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CORROSION PROTECTION AND LATERAL DISPLACEMENT CHARACTERISTICS OF ROCK ANCHORS

H. Fines, W. Slater and R. Sage

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CORROSION PROTECTION AND LATERAL DISPLACEMENT CHARACTERISTICS OF ROCK ANCHORS

by

H. Fines*, W. Slater* and R. Sage**

ABSTRACT

Rock anchors have good potential application in stabilizing open pit slopes. Before they can be widely used their corrosion resistance and behaviour under lateral displacement — likely in a mine slope — must be shown to be satisfactory.

Field tests to examine these characteristics are impractical. Corrosion resistance was examined by studying greased, polyethylene-sheathed strand used in civil engineering. It was concluded this type of strand should be adopted for mine rock anchors where good corrosion resistance is required.

Laboratory tests on greased, sheathed strand were used to examine lateral displacement characteristics. The strand, grouted inside a steel tube, was tensioned and then displaced laterally. A maximum reduction in ultimate strength of about 17% resulted. Because the working load of a rock anchor is about 60% of undisplaced ultimate strength, to account for tension losses during installation, it appears the design working load need not be reduced because of lateral displacement anticipated after tensioning. In effect, a design working load of 60% of undisplaced ultimate strength would become an actual working load of not more than 72% of the displaced ultimate strength, which is acceptable.

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*Chief engineer and President, Conenco Canada (1968) Ltd., **Head, Pit Slope Project, Mining Research Laboratories, Canada Centre for Mineral and Energy Technology, Department of Energy, Mines and Resources, Ottawa, Canada.

PROTECTION CONTRE LA CORROSION ET LES CARACTERISTIQUES DE DEPLACEMENT LATERAL DES ANCRAGES

par

H. Fines*, W. Slater* et R. Sage**

RESUME

Les ancrages ont un bon potentiel d'utilisation pour stabiliser les pentes des exploitations à ciel ouvert. Avant d'être employés couramment, la résistance à la corrosion et le comportement lors de déplacements latéraux, susceptibles de se produire dans les pentes des exploitations, doivent donner des résultats satisfaisants.

Les essais en chantier effectués pour examiner ces caractéristiques ne sont pas pratiques. La résistance à la corrosion a été examinée par une étude d'un toron graissé avec revêtement de polyéthylène employé en génie civil. Les auteurs ont conclu que ce genre de toron devrait être adopté dans les ancrages des exploitations où une bonne résistance à la corrosion est indispensable.

Les essais en laboratoire sur les torons graissés et enduits ont servi à examiner les caractéristiques du déplacement latéral. Ce toron est cimenté à l'intérieur d'un tube d'acier, soumis à des tensions et ensuite déplacé latéralement. On a obtenu une réduction maximale d'environ 17% de la résistance ultime. Comme la charge de travail d'un ancrage se situe à environ 60% de la résistance ultime afin de compenser pour la perte de tension due à l'installation, il semblerait que la charge de travail n'a pas besoin d'être réduite à cause du déplacement latéral anticipé après avoir été soumis à un tension. Au fait, une charge de travail de 60% de la résistance ultime non-déplacée peut être remplacée par une charge de travail actuelle de 72% au plus de la résistance ultime déplacée, ce qui est acceptable.

*Ingénieur en chef et Président, Conenco Canada (1968) Ltd., **Chef, Projet sur les pentes des exploitations à ciel ouvert, Laboratoires de recherches minières, Centre canadien de la technologie des minéraux et de l'énergie, Ministère de l'Energie, des Mines et des Ressources, Ottawa, Canada.

CONTENTS

	Page
ABSTRACT	i
RESUME	ii
INTRODUCTION	1
CORROSION PROTECTION	2
Bridge over the Chippawa Canal	3
Highway Bridge near Niagara Falls	3
Ground anchors at Toronto Metropolitan Zoo	3
Pumping station in Ottawa	3
Ungrouted anchors	3
Cost	4
Conclusions - Corrosion protection	4
LABORATORY TESTS OF LATERAL DISPLACEMENT	4
Test apparatus	5
Discussion of test results	6
Conclusions - Displacement	7

TABLES

1	Tests on undeflected bare strand	7
2	Test to failure with lateral deflection 3-in. (75-mm) wheel, 2-in. (50-mm) (11°) deflection	8
3	Test to failure under lateral deflection 3-in. (75-mm) wheel, 3-in. (75-mm) (17°) deflection	8
4	Test to failure with lateral deflection 3-in. (75-mm) wheel, 4-in. (100-mm) (22°) deflection	9
5	Test to failure with lateral deflection 3-in. (75-mm) wheel, 5-in. (125-mm) (27°) deflection	9
6	Test to failure with lateral deflection 6-in. (150-mm) wheel, 2-in. (50-mm) (11°) deflection	9
7	Test to failure with lateral deflection 6-in. (150-mm) wheel, 3-in. (75-mm) (17°) deflection	9
8	Test to failure with lateral deflection 6-in. (150-mm)	10
9	Bare strand tensioned and then deflected to failure 3-in. (75-mm) wheel	10
10	Greased, sheathed strand, 3-in. (75-mm) wheel	10
11	Greased, sheathed, grouted strand, 3-in. wheel	11

FIGURES

1	Rock anchor must cross potential movement zone in rock	12
2	Performance of rock anchors subject to ground movement	12
3	Steps in installing a fully grouted anchor: (a) anchor inserted in hole (b) bottom grouted (c) anchor tensioned and clamped (d) remainder of hole grouted.	13
4	Rock anchor used at Chippawa Canal	14
5	Rock anchors for bridge abutments (courtesy CONENCO Ltd.)	15

FIGURES (cont'd.)

6	Corrosion resistance of greased and ungreased strand. The upper photograph shows strand exposed to corrosion before cleaning; the bottom photograph shows the strand after cleaning. In each case the lower of the strands is greased. The protection of the grease is clearly visible in the bottom photograph.	16
7	Rubber bushing to accomodate natural movement	17
8	Observation of soldier pile movements	17
9	Rock anchor after natural displacement	18
10	Schematic of deformation in a rock anchor due to lateral displacement. Off-set is about one inch in five inches	18
11	Representative stress-strain curve of strand used for displacement tests	19
12	Detail of test machine	20
13	Test machine	21
14	Detail showing deflected strand tensioned to failure	22
15	Results of tests with lateral deflection mean values with bare strand	23
16	Breaking loads of deflected strands — all test results	24
17	Results of tests with 3-in. (25-mm) wheels	25
18	Results of tests with 6-in. (50-mm) wheels	26
19	Photograph of recovered strand showing damaged polyethylene sheath	27
20	Grout-encased strand before and after testing	27
21	Detail showing grout-encased strand during testing	27
22	Deflected configurations of strand	28

INTRODUCTION

Rock anchors have been used for many years in civil engineering to stabilize slopes and other structures in soil and rock. In recent years, considerable attention has been paid to the application of rock anchors to slope stabilization in open pit mines (1,2,3). Briefly, in some circumstances rock anchors may increase the stability of a slope — that is, it may be possible to excavate a slope supported with rock anchors to a steeper angle than the same slope without rock anchors. In open pit mining, increased slope angles usually substantially reduce the total cost of mining. If rock anchors cost less than the savings resulting from steeper slopes, their use is justified on economic grounds. Several trial installations of rock anchors have been carried out and at least one full scale installation has been completed (3,4).

One of the important questions that has arisen from current mining experience is that of corrosion protection to rock anchors. In particular, rock anchors are susceptible to the phenomenon of stress corrosion, which manifests itself in sudden, brittle fracture at high stress. This question is complicated in the mining context by two factors: first, the mining environment is often chemically complex and may especially favour corrosion; second, the successful corrosion protection measures employed in civil engineering may be prohibitively expensive in the more severe economic restraints of mining. There is therefore a clear need for a simple yet effective corrosion protection mechanism for rock anchors before their use can be widely adopted in mining.

The second significant point to emerge from mining studies is that slopes to be stabilized with rock anchors will always be near the state of limiting equilibrium. That is, the slopes, though stable, will tend to undergo the displacements associated with slopes that are near the point of instability. This arises because if the slope were very stable there would be no point in using rock anchors. Thus the use of rock anchors to stabilize mine slopes requires an understanding of anchor behaviour in potential movement zones. Figure 1 illustrates this point. A rock anchor is typically used to help prevent sliding on a discontinuity. If some movement does occur, there is a tendency for the anchor to undergo lateral displacement. The effect of this on the strength of an anchor is not known. Knowledge of rock anchor behaviour in such circumstances is also necessary before their use can be adopted in mining.

This report is concerned both with corrosion protection and with anchor behaviour under lateral displacement. Ideally, each of these subjects would be studied under actual operating conditions in the field. However, this is difficult. In the case of corrosion, it would necessitate installing anchors with different corrosion protection in a variety of environments, and recovering and examining the anchors at different periods. The required life of rock anchors in practice might be as much as ten years; a field trial would therefore require recovery of anchors at, for example, six months, one year, two years, four years, six years, eight years, and ten years. Such an investigation program would clearly be a major undertaking.

Examining the lateral displacement characteristics of an anchor in the field would be even more difficult, requiring the installation of anchors in a slope known to undergo movement. The anchors would have to be tensioned to loads near their ultimate capacity to determine if displacement resulted in anchor failure.

In view of these difficulties, it was decided to examine the questions of corrosion protection and lateral displacement behaviour separately. It was felt that corrosion would be best investigated by evaluating experience of rock anchor installations, whether in mining or civil engineering, in corrosive environments. Lateral displacement behaviour seemed best appraised by laboratory tests which would simulate, as far as possible, conditions expected in the field.

This report has two parts. The first describes the information collected on corrosion protection and makes recommendations for corrosion protection of rock anchors for use in mining. The second describes laboratory tests to determine the effect of lateral displacement on the strength of rock anchors. The behaviour of rock anchors with corrosion protection was examined in these tests.

CORROSION PROTECTION

The techniques of rock anchors are taken from the prestressed concrete industry. The importance of corrosion protection to prestressing steel is well known; in most cases this protection is afforded by surrounding the tensioned cable with concrete or a cement grout. This has a dual function: if properly placed — that is, if free from voids — it isolates the steel from the surrounding environment; and it creates an alkaline environment at a pH of about 12 around the steel, which tends to inhibit stress corrosion.

In some circumstances, unbonded cables — that is, cables free to move relative to the surrounding structure except at the end anchorage points — have been used in the construction industry. In the 1950's, unbonded cables consisting of 0.5-in. (13-mm) diameter 7-wire strands were used in post-tensioned building slabs. They were protected by coating with non-oxide grease and wrapping in waxpaper (5). Subsequently, the use of unbonded cables in prestressed concrete has increased, and has also been adopted in such applications as external prestressing of steel structures. In this development, the waxpaper wrapping has been replaced first by PVC tube and then in the late 1960's with polyethylene tube. The strands within the tube are coated with a grease known to inhibit corrosion. These methods appear to be satisfactory in providing corrosion protection.

In both civil engineering and mining applications, rock anchors have been protected from corrosion by encasing the anchor in grout. However, there are disadvantages to this method, particularly in the mining context.

First, if movement occurs around the anchor, as is likely in mine slopes, there will be a tendency for the grout to crack, destroying its protective capability. Such movement also has a tendency to increase the stress in the anchor. In the fully grouted anchor, the increase will be local, because the grout prevents the stress being transferred to regions away from the movement zone. The rock anchor may therefore be locally overstressed. This would not happen if the anchor were not fully grouted (Fig. 2).

Second, if the anchor load is to be monitored or if the capability of retensioning the anchor after installation is required, the anchor must be free to move relative to the borehole. It therefore cannot be grouted. An alternative method of protection which leaves the anchor unbonded to the surrounding ground is therefore desirable.

A third disadvantage of the fully grouted anchor is that two grouting operations are required, the second usually up to two weeks after the first. This arises because the bottom anchorage is formed by grouting up to 25 ft (8 m) of the anchor into the borehole. When this grout has reached a required strength — which takes up to 14 days — the anchor can be tensioned, and then completely grouted into the hole (Fig. 3). This second stage grouting increases anchor cost considerably.

There is therefore considerable advantage in using an unbonded rock anchor if adequate corrosion protection can be attained, particularly if cost compares favourably with the fully grouted anchor.

There has already been considerable experience in civil engineering with unbonded anchors. The first widespread use of rock anchors of any type was in France in the 1940's, when André Coyne used them to stabilize rock foundations for arch dams. The ability to measure the load in the anchors from time to time was considered essential, and the anchors were therefore not fully grouted. Corrosion protection was provided by coating the anchors with bitumen or tar (6). This procedure has subsequently been refined by providing a separate sheath around the anchor and with protective grease. Currently, unbonded anchors are often formed with individual strands coated with a corrosion-inhibiting grease and sheathed in polyethylene tube, similar to civil engineering practice. Some examples of the use of these unbonded cables are described below.

Bridge Over the Chippawa Canal

In 1970, the abutments to a bridge over the Chippawa power canal were stabilized with rock anchors. About three hundred anchors were installed, making this the largest Canadian rock stabilization project to date. It was also the first use of unbonded greased strands in polyethylene tubes in Canada. The tendons had a breaking load of 1080 kips (4803 kN), and were tested to 85% of ultimate load. The retaining force applied by the anchors to the sides of the canal was about 133,000 tons (1183 MN). The unbonded length of the anchors was about 20 ft (6 m), with an anchorage length of 25 ft (8 m). Details of the project are shown in Fig. 4.

Highway Bridge Near Niagara Falls

In 1972, rock anchors of 1260 kips (5010 kN) ultimate load were used to stabilize the abutments of a continuous curved concrete bridge. Details of the anchors are shown in Fig. 5. An interesting feature of this project was the testing of proposed unbonded rock anchors to demonstrate that corrosion protection would be undamaged during installation. A single strand tendon, coated with grease and sheathed in plastic, was grouted inside a 30-ft (9-m) high vertical tube, filled with water. After the grout had hardened, the tendon was cut open and examined. It was found that voids between the greased strand and tube were filled with grout. The relatively high pressures in the grout column had enabled the grout to completely displace the water as it penetrated inside the plastic tube. Thus, during installation, grouting the bottom anchorage also resulted in the greased strand being embedded in grout within the plastic tube, which itself would be embedded in the outer layer of grout. The smooth walls of the plastic tube would allow the tendon, coated with grease and embedded in grout, to move when jacked; after tensioning, the tendon would be protected, first by the outer layer of grout, second by the plastic sheath, and third by the grout-grease mixture which would effectively exclude any water or air. The rock anchors were installed and have functioned satisfactorily.

Ground Anchors at Toronto Metropolitan Zoo

In 1973, 44 rock anchors up to 105-ft (32-m) long were installed at the zoo in consolidated till with sand intrusions. The anchors varied in size from five 0.5-in. (13-mm) to 12 0.5-in. (13-mm) strands, each individually greased and sheathed in plastic. The ultimate capacity of these anchors varied from 207 to 496 kips (920 to 2210 kN). One of the problems of the site was a high groundwater table; previous experience had indicated that the sheathed rock anchor, grouted in one stage, would satisfactorily overcome these problems.

Pumping Station in Ottawa

In 1975, rock anchors consisting of four 0.6-in. (15-mm) dia strands were used in the foundations to this pumping station where ground conditions were particularly wet. Again the sheathed, single-stage grouted anchor was felt to be best. In this particular case, Ciment Fondu was used as grout.

Unbonded rock anchors do not seem to have been used widely elsewhere with the exception of the Devonport Naval Base in England (7). In this project, 475 anchors of about 500 kips (2200 kN) working load were used to tie back the concrete retaining walls of a large dock to the surrounding ground. Unbonded anchors with single stage grouting were specified. Stringent conditions were laid down for fabrication of these anchors, and equipment was especially developed to grease and then push the strand into a plastic tube. The anchors ranged in length from 100 to 150 ft (30 to 45 m). A graphited bituminous grease was used, especially designed to resist removal from the strand.

Ungrouted Anchors

Several instances are recorded where greased, sheathed tendons have been used in potentially corrosive environments without the benefit of external grout cover. One example is at the Port Credit Marina, Ontario. Here, a series of floating concrete docks were tied together by tensioned cables. An interesting feature is that the tensioned cables must also accommodate some lateral movement. The cables were installed in 1972. In 1976, storm damage to one of the docks necessitated removing some strands. These were carefully examined for corrosion and were found to be in excellent condition, despite the relatively hostile environment. At the same time, random pieces of cable, which had been ungreased and were lying in the bottom of the concrete docks, were also collected for comparison. These had had no benefit of corrosion protection for a period of five years. Figure 6 shows the protected and unprotected strands. Figure 6(a) shows the strand as collected, the upper strand being unprotected. Figure 6(a) shows the strands after cleaning. At the bottom is the protected strand after the grease had been removed. The grip marks of an end anchorage can still be clearly seen, attesting to the excellent condition of the strand.

Strands have also been recovered from a prestressed concrete water tank, which required repair because of damage from external causes. Again, no evidence of distress to the greased unbonded strand was observed.

Cost

Commercially supplied prestressing strand, indidvidually greased and sheathed in polyethylene tube, cut to length, coiled and tagged, costs about $0.50/ft ($1.60/m). Hardware for surface anchorage is about $4 per strand or $40 for a 10-strand anchor.

Labour and equipment rented for installation, including grout mixer, grout stressing jacks and hydraulic pumps cost approximately $0.55 per ft ($1.80/m) of strand. Drilling and transportation and accomodation for the work force are not included. All costs are those for 1977.

Conclusions - Corrosion Protection

Field experience indicates that a rock anchor fabricated from greased strand encased in polyethylene tubing has excellent corrosion resisting properties. It can be grouted in one stage. Tests show that grout, as well as surrounding the anchor, rises through the polyethylene tube to eliminate voids, thus providing additional protection.

The smooth-walled polyethlene tube means this type of rock anchor is unbonded; it can therefore be monitored, retensioned if suitable jacking arrangements are made (see Ref. 3) and also will not suffer local overstress if movement occurs. It offers a cost saving of 15-20% over anchors protected by 2-stage grouting. This type of anchor is therefore recommended for rock anchors in mining.

LABORATORY TESTS OF LATERAL DISPLACEMENT

There has been little or no research into lateral movement of rock anchors. However, there are several instances where the qualitative behaviour of rock anchors undergoing lateral displacement is known.

One such case is that of cables installed at the Port Credit Marina, described in the first part of this report. The floating docks were designed to permit relative movement. The tensioned cables passed through rubber bushings at the junctions of the docks, as shown in Fig. 7. The bushings allow the docks to rotate and move relative to one another. No measurements of movement are available but the greased, sheathed strands have functioned satisfactorily.

Two unreported instances of slip in an anchored retaining wall indicate the ability of rock anchors to resist lateral displacement. The cases are similar, and are both shown schematically in Fig. B. One occurred at the Simpson's Tower site, Toronto, and the other at the Pickering nuclear generator site. In each case, disturbance at the toe of the retaining wall, which was tied back with temporary rock anchors (tie backs) made from bare (ungreased) strand grouted in the anchorage zone only, resulted in a vertical drop of the wall. Neither wall failed however, indicating the rock anchors continued to function, despite the obvious lateral displacement.

Similar lateral displacement might be expected in a slope stabilized with rock anchors. Figure 1 shows such hypothetical stabilization. Typically, instability would occur through slip on well defined discontinuities — usually joints or faults. The tension in the rock anchor both directly resists the tendency to slide and increases the normal stress on the discontinuity. This increase in normal stress augments the frictional component of the shear strength against sliding on the discontinuity.

If movement occurs on the discontinuity, the sequence of events would be as follows:

a. the rock anchor is installed and tensioned
b. lateral displacement occurs
c. the rock anchor must now function in a a displaced configuration as shown in Fig. 9.

It is reasonable that rock anchors should, if possible, be designed so that any loss either in load or in rock anchor capacity due to such lateral displacement is accounted for. This raises two questions. First, what degree of movement could be expected in practice? and second, what effect would this have on the load on, or the strength of, the rock anchor?

No definite figures for the amount of displacement are available but the order of magnitude to be expected can be estimated from Fig. 10. This shows a 3-in. (75-mm) borehole crossing a line discontinuity. The discontinuity meets the borehole at approximately 40°, which is a typical angle. A 1-in. (25-mm) cable, representing a rock anchor, runs along the centre of the hole; the hole is displaced by 1 in. (25 mm) in the direction of the discontinuity which seems a reasonable displacement for the rock anchor to withstand. The hole is filled with grout. Sketching what seems to be a reasonable post-displacement configuration gives a possible lateral displacement of the anchor of one in five. That is, the anchor displaces 1 in. (25 mm) in a length along its axis of 5 in. (125 mm).

The laboratory tests were therefore planned to determine primarily the reduction in ultimate strength of a rock anchor due to lateral displacement in the order of 1:5, or about 10°.

Tests were divided into three categories.

a. bare strand
b. greased strand sheathed in polyethylene tube
c. greased, polyethylene-sheathed strand grouted into a rectangular steel tube with observation holes and break lines.

The strand used was Stelco 0.62-in. (13.78-mm) 7-wire stabilized from Pak No. 1337-3. Figure 11 is the test certificate supplied by Stelco. Polyethylene tubing was 0.73 in. (18.5 mm) inside diameter with a wall thickness of 0.04 in. ± 0.01 (1 mm ± 0.25). Grease used was Texaco AFB type II.

Test Apparatus

The testing machine devised for these experiments is shown in Fig. 12. It allows a single strand rock anchor to be tensioned from either end, and to be displaced before or after tensioning by means of the vertical jacks and yoke. When displaced, the strand passes over wheels, the diameter of which can be varied. Figures 13 and 14 are photographs of the test apparatus. For the tests described in this report, wheels of 3-in. (75-mm) and 6-in. (130-mm) were used.

Anchor tensioning was done from one end only. The jack used for tensioning the cable was a Titan 30, with a piston area of 9.02 in.^{2} (5820 mm^2). The jacks used for lateral deflection were Simplex RC 315 with a piston area of 6.53 in.^{2} (4210 mm^2). In calculating loads on the anchor, a conservative 2% friction loss in the jacks was assumed, i.e., the load is equal to the piston area multiplied by gauge pressure multiplied by 0.98.

The first group of tests was used to determine the ultimate breaking strength of the strand. Seven strands were tensioned to failure without deflection; the results are shown in Table 1.

The second group of tests determined the reduction in ultimate capacity of the bare strand using both 3-in. (75-mm) and 6-in. (150-mm) dia deflection wheels. The results of these tests are shown in Tables 2 to 8. The deflection values are given in absolute values and as estimated deflection in degrees, assuming a deflected length of 10 in. (250 mm). The wheels described under the heading "Comments" are numbered starting from the tensioning jack — i.e., wheel 1 is nearest the jack.

The mean results of Tables 1 — 8 are plotted in Fig. 15. They show a peak reduction of ultimate tensile strength of 83%, based on mean values. The reduction is also dependent on the size of wheel used as a deflection point.

The third group of tests was carried out using bare strand, greased and sheathed strand, and greased, sheathed, grout-encased strand. Tests were also made with the strand tensioned and then deflected. The results are shown in Tables 9 — 11.

Figure 16 shows all the test results, for comparison. Figures 17 and 18 separate results for the 3-in. (75-mm) and 6-in. (150-mm) diameter wheels respectively.

In all the tests of deflected, sheathed strand, the polyethylene sheath was damaged at the points of deflection. Figure 19 shows an example of damaged sheath.

Figures 20 and 21 show details of the greased, sheathed, grouted strands before, during and after testing. The break and observation points in the steel grout case can be clearly seen. The rule in Fig. 20 is marked in inches. The grout used was a rapid-hardening cement/water mixture with an estimated strength at the time of the tests of 4000 psi (27.6 MPa).

Discussion of Test Results

The main points from the test results are:

a. there is a reduction in ultimate capacity of a strand tested with deflection;
b. samples deflected with 3-in. (75-mm) wheels failed at a lower load than samples deflected the same amount with 6-in. (150-mm) wheels;
c. there appears to be a maximum load reduction of about 83% ultimate strength between 2 in. and 4 in. (50 and 100 mm) deflection for both 3-in. (75-mm) and 6-in. (150-mm);
d. there is a wide but consistent scatter in the test results;
e. greased strand in polyethylene sheath shows no significant difference in strength from bare strand.

Considering the results as a whole, the following mode of failure appears reasonable. If the pattern of the 7-wire strand remains as shown in Fig. 22(a), one or two wires next to the wheel would fail at a fairly low load. If the wires are forced into the configuration shown in Fig. 22(b), then one to four wires next to the wheel would probably fail at a notably higher load. It is probable that intermediate configurations such as Fig. 22(c) would also occur. The force which causes the strand wires to displace would be a function of both the force applied and the contact length as shown in Fig. 17(d). It appears that the wires do not consistently displace until the deflection exceeds 4 in. (100 mm), thereby causing:

a. considerable scatter in results; and
b. an increase in breaking strength above 4-in. (100-mm) deflection.

The tests also show that polyethylene sheath will generally be cut through or crushed when a substantial lateral deformation occurs. This may be a consideration in corrosion protection; however, the test conditions are considered severe, and an actual rock anchor would have the remaining protection of the grease-grout mixture inside the sheath.

The significant factor to emerge from the tests is that there will be a maximum reduction in anchor ultimate strength of 17%, if the kink induced at the point of deflection has a radius not less than 1.5 in. (38 mm). A rock anchor in practice usually consists of several strands encased in a grout column. If displacement occurs, the grout would crush and produce smoothing at the points of deflection; it therefore appears unlikely that a radius below 1.5 in. (38 mm) would occur in reality.

If a maximum 17% reduction in ultimate load occurs, the relevance of this to design load must be considered. The recommended design load of a rock anchor is 60% of guaranteed ultimate tensile strength. This figure is derived as follows:

a. the maximum allowed load in tensioning is 80% of guaranteed strength,
b. losses on anchoring and through short term relaxation amount to a further 20% loss in load, i.e., to 60%.

Note that there is no objection in principle to a rock anchor beirg designed to a working load of 80% of ultimate strength. The 60% figure arises solely from installation considerations.

Once installed, if a rock anchor tensioned to 60% of ultimate strength undergoes displacement, the apparent ultimate strength may drop to 83%. The working load would then become 60/83 = 72% of the apparent ultimate strength. This is an acceptable working load. It thus appears that no reduction in design working load is required to account for strength reductions due to displacement.

Conclusions - Displacement

The results of this study indicate that no reduction in rock anchor working load is required to accomodate possible lateral displacement. This assumes the kink due to displacement would have a minimum radius of 1.5 in. (3.8 mm).

Polyethylene sheath encasing a greased strand will probably be damaged during lateral displacement. This may reduce the corrosion resistance of the anchor although the substantial protection of the grease-grout mixture would remain.

Table 1: Tests on undeflected bare strand

Test no.	Failure load kips	kN	Comments
1	66.3	295	Failed at jack grips
2	68.9	307	Jack load dropped — no break found on dismantling
3	67.2	299	One wire broke at passive anchorage
4	67.6	301	One wire broke at passive anchorage
5	68.9	307	Jack load dropped — no break found on dismantling
6	69.8	311	All wires broke at jack grips
7	67.2	299	All wires broke at jack grips

Mean failure load: 68.0 kips (303 kN)

Table 2: Test to failure with lateral deflection 3-in. (75-mm) wheel 2-in. (50-mm) (11°) deflection

Test no.	Failure load kips	kN	Comments
8	61.0	272	2 wires failed at wheel 1, 1 wire failed at wheel 2
9	60.1	268	4 wires failed at wheel 1
10	56.5	252	1 wire failed at wheel 1
11	58.3	260	1 wire failed at wheel 1
12	58.7	261	1 wire failed at wheel 1
13	57.9	258	1 wire failed at wheel 2
14	56.6	252	1 wire failed at wheel 1
15	61.4	273	1 wire failed at wheel 1
16	61.0	272	1 wire failed at wheel 1
17	57.9	258	1 wire failed at wheel 1

Mean failure load 58.9 kips (262 kN)

Table 3: Test to failure under lateral deflection 3-in. (75-mm) wheel 3-in. (75-mm) (17°) deflection

Test no.	Failure load kips	kN	Comments
18	61.9	276	1 wire failed at wheel 2
19	59.2	264	1 wire failed at wheel 1 and at wheel 2
20	59.2	264	1 wire failed at wheel 1
21	59.2	264	1 wire failed at wheel 1
22	59.2	264	1 wire failed at wheel 1
23	56.6	252	1 wire failed at wheel 1
24	56.6	252	4 wires failed at wheel 1
25	56.1	250	1 wire failed at wheel 1
26	58.3	260	2 wires failed at wheel 1, 1 wire failed at wheel 2
27	56.6	252	1 wire failed at wheel 1

Mean failure load 58.3 kips (260 kN)

Table 4: Test to failure with lateral deflection 3-in. (75-mm) wheel, 4-in. (100-mm) (22°) deflection

Test no.	Failure load kips	kN	Comments
28	59.2	264	1 wire failed at wheel 1 and at wheel 2
29	59.6	265	3 wires failed at wheel 1
30	57.5	256	2 wires failed at wheel 1
31	57.5	256	1 wire failed at wheel 1
32	55.7	248	1 wire failed at wheel 1
33	58.3	260	1 wire failed at wheel 1
34	56.6	252	1 wire failed at wheel 1
35	57.9	258	1 wire failed at wheel 1
36	59.2	264	2 wires failed at wheel 1
37	58.3	260	1 wire failed at wheel 1

Mean failure load 57.9 kips (258 kN)

Table 5: Test to failure with lateral deflection 3-in. (75-mm) wheel, 5-in. (125-mm) (27°) deflection

Test no.	Failure load kips	kN	Comments
38	61.9	276	6 wires failed at wheel 1
39*	63.6	283	No actual failure
40	60.1	268	1 wire failed at wheel 1
41	60.1	268	4 wires failed at wheel 1
42	50.8	226	3 wires failed at wheel 1
43	59.2	258	3 wires failed at wheel 1
44	59.2	258	1 wire failed at wheel 1
45	56.6	252	1 wire failed at wheel 2
46	57.5	256	1 wire failed at wheel 1
47	60.1	268	1 wire failed at wheel 1

Mean failure load 58.9 kips (262 kN)

* Deflection actually 4.75 in.(120.6 mm) (25°)

Table 6: Test to failure with lateral deflection 6-in. (150-mm) wheel, 2-in. (50-mm) (11°) deflection

Test no.	Failure load kips	kN	Comments
48	63.6	283	4 wires failed at wheel 1
49	61.9	276	4 wires failed at wheel 1
50	61.9	276	1 wire failed at wheel 1
51	64.1	285	4 wires failed at wheel 1
52	63.6	283	1 wire failed at wheel 1

Mean failure load 63.0 kips (28 kN)

Table 7: Test to failure with lateral deflection 6-in. (150-mm) wheel, 3-in. (75-mm) (17°) deflection

Test no.	Failure load kips	kN	Comments
53	58.3	259	1 wire failed at passive anchorage
54	59.7	266	1 wire failed at wheel 1
55	64.5	287	1 wire failed at wheel 1
56	64.5	287	1 wire failed at wheel 1
57	64.5	287	4 wires failed at wheel 1

Mean failure load 62.9 kips (280 kN)

Table 8: Test to failure with lateral deflection 6-in. (150-mm)

Test no.	Lateral deflection In.	mm	Degrees	Failure load kips	kN	Comments
58	3.5	89	19	64.5	287	2 wires failed at wheel 1
59	4.0	101	22	64.1	285	1 wire failed at wheel 1
60	4.0	101	22	66.3	295	4 wires failed at wheel 1
61	5.0	127	27	64.1	285	2 wires failed at wheel 1
62	5.0	127	27	66.3	295	4 wires failed at wheel 1
63	5.0	127	27	64.5	287	4 wires failed at wheel 1

Mean failure load at 4-in. (100-mm) deflection 65.2 kips (290 kN)
Mean failure load at 5-in. (125-mm) deflection 65.0 kips (289 kN)

Table 9: Bare strand tensioned and then deflected to failure 3-in. (75-mm) wheel

Test no.	Peak deflection in.	mm.	deg	Initial strand load kips	kN	Failure load kips	kN	Comments
64	2.5	648	14	56.6	252	56.6	252	1 wire failed at wheel 3
65	3.5	89	19	52.1	232	53.9	240	1 wire failed at wheel 3
66	3.5	89	19	49.5	220	53.5	238	1 wire failed at wheel 3
67	3.75	95	21	44.2	197	53.5	238	1 wire failed at wheel 1

Table 10: Greased, sheathed strand, 3-in. (75-mm) wheel

Test no.	Peak deflection in.	mm	deg	Initial Strand load kips	kN	Failure load kips	kN	Comments
68	4.0	102	22	28.7	128	-	-	Strand tensioned and then deflected. Noise at 4-in. deflection, no break found on disassembly
69	4.75	121	25	-	-	60.5	270	Strand deflected and then tensioned to failure. One wire failed at wheel 2
70	3.0	76	17	-	-	59.2	264	Strand deflected and then tensioned to failure. One wire failed at wheel 1

Table 11: Greased, sheathed, grouted strand, 3-in. wheel

Test no.	Description
71	Strand tensioned to 44.2 kips (107 kN) and then deflected. At 4.5-in. (115-mm) deflection (maximum possible) no failure observed; tension increased during deflection (without jacking) to 50.8 kips (226 kN). Tension at 4.5-in. (115-mm) deflection increased by jacking until failure at 57.5 kips (256 kN). Assumed 1 wire failed at wheel 1 (sample not disassembled).
72	Strand tensioned to 52.6 kips (234 kN) (90% of guaranteed ultimate strength and deflected. At 4.5-in. (115-mm) jackload had increased to 53.9 kips (240 kN) without failure. Tension at 4.5-in. (115-mm) deflection increased to failure at 63.6 kips (283 kN). 1 wire failed at passive anchorage.
73	Strand tensioned to 55.7 kips (248 kN) (95% of guaranteed ultimate strength) and deflected to 4.5 in. (115 mm). At 4.5 in. (115 mm) no change in load. Tension at 4.5-in. (115-mm) deflection increased to failure at 64.5 kips (287 kN). Assumed 1 wire failed at wheel 1 (sample not disassembled).

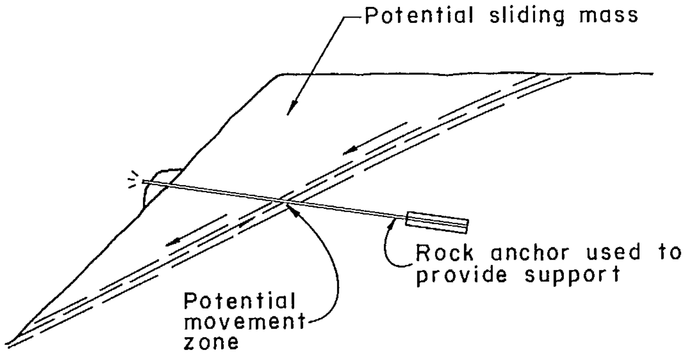


Fig. 1 - Rock anchor must cross potential movement zone in rock

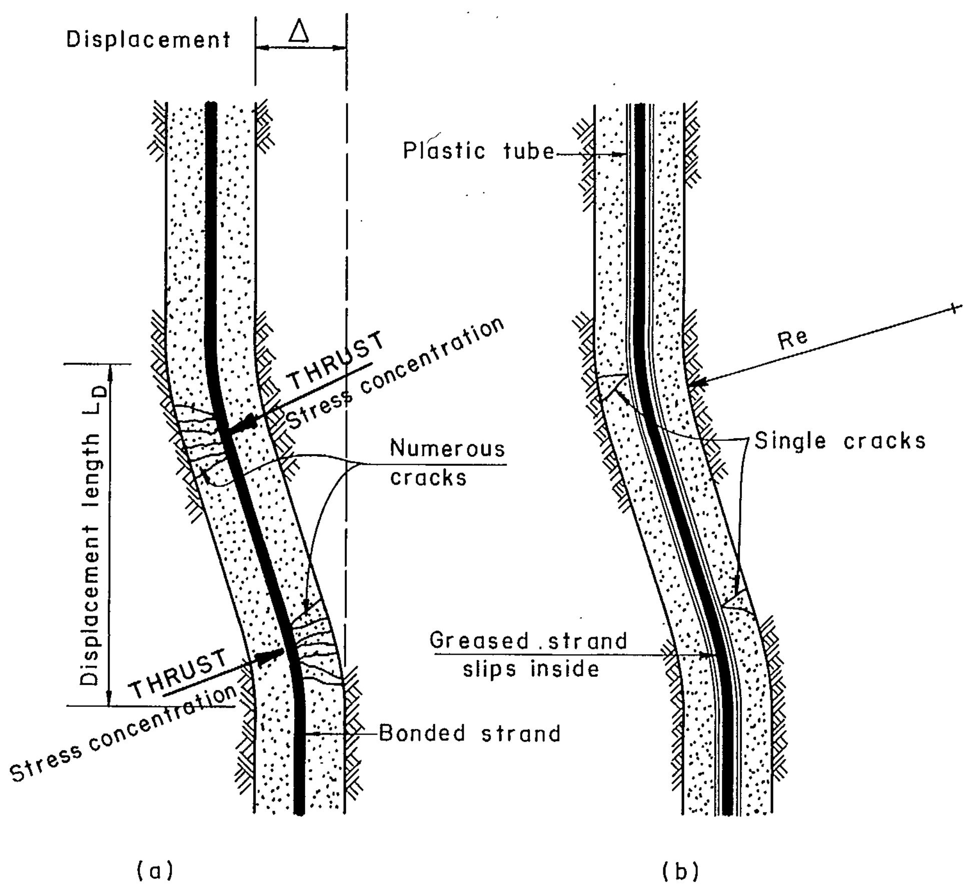


Fig. 2 - Performance of rock anchors subject to ground movement

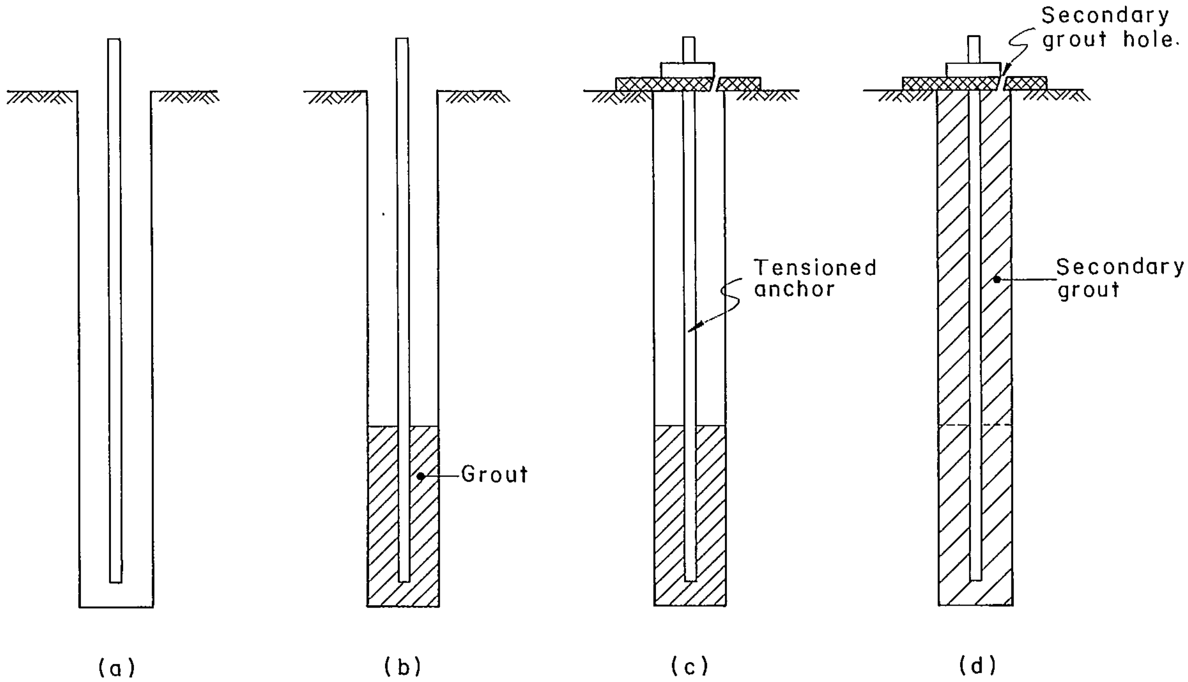
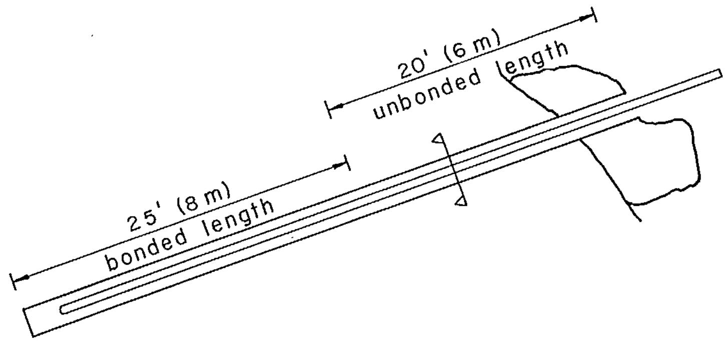
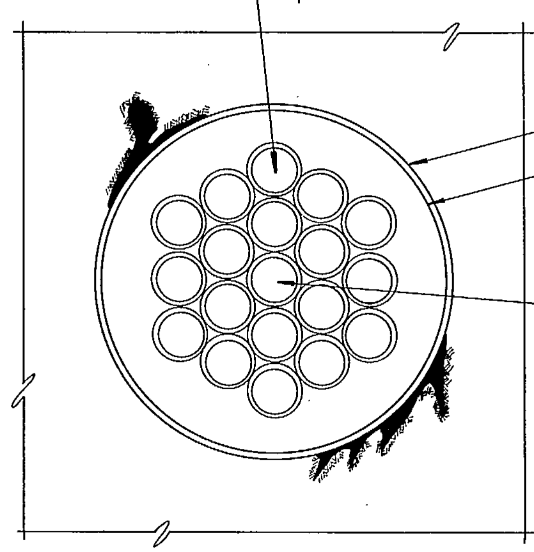


Fig. 3 - Steps in installing a fully grouted anchor: (a) anchor inserted in hole (b) bottom grouted (c) anchor tensioned and clamped (d) remainder of hole grouted





Section

Fig. 4 - Rock anchor used at Chippawa Canal

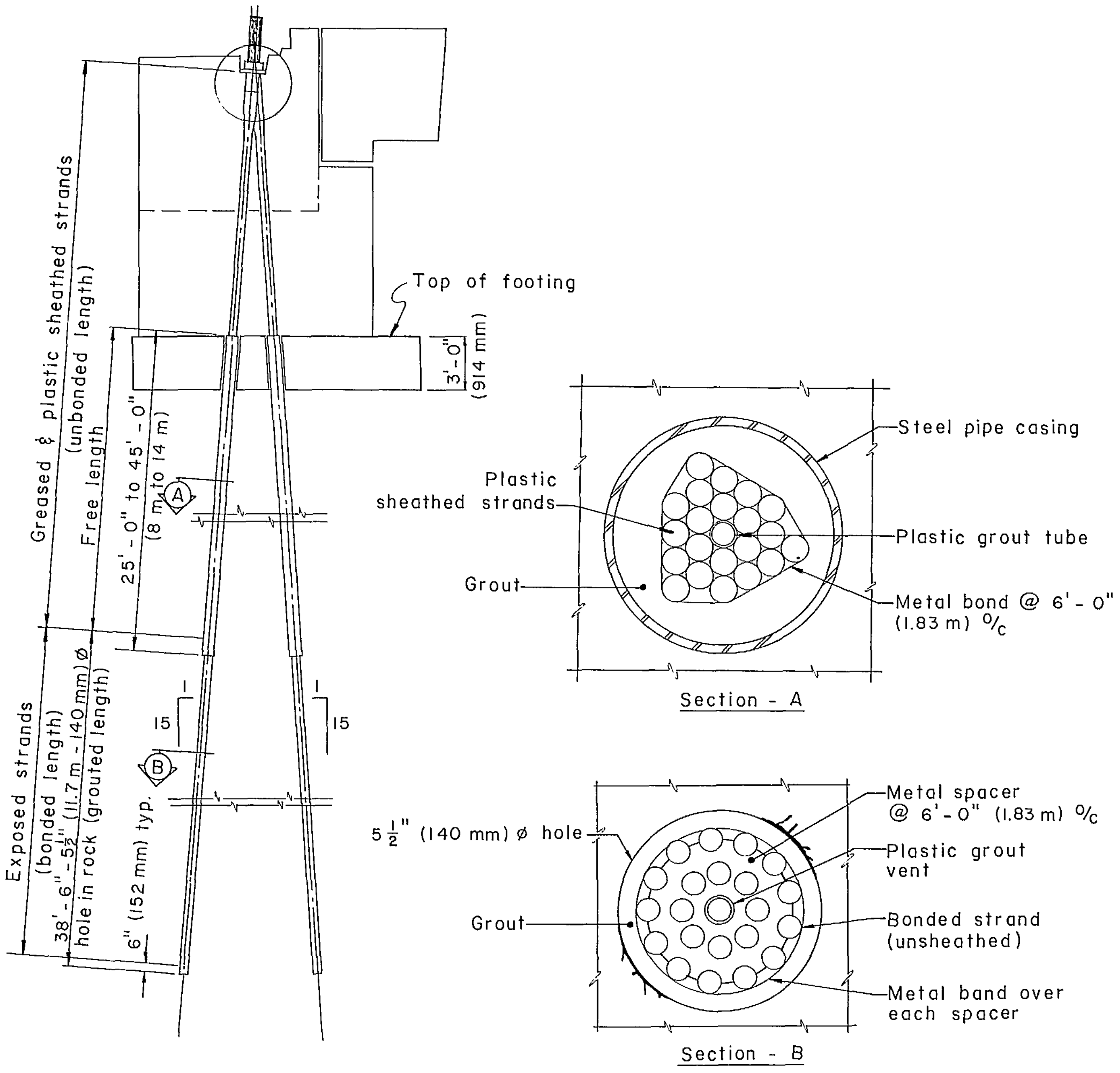


Fig. 5 - Rock anchors for bridge abutments (courtesy CONENCO Ltd.)

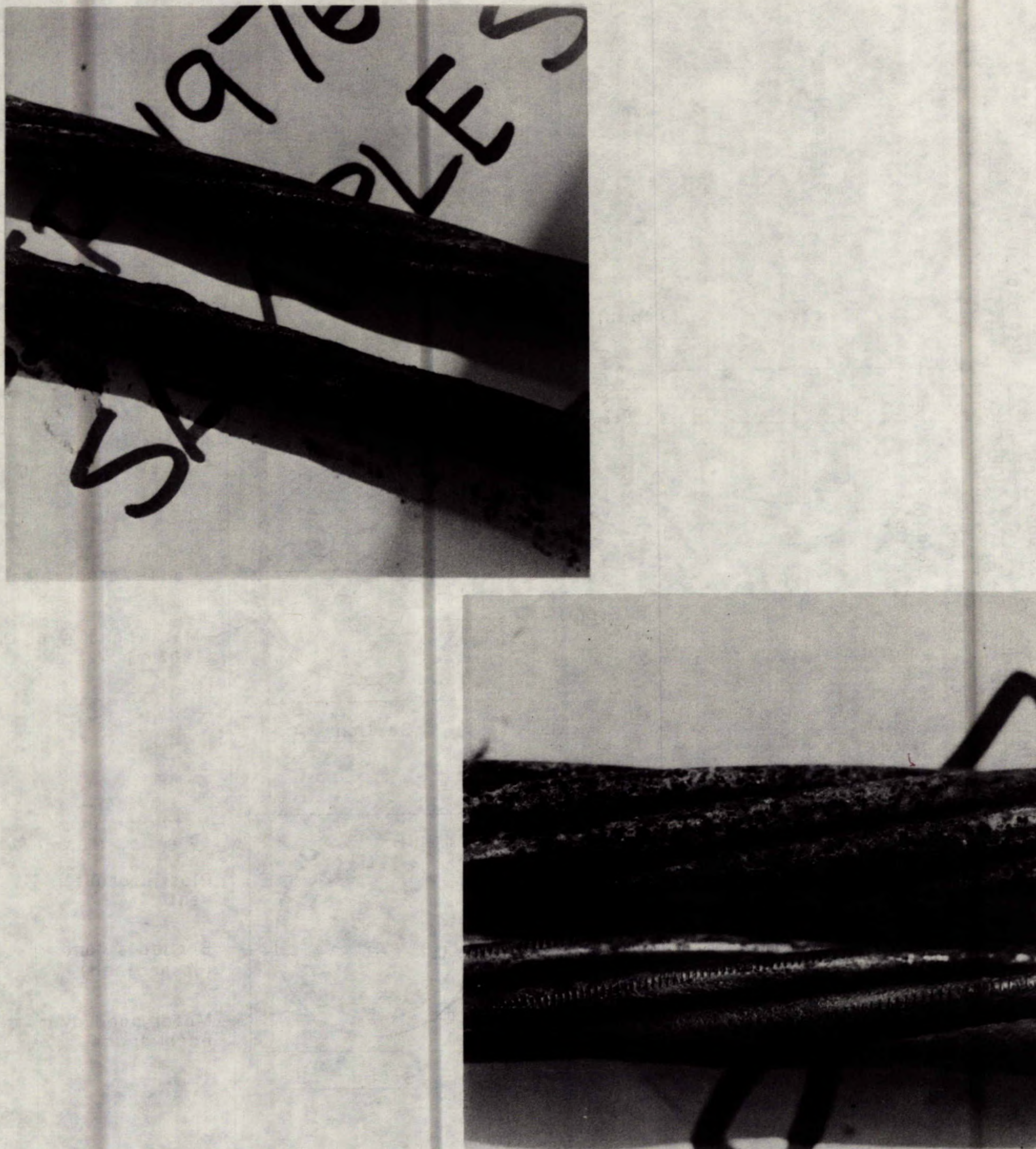


Fig. 6 - Corrosion resistance of greased and ungreased strand. The upper photograph shows strand exposed to corrosion before cleaning; the bottom photograph shows the strand after cleaning. In each case the lower of the strands is greased. The protection of the grease is clearly visible in the bottom photograph

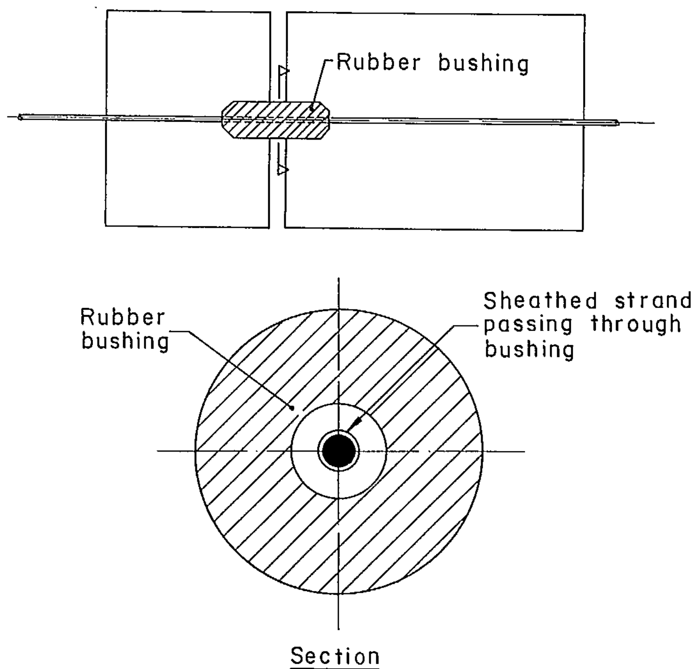


Fig. 7 - Rubber bushing to accomodate natural movement

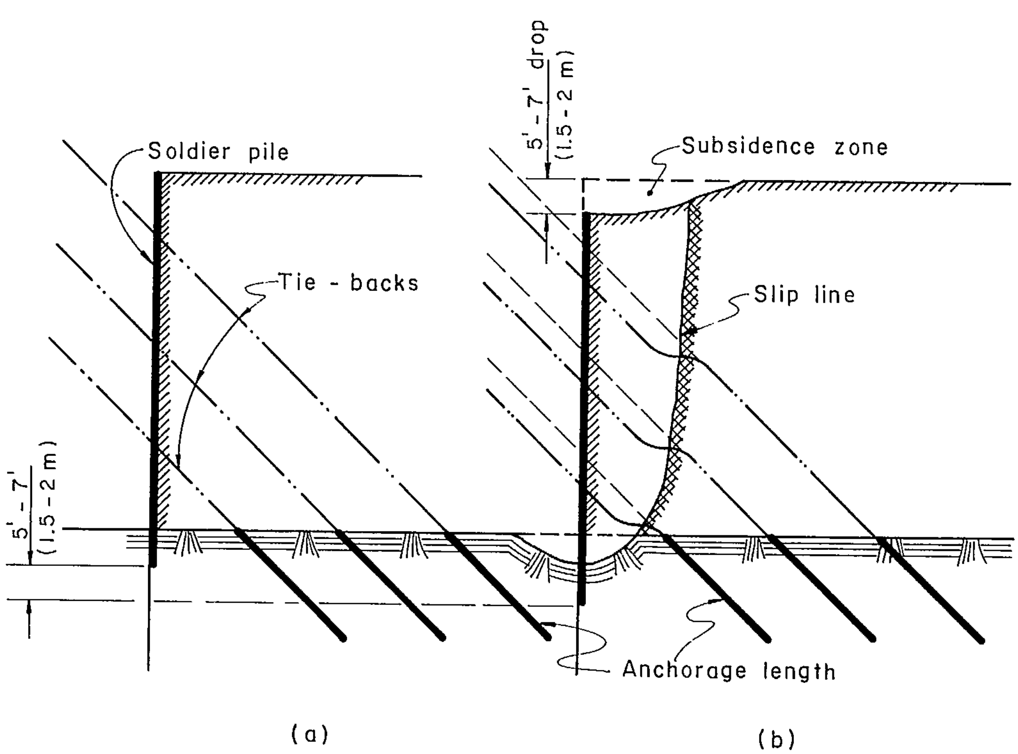


Fig. 8 - Observation of soldier pile movements

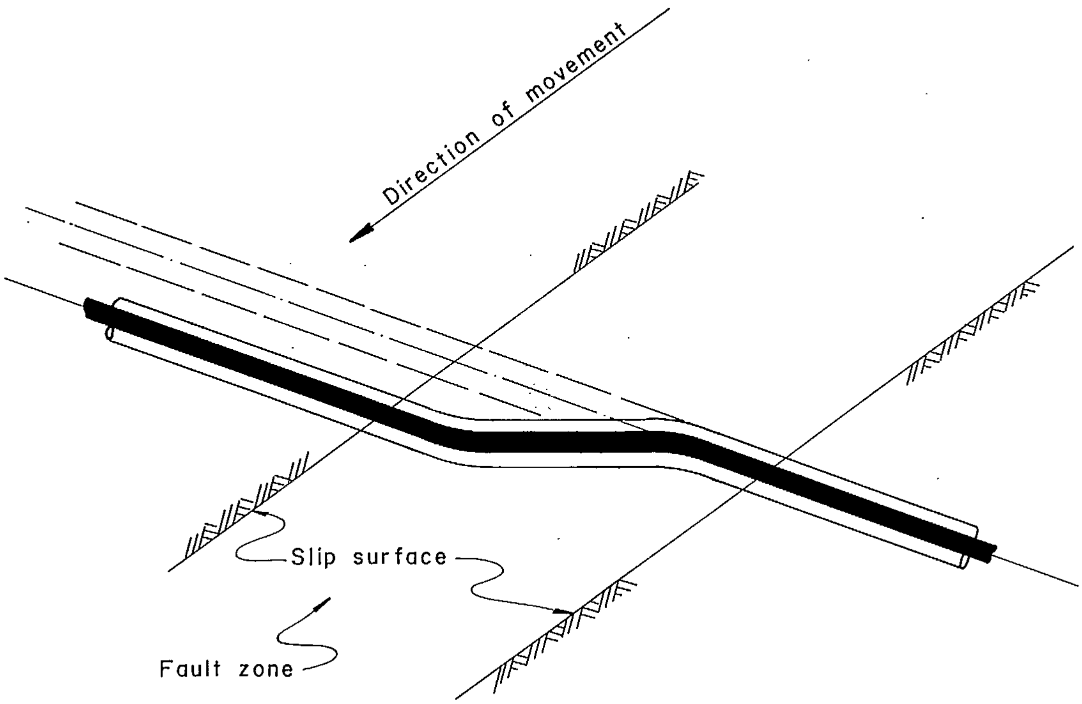


Fig. 9 - Rock anchor after natural displacement

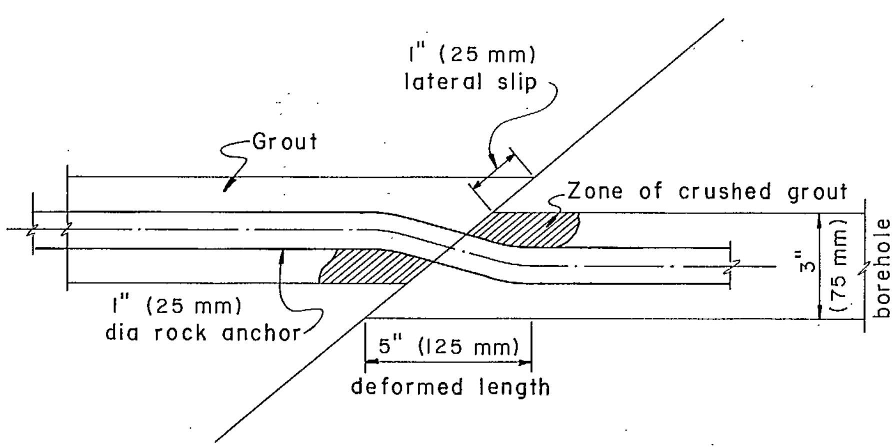


Fig. 10 - Schematic of deformation in a rock anchor due to lateral displacement. Off-set is about one inch in five inches

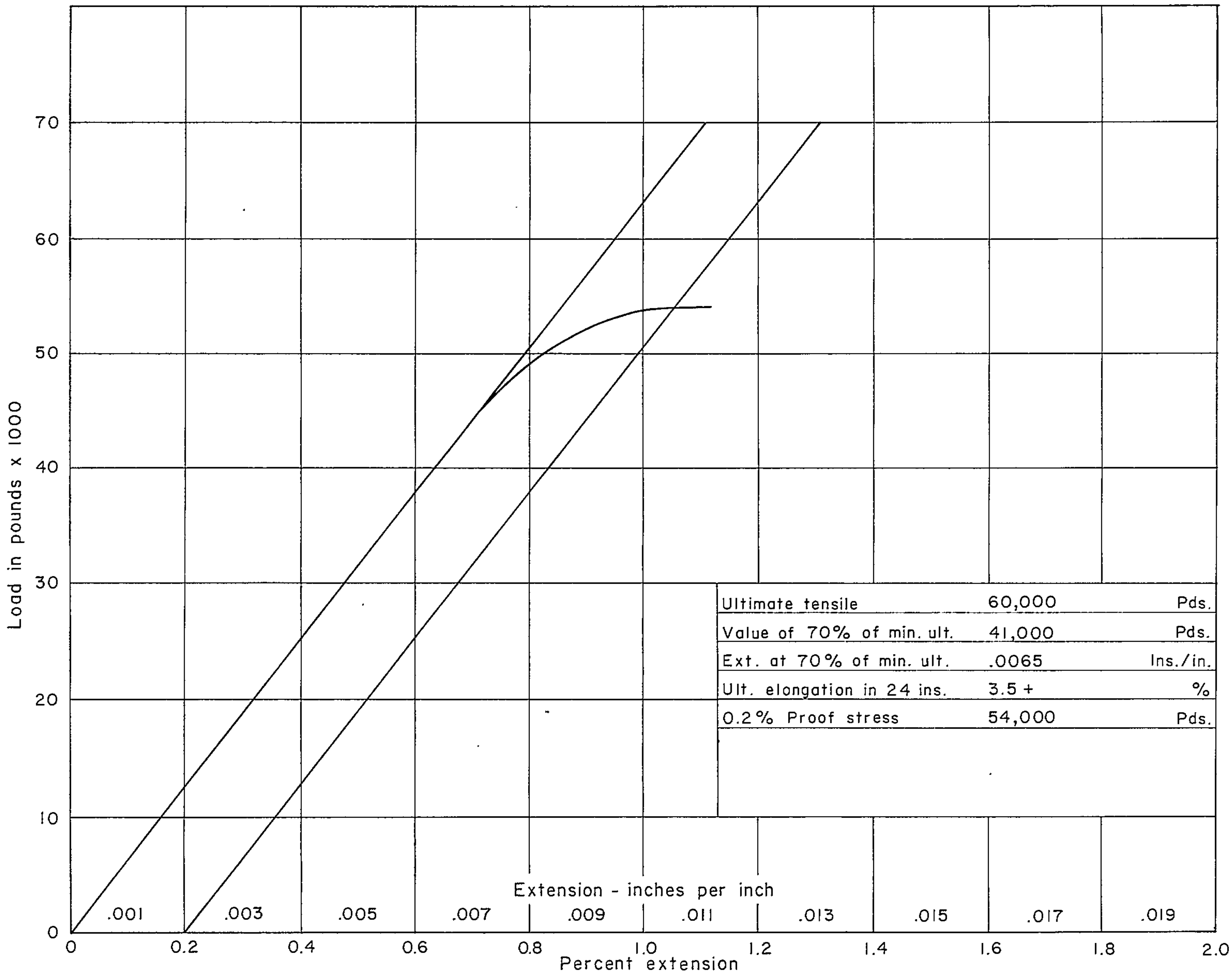


Fig. 11 - Representative stress-strain curve of strand used for displacement tests

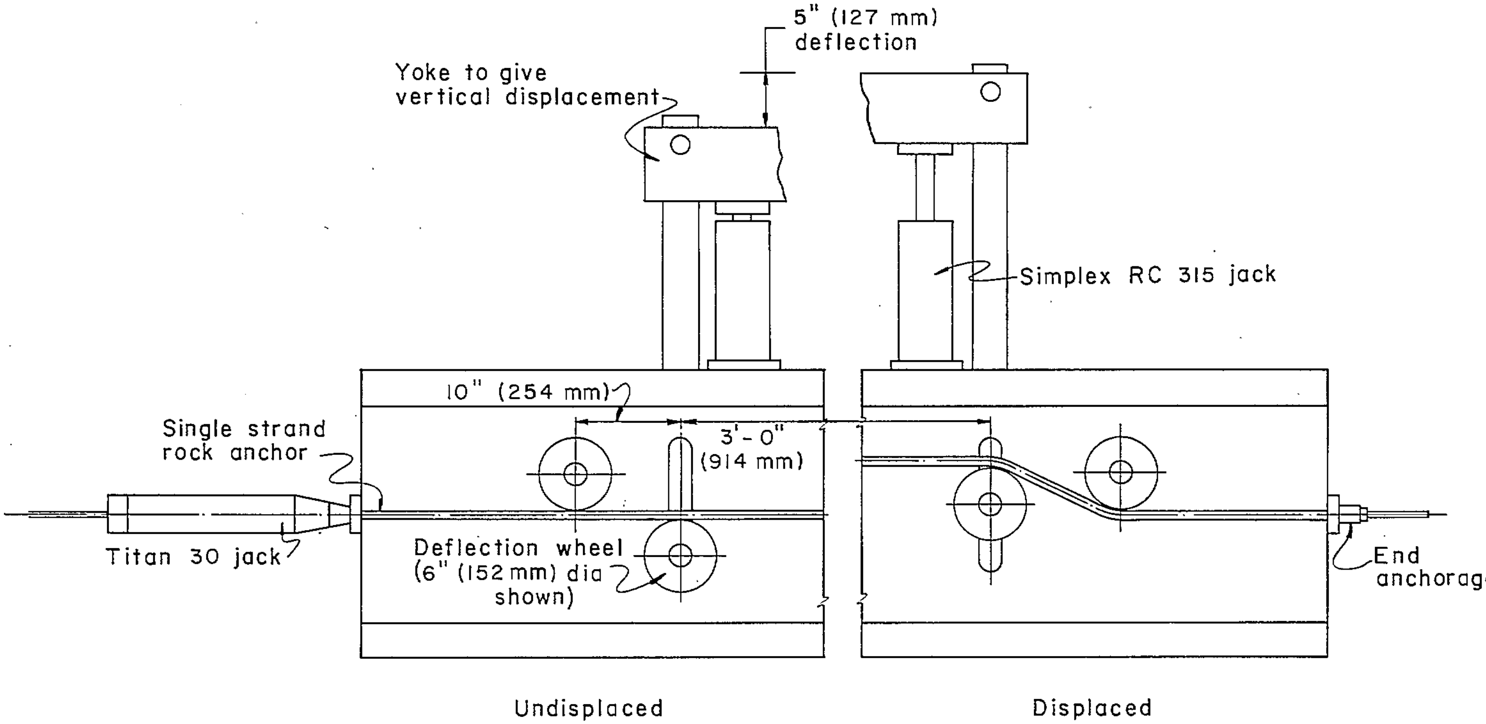


Fig. 12 - Detail of test machine

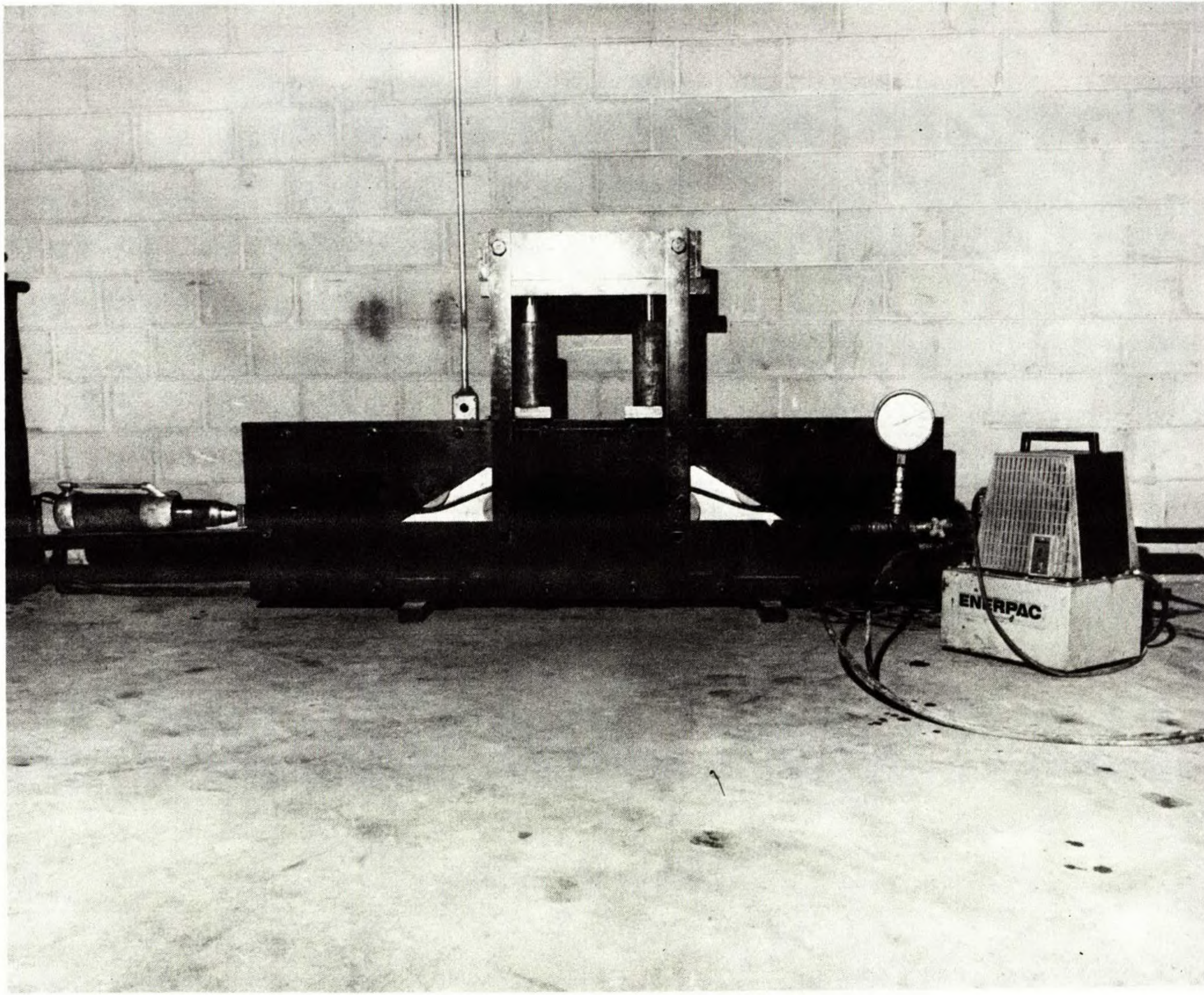


Fig. 13 - Test machine

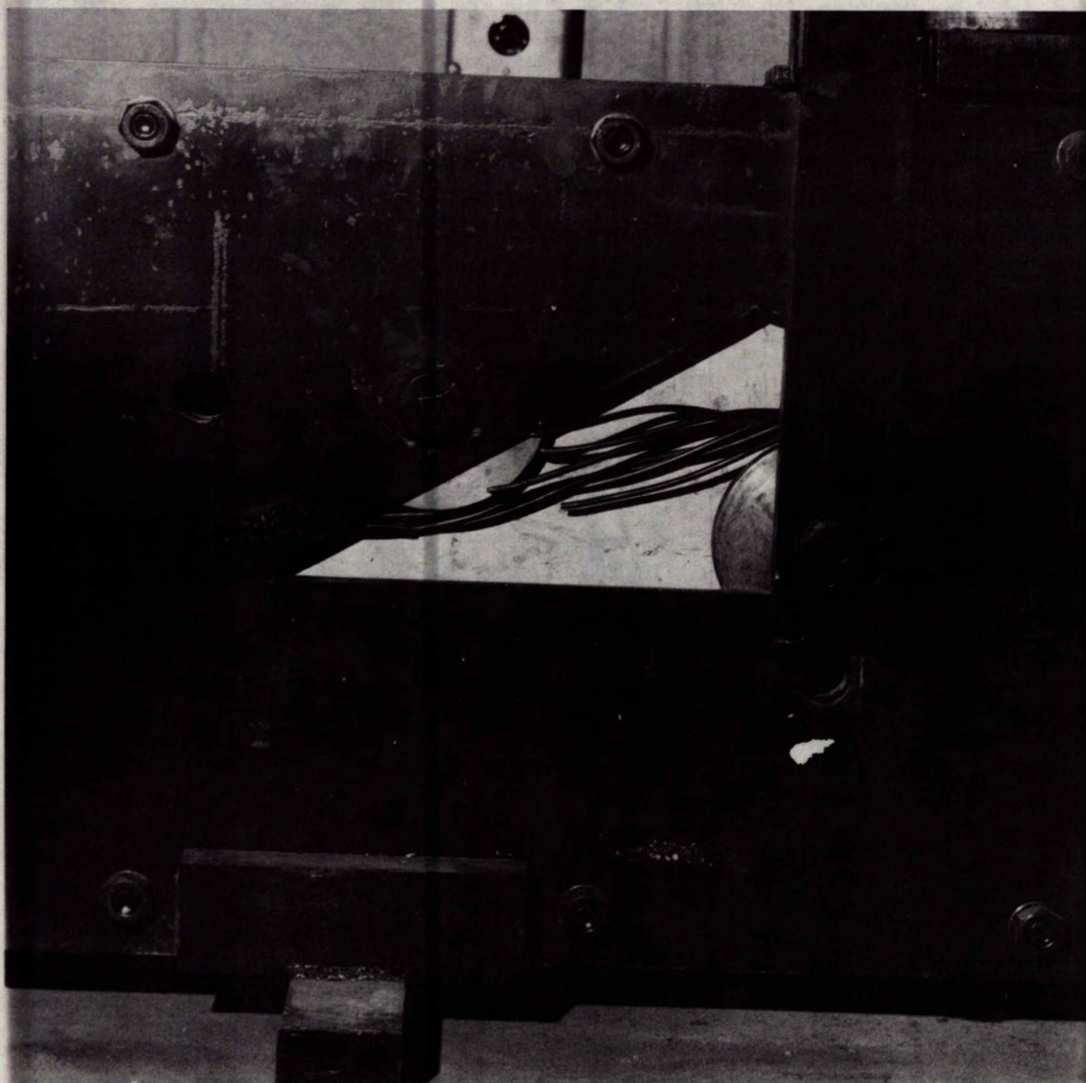


Fig. 14 - Detail showing deflected strand tensioned to failure

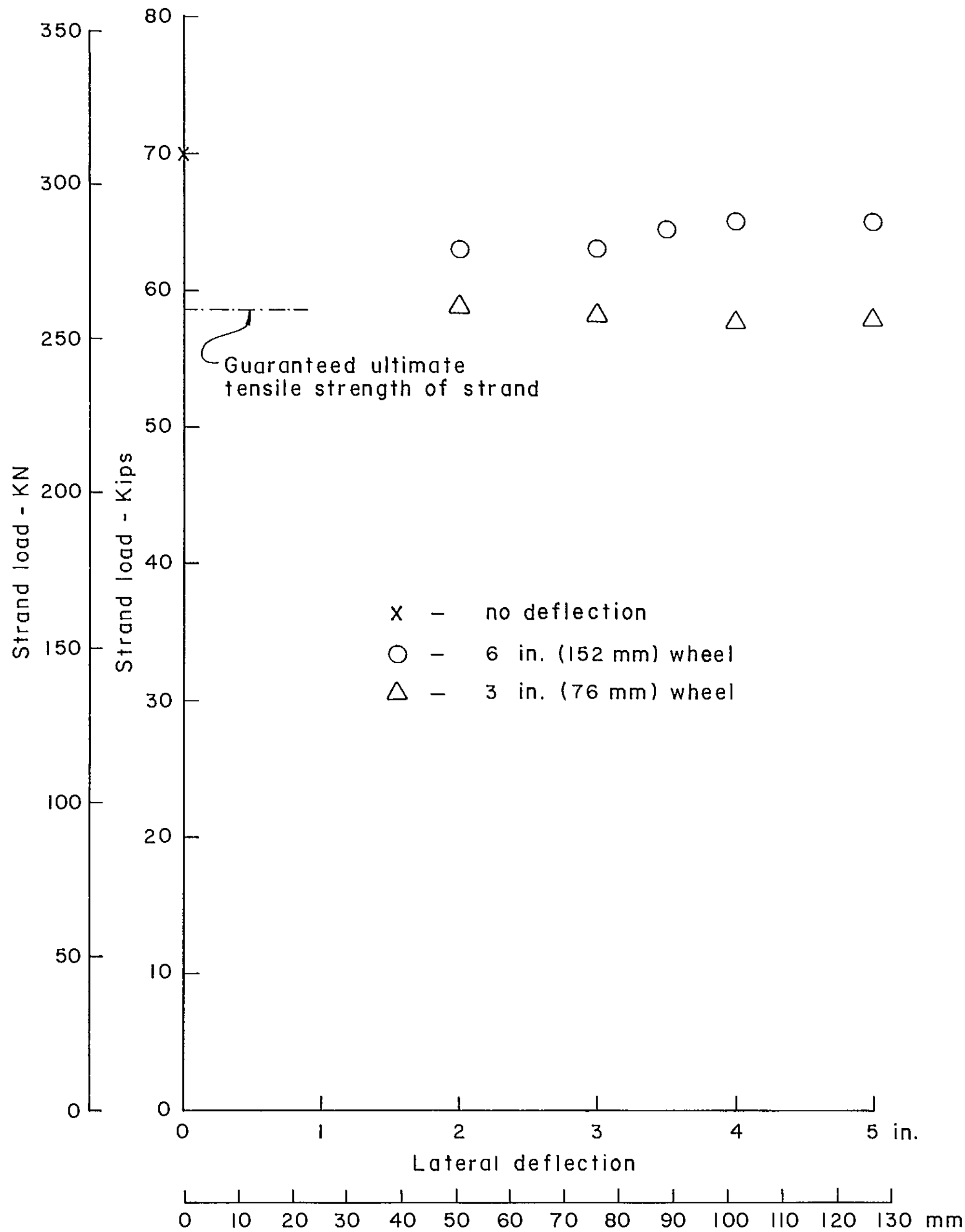


Fig. 15 - Results of tests with lateral deflection mean values with bare strand

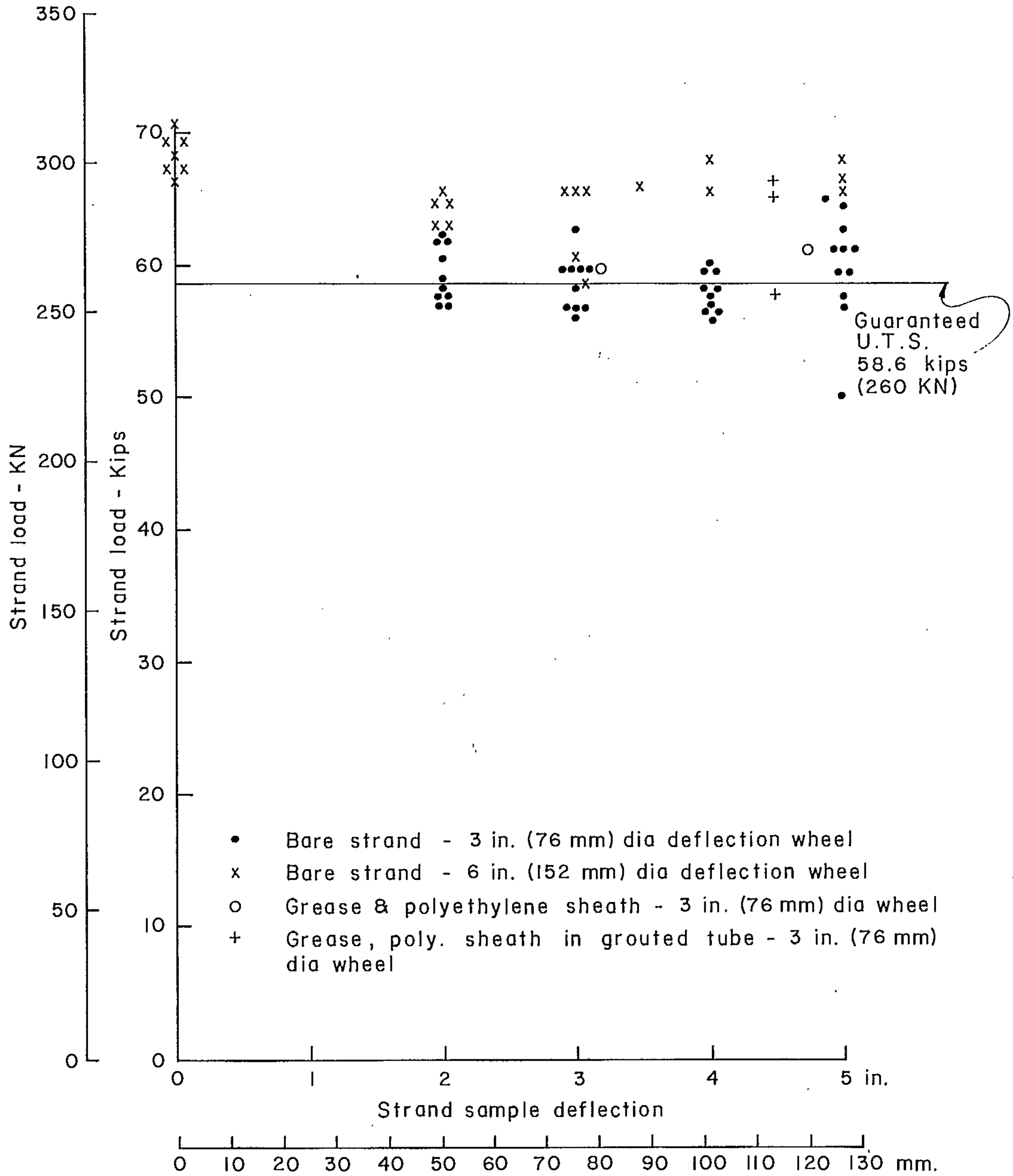


Fig. 16 - Breaking loads of deflected strands — all test results

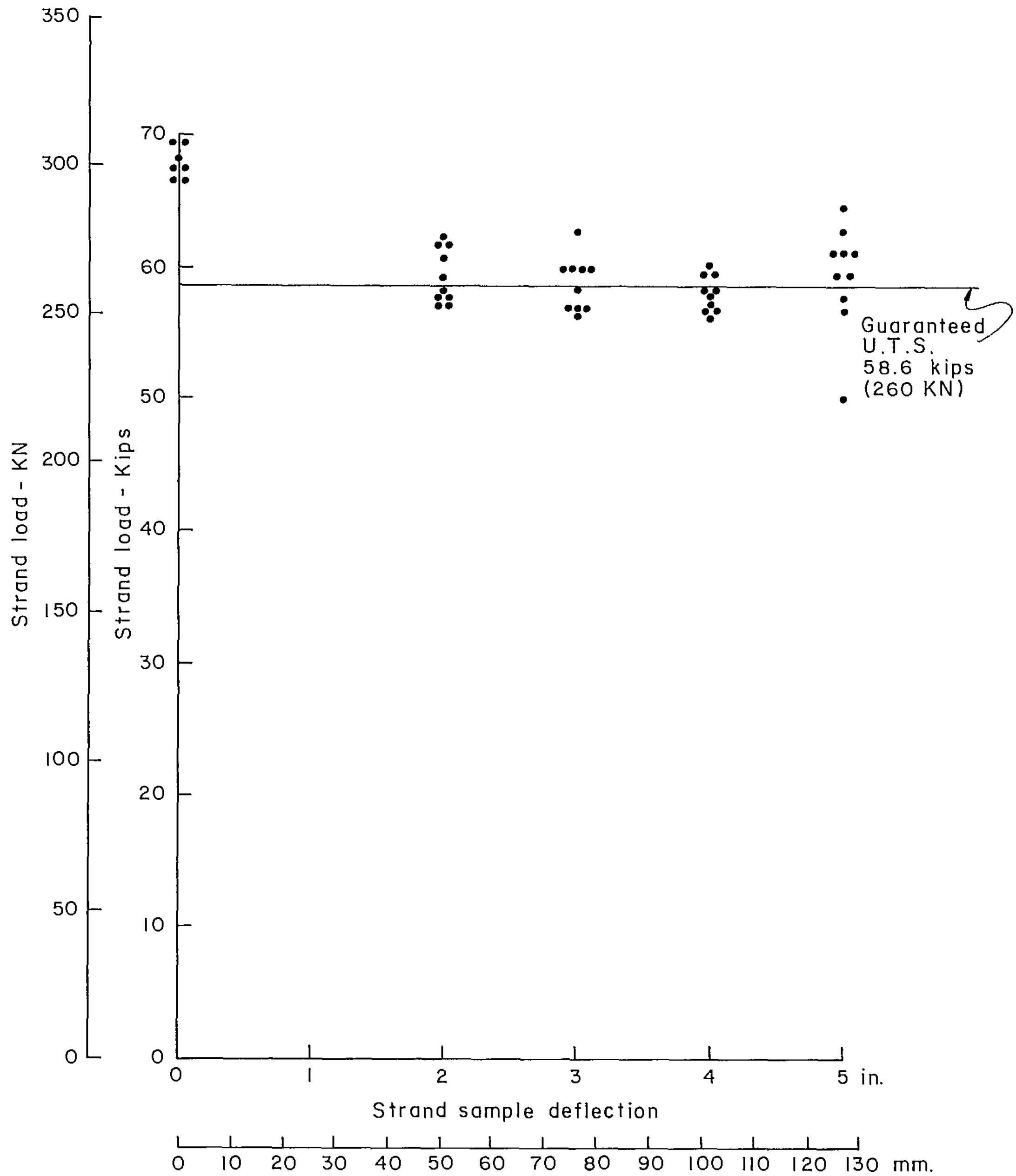


Fig. 17 - Results of tests with 3-in. (25-mm) wheels

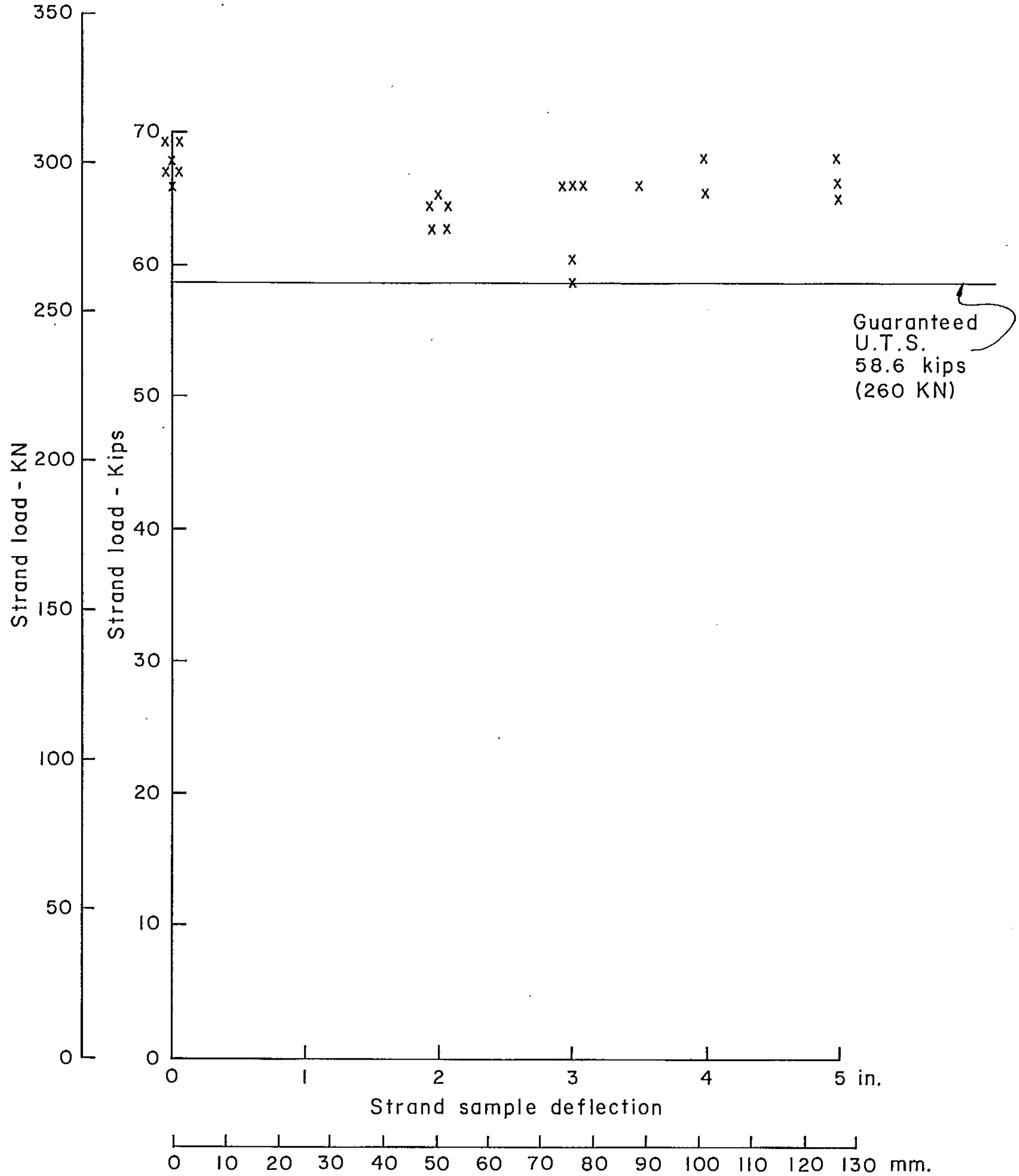


Fig. 18 - Results of tests with 6-in. (50-mm) wheels

Fig. 19 - Photograph of recovered strand showing damaged polyethylene sheath

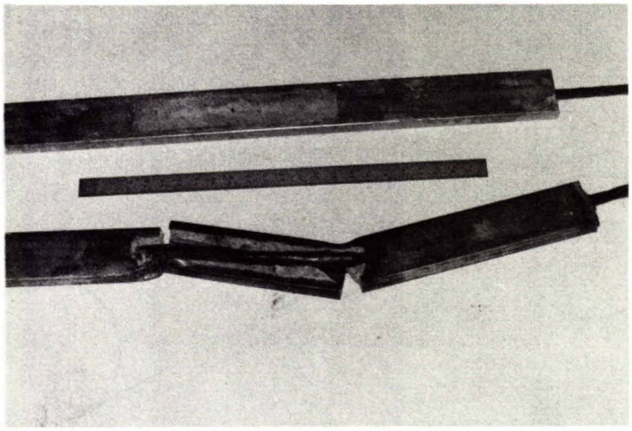


Fig. 20 - Grout-encased strand before and after testing

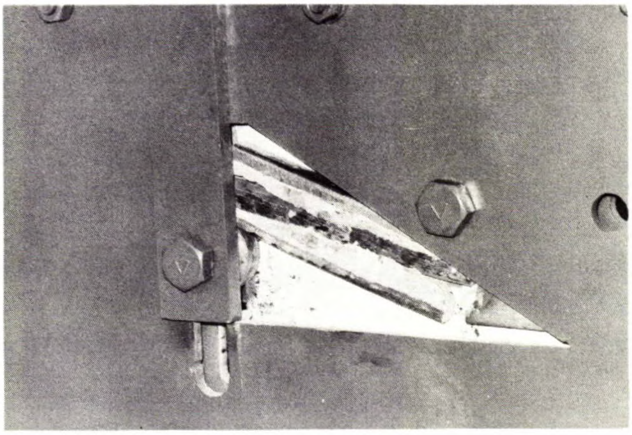


Fig. 21 - Detail showing grout-encased strand during testing

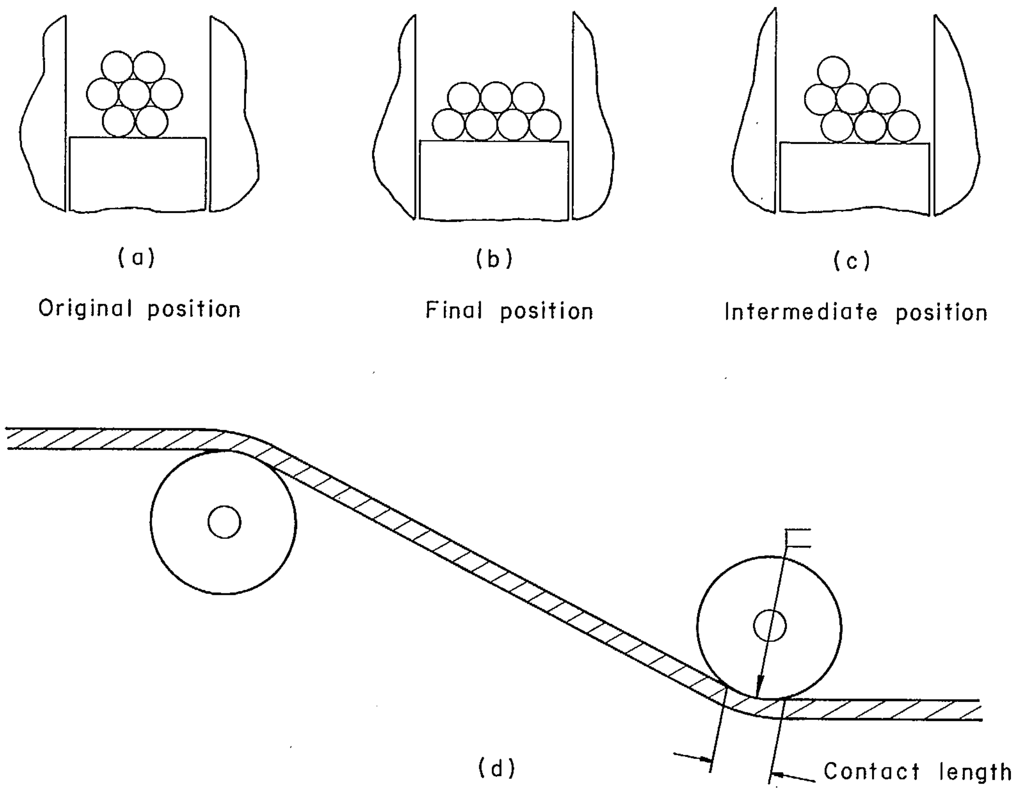


Fig. 22 - Deflected configurations of strand

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