimated because of this lack of knowledge and the psychology of safety.

The problem complex is a large one and it has to be examined step-wise. To start with a rather simple case is studied which is defined in chapter 3.

The experimental and theoretical work described in this report has been carried out by the Author under the guidance of a reference group. Members of the reference group were initially Eddy Abrahamsson, Torbjörn Hahn, Håkan Sundquist, all FortF; Sten G. A. Bergman, Consultant, Sten Bjurström, BeFo; Claes Alberts, Stabilator AB; and later Bengt Broms, Swedish Geotechnical Institute, Stig Sällström, Swedish State Power Board.

The tests were made at the Research Station of the Research Department of FortF. The test rig has been designed by the Design Department of FortF.

2. EARLIER INVESTIGATIONS

The interaction between shotcrete and hard rocks has not been investigated previously in Scandinavia to the knowledge of the author. Östlund (/3/), Sällström (/4/) and Poijärvi (/5/) investigated the properties of the shotcrete itself. Barbo (/6/) has published results from some adhesion tests on shotcrete applied to different minerals. The number of tests is rather limited and of course the structural behaviour of the shotcrete has not been determined in his investigation.

In Austria several investigators have studied the use of shotcrete as tunnel lining. The investigated rock materials were weak, however, so the problem studied is principally the one of a concrete pipe surrounded by soil. (Ref. /7/, /8/, /9/, /10/). See also chapter 3.

This investigation is concerned with high strength jointed rocks.

3. LOADING CASE

A rather simple loading case, the punching of a large rock fragment through a plane shotcrete layer, was chosen for this investigation.

This case was studied by Rotter (/1/) who assumed a shear failure in the shotcrete layer. This led to the conclusion that a 3 cm shotcrete layer could carry 13 times the weight of a rock pyramide having a base area of one m^2 . The consequence of this model was that the carrying capacity of the shotcrete layer was proportional to the layer thickness and the shear strength of the shotcrete.

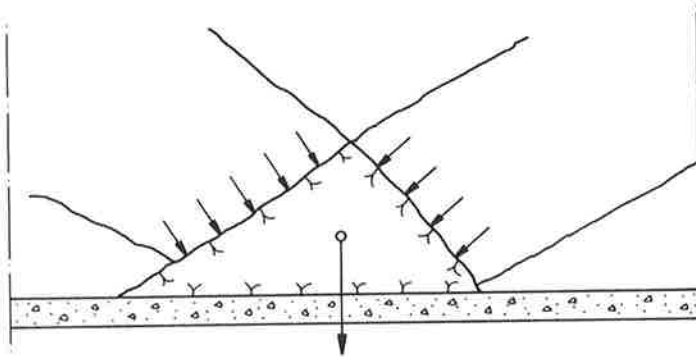


Fig 3 a Loading case in this investigation.

4. TEST RIG AND TEST SPECIMENS

4.1 MAIN TESTS

The main tests were made in the 3.7 m x 1.2 m test rig shown in Fig 4 a. Rock slabs of fine-grained grey granite were tested. The middle slab was pressed upwards during a test for practical reasons. The rig was fixed to the ground with a hinge parallel to the test specimen and was tilted to make the rock surface vertical during the application and curing of the shotcrete.

The different parts of the rig were balanced against each other with respect to the deformations in order to avoid bending of the shotcrete layer. The fulfilment of this was checked with a strain gauge during one of the tests. The load was applied with two parallel-connected handpumped hydraulic jacks.

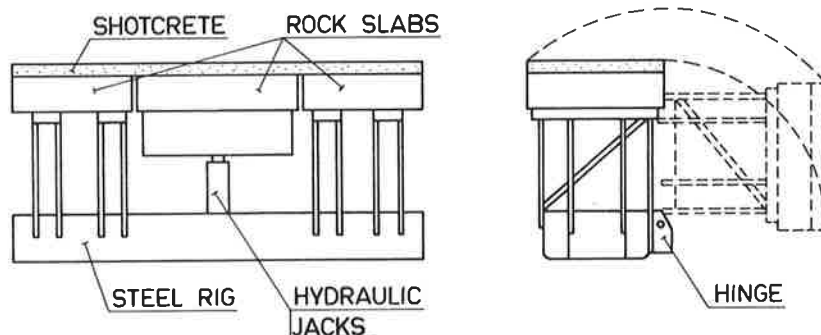


Fig 4 a Test rig.

4.2 CONTROL TESTS

A flat box 45 x 45 cm with the height equal to the thickness of the shotcrete in the main test was shotcreted and stored together with the main test specimen.

On the testing day cubes and beams were sawed out of this shotcrete plate.

4.2.1 Test cubes

The side length of the cubes was the same as the thickness of the shotcrete layer in the main specimen except for the 2 cm layer. In that case 4 cm cubes were used.

The 8 cm cubes were investigated in a 1.0 MN testing machine. A mechanical force gauge was put into the testing machine when the 4 cm cubes were tested. The compressive strengths presented in this report are the mean values of four cubes. No correction for the size of the cube has been made.

4.22 Test beams

The span length was 40 cm for the investigated beams. A two point loading system was used. The load was applied at the third points. The cross section was square and the side length was equal to the thickness of the shotcrete layer in the main specimen, except for the specimens with a 2 cm shotcrete cover where 4 cm beams were used.

The 8 x 8 cm beams were loaded by a hydraulic jack. The applied load was measured with a mechanical force gauge. The 4 x 4 cm beams were loaded with lead weights.

The flexural strengths given in this report represent the mean values for two tests. No correction for beam size has been made.

4.23 Testing of the adhesion strength

Cores with 50 mm diameter were drilled in the shotcrete down to the rock surface. A steel plate with a centric pin was glued to the top of each core with epoxy resin. The cores were loaded in tension. The corresponding force was measured at failure. The adhesion strength, f_{ad} , the force divided by the core area, was calculated. Three cores were tested for each test in the main test series.

5. MIXING OF CONCRETE, SHOTCRETING AND CURING

Standard cement and gravel (0-8 mm) were used for the concrete. The weight ratio was 1:4.5. The amount of water added to the shotcrete was regulated by the nozzle-man and was normally kept as small as possible.

The specimens with 2 cm layer were shotcreted in one operation. The specimens with 4 or 8 cm cover were normally shotcreted in two layers. The second layer was applied about three hours after the first layer.

A few hours after the shotcreting the concrete was covered with plastic sheets and it was then kept damp until the testing.

6. DEFORMATION MEASUREMENTS

The deformations were measured with dial gauges. The accuracy of the readings was about 0.01 mm. The test arrangement is described later in connection with the descriptions of the different tests.

7. TEST DESCRIPTIONS AND TEST RESULTS

7.1 A BASIC DESCRIPTION OF THE TESTS. TERMINOLOGY

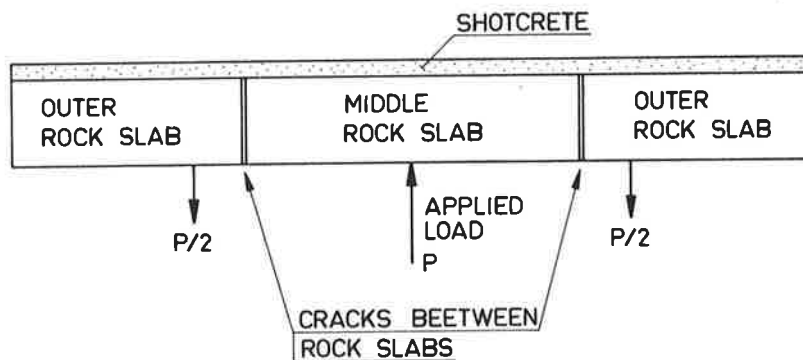


Fig 7 a Principal view of main test specimen.

7.3 REINFORCED SHOTCRETE COMBINED WITH ROCK BOLTS

7.31 Description of reinforced test specimens

Five tests were made with 8 cm reinforced shotcrete combined with rock bolts. The test specimens are shown in Fig 7 f.

Test A1 was made mainly to demonstrate that a plane reinforcement is not effective under the loading conditions chosen if it is not fixed properly to the rock due to the character of the adhesion failure.

Test A2 corresponded to the common case when the reinforcement nets are attached to rock bolts only with thin steel wires.

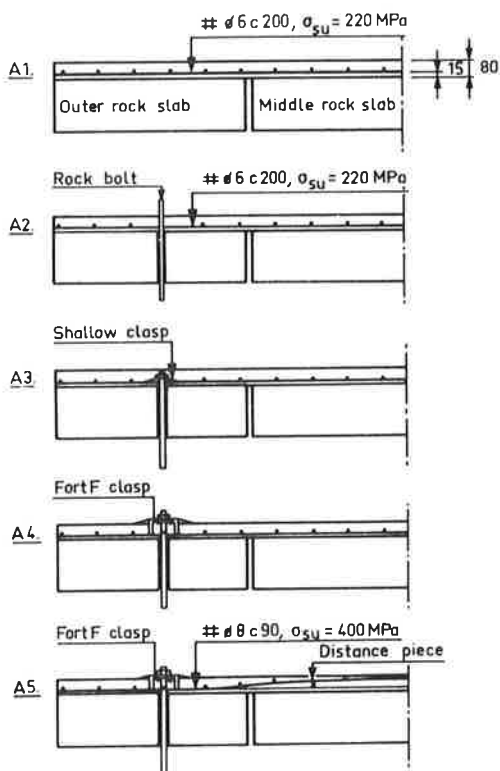


Fig 7 f Test specimens in series A.

The adhesion crack spread along the rock-shotcrete interface at a load of about one third the failure load until the concrete cover failed by bending. See Fig 7 c.

The failure load was not affected by the layer thickness except in the 2-cm tests probably because of the rather large slit (~ 5 mm) between the rock slabs.

The dependence of the adhesion strength was obvious:

$$\delta = \frac{\bar{p}_f}{f_{ad}} \quad (\text{see Fig 7 d})$$

was nearly a constant with a value of about 30 mm. This indicated that only a narrow band of shotcrete at a crack carried the load.

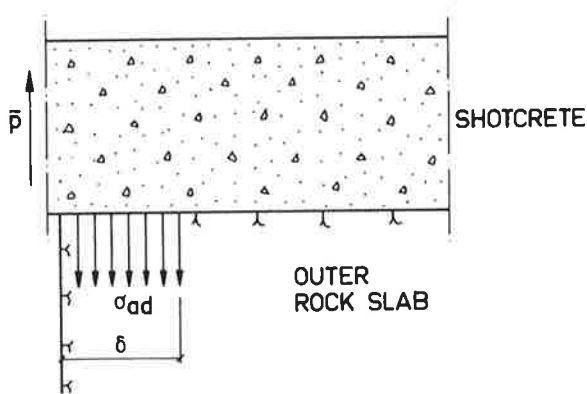


Fig 7 d Nominal adhesion stress block.

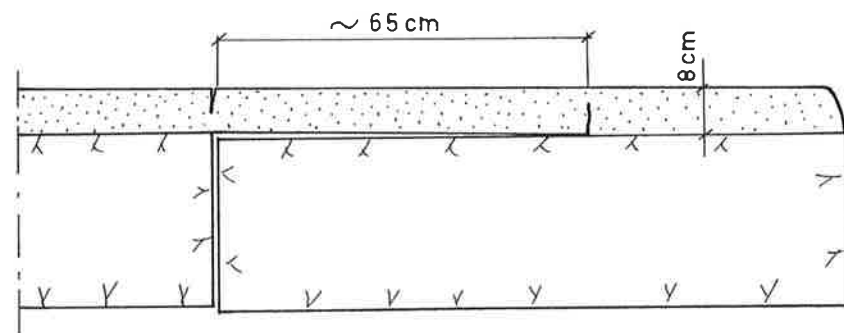
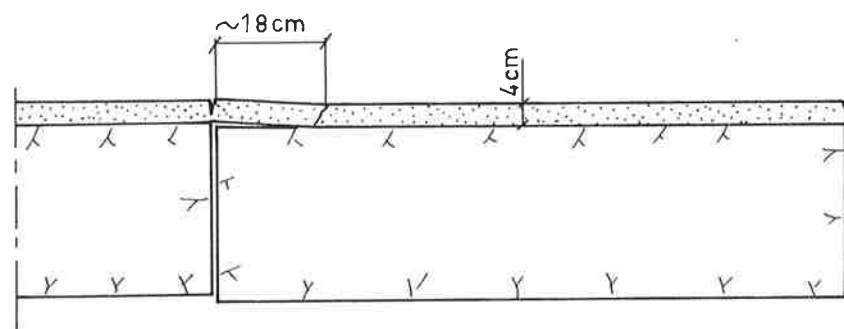
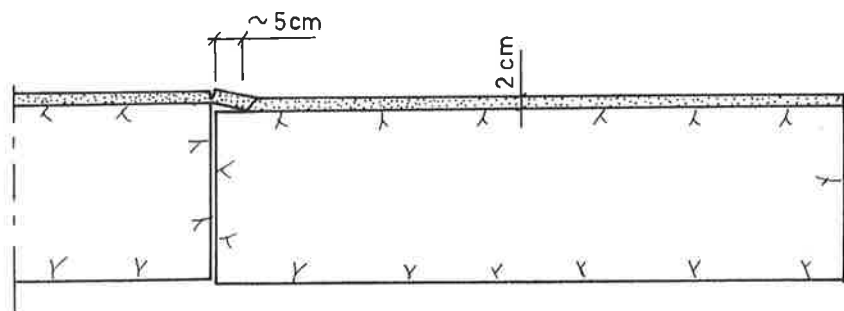
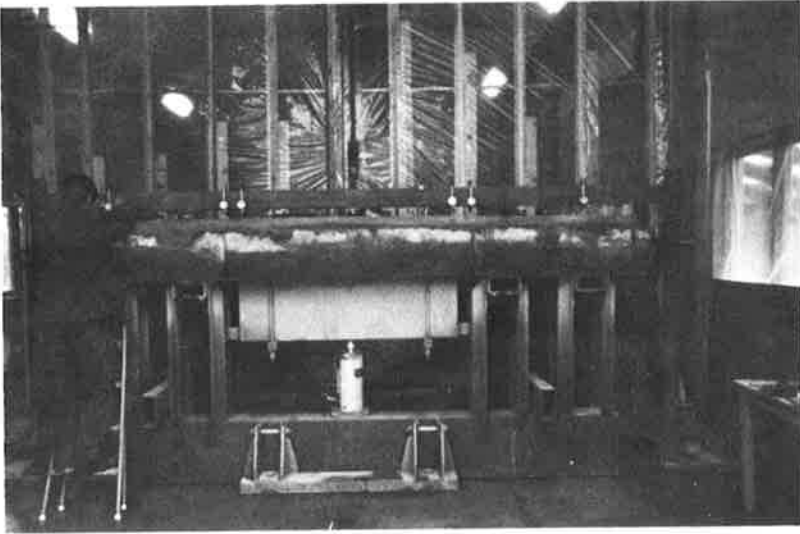
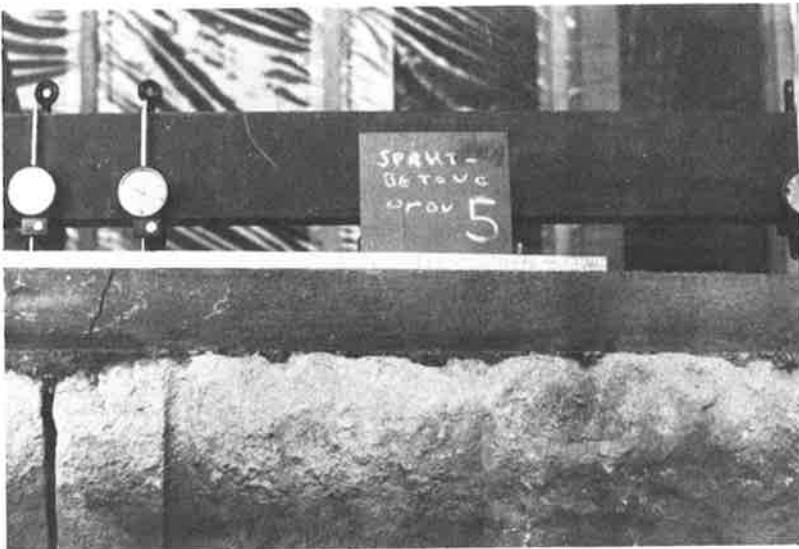


Fig 7 c Shotcrete layers after failure.



Test rig in testing position.



Unreinforced shotcrete layer after failure.

7.21 Tests under normal adhesion conditions

The main results are shown in Table 1. The cause of failure was always too high adhesion stresses between the shotcrete and the rock surface.

Test No	Age days	Nozzle-man	Layer thickness cm	f_c MPa	f_{bt} MPa	f_{ad} MPa	P_f kN/section	$\bar{P}_f$ kN/m	δ mm
1	7	J	4	37.0	6.6	1.4	42	36	26
2	7	J	8	31.6	6.1	1.8	59	52	29
3	2	K	4	23.9	5.4	1.8	35	30	17 ^x
4	2	K	8	31.6	6.3	1.8	-	-	- ^{xx}
5	1	K	8	32.4	6.4	-	34	30	-
6	1	J	4	19.1	4.1	0.9	28	24	27
7	2	J	8	32.8	7.1	1.1	24	60	55 ^{xxx}
8	2	K	2	26.8	5.0	1.3	17	15	11
9	6	H	2	37.0	6.0	1.5	31	27	18
10	1	H	4	22.3	5.7	> 1.8	60	52	< 29
11	1	H	8	36.9	7.5	1.4	55	48	33
12	6	H ₈	4	32.8	6.3	2.3	30	75	33 ^{xxx}
13	6	H	4	31.7	7.6	1.8	53	46	25 ^{xxxx}
A1	7	H	8	47.8	7.2	1.8	59	51	29
A2	7	H	8	45.6	7.0	1.6	69	60	38
A3	7	H	8	45.4	8.0	2.1	92	80	38
A4	6	H	8	35.2	6.9	1.6	84	73	45
A5	7	H	8	43.7	8.0	2.1	74	64	31

x Dry test cores.

xx Malfunction of test rig.

xxx Adhesion partly eliminated $\bar{p} = P/0.4$ instead of $P/1.15$.

xxxx Rock surface dirty by diesel smoke.

Table 1 Main test results.

For the reinforced test specimens it was possible to increase the applied load after the adhesion crack propagation in some cases. For these tests a final failure load or secondary failure load existed.

7.2 TESTS WITH UNREINFORCED SHOTCRETE

Ten tests under normal adhesion conditions and three tests under special conditions were made. The layer thickness and concrete age were varied. Also reinforced test specimens are discussed in this chapter since the reinforcement did not function until after the primary failure.

Dial gauges were used to check the movement of the rock slabs in tests 1-13. The gauges were attached to a steel frame, which was free from the test rig, and pointed to small glass plates glued to the concrete surface.

The angular displacement was about 0.0001 radians for the two outer rock slabs near maximum load in most cases. This deformation did not seem to affect the test results since the strains in the concrete surface above the slit between the slabs were negligible.

The middle rock slab was pressed upwards by the load P during a test. The outer rock slabs were restrained by the loads $P/2$. The applied load was distributed from the middle slab to the outer slabs by the shotcrete layer.

The applied load, P , and the vertical displacement, w , of the middle slab were measured during the test.

At a certain load level, the primary failure load, an adhesion crack formed between the shotcrete and the surface of the outer rock slabs near the slit between the rock slabs. This happened in all tests. The load suddenly dropped to one third or less when the adhesion crack formed. This load was called the adhesion crack propagation load and it remained nearly constant during the crack propagation until a flexural failure took place in the shotcrete layer.

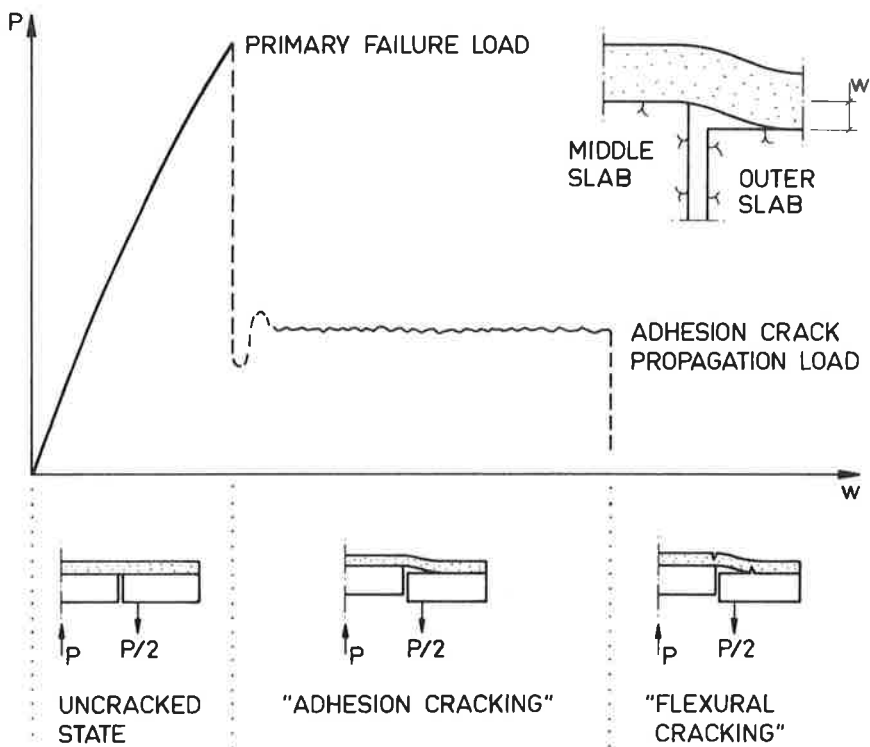
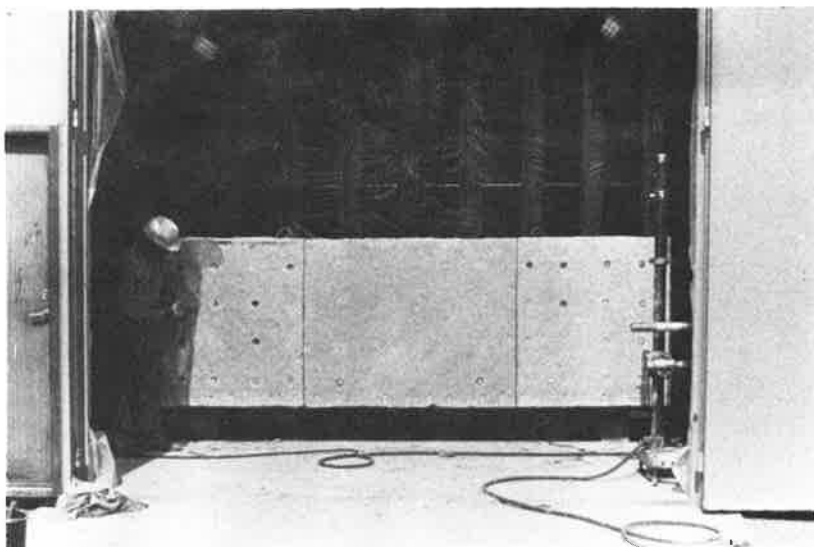


Fig 7 b Principal load-displacement diagram for unreinforced test specimens.



Test rig in shotcreting position .



Operator in shotcreting position .

$$\tau_{\text{nom}} = \frac{2 P}{\pi h^2 \left(\frac{B}{h} + 1 \right)} = 1.2 \text{ MPa}$$

using a modified formula by Nylander and Kinnunen (/11/).

B is a nominal width of the clasp and h is the effective depth of the shotcrete layer. In this case P in the original formula is replaced by 2 P because the rock bolt is subjected to load on one side only.

The theoretical failure stress according to Nylander and Kinnunen (/11/) is 1.4 MPa. Thus the measured failure load was 80 % of the theoretically calculated load. This result indicates that the clasps did not transfer the load quite properly from the concrete to the bolts since the punching theory mentioned above normally underestimates the failure load.

A considerable drop of the load occurred after the adhesion failure in Test A 4. It was, however, possible to increase the load up to 36 % of the primary failure load. The crack pattern was the same as for Test A 3.

The failure was caused by punching. The nominal shear stress at failure was $\tau_{\text{nom}} = 1.3 \text{ MPa}$, which is equal to the theoretical value. This indicates that the FortF clasps transferred the load more properly from the concrete to the bolts and that the failure load corresponded to the amount of reinforcement in agreement with the punching theory mentioned above.

In Test A 5 the decrease of the load was smaller than for the other tests. It was possible to increase the load up to 94 % of the primary failure load (only 75 % of the primary failure load for Test A 3, though). During the reloading several fine flexural cracks developed and a considerable curvature was noticed. The failure was by punching at one side of the test specimen and more like a beam shear failure at the other side. The nominal shear stress at failure was 3.0 MPa, which corresponds 110 % of the theoretical value.

7.33 Deformations

In Fig 7 h the relative displacement of the middle slab is plotted against the applied load. Test specimen A 5 is of special interest. A properly designed reinforcement in combination with an effective clasp gave a very strong and ductile shotcrete cover. The plastic energy in Test A 5 at failure was six times the energy in Test A 3. The energy ratios for Tests A 5, A 4, A 3 were 6:1.6:1.0.

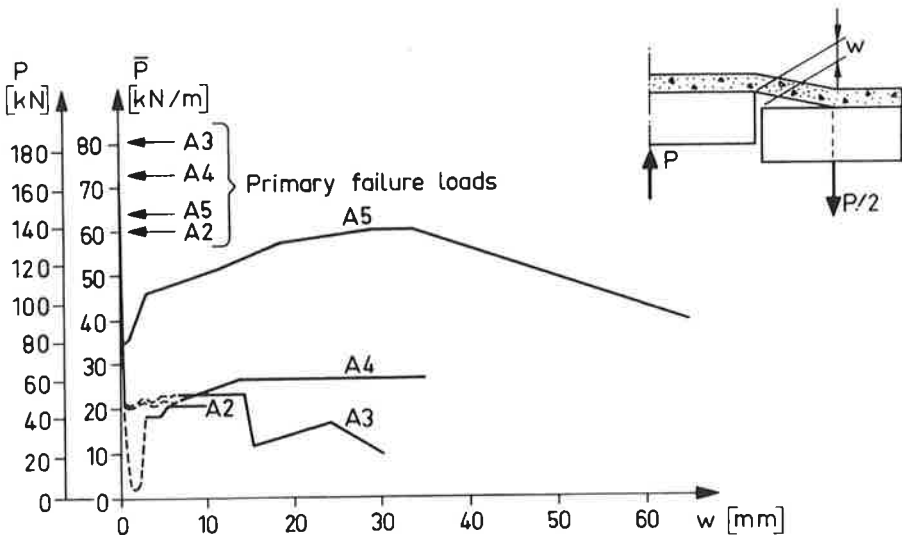


Fig 7 h Load-deformation curves for Tests A 2- A 5.
 $1 M_p = 10 \text{ kN}$, $1 \text{ kp/cm} = 1 \text{ kN/m}$. Arrows show the primary failure loads for the different tests.

8.4 THE CRACKED REINFORCED SHOTCRETE LAYER ACTING AS A CIRCULAR MEMBRANE

The effect of the two-way reinforcement is taken into account by assuming that it acts as radial reinforcement with spacing equal to c when $r = r_o$. It has also been assumed that it reaches some characteristic tension ϵ_B over the distance r_o to R at a failure stress f_B .

The line load $\bar{p}_{mbr}$ which means $\bar{p}$ at membrane failure can then be calculated from

$$\bar{p}_{mbr} \cdot 2 \pi r_o = \frac{2 \pi r_o}{c} A_b \cdot f_B \cdot \sin \psi$$

where A_b is the area of one reinforcing bar and ψ is the slope of the membrane.

The line load $\bar{p}_{mbr} = \frac{A_b \cdot f_B}{c} \cdot \sin \psi$ is thus independent of r_o .

The angle ψ is determined by

$$\sin \psi = \frac{\sqrt{2 \epsilon_B}}{1 + \epsilon_B}$$

Then

$$\bar{p}_{mbr} = \frac{A_b \cdot f_B}{c} \left(\frac{\sqrt{2 \epsilon_B}}{1 + \epsilon_B} \right)$$

If, for example, $A_b = 0.5 \cdot 10^{-4} \text{ m}^2$ ($\emptyset 8$),

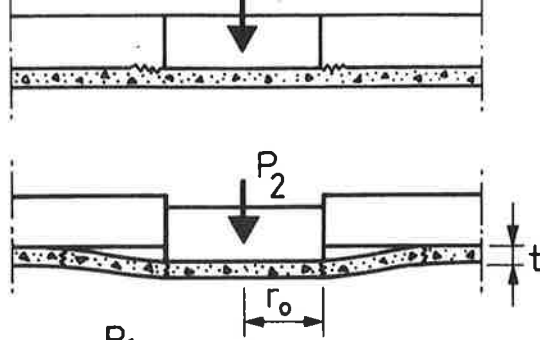
$f_B = 650 \text{ MPa}$, $c = 0.2 \text{ m}$, $\epsilon_B = 0.02$ (2 %)

$$\bar{p}_{mbr} = 32 \text{ kN/m} < 40 \text{ kN/m}$$

If instead ϵ_B is 0.08 then

$$\bar{p}_{mbr} = 60 \text{ kN/m} > 40 \text{ kN/m}$$

It is therefore of great importance to have an accurate value of ϵ_B for bars embedded in concrete.



P_1
 P_2 without reinforcement
 P_2 with reinforcement

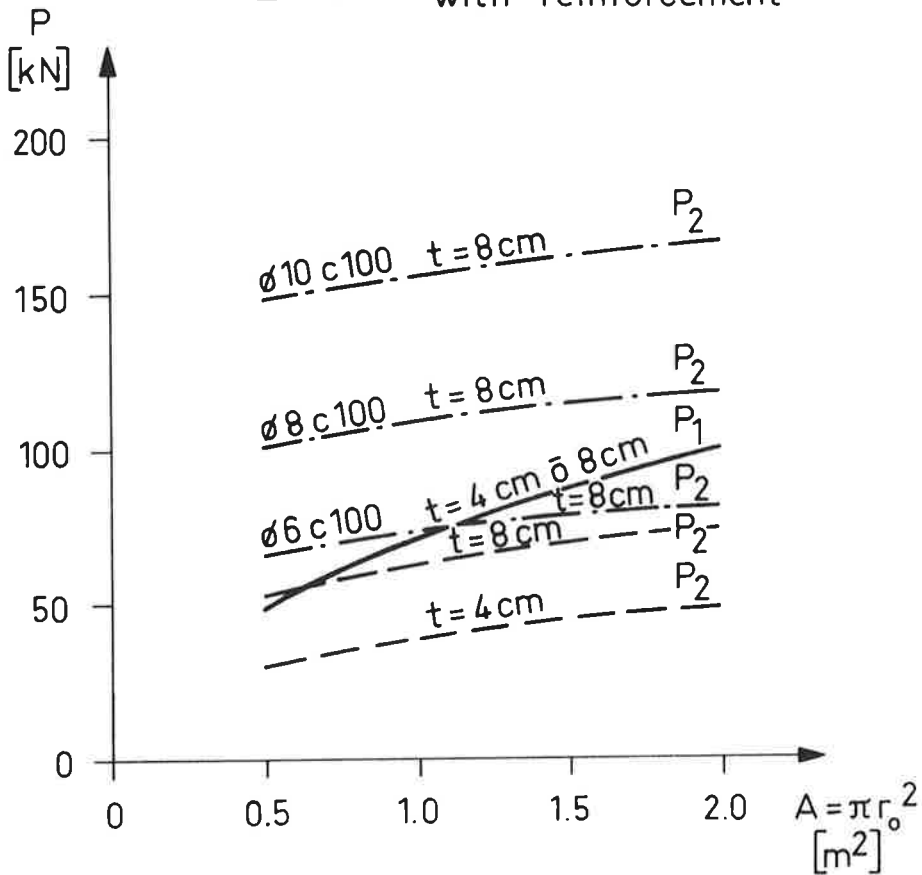


Fig 8e Theoretically calculated primary (P_1) and secondary (P_2) failure loads for plane circular shotcrete layers. P_2 is calculated both for unreinforced shotcrete layers and reinforced ones.

$$f_{bt} = 6.0 \text{ MPa}, f_{ad} \cdot \delta = 20 \text{ kN/m}.$$

Yield strength of reinforcement = 500 MPa.

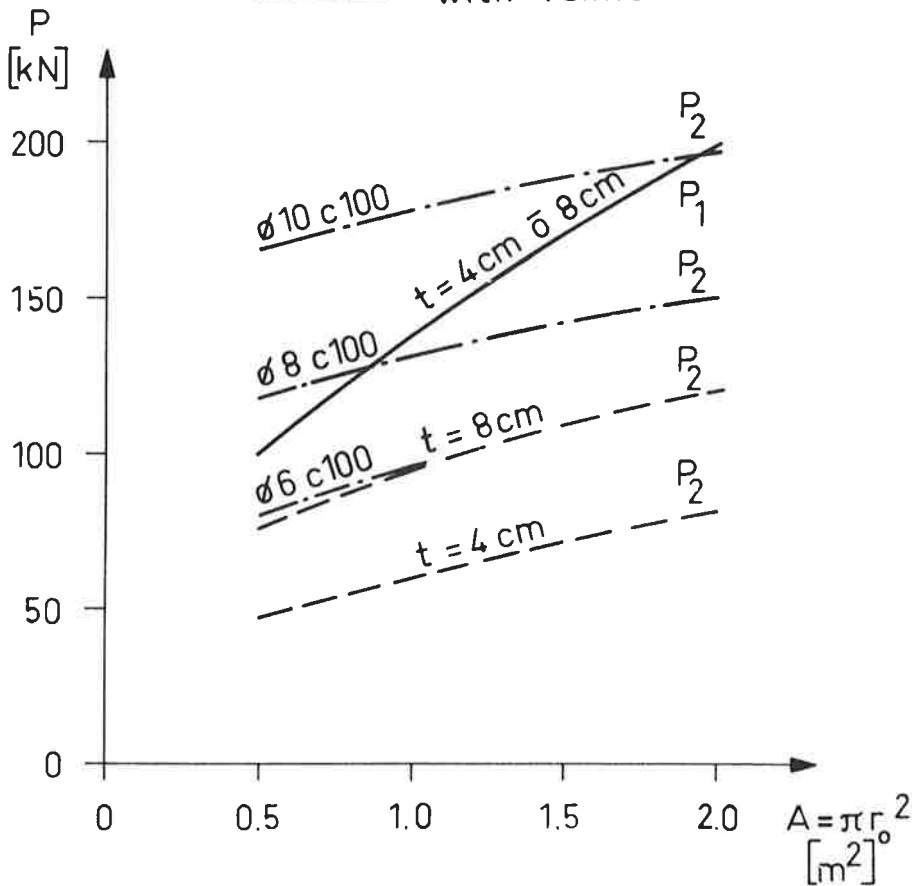
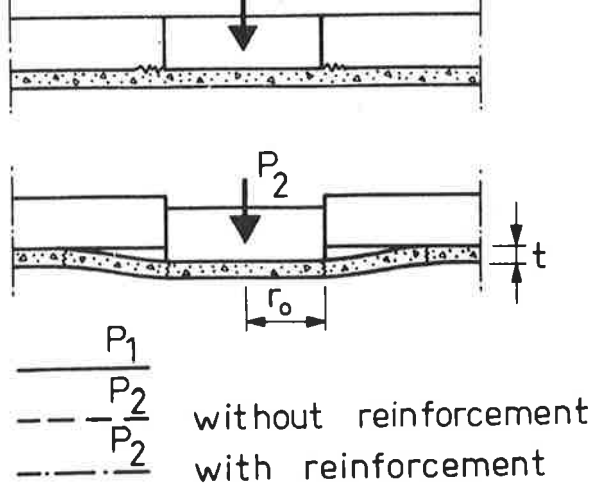


Fig 8d Theoretically calculated primary (P_1) and secondary (P_2) failure loads for plane circular shotcrete layers. P_2 is calculated both for unreinforced shotcrete layers and reinforced ones.

$f_{bt} = 6.0 \text{ MPa}$, $f_{ad} \cdot \delta = 40 \text{ kN/m}$.

Yield strength of reinforcement = 500 MPa.

Area of moving block m ²	Required reinforcement area %	Center distance for bar		
		Ø 6 mm	Ø 8 mm	Ø 10 mm
0,5	0.6	64	114	178
1.0	0.9	44	80	125
2.0	1.3	31	56	88
3.0	1.6	26	46	72
4.0	1.8	22	40	63
5.0	2.0	20	36	56

Table 2 Required two-way reinforcement to maintain plate effect for an 8 cm plane shotcrete layer. It is assumed that the shotcrete - rock interface can carry a line load which is 13.3 kN/m after the primary adhesion failure.

The table shows that it is possible to reinforce against downfall of small blocks and that normally used reinforcement is insufficient under the assumed circumstances.

In Fig 8d - e curves for reinforced shotcrete are plotted as a comparison with equivalent curves for unreinforced shotcrete. The curves show that commonly used amounts of reinforcement are normally insufficient if plate action is to be maintained after the adhesion failure. The figures show, however, that the action of reinforcement bars becomes more efficient at bad adhesion conditions.

Consider one sector element as indicated in Fig 8 c .

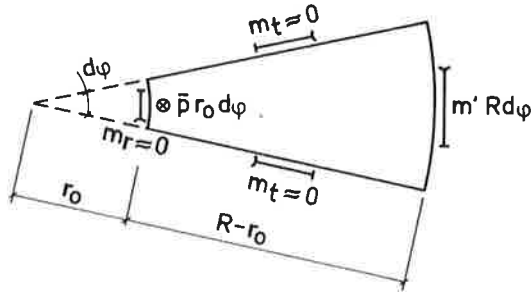


Fig 8 c Sector element in the yield line analysis.

The reinforcement net is normally placed close to the rock surface. The lever arms for the forces in the reinforcement are therefore small where the bending causes compression nearest to the rock surface. The moments of these forces become small, too, and are assumed to be zero in this simplified analysis. The equilibrium equation for the sector element becomes:

$$\bar{p} \cdot r_o \, d\varphi \, (R - r_o) = m' R d\varphi$$

which gives

$$m' = \bar{p} \cdot r_o \left(1 - \frac{r_o}{R}\right)$$

It is interesting to investigate if it is possible to put in the amount of reinforcement required by this equation.

At, for example $r_o/R = 1/3$, $\bar{p} = 40 \text{ kN/m}$, $h = 8 \text{ cm}$, $f_{sy} = 400 \text{ MPa}$ the following reinforcement will be required for a circular block.

In the following $P_1 = 2 \pi r_o f_{ad} \cdot \delta$ represents the load when the adhesion crack forms. During the development of the adhesion crack the load is

$P_r = 2 \pi r \cdot \frac{1}{3} \cdot f_{ad} \cdot \delta$ and especially when the plate fails the load is

$$P_2 = \frac{f_{bt} \cdot h^2}{k}$$

In Fig 8d - e P_1 and P_2 as functions of the block area $A = \pi r_o^2$ are given for two commonly used thicknesses of the shotcrete layer, 4 and 8 cm. Fig 8d represents very good adhesion conditions and Fig 8e represents fair adhesion conditions. It can be seen easily in these figures that P_2 always is much smaller than P_1 , which means that there is no redundancy in an unreinforced plane shotcrete lining.

8.3 THE CRACKED REINFORCED SHOTCRETE LAYER STUDIED WITH THE YIELD LINE THEORY

The loading case is the same as in Fig 8a. The assumed yield line pattern is shown in Fig 8b below.

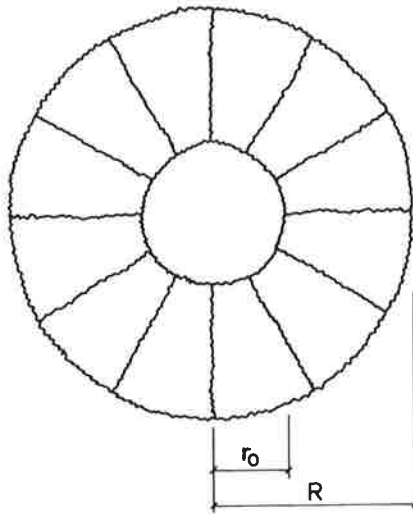


Fig 8 b Yield line pattern.

8.2 THE UNCRACKED SHOTCRETE LAYER ACTING AS AN ISOTROPIC CIRCULAR PLATE

The loading case is indicated in Fig 8 a below.

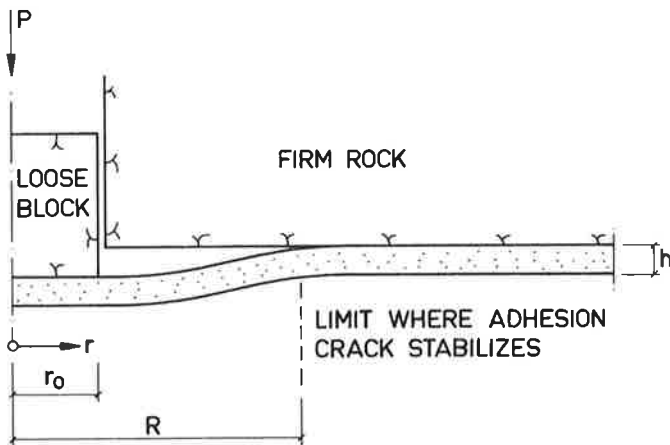


Fig 8 a Loading case for circular plate.

In Timoshenko's "Theory of Plates and Shells" solutions are given for this case.

The maximum bending stress, $\sigma_{\max}$, can be calculated from the relationship

$$\sigma_{\max} = \frac{kP}{h^2}$$

In this equation P is the applied load at the center of the rock piece and k is a constant which has been calculated for different ratios R/r_0 . R is the radius of the circular plate and r_0 is the radius of the moving rock piece. Timoshenko gives k for Poisson's ratio = 0.3, which is not always quite correct for concrete. His values are used here without any corrections, however.

8. THE TWO-DIMENSIONAL PLANE CASE - A THEORETICAL STUDY

8.1 GENERAL REMARKS

The performed tests were one-dimensional, since strips of constant width were tested. In reality the plane case nearly always is two-dimensional. An adhesion crack around a single rock piece spreads more or less like a ring on a water surface. Since the periphery of the failure zone grows larger and larger the bearing capacity has to increase since the failure load has been found to be proportional to the length of the failure zone. The adhesion crack has to stabilize somewhere. It is of interest to study a circular concrete plate and examine which load it can carry.

In the following an attempt will be made to get an idea of what might happen after the primary adhesion failure.

Four cases can be distinguished:

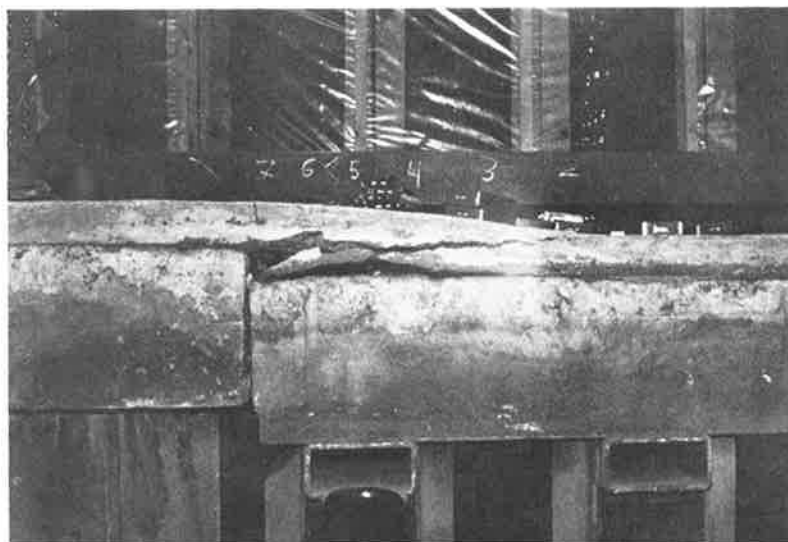
- a) An uncracked circular concrete plate carries the load after the adhesion crack has stabilized.
- b) A cracked reinforced circular concrete plate carries the load.
- c) A circular membrane carries the load.
- d) The reinforcement bars are torn off and the slab collapses.

In the following analysis two fundamental assumptions are made:

- 1) An adhesion crack forms when the distributed shear force, $\bar{p}$, is 40 kN/m. The crack stabilizes when $\bar{p} = 13.3$ kN/m. This assumption has been empirically confirmed by the tests, but is probably not valid for all rock materials.
- 2) The load follows the movements of the concrete plate. This assumption is probably not realistic for all rock loads, especially at large deformations.



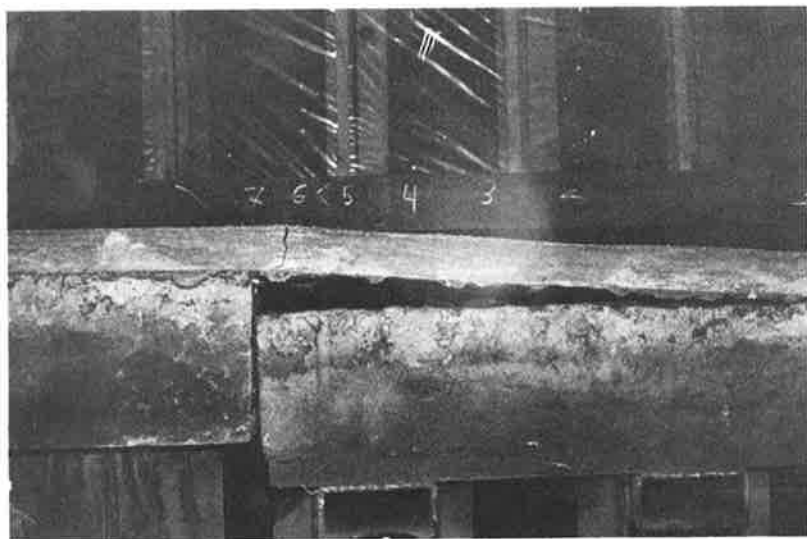
Clasp used in Tests A 4 and A5. (The FortF clasp)
Reinforcement used in Test A 5.



Test specimen A 5 after failure.



Clasp used in Test A 3.



Test specimen A 3 after failure.

Dragprovningarna utfördes i en MTS provningsmaskin vid FOA, Urvik. Armeringsstängerna fästes i provningsmaskinen med hydrauliska klämbäckar, som eliminerar glidning i infästningen. Härigenom kunde provningsmaskinens inbyggda lägesgivare användas för töjningsmätningen.

Styrprogrammet, som var utarbetat av *J Eriksson*, FOA, medgav att töjningshastigheten kunde hållas låg under provets första skede, för att sedan successivt ökas inom ett övergångsområde upp till en högre nivå inom det senare skedet.

Vid prov 1-12 och 17-20 och tillhörande kontrollprov var töjningshastigheten $\dot{\epsilon} = 4.76 \cdot 10^{-5}$ 1/s upp till bedömd sträckgräns varefter den långsamt ökades till $3.13 \cdot 10^{-4}$ 1/s. Vid prov 132-162 med kontrollprov som utfördes vid ett senare tillfälle var motsvarande töjningshastigheter $2.50 \cdot 10^{-5}$ 1/s respektive $1.67 \cdot 10^{-4}$ 1/s.

Detta innebar att sträckgränsen uppnåddes på 45 s vid prov 1-8, på 65 s vid prov 9-12, på 95 s vid prov 132-162 och på 35 s vid prov 17-20.

Under provningens gång registrerades kraft och längdändring kontinuerligt på en x-y-skrivare.

4 PROVNINGSRESULTAT

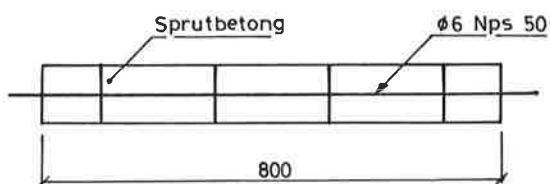
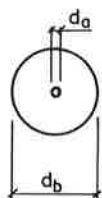
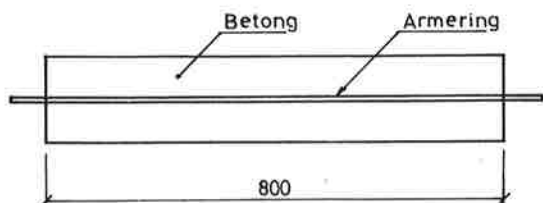
4.1 Betongens hållfasthetsvärden

Kubhållfastheten för betongen vid prov 1-12 och 17-20 var 31.2 MPa. Draghållfastheten var 1.8 MPa. Det skall dock erinras om att den använda provningsmetoden för draghållfasthet rymmer stora felmöjligheter. Kubhållfastheten för sprutbetongen vid prov 132-162 var 40.9 MPa och dess böjdraghållfasthet var 8.5 MPa.

Prov nr	Armerings- kvalitet	Armerings- stång [mm]	Betong- tvärsnitt [mm]	Armerings- procent %	Anm.
1	Ks 40	∅ 8	∅ 80	1.0	
2	"	∅ 8	∅ 80	1.0	
3	"	∅ 8	∅ 120	0.44	
4	"	∅ 8	∅ 120	0.44	
5	"	∅ 12	∅ 120	1.0	
6	"	∅ 12	∅ 120	1.0	
7	"	∅ 12	∅ 216	0.31	
8	"	∅ 12	∅ 216	0.31	
9	Ks 60	∅ 12	∅ 120	1.0	
10	"	∅ 12	∅ 120	1.0	
11	"	∅ 12	∅ 216	0.31	
12	"	∅ 12	∅ 216	0.31	
132	Nps 50	∅ 6	□ 30x100	0.93	sprutbetong
142	"	∅ 6	□ 30x100	0.93	ålder 10 d.
152	"	∅ 6	□ 100x100	0.28	"
162	"	∅ 6	□ 100x100	0.28	"
17	Ss 26	∅ 12	∅ 120	1.0	utgår p.g.a.skada
18	"	∅ 12	∅ 120	1.0	
19	"	∅ 12	∅ 216	0.31	
20	"	∅ 12	∅ 216	0,31	

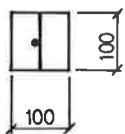
Tabell 1 Sammanställning av armeringskvalitet, stångdiameter, provkroppstvärsnitt och armeringsprocent.

Kind of reinforcement, bar diameter, cross section of the test specimen, percentage of reinforcement. Tests 132-162 were made with shotcrete at an age of 10 days.



Provkropp 132 och 142

[mm]



Provkropp 152 och 162

Fig 2a Provkroppar
Test specimens

Sprutbetongens tryckhållfasthet provades med tre utsågade 8 cm kuber.

Sprutbetongens böjdraghållfasthet provades med två oarmerade böj-
balkar 40 x 8 x 8 cm belastade med punktlaster i tredjedelspunkterna.

1 PROBLEMSTÄLLNING

För att kunna bedöma en armerad betongkonstruktions verkningssätt vid överpåverkan (vapenlast, ras, påkörning etc.) är det av stor betydelse att känna till den energiupptagande förmågan i konstruktionen.

Vad beträffar armeringen föreligger veterligt endast ett mindre försöksmaterial avseende brottförlängningen hos icke betongingjutna armeringsstänger av större längd än mätområdet vid standarddragprov. För ingjutna armeringsstänger saknas i stort grundläggande data beträffande brottförlängning. Föreliggande begränsade, experimentella undersökning syftar till att visa hur σ - ϵ -diagrammet för en lång (längd $\gg 10 \cdot d_a$) armeringsstång påverkas av kringggjutning av betong i ett renodlat dragförsök.

2 PROVKROPPAR

20 st provkroppar enligt fig 2a och tabell 1 tillverkades. Vid prov 1-12 och 17-20 bestod provkroppen av en rak armeringsstång centriskt ingjuten i en 800 mm lång, cirkulär betongcylinder. Vid prov 132-162 bestod provkroppen av en stång klippt ur ett nät med 20 cm maskvidd. Härvid kvarlämnades 10 cm långa tvärpinnar. Nätstrimlan sprutades in med sprutbetong så att en provkropp med rektangulärt tvärsnitt erhöles. Tvärsnittsdimension och ingående armering i de olika provkropparna framgår av tabell 1.

Till varje huvudprov uttogs ett armeringsprov ur samma stång utom vid nätproven. Kontrollproverna vid nätproven bestod av två nätstrimlor klippa ur samma nät som stängerna till huvudproven.

Betongens tryckhållfasthet provades med fyra normenliga kuber.

Betongens draghållfasthet uppskattades med hjälp av tre lossdragningsprov av 42 mm kärnor som borrades ur kuber.

Beteckningar

A_a	area hos armeringsstång	$[m^2]$
A_b	area hos betongtvärsnitt	$[m^2]$
d_a	stångdiameter	$[mm]$
d_b	diameter hos betongcylinder	$[mm]$
ϵ	relativ förlängning hos provkropp	$[\%]$
ϵ_B	rel. förlängning vid brott	$[\%]$
$\dot{\epsilon} = d\epsilon/dt$	rel. förlängning per tidsenhet	$[1/s]$
σ	dragkraft/nominell stångarea	$[MPa]$
σ_B	σ vid brott	$[MPa]$
μ	armeringsarea/betongarea	$[\%]$

Notations

A_a	area of reinforcement bar	$[m^2]$
A_b	area of concrete cross section	$[m^2]$
d_a	bar diameter	$[mm]$
d_b	diameter of concrete cylinder	$[mm]$
ϵ	relative elongation of test specimen	$[\%]$
ϵ_B	ϵ at failure	$[\%]$
$\dot{\epsilon} = d\epsilon/dt$	ϵ per time unit	$[1/s]$
σ	tension force/nominal bar area	$[MPa]$
σ_B	σ at failure	$[MPa]$
μ	reinforcement area/concrete area	$[\%]$

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Förord

Föreliggande undersökning har utförts vid Institutionen för Byggnadsstatik, KTH, med anslag från Civilförsvarsstyrelsen, Fortifikationsförvaltningen och Stiftelsen Bergteknisk Forskning. Behjälpliga vid utförande och redovisning har varit *B Ahlgren*, *E Ask*, *B Karlström* och *T Wall* vid institutionen samt *J Eriksson* och *E Wernqvist* vid Försvarets Forskningsanstalt.

Stockholm 1975-06-13

Jonas Holmgren

DEFORMATIONER HOS INGJUTNA ARMERINGSSTÄNGER

av

Jonas Holmgren

MEDDELANDE NR 114
INSTITUTIONEN FÖR BYGGNADSSTATIK
KUNGL TEKNISKA HÖGSKOLAN
STOCKHOLM 1975

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- c) Conventional reinforcement nets close to the first layer of shotcrete are of little or no value even if they are combined with a reinforcement cage around the rock bolt. The tests showed that the reinforcement must be designed as for ordinary reinforced concrete members and be combined with a proper reinforcement cage around the rock bolt.
- d) Some theoretical considerations indicate that the reserve strength (residual strength) is lower than the strength at the primary adhesion failure for conventionally reinforced shotcrete.
- e) Some tests with accelerating admixtures for shotcrete show certain undesirable effects which have to be examined more closely.

10. NEED OF FURTHER RESEARCH

- a) It is quite natural to investigate also curved shotcrete layers since only plane shotcrete layers were studied in this investigation. The adhesion strength will probably be important for the primary failure also for these tests but the progressive development of the failure has to be studied.
- b) It is also desirable to make a systematic study of the adhesion strength for different minerals since this factor has a decisive influence on the structural behaviour of shotcrete.
- c) The effects of the accelerating admixtures have to be closer examined.

Fiber reinforced shotcrete in combination with rock bolts might also be worth while to study. A lot of money can be saved if the bar reinforcement can be abolished.

Some preparatory tests by the Author have shown, that ϵ_B for bars surrounded by concrete may be $1/3$ to $1/2 \epsilon_B$ for free bars. The investigated bars were 8 mm deformed bars and 5 mm bars from reinforcement nets. The bars from the reinforcement nets had 100 mm pieces of the transverse bars left. The bars were surrounded by 145 mm concrete cylinders. The length of the test specimens was 800 mm. It was also found out that ϵ_B for reinforcement nets could be so small as 0.5 %.

For a reinforcement net:

$$A_b = 0.28 \cdot 10^{-4} \text{ m}^2, \quad f_B = 600 \text{ MPa}, \quad c = 0.1 \text{ m}$$

$$\epsilon_B = 0.005$$

$$\bar{p}_{mbr} = 16.7 \text{ kN/m} < 40 \text{ kN/m}$$

This indicates that reinforcement nets are more unreliable than loose bars.

9. SUMMARY

In this paper there are described 18 large scale tests on plane shotcrete layers in combination with rock blocks. The age of the concrete and the layer thickness were varied.

For five of the test specimens the influence of the reinforcement arrangement and the rock bolts was investigated.

The following main conclusions were drawn.

- a) For the unreinforced plane shotcrete layer failure was always due to adhesion failure along the shotcrete - rock interface. The failure load was independent of the layer thickness within the interval 2-8 cm. For good rock materials, however, the failure load is rather high. A plane shotcrete layer carries a rock cylinder with one square meter base area and five meters height.
- b) After the primary adhesion failure the load dropped to one third the primary failure load or less. The adhesion crack propagated along the rock surface until bending failure took place in the shotcrete.

Vid armeringsstänger ur nät Nps 50 $\emptyset$ 6 och $\mu = 0.93 \%$ var ϵ_B $2/3$ till $1/3$ av ϵ_B för fria stänger. Vid $\mu = 0.28 \%$ var ϵ_B omkring $1/6$ av ϵ_B för fria stänger. ϵ_B för fria stänger Nps 50 $\emptyset$ 6 var 6% .

Vid släta stänger (Ss 26, $\emptyset$ 12) skiljde sig ϵ_B vid ingjutna stänger inte från ϵ_B vid fria stänger p.g.a. att tvärkontraktionen hos stängerna var så stor att all samverkan mellan stål och betong upphörde. Efter brott hade stängdiameteren minskat till 11 mm. ϵ_B för Ss 26 $\emptyset$ 12 var 20-25 %.

Vid samtliga prov med ingjutna stänger, men särskilt vid större betongarea, erhöles större last vid fördelad töjning lika med sträckgränstöjning för fri stång än vad som svarar mot sträckgräns multiplicerad med stångarea. Förklaringen torde vara att armeringsstängerna vid uppsprickningen lokalt kommer upp i konsolideringsområdet intill sprickorna, medan lasten mellan sprickorna tas av armering och betong i samverkan. Detta har säkerligen stor betydelse vid böjning av betongbalk med liten armeringsmängd.

Vid snabba förlopp, såsom vid vapenlaster, påkörning o.d., torde ovannämnda effekt kunna påräknas. Man kan då sannolikt utgå ifrån att momentupptagande förmågan blir större än vad som anges av sträckgränskraft i armering multiplicerad med inre hävarm. Huruvida fenomenet uppträder vid långvarig belastning är inte utrett i och med denna undersökning. Det förefaller emellertid inte osannolikt att viss krypning och uppsprickning i den betong som omger armeringsstängens kammar närmast intill en spricka skulle kunna ge större töjningsvärden vid långtidsprovning än vid denna undersökning.

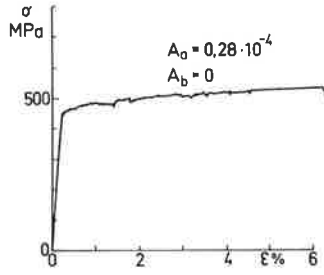
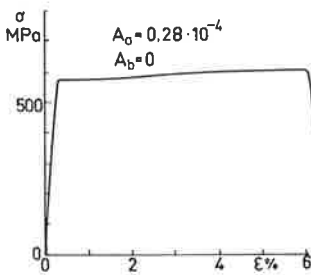
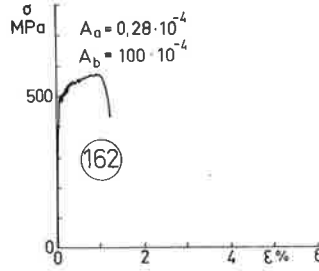
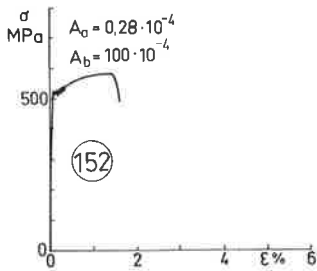
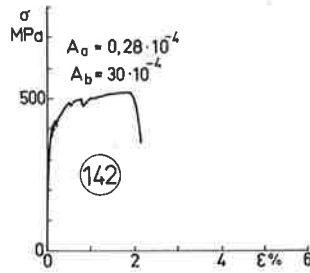
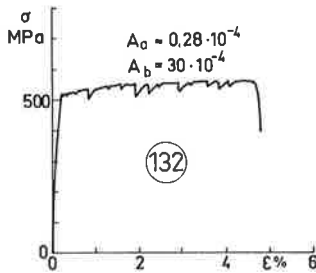


Fig 4e σ - ϵ -diagram för stänger ur nät Nps 50 ϕ 6 omgivna av sprutbetong samt tillhörande kontrollstänger. A_a = stångarea, A_b = sprutbetongarea.

Stress-strain relationship for bars from reinforcement nets Nps 50 ϕ 6 surrounded by shotcrete and the corresponding control bars. A_a = area of the bar, A_b = area of the surrounding shotcrete.

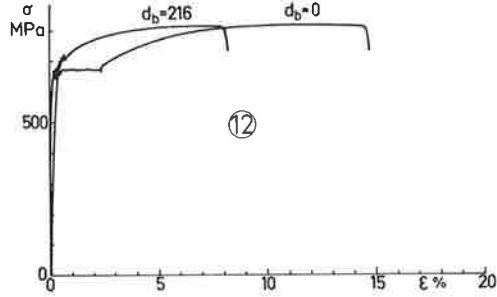
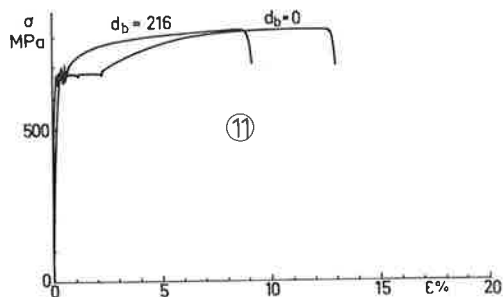
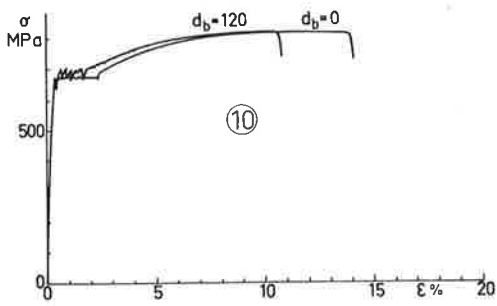
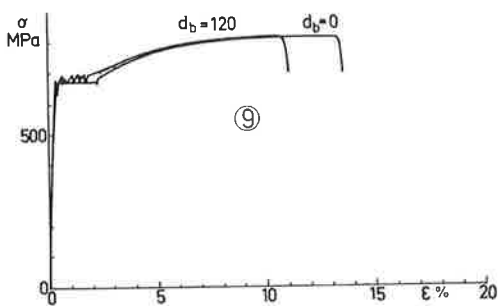


Fig 4c σ - ϵ -diagram för ingjutna stänger Ks 60 ϕ 12 och tillhörande kontrollstänger. d_b = betongcylinderns diameter.

Stress-strain relationship for deformed reinforcement bars Ks 60 ϕ 12 surrounded by concrete and the corresponding control bars. d_b = diameter of the concrete cylinder.

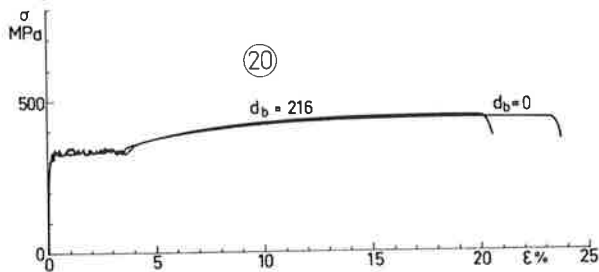
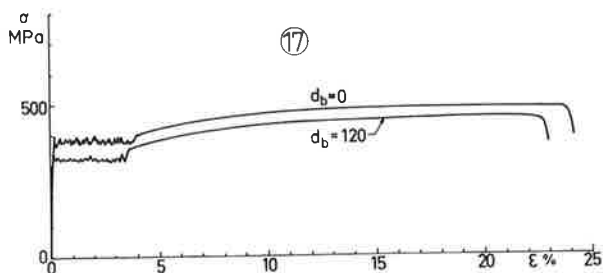


Fig 4d σ - ϵ -diagram för ingjutna stänger Ss 26 ϕ 12 och tillhörande kontrollstänger. d_b = betongcylinderns diameter.

Stress-strain relationship for smooth bars Ss 26 ϕ 12 and the corresponding control bars. d_b = diameter of the concrete cylinder.

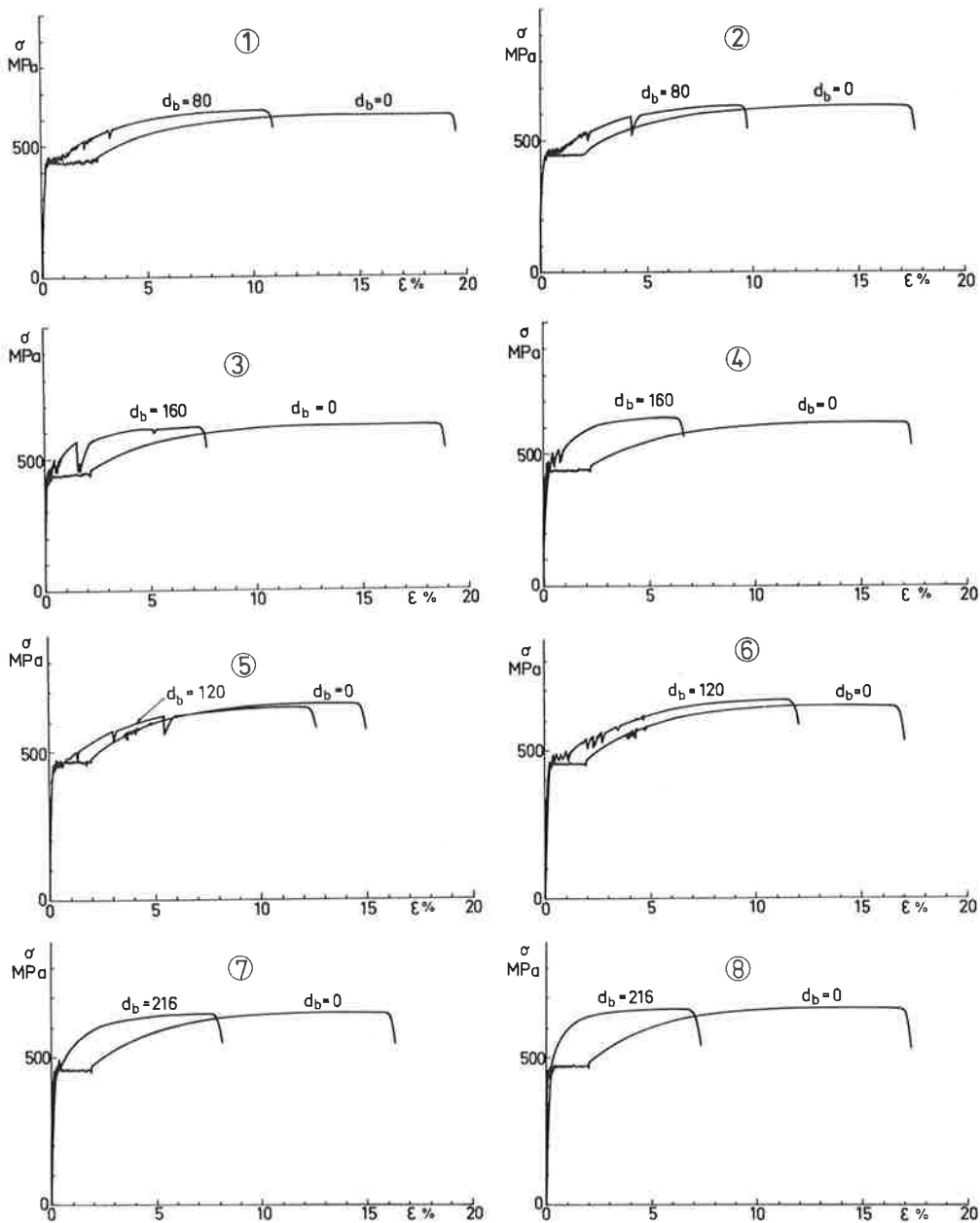


Fig 4b σ - ϵ -diagram för ingjutna stänger Ks 40 och tillhörande kontrollstänger. Vid försök 1-4 var $d_a = 8$, vid försök 5-8 var $d_a = 12$. d_a = armeringsstångens diameter, d_b = betongscylinders diameter i mm.

Stress-strain relationship for deformed reinforcement bars Ks 40 surrounded by concrete and the corresponding control bars. At tests 1-4 $d_a = 8$, at tests 5-8 $d_a = 12$. d_a = diameter of the bar, d_b = diameter of the concrete cylinder in mm.

4.2 Sprickbildning

Några sprickbilder redovisas i fig 4a. Den för ögat synliga uppsprickningen började oftast vid last motsvarande armeringens sträckgräns. Vid stormängd omgivande betong erhöles ett fåtal sprickor, vid liten mängd var antalet sprickor större. Vid provning av betonginsprutade nätstrimlor uppstod de första sprickorna vid armeringens tvärpinnar. Vid brott var sprickbredderna av storleksordningen cm.

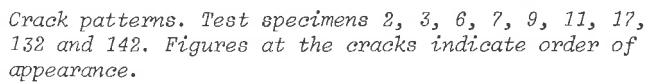
4.3 Spännings- töjningsdiagram

I fig 4b-e redovisas σ - ϵ -diagram för de ingjutna provkropparna med tillhörande kontrollstänger. σ är beräknat såsom registrerad last dividerad med nominell stångarea och ϵ är total förlängning hos provkroppen dividerad med provkroppens ursprungliga längd (800 mm).

Trots kraftig uppsprickning i många fall inverkade betongingjutningen avsevärt på brottförlängningen (ϵ_B). Vid armeringsstänger Ks 40 $\emptyset$ 8 minskade ϵ_B till 0.6 å 0.35 av ϵ_B för fria stänger. Den större minskningen hänför sig till större mängd ($\mu = 0.44\%$) omgivande betong.

Vid Ks 40 $\emptyset$ 12 var minskningen av ϵ_B vid liten mängd ($\mu = 1\%$) omgivande betong mindre än vid $\emptyset$ 8, förmodligen beroende på sämre effektivitet hos kammarna vid denna stängdiameter. Vid den större mängden omgivande betong var ϵ_B ungefär densamma vid $\emptyset$ 12 ($\mu = 0.31$) som vid $\emptyset$ 8 ($\mu = 0.44\%$). ϵ_B för fria stänger Ks 40 var vid $\emptyset$ 8 17-19% och vid $\emptyset$ 12 14-16%.

Vid armeringsstänger Ks 60 $\emptyset$ 12 reducerades ϵ_B till omkring 3/4 av ϵ_B för fria stänger vid $\mu = 1\%$. Vid $\mu = 0.31\%$ var ϵ_B för ingjutna stänger omkring 0.6 av ϵ_B för fria stänger. ϵ_B för fria stänger Ks 60 $\emptyset$ 12 var 13-14%.



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CURVED SHOTCRETE LAYERS SUBJECTED TO PUNCH LOADS

by

Jonas Holmgren

A simple example shows that suspension action and membrane action can take a lot more energy than beams and plates in bending. The example is strongly simplified but shows all the same that these actions should be regarded when designing a structure which might be subjected to extreme loads of disastrous character.

REFERENS

Birke, H: "Kupoleffekt vid betongplattor". Meddelande nr 108 från Institutionen för Byggnadsstatik, Kungl Tekniska Högskolan, Stockholm 1975.

19 tension tests on reinforcement bars surrounded by concrete were made. The stress-strain relationship for bars of 800 mm length was registered for the cast-in bars as well as for the corresponding control bars.

The loading velocity was very low compared with the velocities used in rapid tests with steel. On the other hand the velocity was not low if the time-dependent characteristics of concrete are taken into consideration.

At deformed bars and bars from reinforcement nets the failure strain decreased when the bar was surrounded by concrete. The failure strain was dependent on the bar diameter and the amount of surrounding concrete. When the bar diameter increased the failure strain increased if the reinforcement percentage was kept constant. When the reinforcement percentage decreased the failure strain decreased.

At smooth bars the bond between steel and concrete disappeared totally at large strains even at small reinforcement percentages.

At strains corresponding to the yield interval for free bars the load was higher for cast-in bars than for free bars. The explanation is probably that the steel locally at the cracks reaches the strain hardening stress. Between the cracks the load is carried by the steel and the concrete together at lower stress and strain levels. This phenomena did not appear at smooth bars.

When designing concrete members for disaster loads the failure strain can be put equal to 8 % for deformed bars. If smooth bars are used the failure strain can be put equal to 18 to 20 %.

Reinforcement nets are unreliable in this respect. At pre-tests for this investigation a net was found that had a failure strain of only 1.5 % for a free bar and 0.4 % for a cast-in bar. Since nets according to code requirements shall have a failure strain of at least 3 % the failure strain for cast-in nets might be put equal to 1 %.

19 dragprovningar av armeringsstänger omgivna av betong utfördes. σ - ϵ -diagram för 800 mm mätlängd registrerades för de betongingjutna stängerna liksom även för motsvarande oingjutna kontrollstänger.

Belastningshastigheten var mycket låg jämfört med vad som brukar anses vara snabba försök med stålmaterial. Däremot kan den inte anses vara låg vid beaktande av betongs tidsberoende egenskaper.

Vid kamförsedd armering samt nät minskade brotttöjningen när stängen var ingjuten i betong. Brotttöjningen var beroende av stängdiameter och mängden omgivande betong. Vid ökande stängdiameter blev brotttöjningen större än vid mindre stängdiameter och samma armeringsprocent. Vid ökande mängd omgivande betong minskade brotttöjningen.

Vid släta stänger upphörde samverkan mellan stål och betong fullständigt vid stora töjningar oberoende av mängden omgivande betong. Vid töjning motsvarande sträckgränsområde för icke ingjuten armering var lasten vid ingjuten armering högre än vid icke ingjuten armering, vilket förklaras av att stålet lokalt vid sprickorna kommer upp i konsolideringsspanning medan lasten tas av stål och betong i samverkan mellan sprickorna. Detta fenomen uppträdde inte vid släta stänger.

Vid dimensionering med avseende på överpåverkan kan brotttöjningen för ingjutna kamstål sättas till 8 %. Vid släta stänger kan brotttöjningar upp till 18 à 20 % utnyttjas.

Nät bör användas med försiktighet såsom energiupptagande element i betongkonstruktioner. Brotttöjningen kan variera starkt, exempelvis erhöles för en ingjuten nätstäng en brotttöjning på endast 0.4 % vid förprovningar till denna undersökning. Detta värde gällde ett nät med fri brotttöjning på endast 1.5 % vilket är lägre än normerat värde 3 %. Om nät används som energiupptagande armering bör brotttöjningen ej sättas högre än 1 %.

I ett enkelt exempel visas att lin- och membranverkan hos balkar och plattor är väsentlig att söka utnyttja vid konstruktioner som kan ut-sättas för överpåverkan. Dessa effekter kan ge större bidrag till energiupptagningen än balk- och plattverkan.

I fig 5b redovisas last-nedböjningskurva dels för en lina bestående av en ingjuten armeringsstång med σ - ϵ -diagram enligt prov 8, dels för en lina bestående av en icke ingjuten stång med samma σ - ϵ -diagram som kontrollprovet till prov 8. (Se fig 4b).

Av figuren framgår att den procentuella skillnaden i lastupptagande förmåga vid viss nedböjning är stor (ända upp till 25 %) mellan en ingjuten och en icke ingjuten lina i töjningsområdet 1/2 - 2 %.

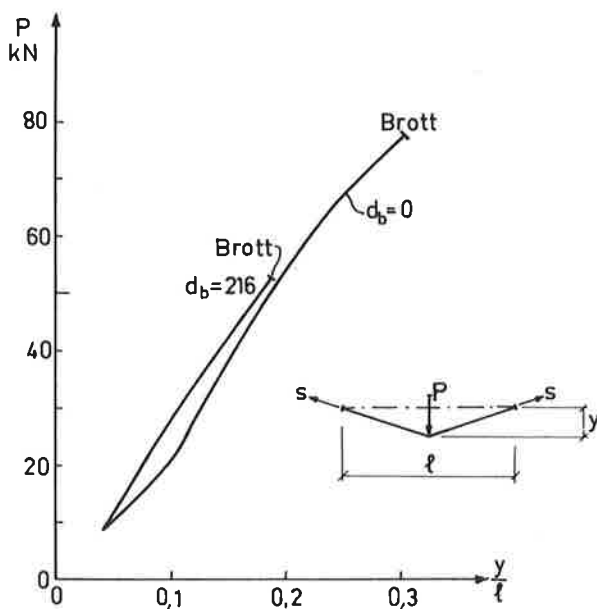


Fig 5b. Last-nedböjningskurvor för lina av ingjuten ($d_b = 216$) resp icke ingjuten ($d_b = 0$) armeringsstång $\phi 12$ Ks 40 med σ - ϵ -diagram enligt fig 4b, prov 8 med kontrollprov.

Load-deflection curves at suspension action of embedded ($d_b = 216$) and non-embedded ($d_b = 0$) bars $\phi 12$ Ks 40 with stress-strain relationships according to fig 4b, tests 8.

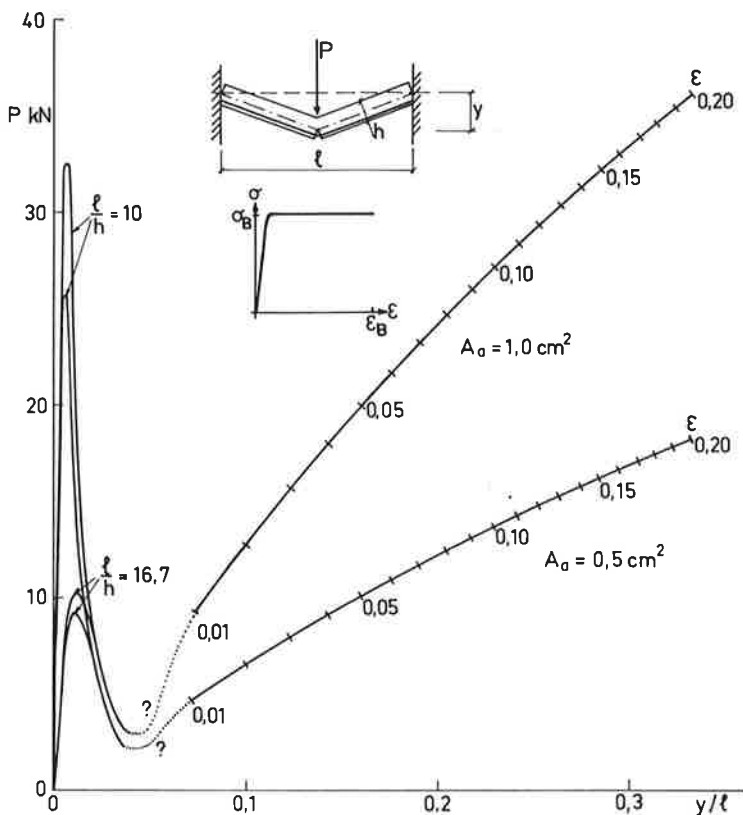


Fig 5a. Exempel. Till vänster i diagrammet last-nedböjningskurvor erhållna från försök av Birke [1] med armerade modellbalkar. I diagrammet har inlagts beräknade last-nedböjningskurvor vid linverkan hos armeringen (graderade med förlängning ϵ). Antaget, förenklat σ - ϵ -diagram för det ingjutna stålet visas i fig.

Example. To the left load-deflection curves from tests by Birke [1] on reinforced model beams. The figure also shows calculated load-deflection curves assuming suspension action of the reinforcement. The strain ϵ is shown. The assumed, idealized stress-strain relationship for the embedded bar is also shown.

Vid sådan förankring av underkantsarmeringen att horisontell dragkraft vid balkupplag kan upptas, torde förutsättningar för linverkan hos armeringen i ett sent brottskede föreligga. Balk med punktlast på mitten studeras under förutsättning enligt ovan.

Vid linverkan är sambandet mellan last P och förlängning av armeringen:

$$P = \sigma_B \cdot A_a \frac{2 \sqrt{2\varepsilon + \varepsilon^2}}{\sqrt{1 + 2\varepsilon + \varepsilon^2}}$$

där σ_B är brottspänning för armeringen, för enkelhets skull förutsatt konstant över hela töjningsområdet. A_a är armeringens area och ε är fördelad töjning över armeringens hela längd. Härledningarna är gjorda under antagande av rätlinjigt ökande nedböjning från upplag till balkmitt. Det förutsätts även att töjningen, ε , är jämnt fördelad från upplag till balkmitt.

Mittnedböjningen y erhålls ur

$$\frac{y}{l} = \frac{1}{2} \sqrt{2\varepsilon + \varepsilon^2}$$

Sambanden enligt ovan har tillämpats på ett exempel i fig 5a. Där visas last-nedböjningskurvor för fyra armerade modellbalkar provade av Birke [1]. I samma diagram är inritade beräknade last-nedböjningskurvor för armeringen under förutsättning av linverkan. Exemplet visar tydligt att stora vinster beträffande energiupptagande förmåga hos balkar och plattor är att göra om konstruktionerna utformas med målsättningen att i möjligaste mån nyttiggöra lin- eller membranverkan.

Det skall dock påpekas att exemplet är idealiserat såtillvida att det förutsätts att förhållandena vid dragprov med centriskt ingjuten armeringsstäng är direkt överförbara till det studerade fallet.

Test no	t cm	f_c MPa	$\bar{p}_f$ kN/m	$\bar{h}_f$ kN/m	$\bar{r}_f$ kN/m	$\sigma_{nom} = \frac{\bar{r}_f}{t}$ MPa	$\frac{\sigma_{nom}}{f_c}$
10	4	29,6	171 ^x	305	350	8,8	0,30
11	4,5	23,9	245 ^o	418	485	10,8	0,45
12	4	35,4	300 ^o	579	652	16,3	0,46
13	4	37,2	335 ^x	621	705	17,6	0,47
14	4	26,1	283 ^x	496	571	14,3	0,55
15	8	40,9	613 ^o	1046	1212	15,2	0,37
16	8	38,1	557 ^o	877	1039	13,0	0,34

Table 3 Failure loads at Tests 10~16.

Remarks

^x Buckling of middle part.

^o Shear failure.

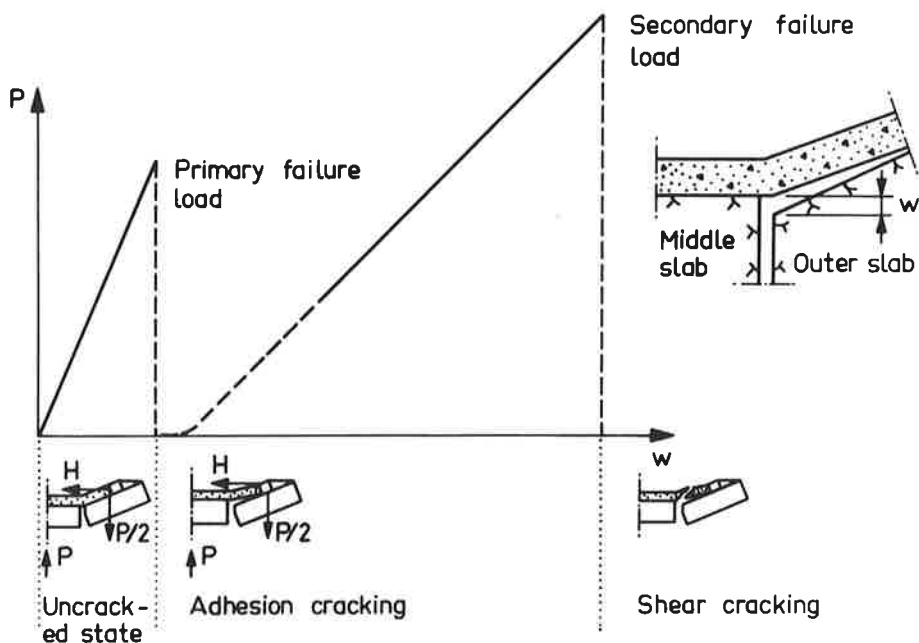


Fig 6c Principal load-displacement curve and failure modes for shotcrete arch with end supports.

In Tests 10, 13, 14 and 15 the adhesion crack spread to the horizontal part of the shotcrete layer and the whole arch turned free from the rock blocks. At the 4 cm-tests this lead to buckling of the horizontal part. In Tests 11, 12, 15 and 16 the failure was a shear failure because of high compressive stresses in the "leg" of the shotcrete arch. See fig 6c. The most important test results are shown in Tables 2 and 3.

Test no	t cm	Vertical load at adhesion failure $\bar{p}_{f,ad}$ kN/m			$\bar{p}_f$ kN/m	$\frac{\bar{p}_{f,ad, min}}{\bar{p}_f}$
		Left side	Middle block	Right side		
10	4	100	100	100	171	0,58
11	4,5	165	-	145	245	0,59
12	4	140	-	130	300	0,43
13	4	75	120	75	335	0,22
14	4	75	210	75	283	0,27
15	8	125	315	125	613	0,20
16	8	135	-	135	557	0,24

Table 2 Adhesion failure loads for different parts of the test specimen. Tests 10-16.

error in the vertical force is +5% and in the horizontal force -5% the error in the eccentricity will be about 2 cm.

The test results show, however, that the carried vertical line load, $\bar{p}$, is highly influenced by the horizontal line load, $\bar{h}$, at the joint (Fig 6b).

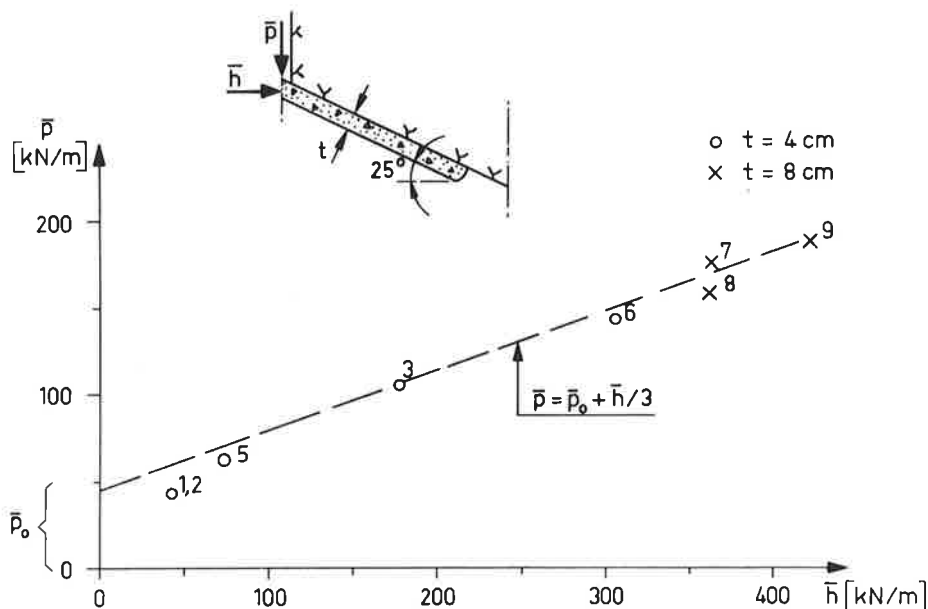


Fig 6b Applied vertical line load, p_f , at failure, as a function of the corresponding horizontal line load, h_f . Tests 1-9. When plotting the values for Test specimens no 1 and 8 their low adhesion strengths has been accounted for by proportioning the failure values.

6.2 ARCHES WITH END SUPPORTS

The tests were carried out in the same way as the tests with arches without end supports.

At the same load levels as in Tests 1-9 an adhesion failure took place at one of the joints. The adhesion crack propagated all the way to the end support. After that the load could be increased again. At a somewhat higher load level an adhesion crack developed at the other joint.

Test	Layer thickness cm	f_c MPa	f_{bt} MPa	f_{ad} MPa	P_f kN/section	$\bar{P}_f$ kN/m	β °	e cm
1	4	33.1	8.2	1.3	24	28	20	-
2	4	59.2	12.8	2.0	35	40	20	-
3	4	28.1	8.8	1.8	89	102	5	2.3
4	4	30.3	9.7	2.0	-	-	-	-
5	4	34.1	8.6	1.9	54	62	16	0.4
6	4	30.0	4.6	2.0	124	142	0	4.1
7	8	44.5	7.1	1.9	154	176	1	1.7
8	8	45.1	8.4	1.3	94	107	9	-1.8
9	8	46.9	8.0	1.6	164	187	-1	-0.2

Table 1 Main test results for shotcrete arches without end supports.
 β and e are defined in chapter 6.1.

The angle β in Table 1 is calculated as $\arctan (V/H) - 25^\circ$ where V and H are the vertical and horizontal forces at failure. β thus represents the angle between the resultant support force and the inclined, lined rock surface. See also Fig 7g, p 23. The eccentricity, e, is then calculated from the value of β and geometrical data for the test rig.

In Tests 1 and 2 the rock blocks partly rested on their supports by mistake during the whole test. The measurement of the reaction forces then became erroneous and the angle β could only be judged to be less than 20° .

The test results show that some variation of the angle β has been obtained. The calculated values of the eccentricity, e, however, are in some cases larger than the maximum eccentricities obtained from calculations assuming a linear stress distribution over the shotcrete section and material strengths equal to those obtained from control tests. One explanation is that the calculated value of the eccentricity is very sensitive to small errors in the measured forces in the steel bars. For instance if the

From that on the load was increased about 8 kN in each step. While the load was kept constant in each step gauge readings were taken.

The failure was always sudden. It was always a spalling failure of the shotcrete layer from one of the fixed blocks. See Fig 6a.

There was no clear indication that the thickness of the layer influenced the carrying capacity.

In Table 1 the most important test results are shown.

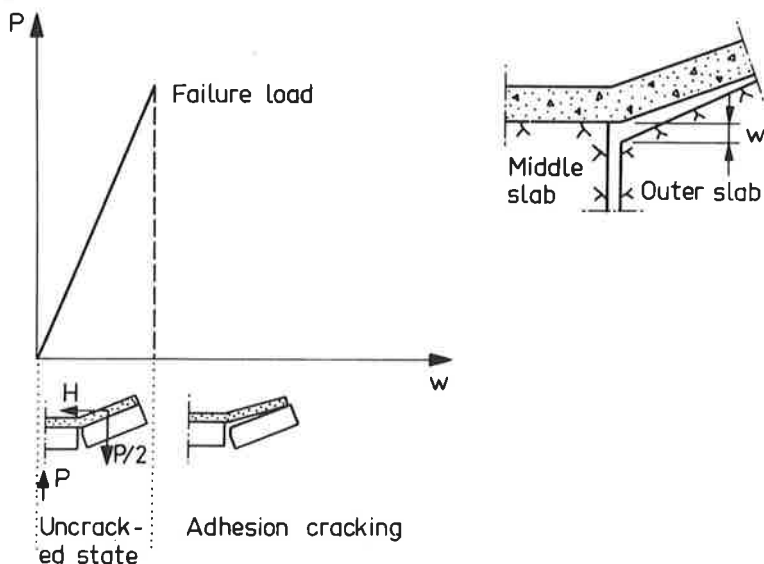


Fig 6a Principal load-displacement curve and failure mode for shotcrete arch without end supports.

4. MIXING OF CONCRETE, SHOTCRETING AND CURING

Standard cement and gravel (0-8 mm) were used for the concrete. The weight ratio was 1:4.5. The amount of water added to the shotcrete was regulated by the nozzleman and was normally kept at a practical minimum.

The specimens were normally shotcreted in two layers. The second layer was applied about three hours after the first one.

5. MEASUREMENTS

The load was applied through hydraulic jacks and measured by a manometer which was connected to the hydraulic system. The jacks and the same manometer were calibrated in a laboratory testing machine.

The reaction forces in the steel bars were measured with strain gauges coupled as a full bridge. The evaluation of the strains was done by reading the unbalance of the gauge bridge on a digital voltmeter with a sensitivity of 0.01 mV. The steel bars were calibrated in a laboratory testing machine together with the same voltmeter and constant-voltage source.

In Tests 13-16 the vertical displacement between the fixed rock blocks and the loaded one was measured with dial gauges with a sensitivity of 0.01 mm.

6. TEST DESCRIPTIONS AND TEST RESULTS

6.1 ARCHES WITHOUT END SUPPORTS

At the beginning of each test zero readings were taken. Then the horizontal steel bars were tensioned to about 3 kN while the rock blocks were still resting on their supports. Then the load was increased to about 40 kN - a level corresponding to four times the weight of the rock blocks and the shotcrete.

3.2 CONTROL TESTS

A flat box 45 x 45 cm with the height equal to the thickness of the shotcrete in the main test was shotcreted and stored together with the main test specimen. On the testing day cubes and beams were sawed out of this shotcrete plate.

3.21 Testing of the compressive strength

The 8 cm cubes were investigated in a 1.0 MN testing machine. A mechanical force gauge was put into the testing machine when the 4 cm cubes were tested. The compressive strengths presented in this report are the mean values of three cubes. No correction for the size of the cube was made.

3.22 Testing of the flexural strength

The span length of the investigated beams was 40 cm. A two point loading system was used. The loads were applied at the third points. The cross section was square and the side length was equal to the thickness of the shotcrete layer in the main specimen.

The flexural strengths given in this report represent the mean values of two tests. No correction for beam size was made.

3.23 Testing of the adhesion strength

Cores with 42 mm diameter were separated from the shotcrete by drilling down to the rock surface. A steel plate with a centric pin was glued to the top of each core with epoxy resin. The cores were loaded in tension, and the force was measured at failure. The adhesion strength, f_{ad} , the force divided by the core area, was calculated. Three cores were tested for each test in the main test series.

As earlier the test rig was fixed to the ground with a hinge parallel to the long side. This arrangement made shotcreting and curing possible with the rock surface in a vertical position while the test were performed with the free block in a horizontal position loaded upwards.

In Tests 1-14 the width of the 4 cm shotcrete layer was 87 cm and in Tests 15-16 with 8 cm thickness 48 cm. The 48 cm layer was divided into two strips of 24 cm each. The reason for having a smaller width of the thicker layers tested with end supports was the limited load capacity of the hydraulic jacks.

In Tests 1-10 the adhesion between the shotcrete and the middle rock slab was prevented at two 12 cm wide strips across the slab at each joint. The purpose was to get an even distribution of the compressive stresses in the shotcrete at the joints.

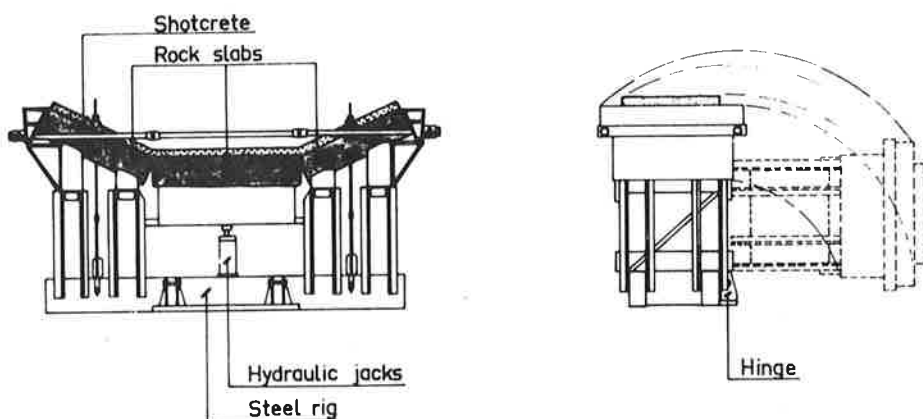


Fig 3a Test rig. Fig. shows the test rig when testing shotcrete arches without end supports.

tal plane. Horizontal and vertical steel bars $\varnothing 50$ and $\varnothing 26$ mm respectively were introduced to take the horizontal and vertical reactions during loading. The steel bars were equipped with strain gauges for the evaluation of the respective forces. The horizontal bars could be placed on three different levels - by this arrangement the direction of the resultant support reaction could be varied, though within a limited range.

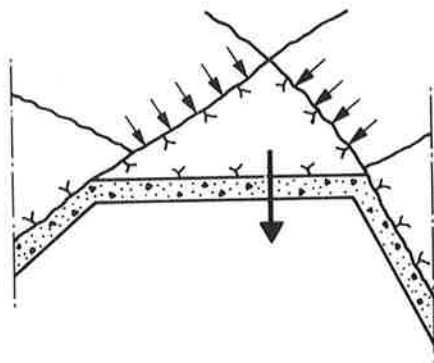


Fig 2a Loading case studied in Tests 1-9.

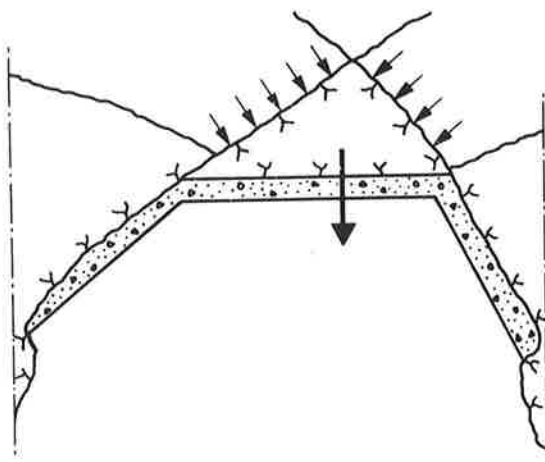


Fig 2b Loading case studied in Tests 10-16.

1. INTRODUCTION

This report deals with the second step of a large Swedish shotcrete investigation. The first step of it was presented and reported in /1/.

In /1/ a description is given of large scale tests made with plane shotcrete layers, both unreinforced and reinforced. The report also contains results of calculations on some theoretical models.

The purpose of the first step was to study the failure mechanism of plane shotcrete layers subjected to punch loads from penetrating blocks. By the combination of tests and theoretical calculations used in /1/ it was possible to distinguish between a good design and a less good design of a shotcrete lining in different situations. The results presented in /1/ are assumed to be valid even for linings with a slightly curved shape.

The purpose of the present part of the investigation was to study the failure mechanism of curved shotcrete layers subjected to punch loads.

2. LOADING CASE AND SUPPORT CONDITIONS

In this step of the investigation the tested shotcrete structures were subjected to a punch load from a single block.

Two different support arrangements were used:

In the first 9 tests the shotcrete lining was fixed to the rock blocks by its adhesion only (Fig 2a).

In tests 10-16 the shotcrete lining also had fixed end supports (Fig 2b).

3. TEST RIG AND TEST SPECIMENS

3.1 MAIN TESTS

The main tests were made in the 3,7 m x 1,2 m test rig used in the first step. It was now modified to be suited to the new geometrical conditions. The outer rock slabs were placed in 25° angle to a horizon-

NOTATIONS

E_c	modulus of elasticity for concrete	(MPa)
E_r	modulus of elasticity for rock	(MPa)
e	eccentricity	(cm)
f_{ad}	adhesion strength	(MPa)
f_{bt}	flexural strength	(MPa)
f_c	compressive strength	(MPa)
$\bar{h}$	horizontal line load along a joint	(kN/m)
H	horizontal force	(kN)
$\bar{p}$	vertical line load along a joint	(kN/m)
P	vertical load	(kN)
$\bar{P}$	resultant line load along a joint	(kN/m)
R	resultant force	(kN)
t	thickness of shotcrete layer	(cm)
V	vertical reaction force	(kN)
β	angle	(-)
σ	normal stress	(MPa)
τ	shear stress	(MPa)

indices: f for failure, h for horizontal, v for vertical

7.42 Influence of the eccentricity

7.5 SOME CONCLUDING REMARKS ON THE FE-CALCULATIONS

8. SUMMARY

REFERENCES

$$\sigma_{\perp} = \frac{(F_{1y} + F_{2y}) \cos \alpha + (F_{1x} + F_{2x}) \sin \alpha}{l}$$

$$\tau_{//} = \frac{(F_{1x} + F_{2x}) \cos \alpha - (F_{1y} + F_{2y}) \sin \alpha}{l}$$

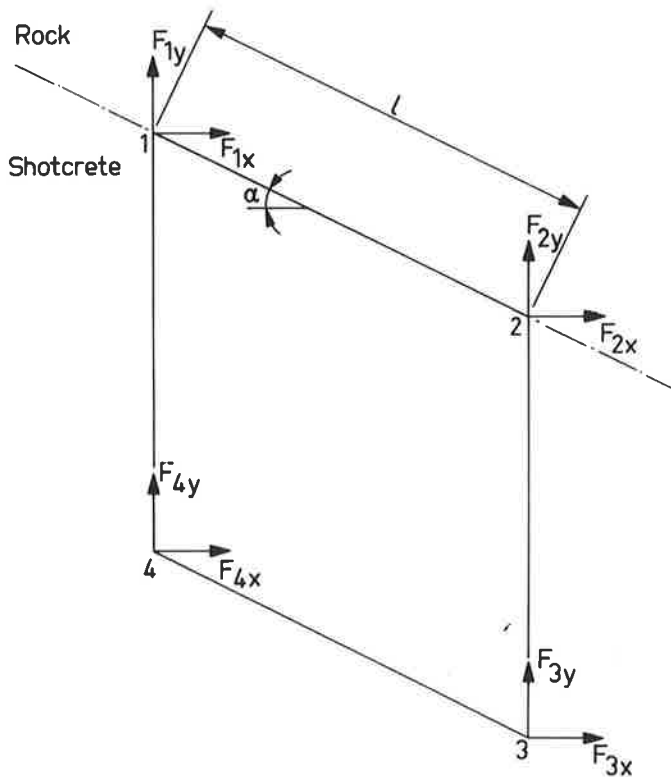


Fig 7 c Transformation of nodal forces to stresses along the element boundary.

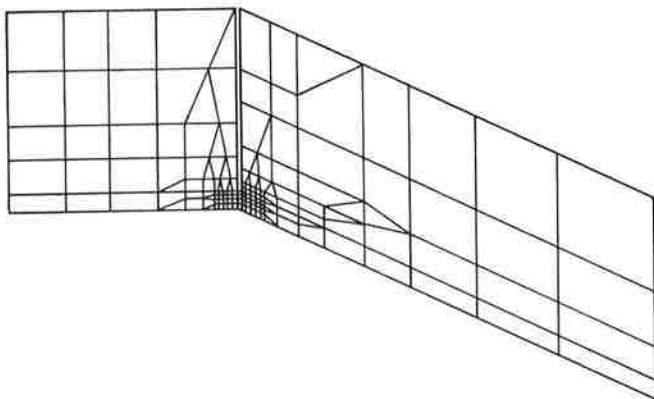


Fig 7 a "Old" FE-mesh.

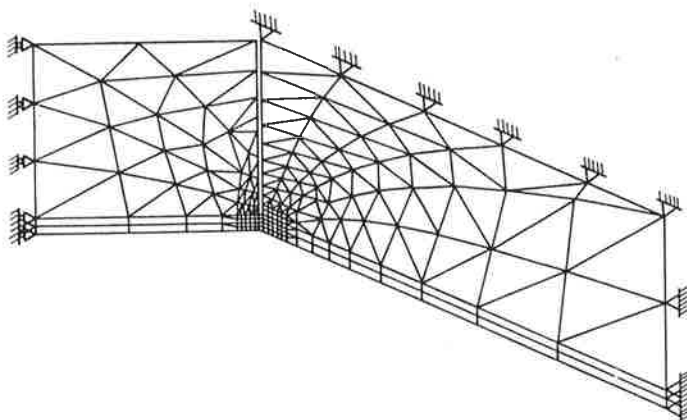


Fig 7 b "New" FE-mesh.

of the elements at the joint along the contact surface between the lining and the fixed slab.

The resultant of the restraint forces on the free block and the applied load was calculated. The resultant line load, $\bar{F}$, is presented in the figures for each loading case together with the corresponding vertical and horizontal line loads, $\bar{p}$ and $\bar{h}$. The nodal forces on the elements situated in the interesting area were transformed manually to distributed forces (stresses) along the element boundary perpendicular and parallel to the rock surface (Fig 7 c).

7. STRESS SITUATION AT SHOTCRETED ROCK

JOINTS STUDIED BY THEORY OF ELASTICITY

7.1 INTRODUCTION

The test described in /1/ were performed with the transferred load from shotcrete to rock perpendicular to the rock surface. The tests described in this report were performed with the resultant force at the joint acting nearly parallel to the rock surface.

In the first case it is obvious that the tensile adhesion strength is decisive for the failure. In the second case it could be the tensile or the shear adhesion strength or a combination of them which governs the failure.

In order to illustrate this subject some finite element calculations were made of the stress state near the joint between two elastic slabs lined with a layer of elastic material.

7.2 FE-MESH AND METHOD OF EVALUTION

The first FE-calculations were made with a manually constructed mesh (Fig 7a). Later on this mesh was replaced with another one which was automatically constructed by a set of programs for the construction of FE-meshes, which was developed at the Research Dept. of the Royal Swedish Fortifications Administration /2/ (Fig 7b).

The calculations were made by a commercially available program (STARDYNE). The assumption for the calculations were plane state of stress and homogenous, isotropic, elastic material. The default value 0.3 for Poisson's ratio was used.

The computer program gave element stresses, nodal displacements and nodal forces for the whole structure. In this treatment attention was paid only to the restraint forces at the supports and the nodal forces

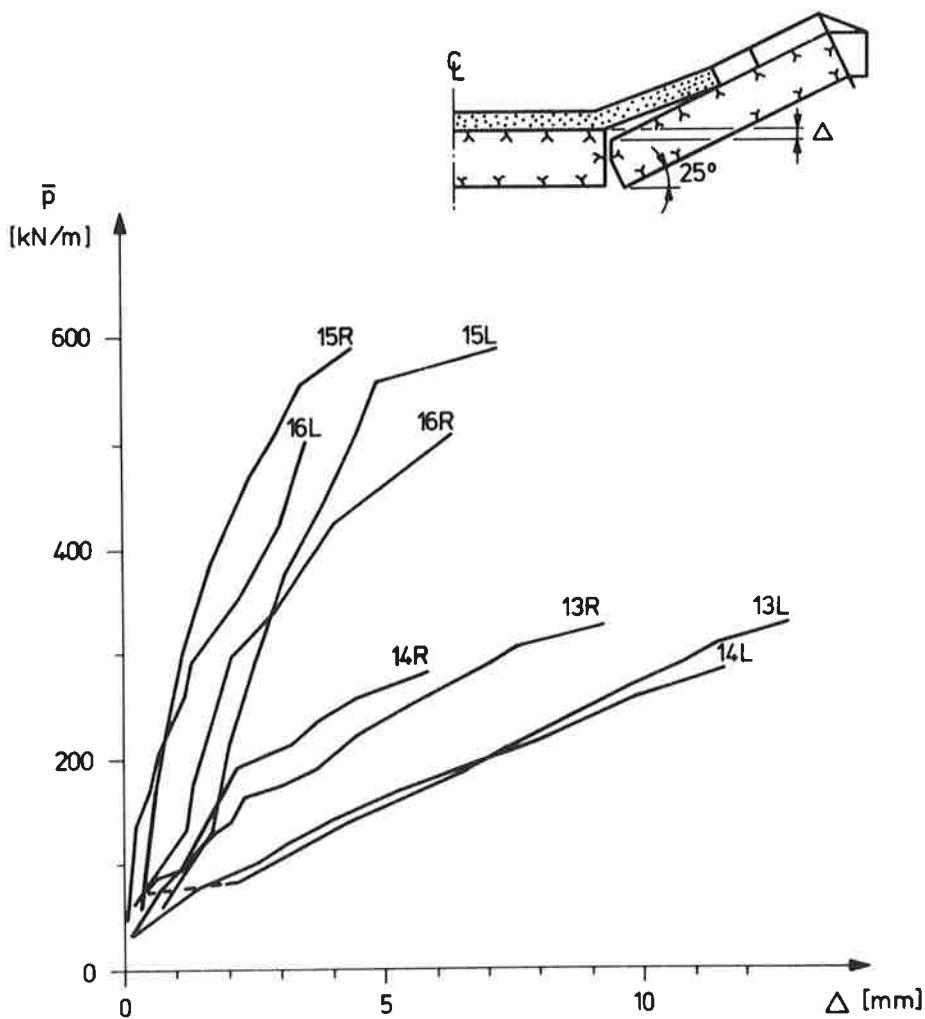
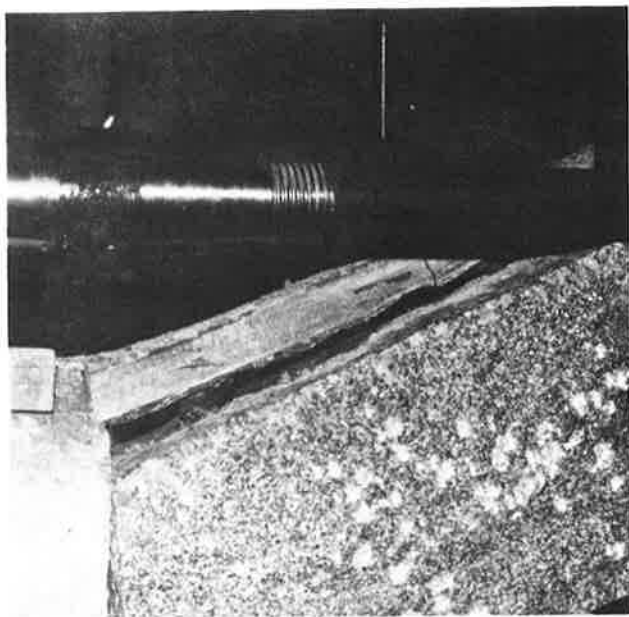


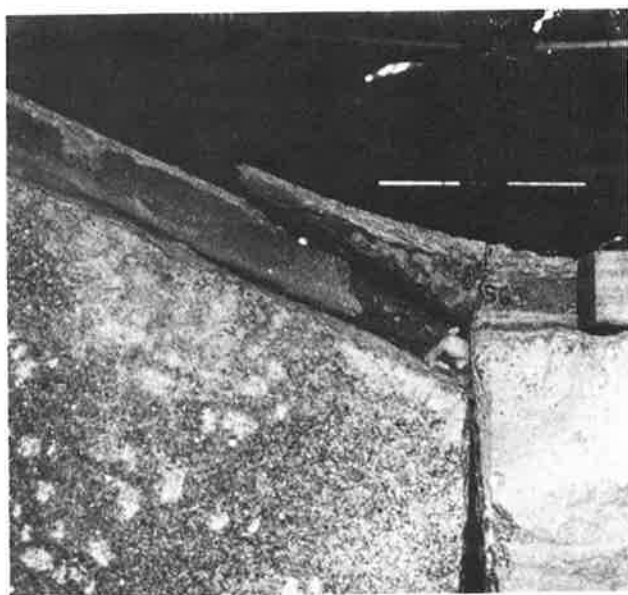
Fig 6d Vertical displacement, Δ , of the middle block as a function of the applied vertical line load, $\bar{p}$. Numbers at curves indicate test number. R stands for right side and L for left side.

The test specimens 11-16, which were not provided with adhesion preventive tape on the middle rock nearest to the joint, showed the same behaviour at adhesion failure as test specimens 1-10. The adhesion failure load level was also the same for these two groups.

The deformation measurements are shown in Fig 6d. The curves show that no special indication of an imminent failure is to be expected in cases similar to the ones tested here. The gap between the shotcrete and the rock was several millimeters at loads considerably lower than the failure load. These results suggest that shotcrete linings may stand free from the rock without any noticeable signs of being heavily loaded. The pressure of water or ice in such a gap may become fatal in the long run.



Failure of Test specimen no 6.



Failure of Test specimen no 11.

7.3 FORCE INCLINED TO THE ROCK SURFACE

7.31 Influence of different moduli of elasticity.

One loading case (Fig 7d) was treated with the values 5 and 1 of the ratio modulus of elasticity for rock to modulus of elasticity for concrete (E_r/E_c). The results are shown in Figs 7e-f. The maximum tensile adhesion stress is only slightly influenced by the variation of E_r/E_c , while the maximum shear adhesion stress is obviously influenced.

The shear stress level, however, is lower than the adhesion stress level in both cases. The shear strength of a normal, rough rock-shotcrete interface can be judged to be much higher than the tensile strength. Because of that it can be assumed that the tensile adhesion strength alone is decisive for the failure. The influence of the ratio E_r/E_c was judged to be of less importance than other parameters and thus this ratio was kept constant and equal to 1 in the continued part of the investigation.

7.32 Influence of different FE-meshes.

As the mesh happen to be altered during the investigation an opportunity was given to study the effect of this change and thus get an idea of the reliability of the FE-calculations in this respect.

In the old mesh the number of nodes and elements were 174 and 156 respectively. In the new one the corresponding numbers were 202 and 261.

A comparison of Figs 7f and 7i shows that the maximum tensile adhesion stress at the contact surface obtained with the old grid is 93% of the stress obtained with the new one. The corresponding percentage for the shear adhesion stresses is 64%.

A comparison of the principal stresses in elements equally situated in the two meshes near the joint show that the difference between the results of the two meshes is relatively small as far as the radii and angular position of the Mohr circles are concerned. Where the absolute value of one principal stress, however, is much smaller than the other one, large relative differences in those smaller principal stresses are of course obtained when the two grids are compared.

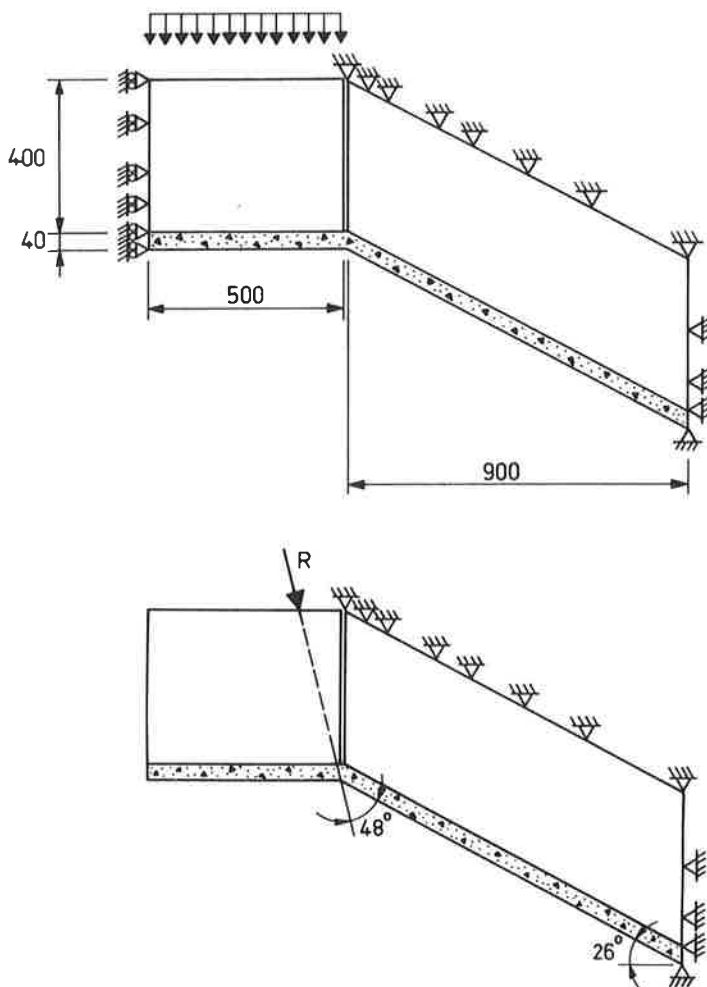


Fig 7 d Loading case A and resultant force in this case for $E_r/E_c = 5$.

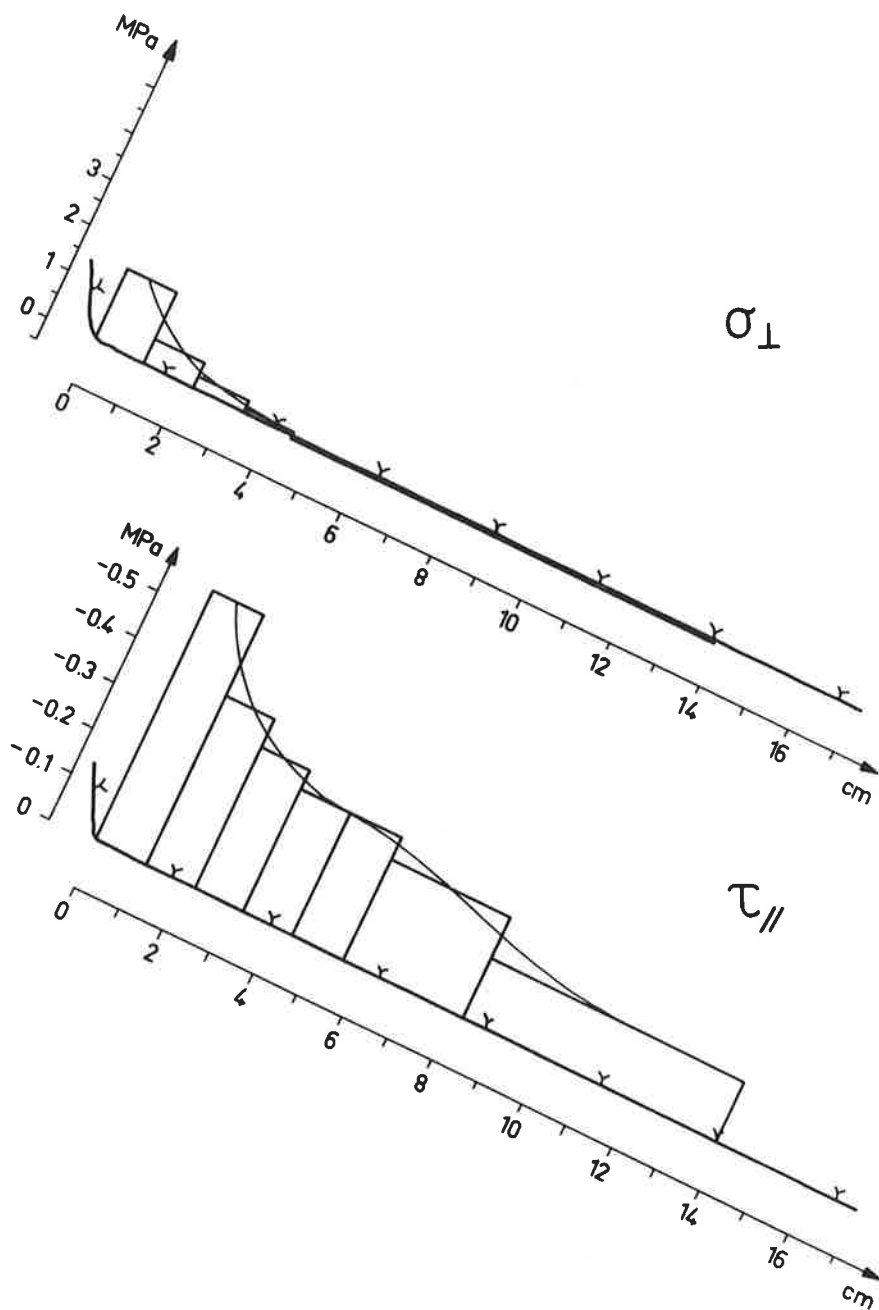


Fig 7e Stress distributions at the joint. Loading case A. $E_r/E_c = 5$.
 $\bar{p} = 50 \text{ kN/m}$, $\bar{h} = 13.7 \text{ kN/m}$, $\bar{r} = 51.9 \text{ kN/m}$.
 Notice different scales for σ and τ .

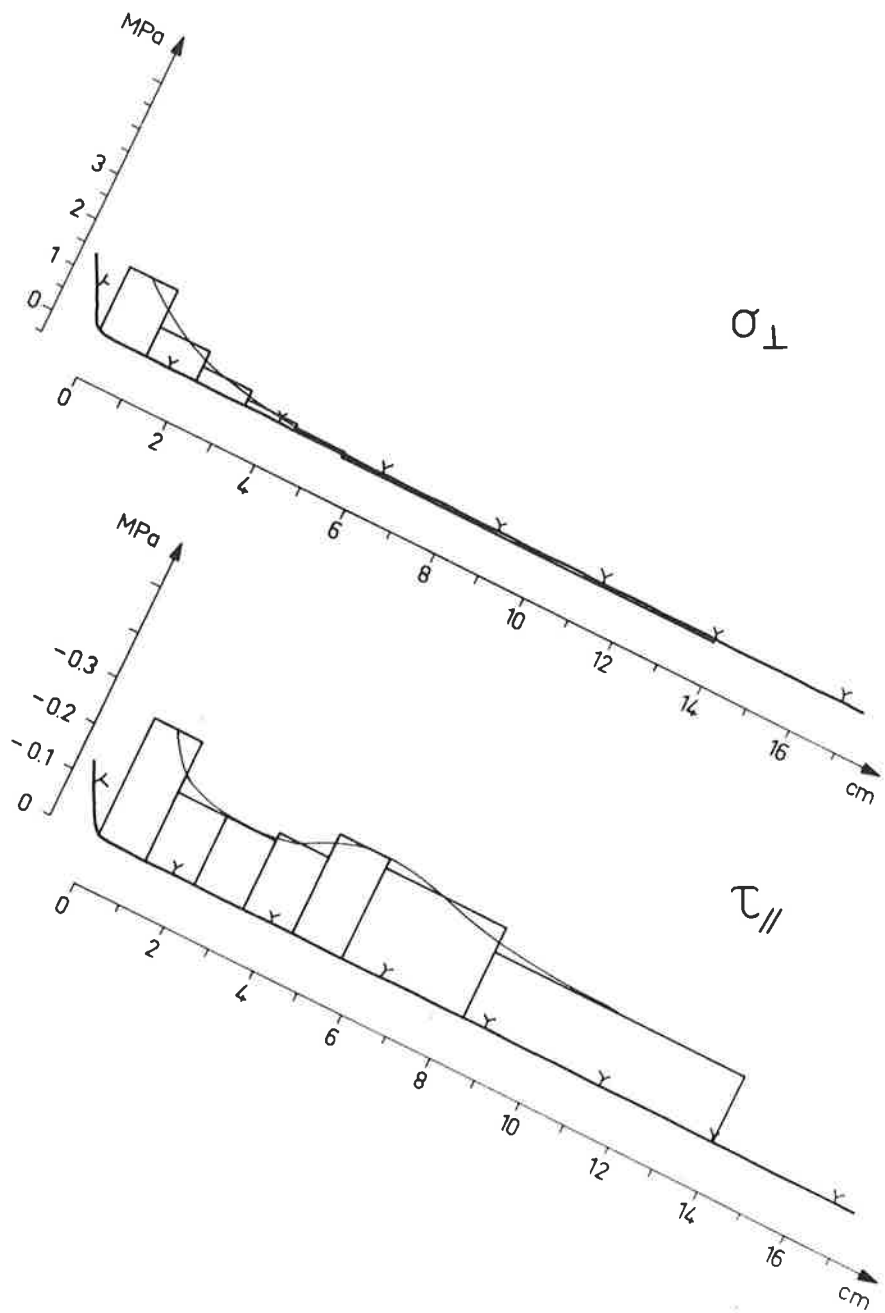


Fig 7f Stress distributions at the joint. Loading case A. $E_t/E_c = 1$.
 $p = 50 \text{ kN/m}$, $\bar{h} = 16.7 \text{ kN/m}$, $F = 52.7 \text{ kN/m}$.
 Notice different scales for σ and τ .

A comparison of this kind can of course only give an indication if the FE-solution is sensitive for the subdivision of the structure into elements. The results show that the choice of mesh does not seriously influence the results in areas where the stresses are of an interesting magnitude.

7.33 Influence of the force inclination.

When the middle slab of the FE-structure is loaded, the curvature and the prevented horizontal movements at the supports and at corner nearest to the shotcrete joint cause a horizontal force. This horizontal force together with the applied vertical load defines the loading case at the joint. By changing the load position and thus the moment curve some variation of the inclination of the resultant force was obtained. The angle β (see Fig 7 g) was 57.0° , 46.6° and 39.6° in the three cases studied. The variation of the maximum tensile adhesion stress because of the changed angle of inclination is rather small (Figs 7 h to j). The variation of the maximum shear stress is larger but the stress level is low in all three cases.

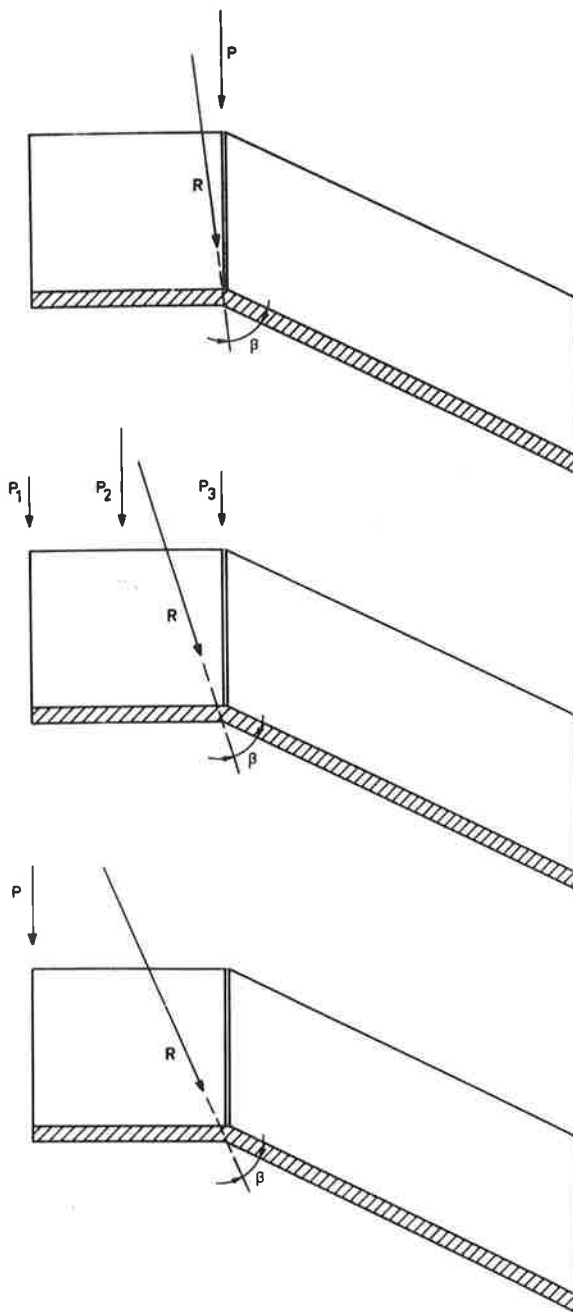


Fig 7g Loading cases B, C and D and the corresponding resultant forces for $E_t/E_c = 1$. $P = P_1 + P_2 + P_3$.

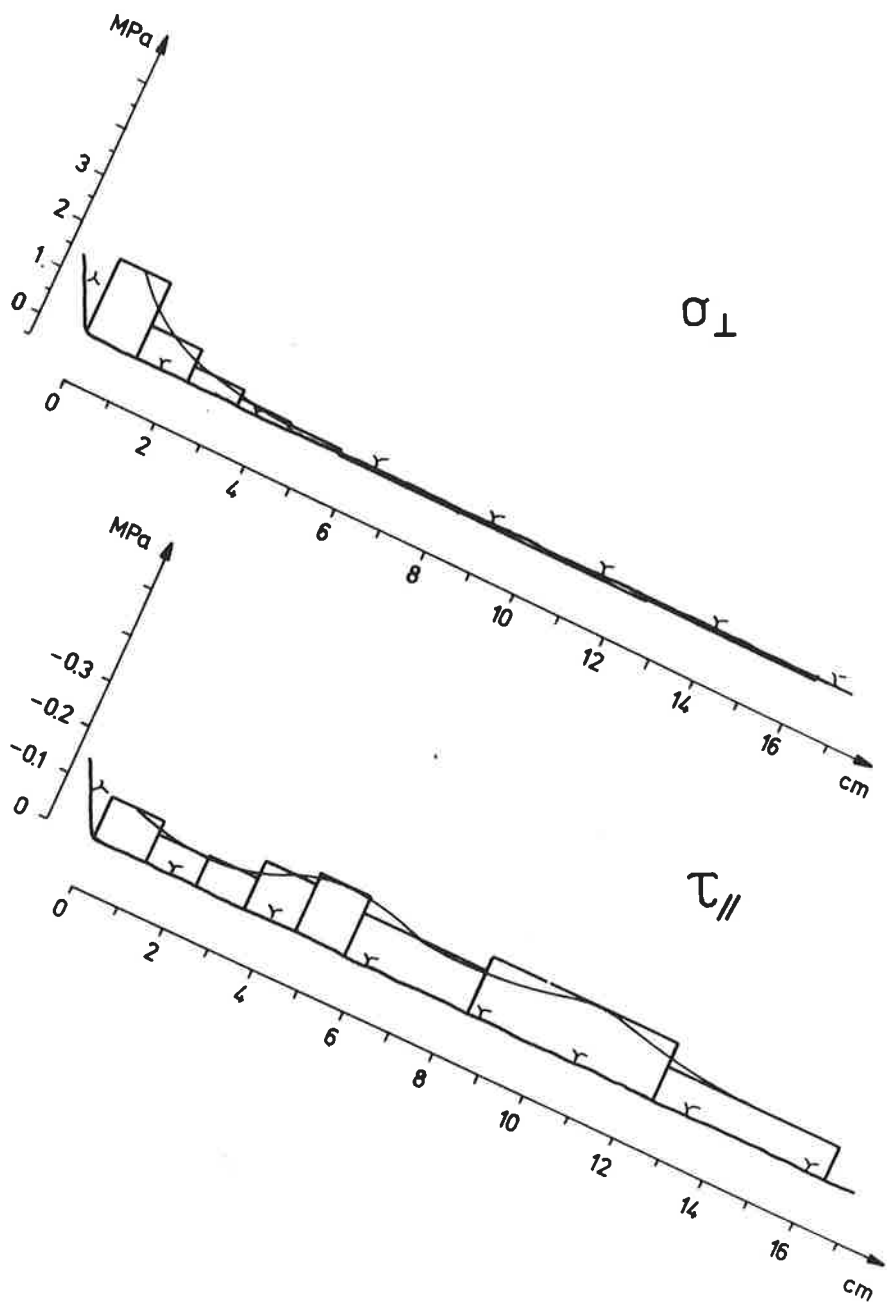


Fig 7h Stress distributions at the joint. Loading case B. $E_r/E_c = 1$.
 $\bar{p} = 50 \text{ kN/m}$, $\bar{h} = 5.5 \text{ kN/m}$, $\bar{r} = 50.3 \text{ kN/m}$.
 Notice different scales for σ and τ .

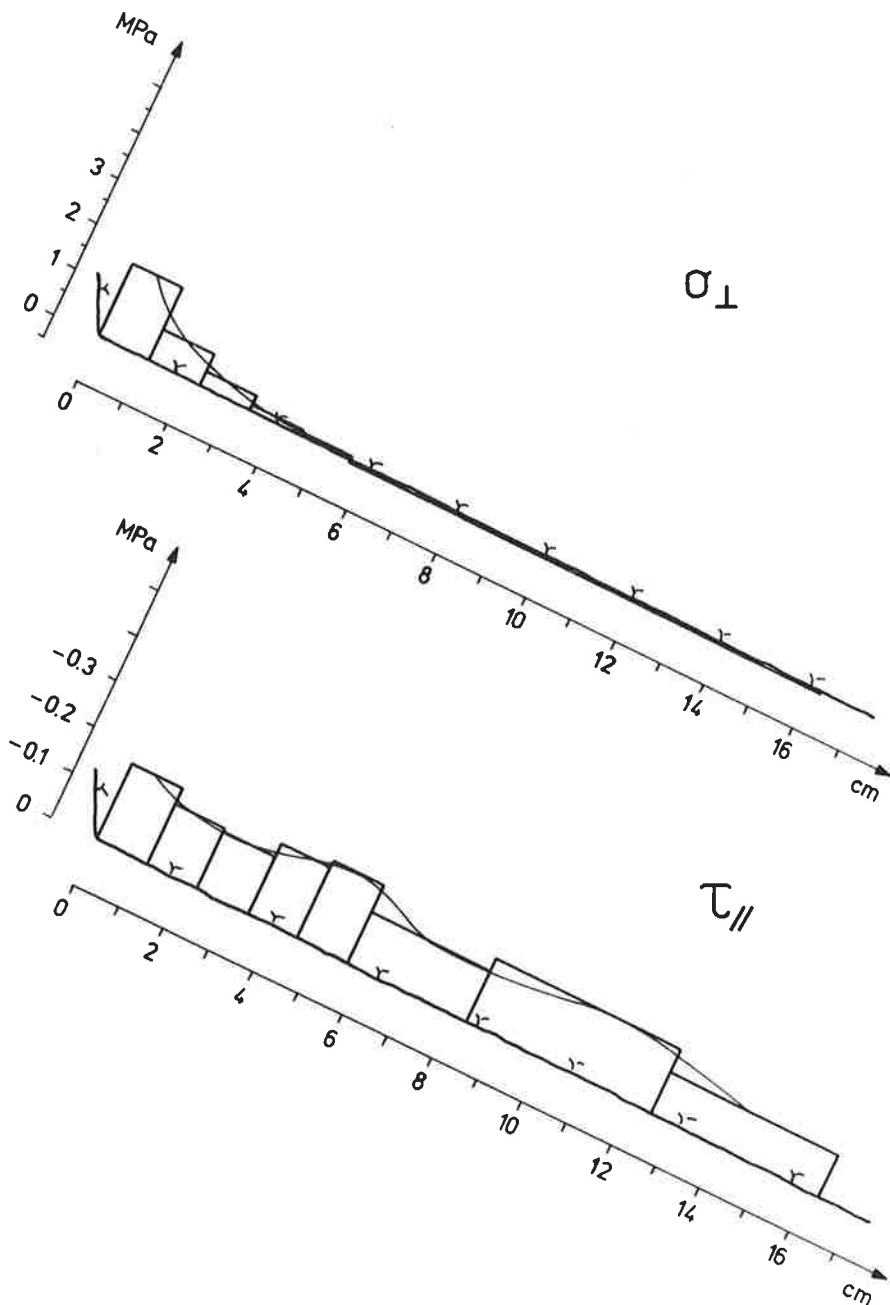


Fig 7i Stress distributions at the joint. Loading case C. $E_f/E_c = 1$.
 $\bar{p} = 50 \text{ kN/m}$, $\bar{h} = 15.9 \text{ kN/m}$, $\bar{r} = 52.5 \text{ kN/m}$.
 Notice different scales for σ and τ .

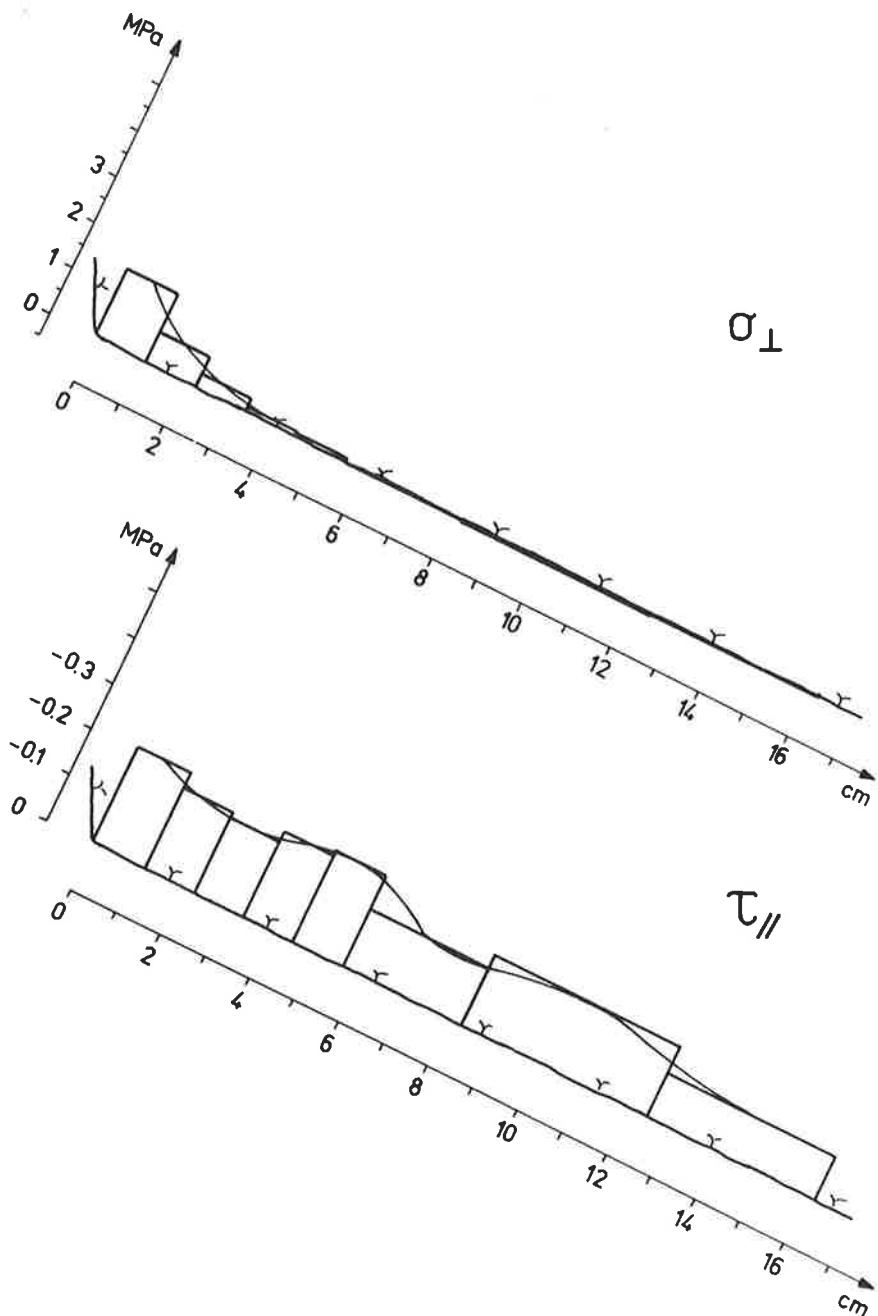


Fig 7j Stress distributions at the joint. Loading case D. $E_r/E_c = 1$.
 $\bar{p} = 50 \text{ kN/m}$, $\bar{h} = 22.9 \text{ kN/m}$, $\bar{r} = 55.0 \text{ kN/m}$.
 Notice different scales for σ and τ .

7.4 FORCE PARALLEL TO THE ROCK SURFACE

7.41 General remarks.

For the calculations in this case one boundary condition was changed compared with the earlier cases. To obtain a resultant force parallel to the rock surface it was not possible to keep the condition of the left vertical boundary being a section of symmetry. The restraints along this boundary were released and two of the nodes were loaded with horizontal forces which produced the desired resultant force together with the vertical load. By variation of the ratio between the two horizontal forces the eccentricity of the resultant force was varied relative to the cross section of the shotcrete layer.

The calculations were performed for the "old" mesh with equal moduli of elasticity for the two materials and with three values of the eccentricity: $+2.0$, ± 0.0 and -1.2 cm. Here the minus sign implies that the resultant force acts above the midpoint of the cross-section of the shotcrete layer, see Fig 7k.

These eccentricities are chosen smaller than the eccentricity at tensile cracking. At tensile cracking of a 4 cm section the eccentricity of an axial force equal to the one used in the FE-calculations is 2.4 cm when the flexural tensile strength is 8 MPa.

7.42 Influence of the eccentricity.

The results of the FE-calculations are shown in Figs 7l to n. The stresses are only slightly affected by the variation of the eccentricity.

From the stress magnitudes and distributions in this cases it can be concluded that the tensile adhesion strength alone is decisive for the failure. There are two reasons for such a statement:

a) The tensile adhesion stress obtained by this analysis is larger than the maximum shear stress.

b) The tensile adhesion strength is probably always lower than the shear strength because of the roughness of the contact surface.

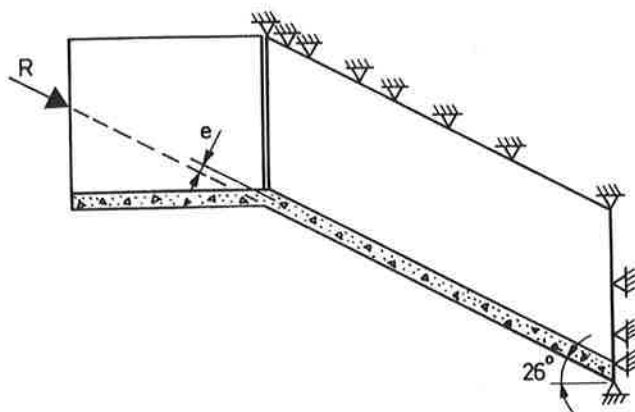


Fig 7k Loading case E. $\bar{p} = 50 \text{ kN/m}$, $\bar{h} = 107 \text{ kN/m}$, $\bar{r} = 118 \text{ kN/m}$.

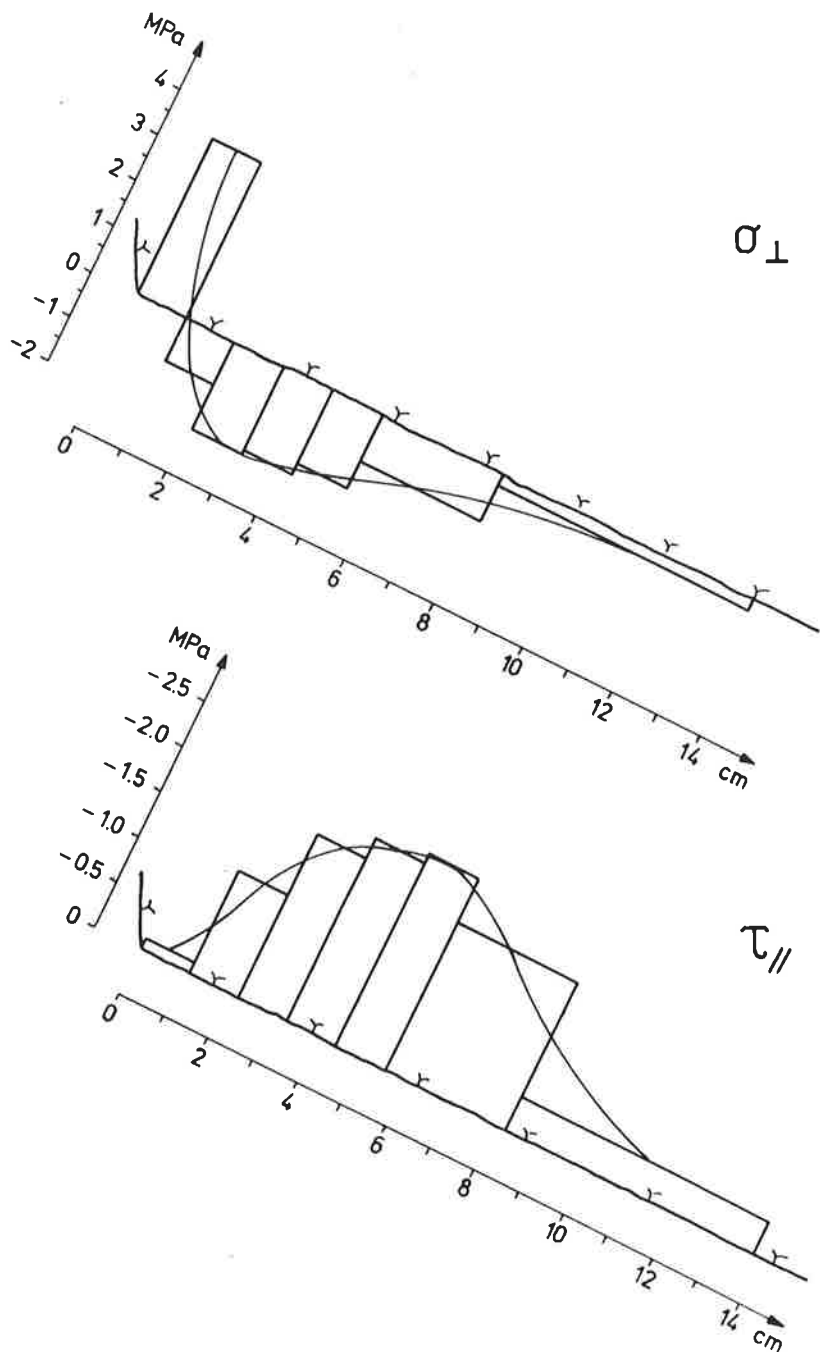


Fig 71 Stress distributions at the joint. Loading case E. $E_f/E_c = 1$.
 $e = +2.0$ cm.
 Notice different scales for σ and τ .

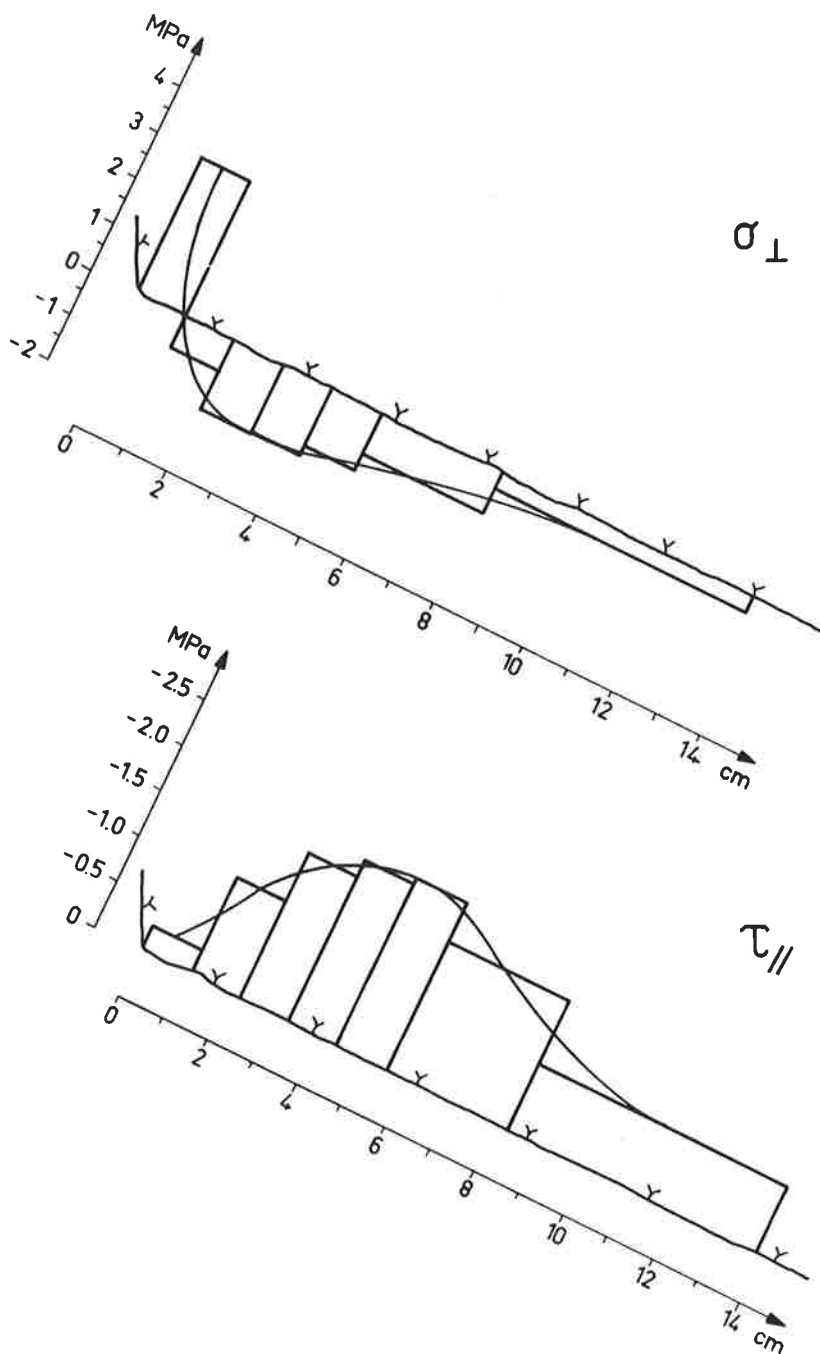


Fig 7m Stress distributions at the joint. Loading case E. $E_r/E_c = 1$,
 $e = \pm 0$.

Notice different scales for σ and τ .

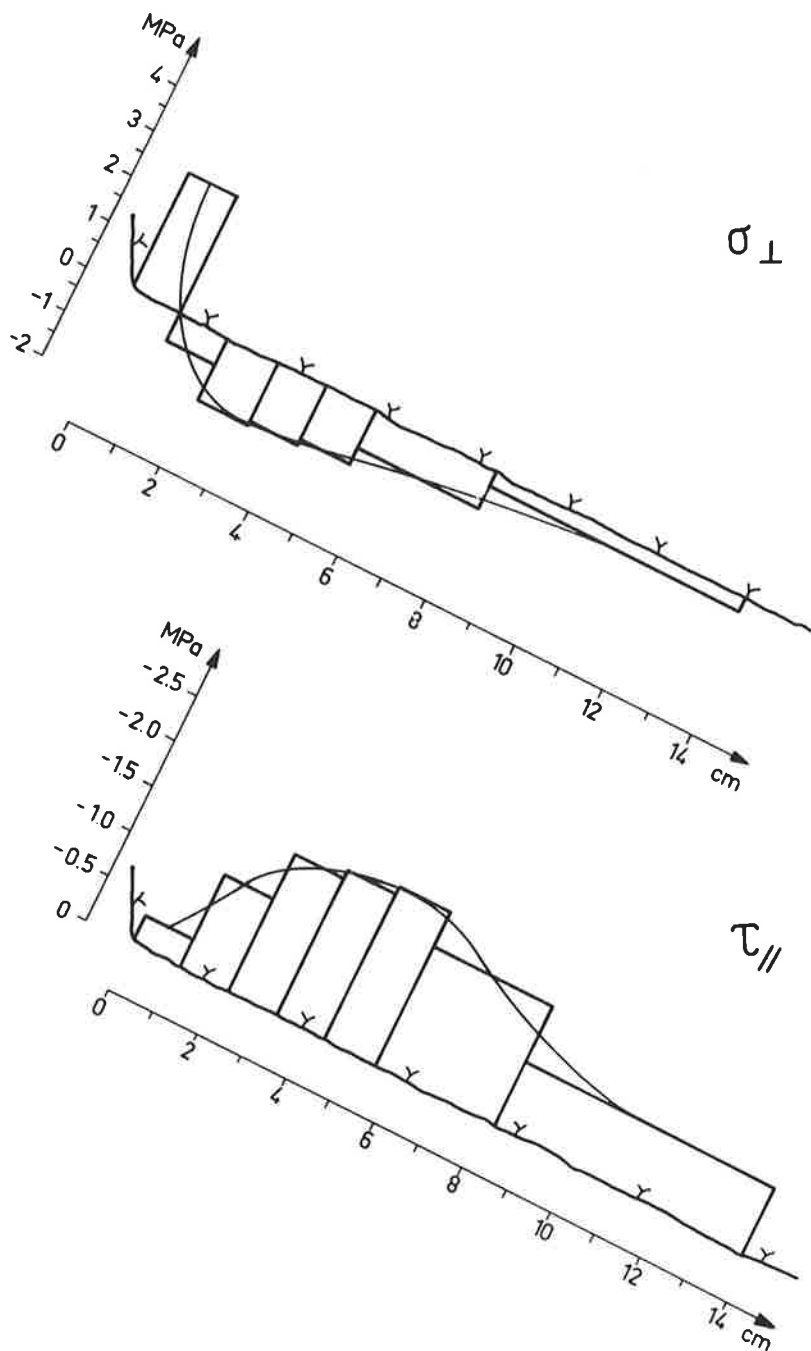


Fig 7n Stress distributions at the joint. Loading case E. $E_r/E_c = 1$.
 $e = -1.2$ cm.
 Notice different scales for σ and τ .

7.5 SOME CONCLUDING REMARKS ON THE FE-CALCULATIONS

It must be clearly stated that the tensile adhesion peak stress is highly influenced by the local geometry at the joint. The exact solution for a sharp corner in an elastic medium is an infinite tensile adhesion stress. The FE-calculations, however, give average stresses which are considered valid for each element as a whole. These elements are not small compared to the area where the stress peak is situated. Of course this means that the absolute values of the maximum adhesion stress cannot be used for a comparison with values of the adhesion strength obtained from core tests. The FE-results represent some unknown stress concentration factor and the core tests an other unknown one. At the large scale tests the local geometry probably was favourable for the stress distribution. The joint between the rock blocks was filled with fabric before shotcreting but this could not prevent the shotcrete from penetrating some millimeters into the joint and thus forming a smooth transition from shotcrete to rock.

It is uncertain if the different FE-results can be compared with each other when regarding the "average peak stress" because of the big difference in stress gradient. The stress gradient seems to be much larger at a force parallel to the rock surface than at a force inclined to the surface.

What can be concluded from the FE-calculations, however, is that tensile adhesion stresses probably always governs the failure at shotcreted rock joints subjected to punch loads.

The shear stress obtained by this analysis is of a smaller magnitude than the tensile stress. It is also distributed over a larger length than the tensile adhesion stress.

8. SUMMARY

16 curved shotcrete layers interacting with rock blocks were tested in a large scale.

The tests showed that the primary failure of the shotcrete-rock structure always is an adhesion failure. The primary failure load is in practice independent of the layer thickness. For the tested cases the primary failure is also the final failure unless end supports for the shotcrete arch are provided.

A shotcrete arch with end supports subjected to large compressive forces may stand free from the rock over large areas.

Finite - element calculations of the stress state at the rock-shotcrete interface near a joint indicate that the governing parameter determining the failure is the tensile adhesion strength rather than the interface shear strength. This statement seems to be valid for all loading cases when shotcrete interacts with hard, jointed rock masses.

REFERENCES

- /1/ Holmgren J. "Plane shotcrete layers subjected to punch loads". Royal Swdeish Fortifications Adm., Research Dept., Report 121:4. Stockholm 1975.
- /2/ Keijer U. "Hjälpmedel för generering av indata för finita elementprogram" (Input data generators for finite element programs). FoU bygg 1 (1978) : 1, pp 4-7. Stockholm 1978.