KBS TEKNISK RAPPORT

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Experimental determination of the stress/strain situation in a sheared tunnel model with canister

Roland Pusch

Högskolan i Luleå 1978-03-02

KÄRN-BRÄNSLE-SÄKERHET

POSTADRESS: Kärnbränslesäkerhet, Fack. 102 40 Stockholm. Telefon 08-67 95 40

EXPERIMENTAL DETERMINATION OF THE STRESS/STRAIN SITUATION IN A SHEARED TUNNEL MODEL WITH CANISTER

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TEKNISK RAPPORT KBS 16

REPORT ON

EXPERIMENTAL DETERMINATION OF THE STRESS/STRAIN SITUATION IN A SHEARED TUNNEL MODEL WITH CANISTER

Luleå 1978-03-02
Div. Soil Mechanics, University of Luleå
R PUSCH

HÖGSKOLAN I LULEÅ
MASKINTEKNIK · GEOTEKNOLOGI · ARBETSVETENSKAP

EXPERIMENTAL DETERMINATION OF THE STRESS/STRAIN SITUATION IN A SHEARED TUNNEL MODEL WITH CANISTER

INTRODUCTION

A previous KBS report (PUSCH, 1977) concerned a technical matter which could be of great importance as regards the mechanical strength of canisters embedded in a bentonite/quartz buffer mass, i.e. the effect of a differential movement triggered by a critical deviatoric stress condition (Fig. 1).

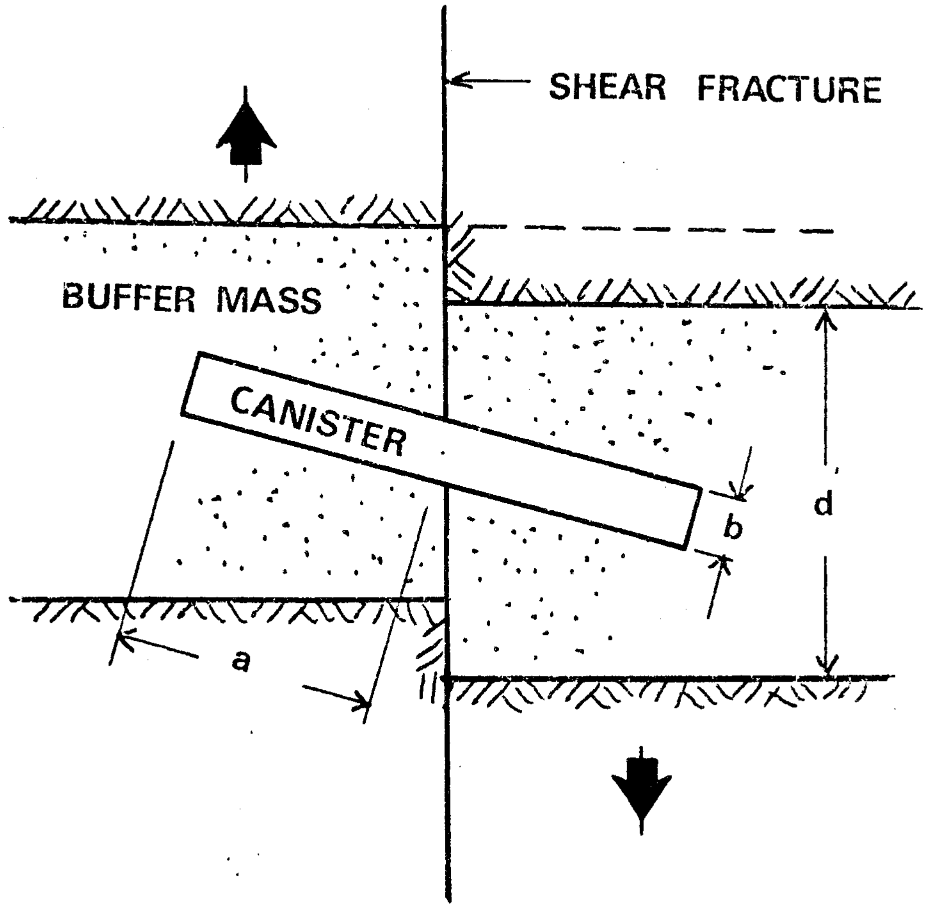

Fig. 1. The considered shear case

The report described the character and range of possible future displacements in Swedish bed rock. The land rise and descent (Skåne) of Sweden were shown to be the only major movements, both being associated with a low frequency of very moderate earth shocks. There are two reasons for the movement: 1) a visco-elastic recovery of the earth crust from the depressed state caused by the glacier loads, and 2) large scale tectonic processes.

The exact location and type of presently occurring and possible future movements is not known and it was therefore suggested in the report to make a theoretical approach.It was considered to be logical firstly to define the present stress situation and then to determine the modes of deformation that might be caused in future as a result of possible stress changes.

It was whown, considering the HAST and K_o cases, that a condition of general shear failure can be achieved only by increasing the horizontal stress. The application of MOHR/COULOMB's criterion and the theory of plasticity indicated that, for high quality rock, the existing horizontal pressure is only about 10-15% of the maximum horizontal pressure at failure at 1000 m depth. This means that very large stress changes - associated with tectonic processes which are improbable for our part of the Russian Platform - are required to yield a failure condition. Even for very bad, fissured rock there is in fact a considerab safety factor at larger depths. However, if we take the weakest existing parts of a rock mass into consideration (clayey or chloritic zones) we find that a failure condition exists already today where such zones are continuous and oriented in a critical manner.

It was concluded that if sudden and large strain will ever occur at the depth where the deposition plants are going to be located (about 500 m) it will take place along already existing continuous weak zones. If no canisters are placed in or close to weak rock zones crossed by the tunnels the situation illustrated by Fig. 1 will therefore never occur.

Yet, "the incredible case" of an unexpected shear failure through intact, high quality rock was considered in the report. The major problem was to find out the deformation pattern and for this purpose a pilot test was made by using a simple shear apparatus consisting of two plexi-glass tubes with an internal diameter of 50 mm. The tubes were filled with an air-dry mass of 10% (by weight) sodium bentonite and 90% quartz with a bulk density of about 1.4 t/m^3 ($w \sim 5\%$). Lead shots were applied in the mass for X-ray determination of the deformation patter the canister being represented by a 0.8 cm steel axis with a length of 7 cm. The device was submerged in water until the water uptake was completed and step-wise shearing was then made with 4% of the tunnel diameter d up to a total displacement of 0.5 d.

The deformation pattern indicated that the contact pressure is distributed as shown in Fig. 2 (hatched areas). Approximately, a uniform pressure according to the broken lines can be assumed. It was observed that a shear strain of more than about 10% produced local empty spaces close to certain parts of the "canister". When the displacement approached 0.5 d it was associated with a system of open shear zones with a tendency to form a direct connection between the canister and the rock. This represents a critical state.

It was stated in the report that the distribution and magnitude of the contact pressure should be fairly independent of the displacement, which means

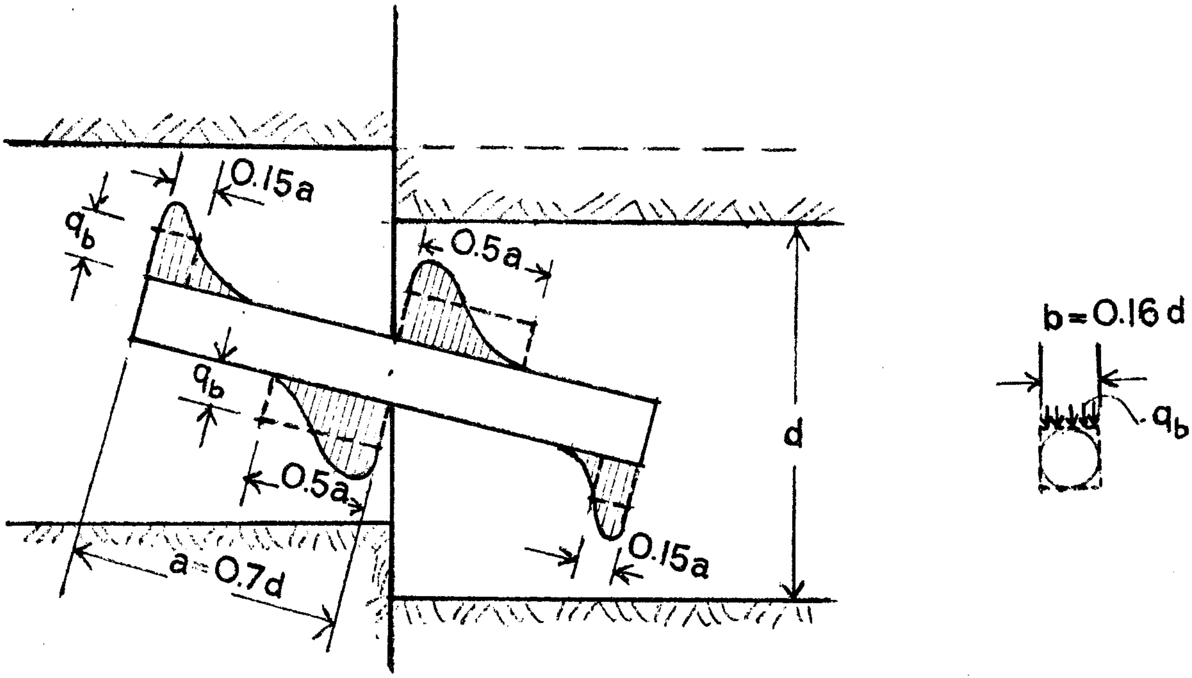

Fig. 2. Contact stress distribution (PUSCH, 1977)

that also moderate movement (a few % of d) would produce a contact pressure of the order of the bearing capacity (ultimate contact pressure) q_b. Since it was considered to be of interest to estimate the mechanical stresses in a canister an attempt was made to determine this bearing capacity by applying theoretical soil mechanics. MEYERHOF's (Fig. 3) and BEREZANTZEV's theories for deeply buried foundations were tried for this purpose.

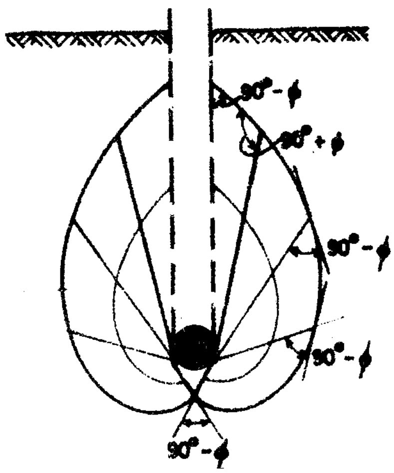

Fig. 3. Failure pattern according to MEYERHOF

This latter theory is in fact less reasonable because it implies that only local failure close to the canister is considered. According to the first-mentioned theory $q_b = \lambda g \rho' \frac{b}{2} N_{\rho q}$ where ρ' = effective bulk density, ϕ' angle of internal friction, b width of loaded area (in m) and $N_{\rho q}$ a parameter. Reasonable values yield:

$$q_b = 3 \cdot 10^4 b \text{ kPa} \qquad (1)$$

The calculation of the maximum bending moment (about $0.075 \cdot q_b \cdot a^2 b$ at a distance of about 0.3 a from the canister's center) and the maximum shear force (about $0.35\ aq_b \cdot b$ at the canister's center) is readily made by applying Fig. 2.

EXPERIMENTAL DETERMINATION OF MECHANICAL STRESSES IN A CANISTER MODEL[1)]

Even if "the incredible case" is extremely unlikely to occur it was considered to be of importance to verify the theoretical expressions for the maximum bending moment and maximum shear force. A special reason was to test the hypothesis that the contact pressure would soon reach a high value and then stay fairly constant when the displacement increased.

Equipment

A brass shear box was used for the test. The inner diameter was 70 mm and its height 125 mm. The lower part of the box was equipped with a porous stone for the water uptake (Fig. 4).

The "canister" (length 96 mm and diameter 16 mm) was delivered by ASEA Atom. It consisted of steel 1550-01 with strain gauges glued in the positions shown in Fig. 5. This arrangement made possible a determination of the strain and stress in 4 points in a certain plane.

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1) The experimental work and the interpretation of stresses and moments were made by Lars G Eriksson, Div. Soil Mechanics, University of Luleå.

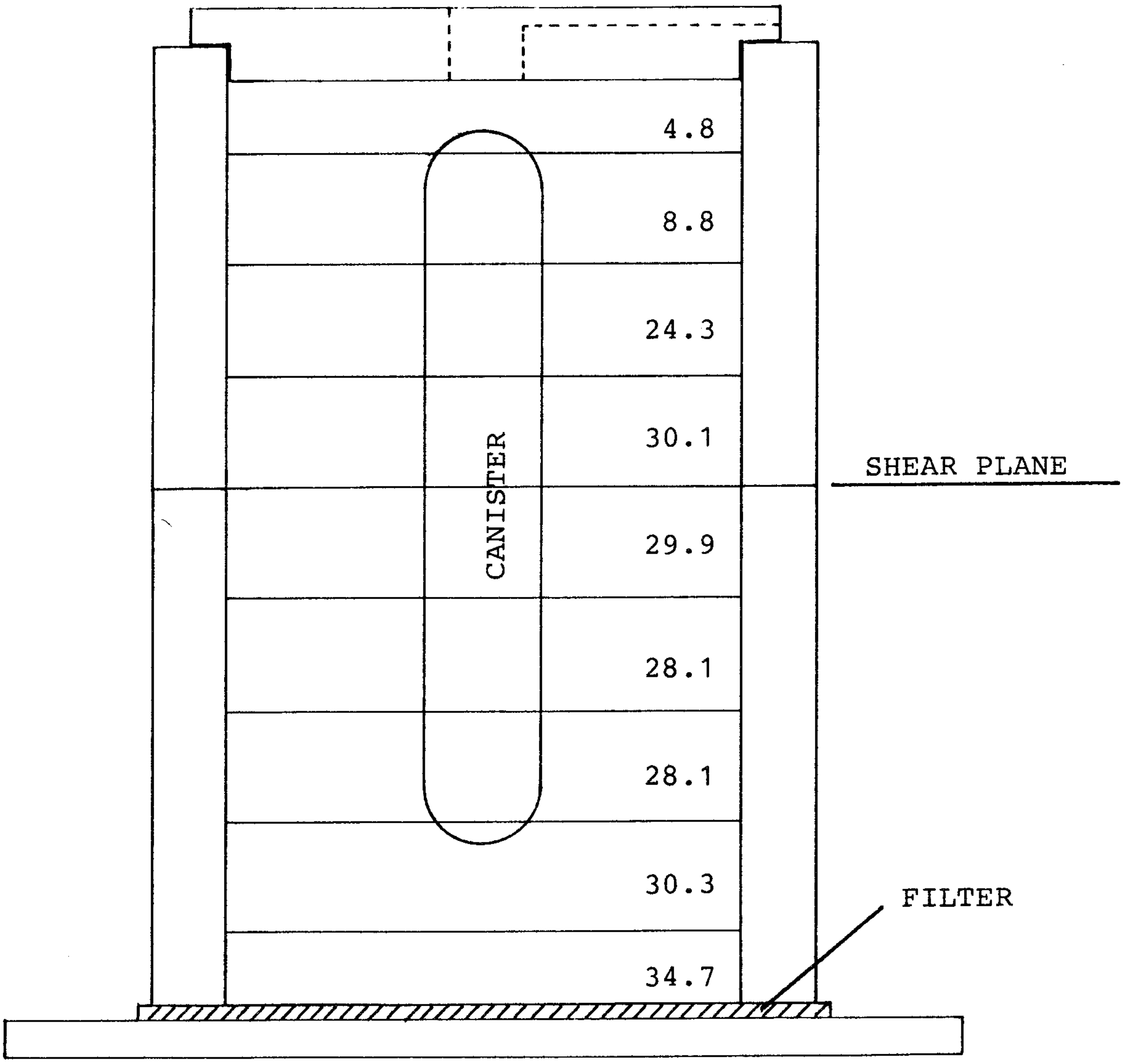
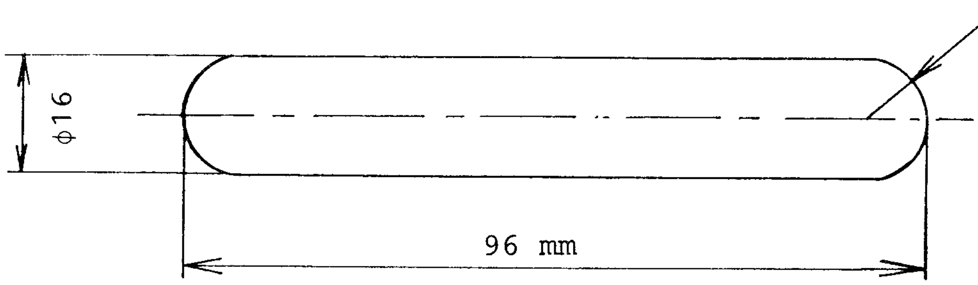


Fig. 4. Schematical picture of the shear box with canister. The figures refer to the water content of the buffer mass.

Canister model III



Steel 1550-01

Position of strain gauges

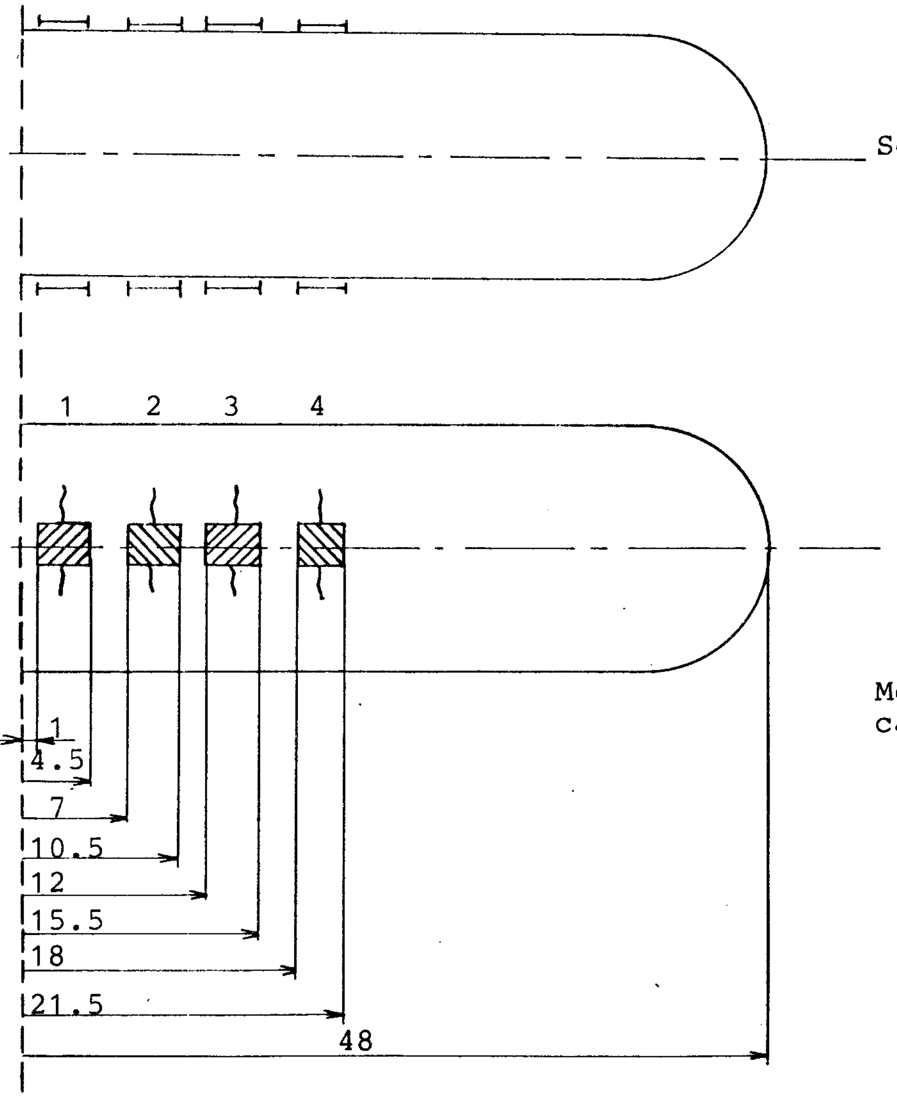


Fig. 5. Position of strain gauges

The shear box was filled with an air-dry "buffer substance" consisting of 10% (by weight) bentonite and 90% Pite silt which was compacted to a bulk density of about 1.4 t/m^3.

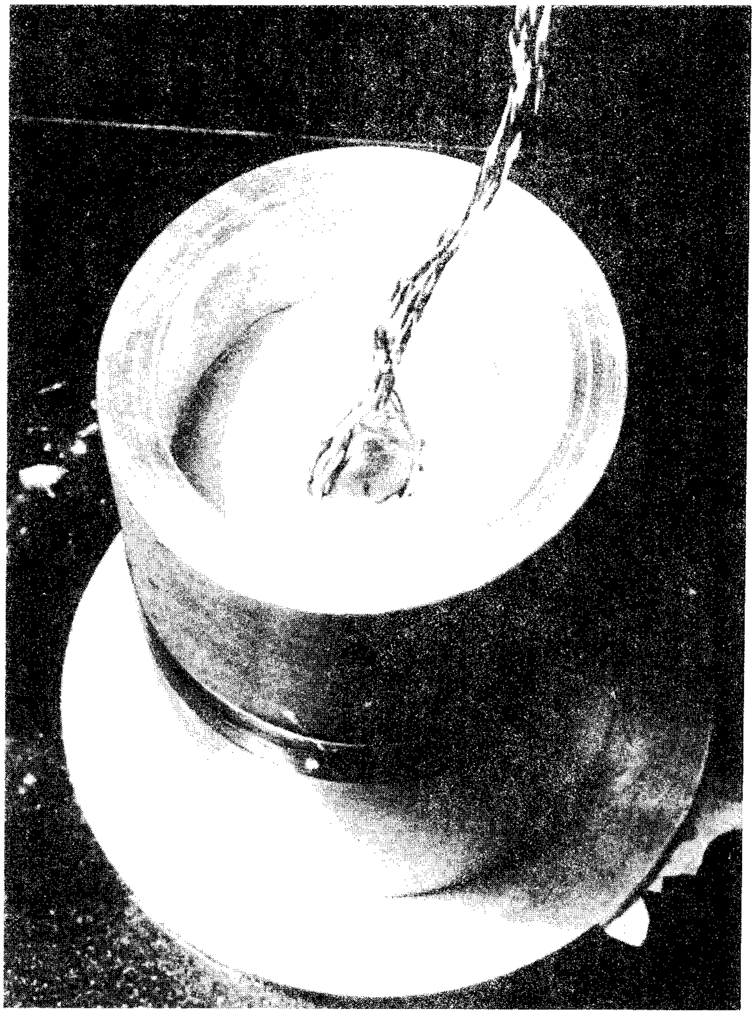
The box with the buffer substance was placed in a container with artificial ground water (cf. JACOBSSON & PUSCH, 1977) for 2 1/2 weeks. Water was taken up to about 50% water saturation. In the upper part of the fill the water content was much lower than in the rest of the box due to the long distance from the filter stone (Fig. 4) but this inhomogeneity cannot have affected the results noticeably. The important thing is that the water content and state of the majority of the mass were similar to those of the fill used in the previous test where the deformation pattern was investigated.

During the water uptake a vertical pressure of 30 kPa was applied to prevent swelling.

The shearing was made by means of a Geonor device (Fig. 7). Ten step-wise displacements corresponding to 0.5 % of the shear box diameter were made and they were followed by ten displacements each corresponding to 1% of the box diameter. A pressure was applied to eliminate vertical movement of the upper box half. This pressure had to be successively increased in course of the displacement.

Each shear displacement was kept constant for 1-2 minutes. During this time the recorded stress was constant. Stress relaxation is probably very small for buffer masses poor in clay.

The canister strain was measured by means of a PEEKEL B 105 (1.25 V) bridge. A pilot test with unloaded canister showed that the accuracy of the measuring device was ±0.1-0.2 μ-S.



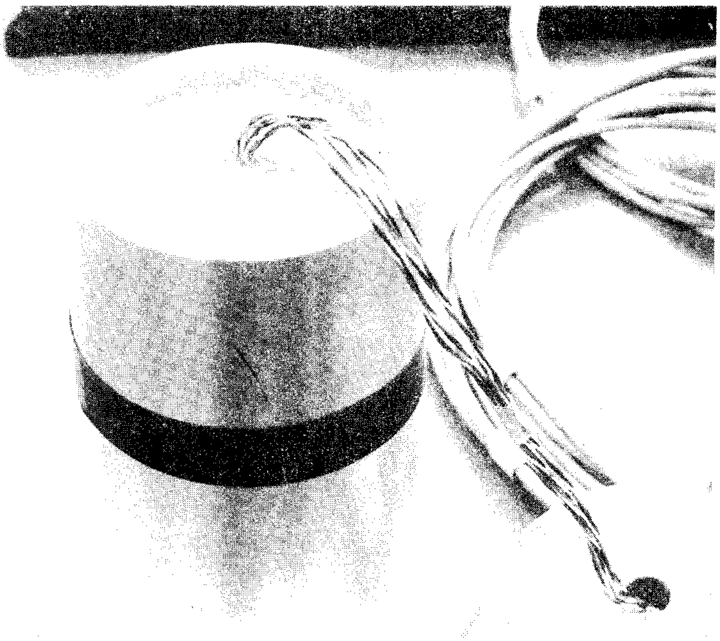
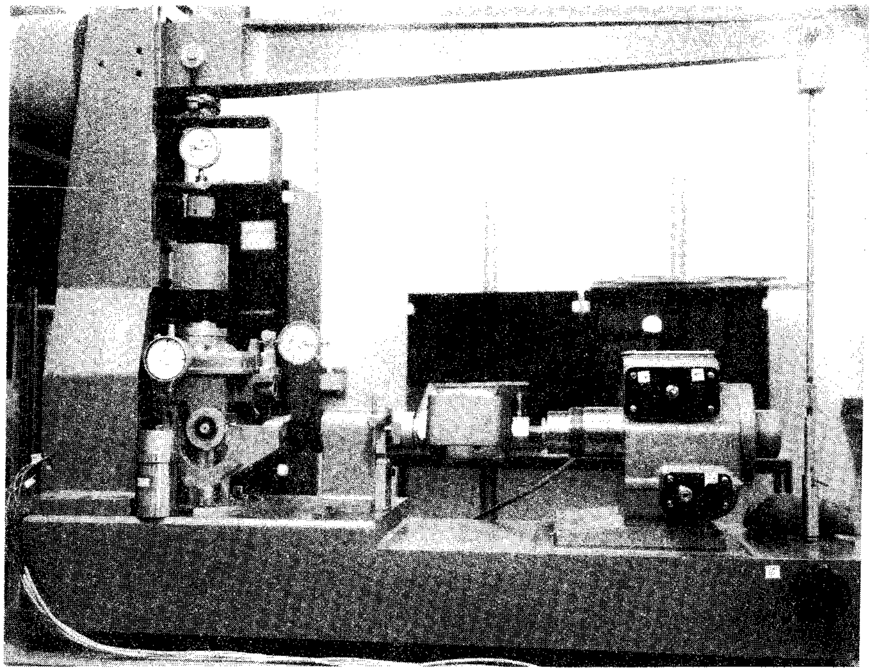


Fig. 6. Application of buffer mass in the shear box. The upper end of the "canister" with cables from strain gauges is seen in the upper picture.



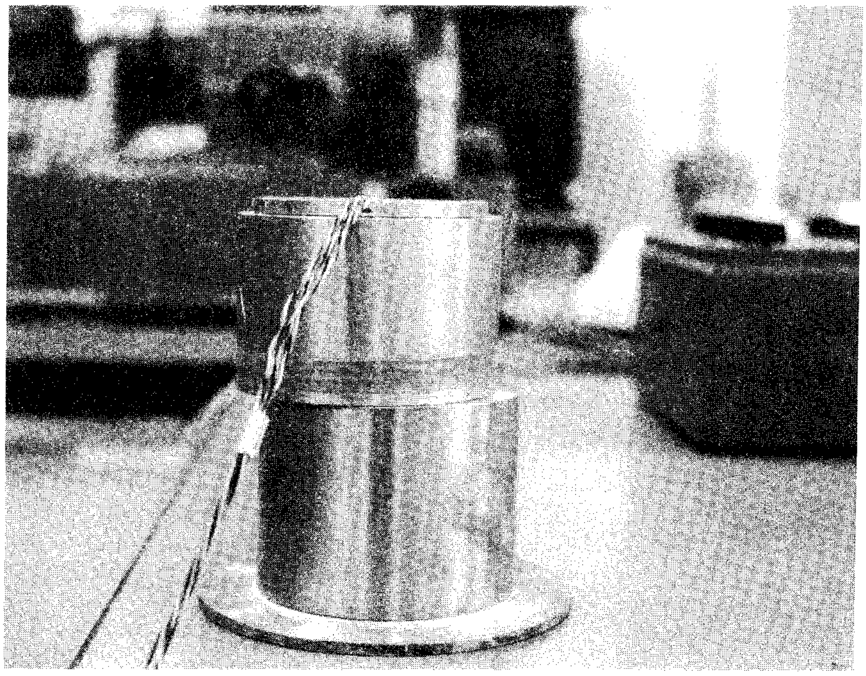


Fig. 7. Upper picture: View of shear apparatus.
Lower picture: Shear box at 15% displacement.

Test results

Assuming the E-modulus of the canister steel to be $210 \cdot 10^6$ kPa the stress values in Table 1 were obtained.

Table 1. Stress values at various shear displacement

% Displ.	Gauge 1		Gauge 2		Gauge 3		Gauge 4	
	μ-S	σ kPa	μ-S	σ kPa	μ-S	σ kPa	μ-S	σ kPa
0	0	0	0	0	0	0	0	0
0.5	0.80	168	0.55	116	1.00	210	0.60	126
1.0	1.15	242	1.55	326	2.35	494	1.60	336
1.5	2.65	556	2.70	567	4.20	882	3.15	662
2.0	4.95	1040	4.95	1040	6.30	1323	4.65	976
2.5	6.60	1386	6.50	1365	7.80	1638	5.55	1166
3.0	7.15	1502	6.90	1449	8.55	1796	5.95	1250
3.5	7.50	1575	7.15	1502	8.85	1858	6.25	1312
4.0	8.00	1680	7.75	1628	9.50	1995	6.55	1376
4.5	8.30	1743	8.25	1732	9.90	2079	6.85	1438
5.0	8.30	1743	8.50	1785	10.50	2205	7.10	1491
6.0	8.90	1869	9.30	1953	11.30	2373	7.40	1554
7.0	9.90	2079	10.05	2110	12.15	2552	8.40	1764
8.0	10.00	2100	10.85	2278	12.60	2646	8.75	1838
9.0	11.15	2342	12.00	2520	13.60	2856	9.40	1974
10.0	11.40	2394	12.40	2604	14.20	2982	10.00	2100
11.0	11.90	2499	13.35	2804	14.75	3098	10.25	2152
12.0	12.35	2594	14.30	3003	15.20	3192	10.50	2205
13.0	12.60	2646	14.95	3140	15.90	3339	10.95	2300
14.0	13.25	2782	15.60	3276	16.65	3496	11.30	2373
15.0	13.65	2866	16.15	3392	17.05	3580	11.60	2436

Figs. 8-11 show the recorded stress values of the four gauges for various shear displacement. It is interesting to see that a failure is reached when the displacement is only 1.4-1.8% of the box diameter. This failure corresponds to the bearing capacity (ultimate contact pressure) q_b. We see that the stress increase is 700-1100 kPa/% displacement before failure and only 150-175 kPa/% displacement after failure. This confirms the previous statement that q_b reaches a certain high value at a small displacement and that it is only very moderately increased at further displacement.

Table 2 gives the tensile stress when the bearing capacity q_b is reached.

Table 2. Tensile stresses and bending moments in canister

Gauge	σ, kPa	M_i kNm	q_b kPa
1	1 430	$5.7 \cdot 10^{-4}$	790
2	1 310	$5.3 \cdot 10^{-4}$	530
3	1 660	$6.7 \cdot 10^{-4}$	660
4	1 170	$4.7 \cdot 10^{-4}$	460

Since we now know the tensile stress and the section modulus is also known ($W = 402 \cdot 10^{-9}$ m³) the bending moment M_i for each gauge position is easily obtained. These values, which are also given by Table 2, make possible a determination of q_b if the contact pressure distribution in Fig. 2 is applied.[1] The last column of Table 2 gives the q_b-values for the four gauge positions. We can conclude that the relatively constant pressure for all the positions support the assumption that the contact pressure distribution can be approximated as in Fig. 2.

[1] It should be noticed that the diameter/length ratio is somewhat different from that of the previous test but the deformation pattern and pressure distribution are about the same.

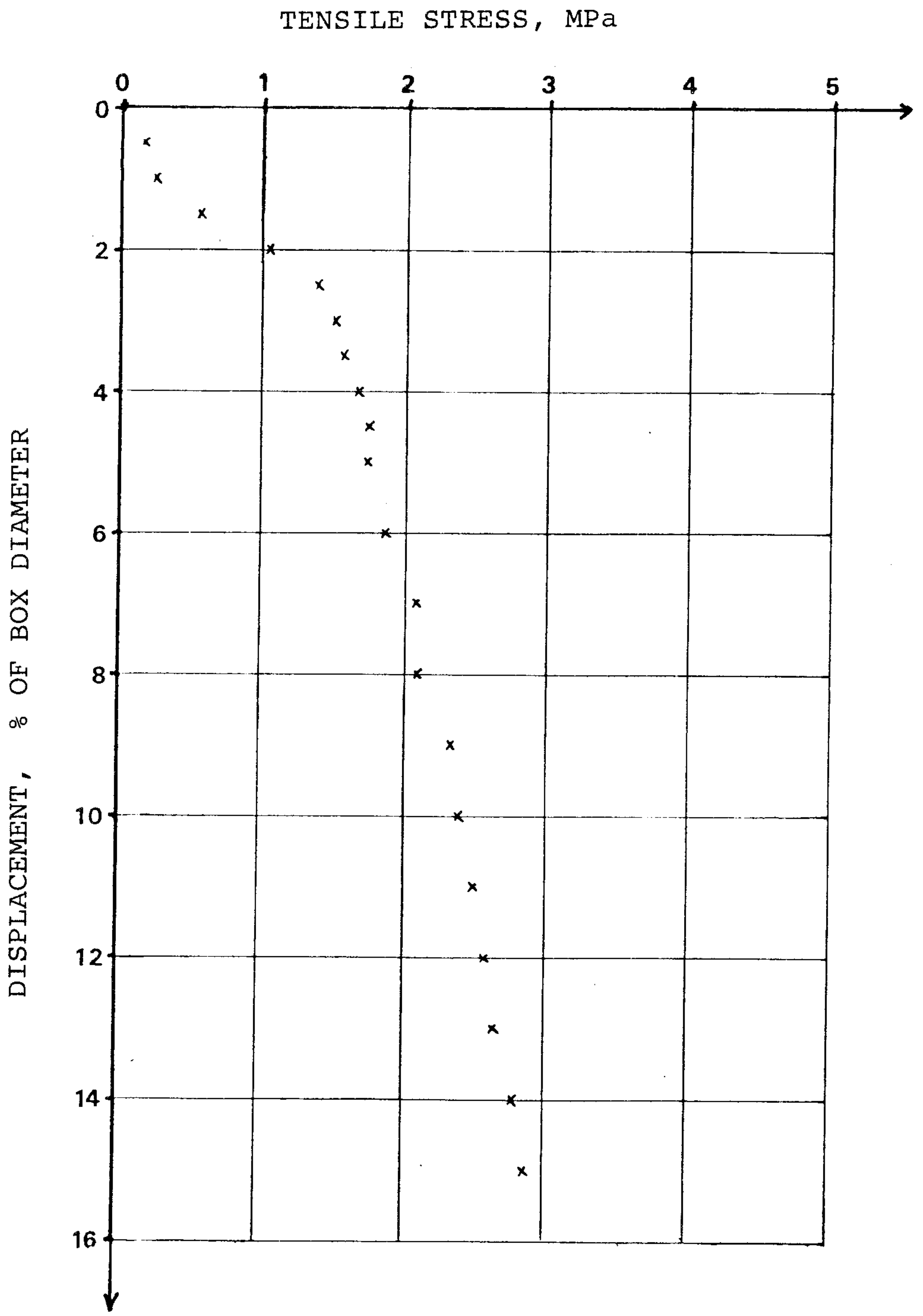


Fig. 8. Stress/displacement relationship
for Gauge 1

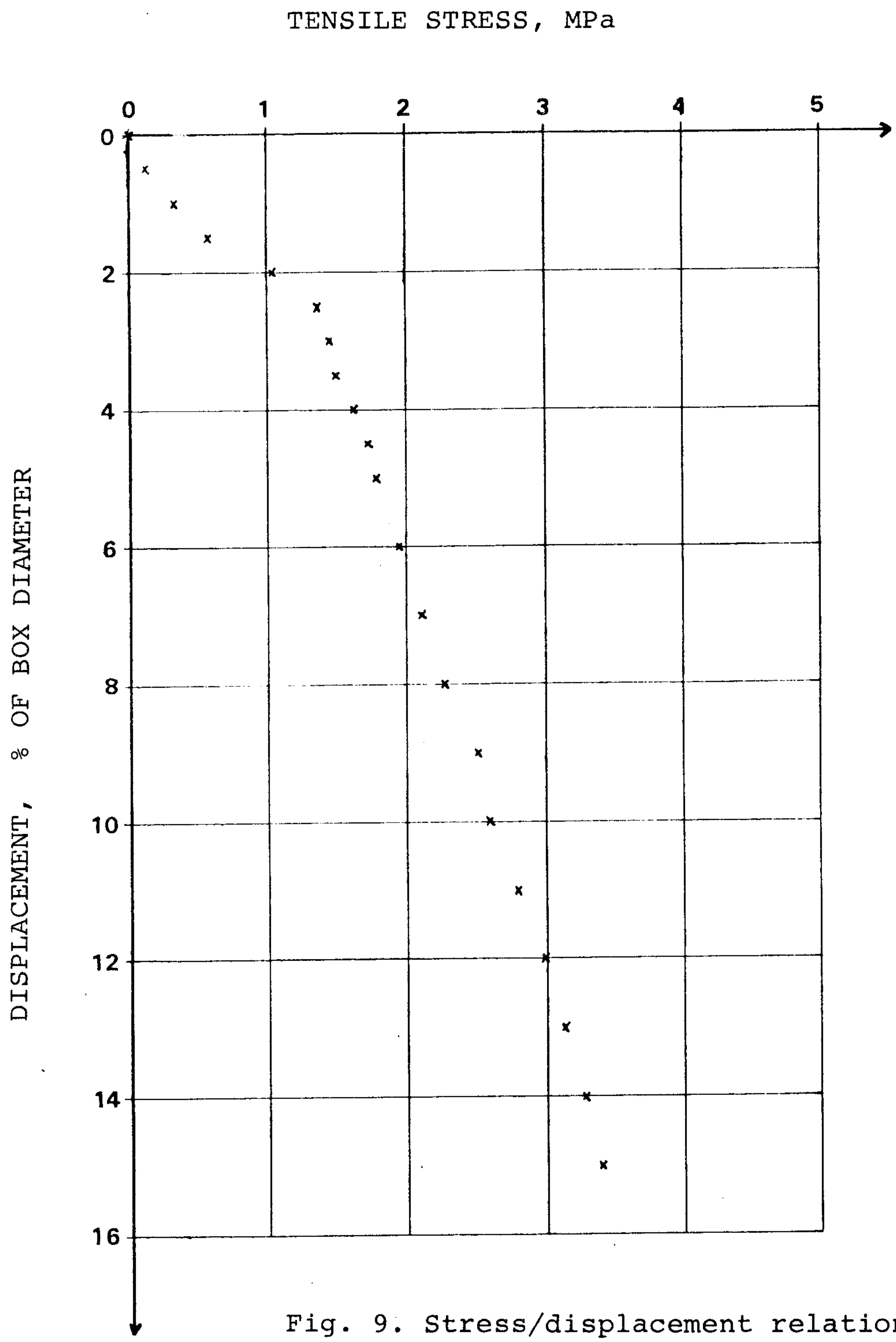


Fig. 9. Stress/displacement relationship for Gauge 2

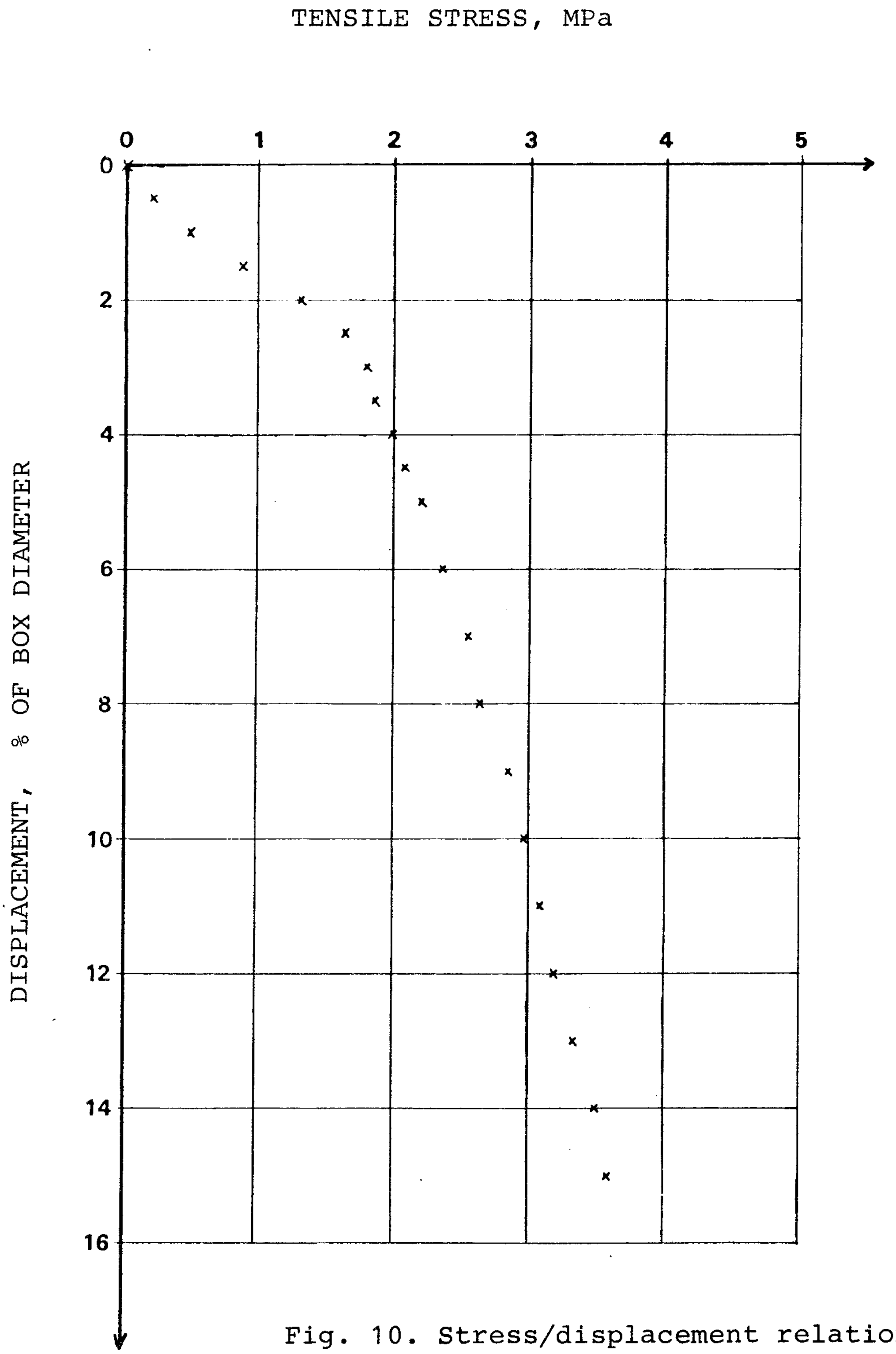


Fig. 10. Stress/displacement relationship for Gauge 3

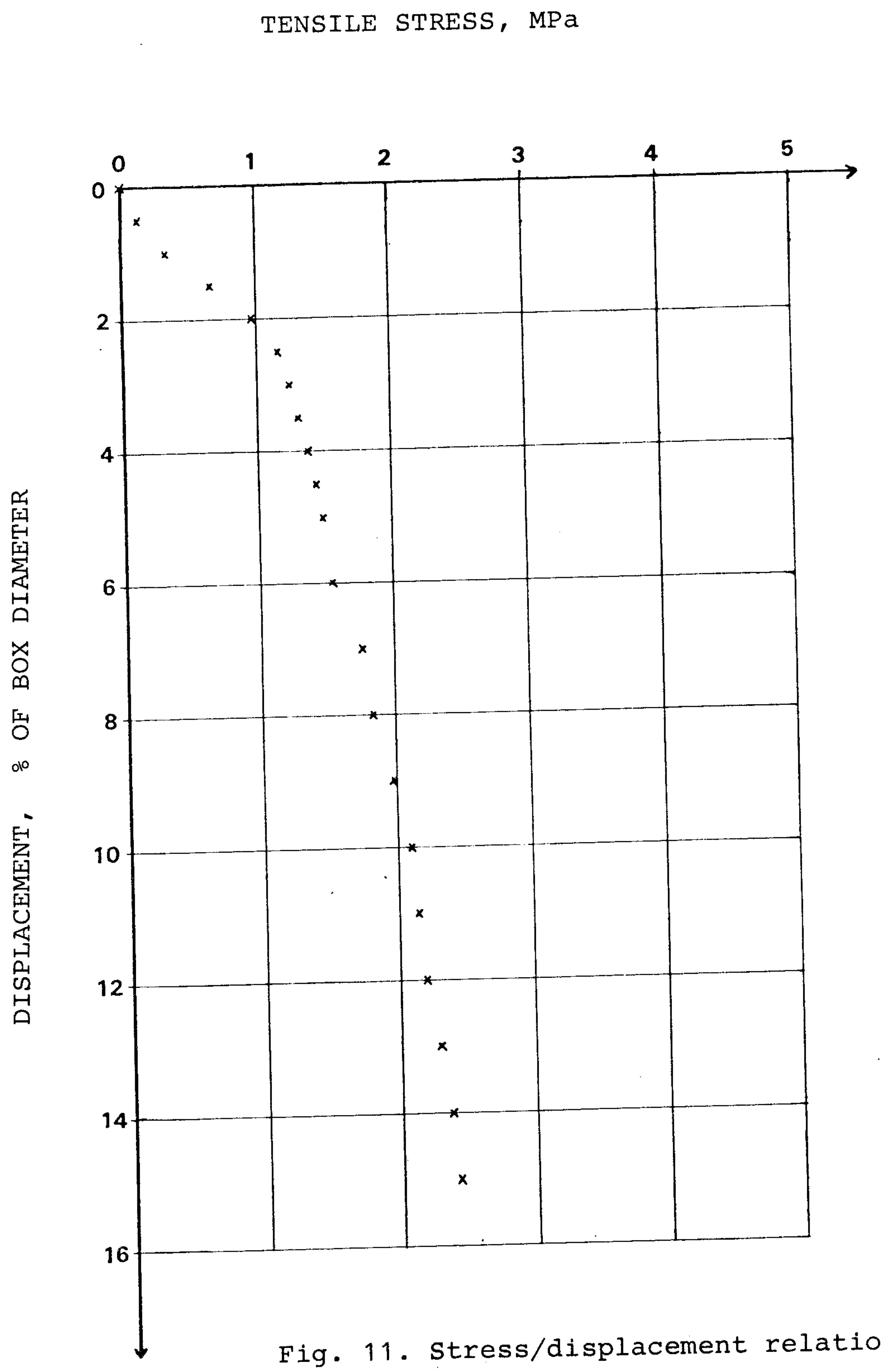


Fig. 11. Stress/displacement relationship for Gauge 4

The possibility of determining q_b by applying MEYERHOF theory can now be investigated. If we use Eq. (1) q_b = 480 kPa which is in fairly close agreement with the measured q_b-values. The conclusion should therefore be that MEYERHOF's theory can be used for a rough estimation of q_b also for the case of a real canister resting in a buffer mass which fills a tunnel. A closer examination shows, however, that it is difficult to make a safe prediction of q_b. Firstly Eq. (1), which was suggested to be valid for the full scale case in the previous report, implies a stress situation in the fill before the displacement and a bulk density which is not valid for the model. If relevant values are introduced for the model case it would yield q_b = 800-1000 kPa instead of 480 kPa. This is higher than the measured q_b-values but since there is still a reasonable agreement the theory is not disqualified. A more serious objection would be that at least one parameter ($N_{\rho q}$) is an unknown function of the geometry of the tunnel/canister/fill system. It may be higher than assumed here and in the previous report ($N_{\rho q}$ = 2500) but it is very probably lower. In the latter case Eq. (1) represents a conservative expression, which again, means that the theory can be used for practical application. Since q_b is very sensitive also to small variations in density, angle of internal friction and so forth, the application of MEYERHOF's theory requires soil mechanical expertness.

If there is a need to know more exactly the canister stresses, the problem should be solved by applying finite element analysis and/or a half-scale shear test. The theoretical approach requires that the stress/strain properties of the fill are thoroughly investigated and described in terms of a mathematical model. Experience shows that this may be a tedious and difficult task.

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Roland Pusch

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54:05 Repository domain groundwater flow analyses
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54:06 Final report
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