Connections in Cold-formed Sections and Steel Sheets

A. W. Tomà

Jan W. B. Stark

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by J.W.B. Stark[1] and A.W. Tomà[2]

1. INTRODUCTION

Joints are an important part of every structure, not only from the point of view of structural behaviour but also in relation to production cost. It has been shown that, for a structure of hot-rolled sections, about 30 percent of the total costs are directly or indirectly influenced by the connections. There is no reason to assume that this cost item will be much lower for light-weight structures. Economically speaking the joining process will tend to raise the cost. A variety of joining methods are available for light-weight structures. Correct selection is governed by a large number of factors; one is structural behaviour. This paper focuses on this point, though structural behaviour is not the only vital factor. For a long time, even in the fairly recent past, steel structures would be composed of hot-rolled steel members. These members have relatively high plate thicknesses. The last decade has seen cold-formed sections being used more and more. The cold-formed structural members differ from the hot-rolled ones by their reduced thickness the shape of their sections and, by definition, the forming process involved. The structural properties of hot-rolled and cold-formed members differ accordingly. Mechanical properties of the material change during cold-forming. The structural behaviour of members in bending or compression depends on material thickness and section-shape.

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[1] Head of the Department of Steel Structures, Institute TNO for Building Materials and Building Structures, Delft, Netherlands.

[2] Research Engineer at the Department of Steel Structures, Institute TNO for Building Materials and Building Structures, Delft, Netherlands.

Fasteners used for connections in hot-rolled sections are especially developed for use in thicker plates. These fasteners can also be used for thinner plates, but sometimes there will then be some different structural behaviour. Furthermore, special fasteners are available; they are sometimes preferable because of their simplicity in assembling, or just for economic reasons. It is clear that such a new development calls for study and research. The Institute TNO for Building Materials and Building Structures (= IBBC-TNO) carries out research programmes in the field of connections with financial support from the ECSC and from Dutch industry.

2. REVIEW OF OUR RESEARCH PROGRAMMES

Three of IBBC-TNO's programmes are related to connections in cold-formed sections and steel sheets:

a) Welded connections in cold-formed sections

The objective of this programme is to draft structural requirements for welded connections in cold-formed sections. At this moment it is not yet possible to give results because the programme has just started.

b) Mechanical connections in cold-formed sections

This programme was completed in 1976 (ref. [7]). A summary of its results will be given in section 4 of the present paper. The aim of the programme was to draft structural requirements for mechanical connections in cold-formed sections. The programme was limited to bolts, screws and rivets. Three countries contributed in the execution of the inherent research, namely: United Kingdom, Austria and the Netherlands. Figure 1 gives an outline of the organisational structure and shows the division of work between the contributing countries. The results of the inventory and survey of literature are presented in [1]. A review is given of a number of types of connections or fasteners, plus test results from manufacturers and research institutes in Germany, Sweden (Baehre) and the United States (Winter, Peköz). Using this information, a standard-test procedure was formulated. In all three countries, a serie of tests was carried out according to this standard procedure (refs [2], [3], [4] and [5]). From these tests, design rules have meanwhile been developed. The results of tests on simple standard specimens have been used to calculate complete sonnections. These connections have been tested, too, and the experimental and theoretical results compared (ref. [6]). Finally, the results have been used to draft the relevant clauses for the European Recommendations for cold-formed sections.

c) <u>Fastening of steel profiled sheets for walls and roofs on steel structures</u>

This programme includes the following items:

I Inventory and literature survey.

II Requirements for connections

- diaphragm action of sheets;
- the influence of repeated loads on the failure strength of connections;
- the influence of secondary forces on connections such as accidental fixing moment in the connection and temperature influences.

III Drafting of recommendations.

Ref. [8] gives the results of the inventory of the field of fastening of steel sheets.

Ref. [9] describes the influence of secondary forces on fasteners caused by accidental fixing moment in the connection. Here the background of the item will be given; section 5 of the present paper treats it more in detail.
Two aspects are important when a fastening system for sheets (loaded perpendicularly to the sheet's plane) has to be designed:

1) Determination of the design strength of connections.

2) Determination of the forces in connections during loading.

As to 1): In view of the large number of types of fasteners and sheets, the report proposes that the characteristic strength of connections should be determined according to the "European recommendations for the testing of connections in profiled sheeting and other light gauge steel components", which is drafted by Committee 17 of the European Convention for Constructional Steelwork (ECCS).

As to 2): To determine the sheet profile in a wall or roof, the sheeting is considered as a simply supported statical system (Fig. 2). But in reality, at the supports, the sheets are more or less clamped. For the design of the sheets it is realistic and safe to neglect the clamping effect. For the design of connections (fasteners), this simplification can be dangerous. The tension force in the fastener under uplift load differs from the value which follows from the caluclation of the sheeting. Even under downward load, the fastener will be loaded in tension (Fig. 3). Deflection of the sheet will make it contact the support at places A or B. This will cause accidental fixing moment of the sheet, which generate extra tension forces in the fasteners.

3. GENERAL REQUIREMENTS FOR CONNECTIONS

The function of a fastener may involve a number of considerations. The main groups of requirements can be summarized as follows:

Structural requirements
- strength
- stiffness
- deformation capacity

Non-structural requirements - economy
- . price
- . assembling
- . demountability
- . number of components
- . required skill

- durability
- . corrosion
- . temperature
- . sensitivity to moisture
- . sensitivity to chemicals

As far as the non-structural group is concerned, the price of a fastener is important. Even more important is the pertinent assembling procedure. Sometimes there may be a requirement for a particualr assembly to be taken apart in its life. The degree of difficulty in the disassembly and reassembly operation can dictate the type of fastener which is most appropriate. Also of importance can be the number of components, and last but not least the required skill of workmen. A large percentage of fasteners is fixed in a shoddy way. For some fasteners this implies a drastic reduction in strength.

Choice of fastener material and finish are largely dictated by environmental conditions in which the fastener will have to operate. The effects of temperature, corrosion resistance, humidity, chemicals are factors which have to be borne in mind. In this respect, fire resistance can be of extreme importance.

In the research programmes of IBBC-TNO, attention is focussed on three structural requirements explained below. Thick-walled structural steel and also the light gauge steel sheet clearly possess such properties as strength, stiffness and deformation capacity (Fig. 4). Therefore, this material is suitable for use in structures. Ancillary parts of structures, e.g. connections, ought to have the same properties (Fig. 5). Evidently, this applies to their strength and stiffenss. It is less known, however, that their deformation capacity should also meet certain requirements.

- Strength

  For each type of connection, a characteristic strength can be determined by theoretical or experimental research. This strength can be influenced by the choice of section, and by the type and number of fasteners.

- Stiffness

  Stiffness of a connection is important because it determines the stiffness of the whole structure, or that of its components. Stiffness can also influence the forces in a connection. This is for instance the case for connections of lateral bracings to purlins and bracings in nonsway frames. The stiffness of a connection also determines the distribution of the loads. Stiffness has to be considered in relation to the whole structure. It is important that the stiffness of a connection is known, so that the stiffness of the whole structure can be calculated.

- Deformation capacity

  Deformation capacity of a connection is important. A connection with no deformation capacity can cause a brittle fracture of a structure or element. This primarily applies to static indetermined structures, where such influences as settling and fluctuating temperatures normally are not included in a design calculation. Local overloading can be eliminated, if the connection can deform sufficiently. In the case of static determined structures, the deformation capacity of each individual fastener can be important, if more fasteners are used. Figure 6 gives two examples which demonstrate the importance of this requirement.

  A failure mode with little strain capacity has disadvantages, if more fasteners have been placed in a row in the direction of load. In a calculation, the force is divided into equal parts applied to each fastener. Theoretically this is not correct. However, this assumption may be used if plastic redistribution can take place.

The same requirement is necessary for connections in trusses, which are calculated as trusses with pin-ended joints. It is well known that secondary stresses are always introduced. A simplified design method is therefore permitted, if the connections can deform plastically in order to limit the influence of the secondary stresses.

4. MECHANICAL CONNECTIONS IN COLD-FORMED SECTIONS

About 1200 shear tests have been carried out on simple standard specimens with normal bolts, friction grip bolts, screws and rivets. Figure 7 shows a review of the design shear strengths of these fasteners. For the screws these design strengths will now be treated in some detail.

The strength of a connection is dependent on the failure mode. For screws, five failure modes can be distinguished.

Failure type 1: Inclination of the fastener (Fig. 8).

This failure mode appears when the thickness of the connected sheets is relatively small. The design equation given in Figure 7 is an empirical one:

$$P_v = k_s\ (d + 10)(t_1^2 + 0.22)\ \sigma_e$$

where: P_v = design strength of a connection for the failure mode inclination of fastener, in N

d = diameter of screw, in mm

t_1 = thickness of sheet in which is screwed, in mm

This is always the thickest sheet

σ_e = calculation value of yield stress of sheet material in N/mm^2

k_s = empirical constant according to:

$$k_s = 0.156\ \left(\frac{t_1}{t_2} - 1\right)^2 + 0.35 \text{ with } k_s \leq 0.7$$

t_2 = thickness of sheet against which head of screw is placed.

Failure type 2: Edge failure.

The screw remains perpendicular to the direction of the applied load. The shear planes are parallel and a distance apart that is equal to the screw diameter.

A design equation must have the following form:

$$P_u = 2t\, c_1\, \tau_e$$

where: P_u = design strength of connection for edge failure

t = thickness of thinnest sheet

c_1 = edge distance in direction of load

τ_e = calculation value of shear stress. According to the tests, $\tau_e = 0.5\,\sigma_e$

Failure type 3: Hole bearing (Fig. 9)

A bearing failure of the sheet material, with shearing-bearing action along two distinctly inclined planes, caused the sheet material to pile up in front of the screw. For this failure mode, the ultimate load is independent of the edge distance.

A design equation must have the following form:

$$P_s = \eta\, d\, t\, \sigma_e$$

where: P_s = design strength of connection for failure mode hole bearing

d = diameter of screw

t = thickness of thinnest sheet

σ_e = calculation value of yield stress of sheet material

η = empirical factor. The tests showed that for screws this factor can be taken as 2.1

Failure type 4: Sheet failure (Fig. 10)

Transverse tearing takes place at the net section and is at right angles to the load direction. From the tests it appeared that for screws the design strength can be calculated by multiplying the net section with the calculation value of the yield stress of the sheet material. A different equation is given for bolts because the strength of a bolted connection is larger, and in more cases this failure mode will appear. In the combinations where the equation for bolts and that for screws are different, another failure mode will occur in the case of screws (so not sheet failure).

Failure type 5: Shear failure of screw (Fig. 11)

A design equation must have the following form:

$$P = A\,\tau$$

where: P = shear strength of screw

A = area of section of screw ($\frac{1}{4}\pi d^2$)

τ = calculation value for shear strength of screw material

It is difficult to determine τ, but in most national codes the ultimate torque moment of screws is given, M_B. If a full-plastic shear stress distribution is assumed at failure, then $M_B = \frac{1}{12}\pi d^3 \tau$.

$$\longrightarrow P = \frac{1}{4}\pi d^2 . \frac{M_B}{\frac{1}{12}\pi d^3} = 3\,\frac{M_B}{d}$$

A factor of safety must be taken between the ultimate strength and the design strength and, furthermore, the number of shear planes must be taken into account.

$$\longrightarrow P_a = \frac{2\,n\,M_B}{d_c}$$

where: P_a = design strength of screw for failure mode shear of screw

n = number of shear planes

M_B = ultimate torque moment of screw

d_c = diameter of core of screw

The load-deformation diagram shown in Figure 11 illustrates the small deformation capacity. Therefore this failure mode is not permitted: the value of P_a has to be 25% larger than any other design strength.

It is appropriate to consider how the test results of standard test specimens can be used to calculate the properties of complete connections. From the actual measured load-deformation diagram, a bi-linear design diagram is derived; it is shown in Figure 12a. The values of stiffness and strength are now known. For a T-connection loaded in bending and shear as shown in Figure 12b, a moment-curvature relation can be calculated (Fig. 12c). This type of calculation was affected for a T-connection between cold-formed top-hat sections and channel sections. Figure 13 gives an impression of the test set-up of a complete connection. Figure 14 shows the results for one case. It can be seen that, under working load, agreement between calculation and test result is good. Furthermore there is a relatively high reserve of strength. This is caused by redistribution of forces between the fasteners after reaching the ultimate load in the most heavily loaded fastener. In this case, the failure mode was inclination of the fastener (Fig. 15a). Figures 15b and 15c show other modes of failure, namely bearing failure and shear failure. The same kind of complete connection tests has been carried out with rivets and bolts. This has given similar agreement as that for screws.

As to connections with high-strength bolts loaded in shear, the following be said. The Austrian research activities (ref. [2]) has resulted in some conclusions.

- The applied sections are made of sheets with small thickness and they are galvanized. Due to the low amount of forces that can be transmitted in these sections by friction, it is not appropriate to design galvanized high-strength bolted connections in cold-formed sections as friction-type connections. Moreover, tests of 70 days have shown that prestress losses occur in the bolts, obviously caused by creeping of the zinc layer. They amount to an average of 20% for bolt M6, 10% for M10 and 4% for M16. Particularly in the smaller bolts, M6 and M10, influences occur in the course of time which would further reduce the forces that can be transmitted by friction.

- Tightening of high-strength bolts to the initial stressing forces asspecified in the European rules for the application of high-strength tightened bolts, 1971, has produced no perceptible increase in the load-carrying capacity of the connections in comparison with hand-tightened connections.

5. TENSION FORCES IN FASTENERS BETWEEN SHEETS AND UNDERSTRUCTURE LOADED BY WIND SUNCTION OR TRANSVERSAL LOAD

In Section 2c we already discussed the existance of extra tension forces in the fasteners, as caused by accidental fixing moments of the sheet. These extra tension forces are called prying-forces.
The value of the prying-forces depends on the following parameters:

- Flexibility of sheet in span direction
- Flexibility of sheet in cross section near fastener
- Diameter of head of fastener or diameter and stiffness of washer
- Distance between fastener and contactpoints A and B
- Place of fastener in cross section of sheet
- Torsional flexibility of support.

All these aspects can be jointly expressed in a prying-force factor. The prying-force factor is defined as: the factor with which the support reaction of the simply supported statical system has to be multiplied to obtain the tension force in the connection. In ref. [9], formulae are derived to determine prying-force factors for sheets on two or three supports under uplift or downward load.
For determination of the prying-force factor it was necessary to acquire information about the deformation of the sheet near the fastener. Therefore, 44 detail tests have been carried out in which sheet profile and fastener were varied. It was possible to derive a formula to predict this flexibility.

For checking the theory, two large-scale tests have been carried out. It appeared that theory and tests are in good agreement.

For real structures, the prying-force can have a value of 3. Thus far, connections have been designed with the simply supported system, their factor of safety usually varies between 2 and 3. In some cases the factor of safety will be fully used by the simplification of the calculation model, because of the prying-force.

As was discussed before, it is important to have a calculation model for the design of the fastening system of steel sheets for loads perpendicular to the sheets. For the design of the sheets, the model according to Figure 2 is correct. For the design of the fastening system for a sheet on two supports under downward load, the calculation model according to Figure 16 is suitable.

The calculation model of Figure 16 has been based on the following assumptions:

a) The upper flange of the support does not rotate.

b) The compression force between sheet and support is concentrated in one line: pressure-line (places, A and A' in Fig. 16). The cross section of the sheet near the pressure-line does not deform.

c) The tension force in a fastener is dependent on the deformations of the cross section of the sheet near the fastener. It is assumed that the deformation of the sheet can be characterized by a spring constant c according to:

$$P = c.\delta$$

where: P = tension force in spring

δ = elongation of spring caused by P

c = spring constant

According to the calculation model of Figure 16, the force in the spring is equal to:

$$F = \frac{1}{2}\, q\, L\, \frac{1}{n}\, \frac{a_1(L^3 - 6a_1^2\, L - 3a_1^3)}{\frac{12\,EI\,L}{n\, c} + 12a_1^2\, L(3L + 2a_1)} \quad \text{------} \quad (5.1)$$

where: F = tension force in spring

q = load on sheet per unity of area

$\left.\begin{matrix} L \\ a_1 \end{matrix}\right\}$ = see Figure 16

n = number of fasteners per unity of length of support

EI = stiffness of sheet per unity of width

c = spring constant per fastener

Equation (5.1) can be rewritten as:

$$F = \frac{R}{n} f \qquad (5.2)$$

where: R = support reaction of a simply supported beam

f = prying-force factor

Because in reality a_1 is small in comparison with the span, a_1 shall be neglected in comparison with L.

Furthermore, the following definitions are introduced:

$$\alpha_1 = \frac{a_1}{L} \qquad (5.3)$$

$$S = \frac{6\,EI}{n\,c\,L^3} \qquad (5.4)$$

The equation for the prying-force factor then is:

$$f = \frac{a_1}{2S + 6\alpha_1^2} \qquad (5.5)$$

Figure 17 shows the relation between f and α_1 for certain values of S.
It will be seen that the prying-force factor will have a maximum for a certain value of α_1. Because in practice the value of a_1 is not prescribed (between certain limits the fastener can be placed everywhere in the flange of the support in the direction of the sheet-span), a design rule has to be based on the maximum value of the prying-force factor, with a check afterwards if the value of α_1 belonging to f_{max} can occur in practice.

The maximum value of f will apply, when:

$$\frac{df}{d\alpha_1} = 0 \quad \text{-------------------} \quad (5.6)$$

From (5.5) and (5.6) follows:

$$\alpha_1 = \sqrt{\frac{S}{3}} \quad \text{------------------} \quad (5.7)$$

$$f_{max} = \frac{1}{4\sqrt{3S}} \quad \text{-----------------} \quad (5.8)$$

When for instance S = 0.005, then according to equation (5.8): $f_{max} = 2.04$ and according to equation (5.7): $\alpha_1 = \frac{1}{24.5}$. When the span of the sheet L = 4000 mm, and the width of the flange of the support is 300 mm, then α_1 can vary from 0 to $\frac{1}{13}$ and it is possible that f_{max} appears. But when the width of the flange is 100 mm, α_1 can vary from 0 to $\frac{1}{40}$ and f_{max} cannot appear. According to Figure 17, the extreme value of f appears for $\alpha_1 = \frac{1}{40}$. Substitution of this value of α_1 in (5.5) gives:

$$f_{ext.} = \frac{\frac{1}{40}}{2 \times 0.005 + 6(\frac{1}{40})^2} = 1.82.$$

From all this it can be concluded that if α_1 has a value smaller than α_1 for which f_{max} appears, one has to substitute the maximum real value of α_1 in equation (5.5) in order to determine the extreme real value of the prying-force factor.

In ref. [9], the influence of simplifications to derive (5.5) from (5.1) is analyzed. From this we can conclude that, for downward load, the simplifications give safe results but for uplift load a correction factor is necessary.

From the preceding part of this section, the importance of the spring constant will be obvious. To obtain information concerning this point, 44 detail tests according to Figure 18 have been carried out, in which type of fastener and sheet profile are varied. The test set-up is according to ref. [10]. The deformation Λ of the connection is defined as: the displacement of the head of the fastener with regards to the webs of the sheet.

$$\Delta = \delta_2 - \frac{\delta_1 + \delta_3}{2} \quad \text{----------} \quad (5.9)$$

where δ_1, δ_2 and δ_3 are the deflection at 1, 2 and 3 respectively in Figure 18b. As spring constant c is defined arbitrarily: 0.6 times the ultimate load (working load) divided by the deformation Δ belonging to that load.

$$c = \frac{0.6\ P_u}{\Delta_{0.6\ P_u}} \quad \text{---------------} \quad (5.10)$$

Figure 19 gives a load-deformation curve which is representative for all the detail tests. Note that after ultimate test-load there is a little deformation until failure. For this reason it is realistic to consider a fastening system as having failed when the force in a fastener under design load is equal to the characteristic strength of a connection. From the test results, an empirical formula is derived to determine the spring constant for fasteners with a head which is stiff in comparison with the sheet thickness.

$$c = 0.0223\ E\ t\sqrt{\frac{t}{b_3 - k}} \quad \text{---} \quad (5.11)$$

where: c = spring constant per fastener

E = modulus of elasticity

t = thickness of steel of sheet

b_3 = flat part of flange of sheet through which fastening is done

k = diameter of stiff head of fastener

Figure 20 shows the comparison of equation (5.11) with test results. From the figure it appears that in all cases the empirical formula gives higher values for c than the tests. This is done for reasons of safety; a high spring constant gives a high prying-force factor.

To verify the theoretical formulas and their assumptions (including the definition of spring constant) a full-scale test has been carried out (Fig. 21). The test specimen failed by exceeding the bearing capacity of the sheet in the midspan cross-section because the fasteners were overdimensioned.

Figure 22 shows the relation between the applied test load and the tension forces in the fasteners. Furthermore, calculated values according to equation (5.1) are given. These calculations are based on a value of c which is determined through the detail test. For the moment of inertia of sheet I, two different calculated values are taken:

- The moment of inertia of the full cross-section of the sheet.
- The moment of inertia belonging to the real bending moment at midspan. The moment of inertia is calculated according to the Dutch recommendations for the calculation of profiled sheets.

Because there is only a small difference in these two sets of calculation results, for reasons of simplicity we take the non-reduced moment of inertia for calculation of the prying-force factor.
From Figure 22 it appears that, if the supports do not rotate, the formulas to determine f and the defenition of c give calculation values which are in good agreement with test results.

6. CONCLUSIONS AND DISCUSSION

For a fastening system in cold-formed sections and steel sheets, two aspects are important:

1) determination of the design strength of connections
2) determination of the forces in connections during loading

At IBBC-TNO, three research programmes related to these items are carried out. This paper deals with the mechanical connections in cold-formed sections and the fastening of steel profiled sheets for walls and roof on steel structures.
The programme of mechanical connections in cold-formed sections has already resulted in calculation rules for European recommendations. The objective of the other programmes is also to yield calculation rules for European recommendations.

APPENDIX I - REFERENCES

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APPENDIX II - NOTATION

a_1 = edge distance of a fastener in a support

$\alpha_1 = \frac{a_1}{L}$

b_3 = flat part of flange of sheet

c = spring constant

c_1 = edge distance of fastener in direction of load

d = diameter of fastener

d_c = diameter of core of screw

EI = stiffness of sheet per unity of width

F = tension force in fastener

f = prying-force factor

k = diameter of head of fastener

L = span of sheet

M_B = ultimate torque moment of screw

n = number of shear planes in fastener or
number of fasteners per unity of length of support

P = design strength of connections loaded in shear load

q = load per unity of area

R = support reaction of a simply supported beam

$S = \frac{6\ EI}{n\ c\ L^3}$

t = thickness of steel sheet

σ_e = calculation value of yield stress

RESEARCH PROGRAMME

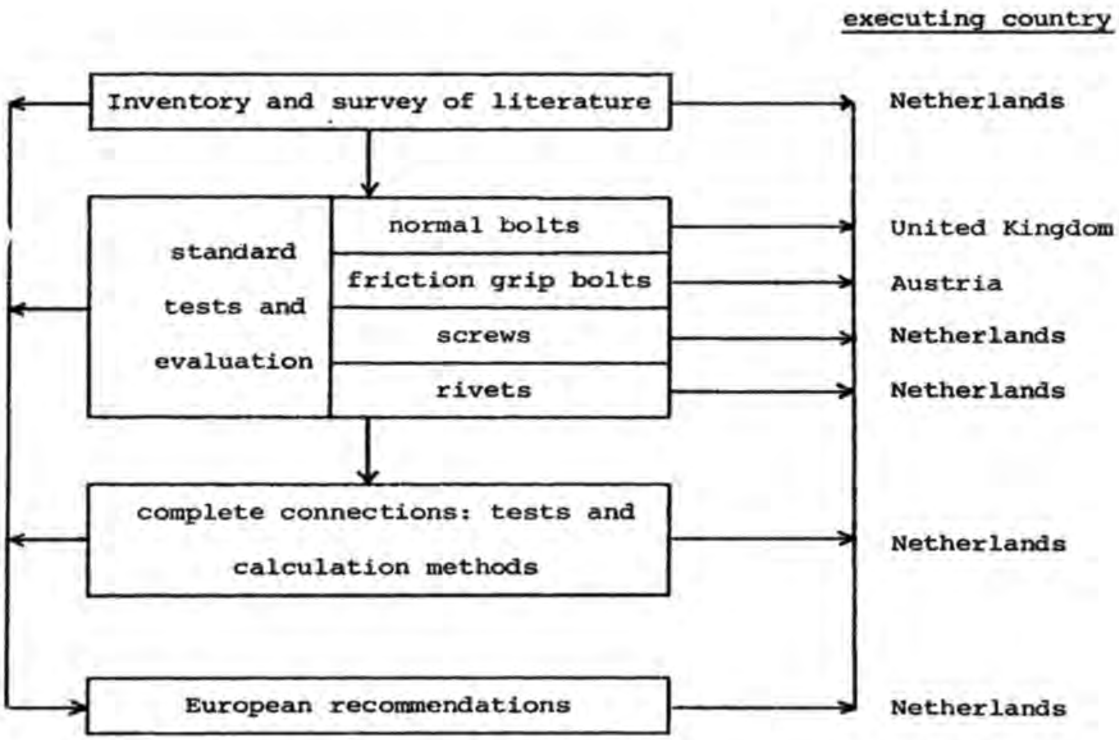


Fig. 1

Organizational structure of the research programme for mechanical connections in cold-formed sections

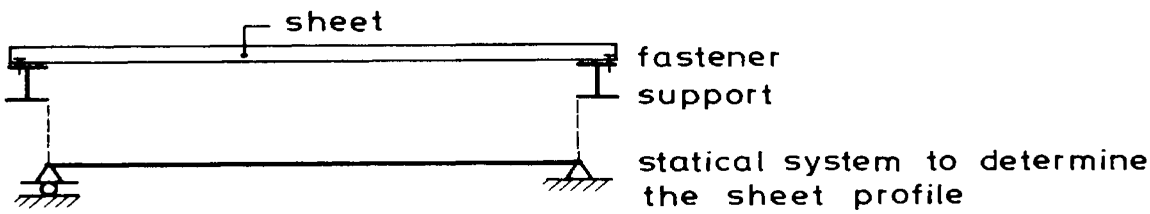


fig. 2

Fig. 3 The support of a sheet in detail

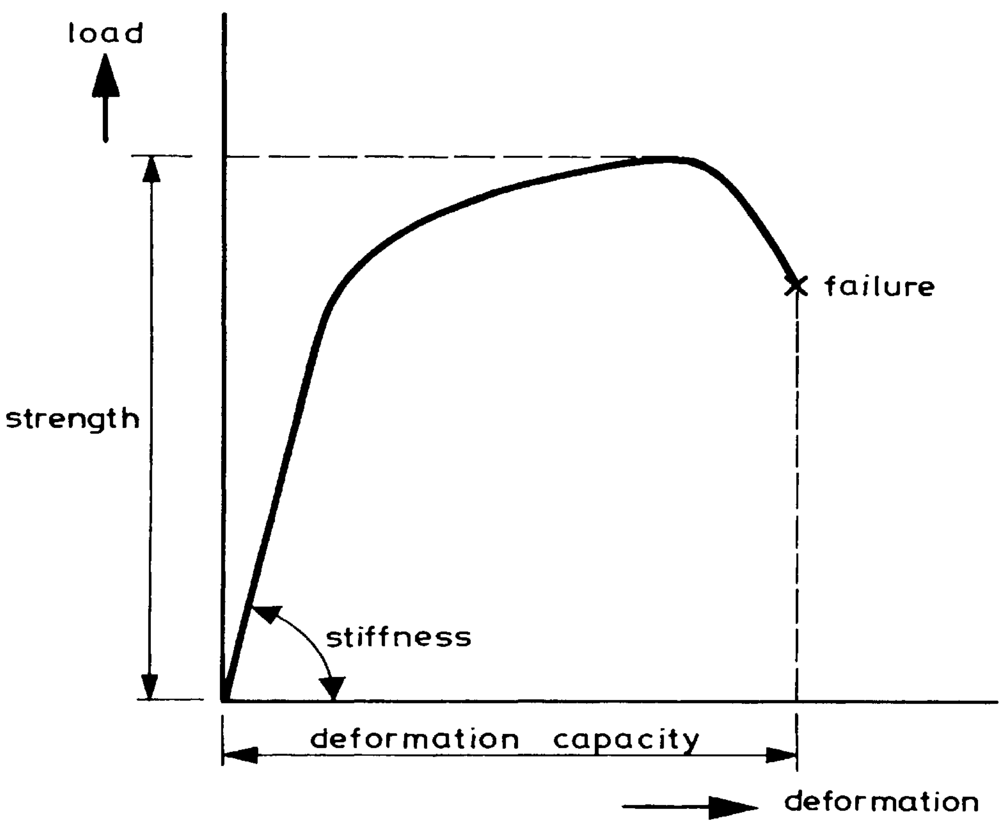
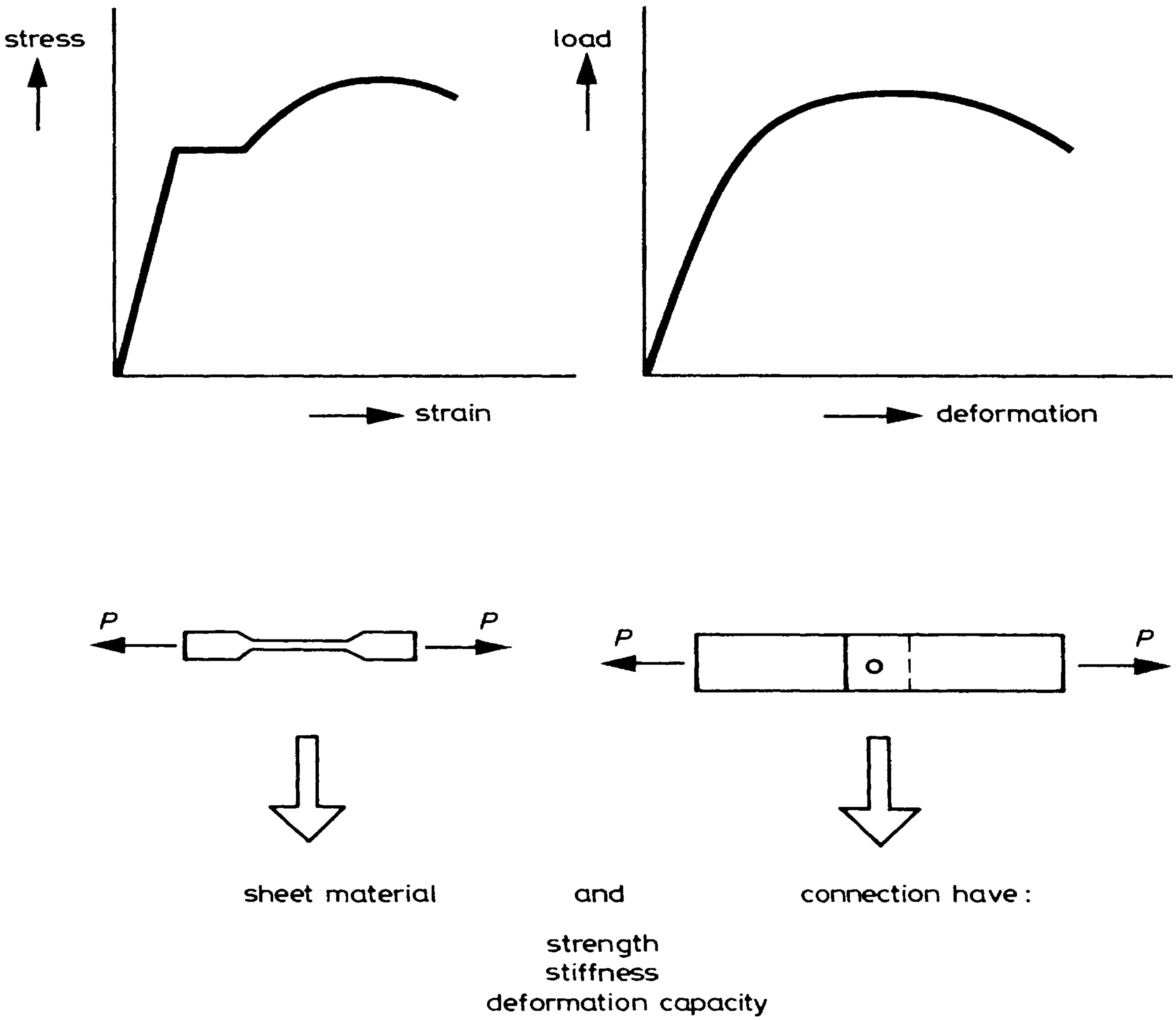


fig. 4


stress
strain
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sheet material
and
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fig. 5

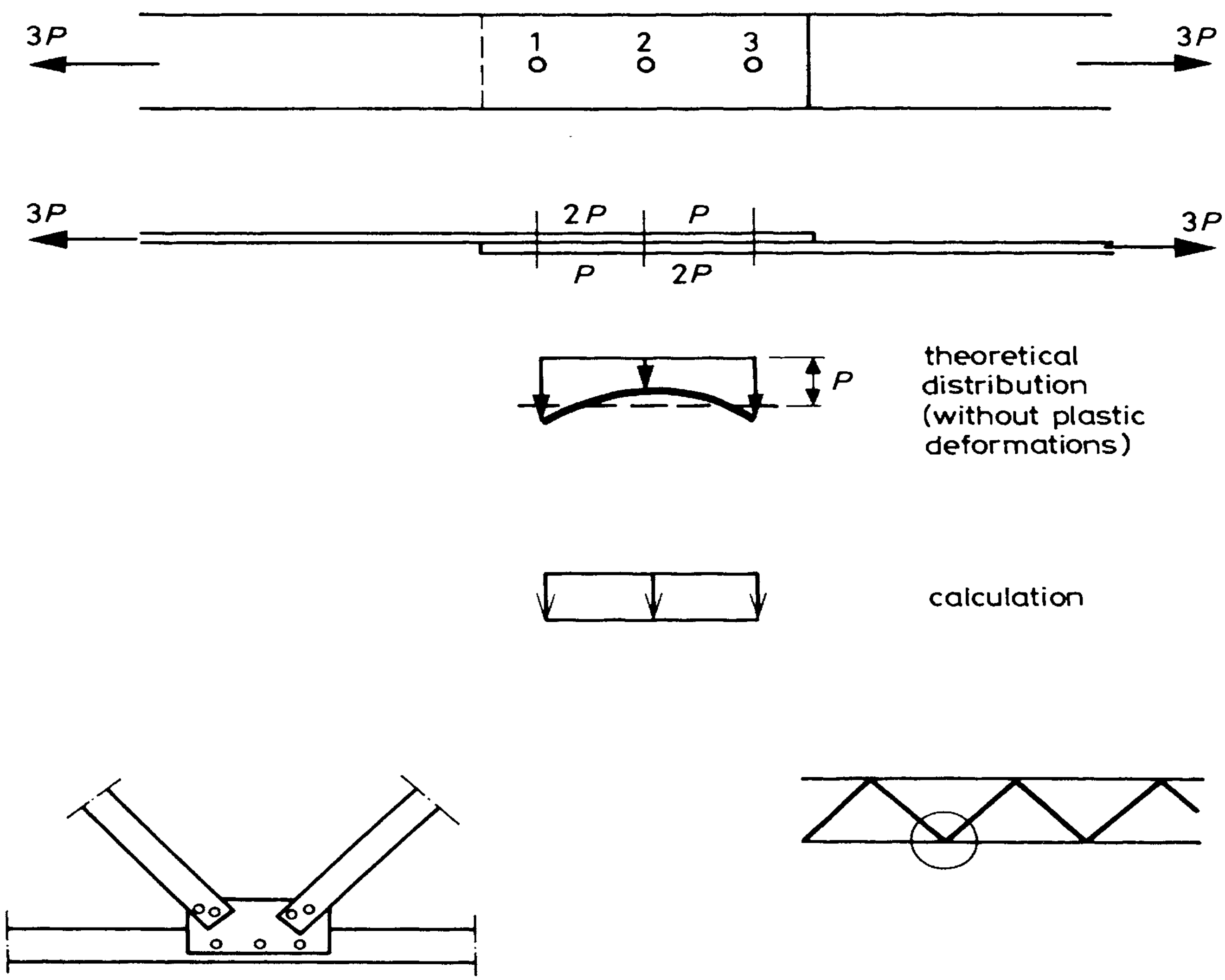
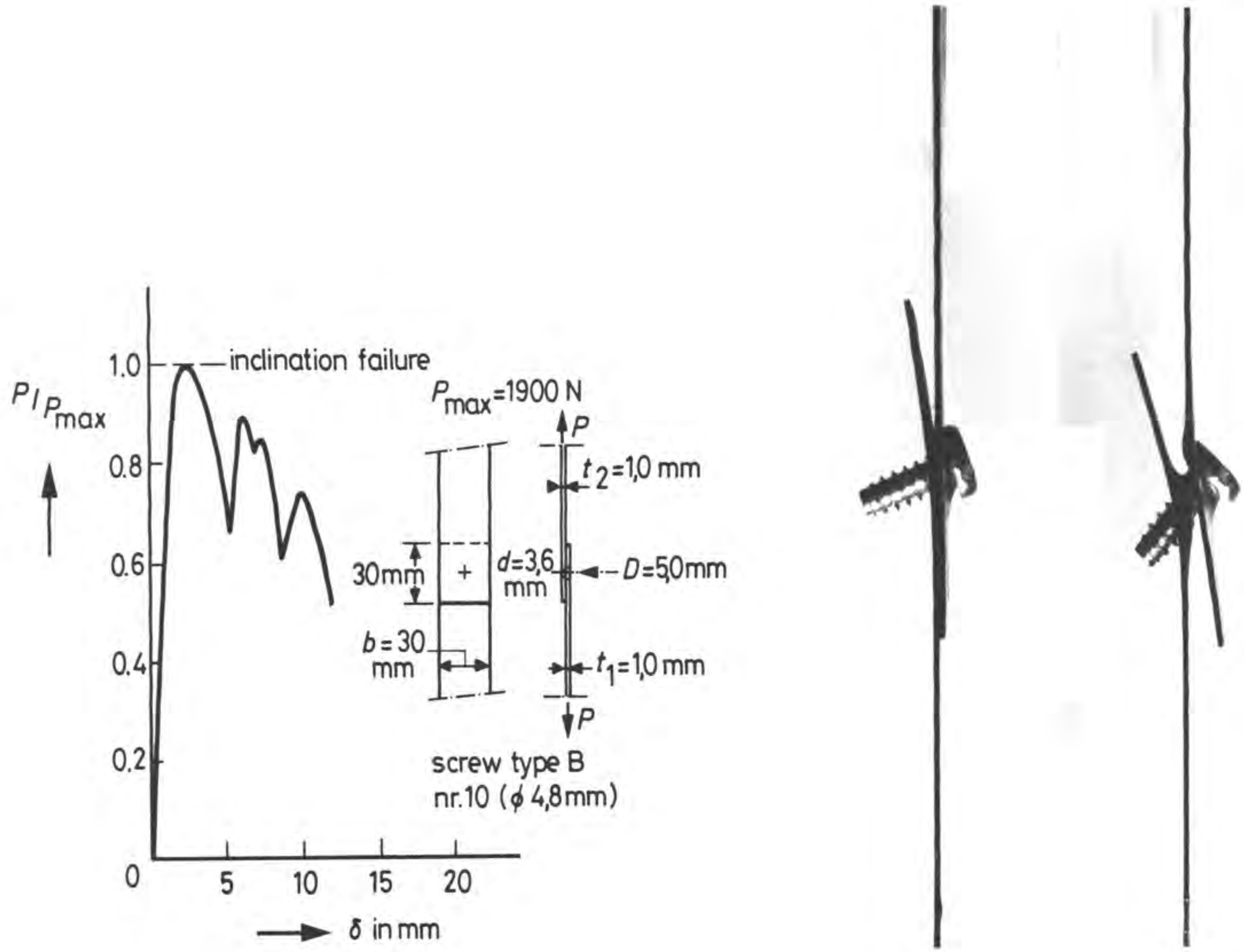

3P
1
2
3
3P
3P
2P
P
P
2P
3P
P
theoretical distribution (without plastic deformations)
calculation

fig. 6

fastener	failure modes and design shear strengths in N				
	inclination of fastener	edge failure	hole bearing	yielding of net section	shear of fastener
bolts and high-strength bolts	not possible	$P_u = 2t\ c_1\ 0.5\ \sigma_e$	$P_s = 2.1\ dt\ \sigma_e$	$P_n = A_n\ \sigma_n < A_n\ \sigma_e$ $\sigma_n \leqq (1 - 0{,}9r + 3r\ \frac{D}{S})\sigma_e$	$P_a = 0.7\ n\ A_s\ \sigma_e$ $P_a \geqq 1.25$ the smallest of other failure modes
screws	$P_v = k_s (d + 10)(t_1^2 + 0.22)\sigma_e$ $k_s = 0.156(\frac{t_1}{t_2} - 1)^2 + 0.35$ $k_s \leqq 0.7$	$P_u = 2t\ c_1\ 0.5\ \sigma_e$	$P_s\ \ 2.1\ dt\ \sigma_e$	$P_n = A_n\ \sigma_e$	$P_a = \frac{2nM_B}{d_c}$ $P_a \geqq 1.25$ the smallest of other failure modes
	$P_v = k_r (d + 5)(t_1^2 + 0.22)\sigma_e$ $k_r = 0.111(\frac{t_1}{t_2} - 1)^2 + 0.65$ $k_r \leqq 0.9$	$P_u = 2t\ c_1\ 0.5\ \sigma_e$	$P_s = 2.1\ dt\ \sigma_e$	$P_n = A_n\ \sigma_e$	P_c = characteristic shear strength deter determined by tests. $P_c \geqq 1.75$ the smallest of other failure modes

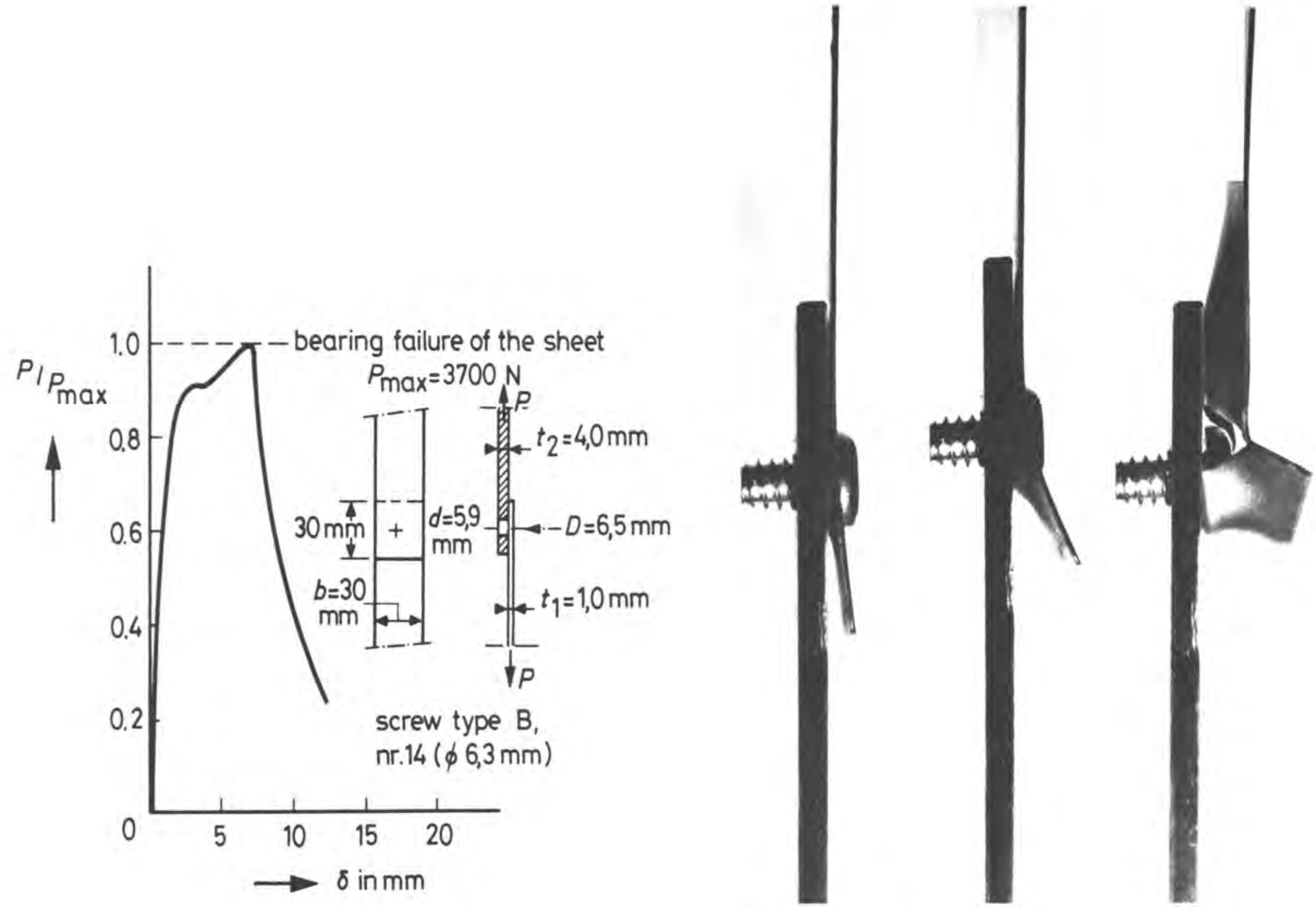
Fig. 7

Summary of the design shear strength of connections



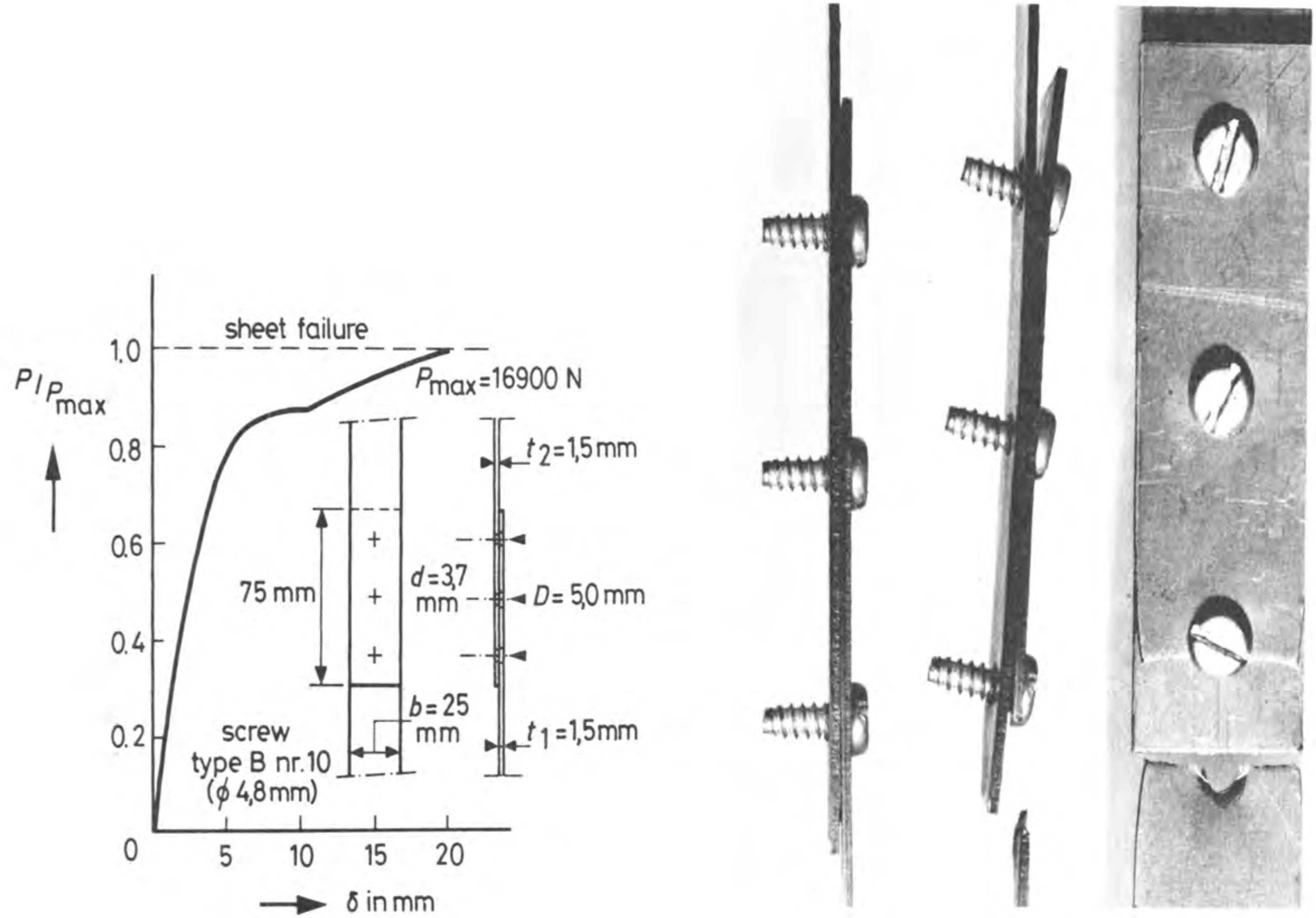
Test on a screwed connection
Failure mode : inclination of fastener

Fig. 8



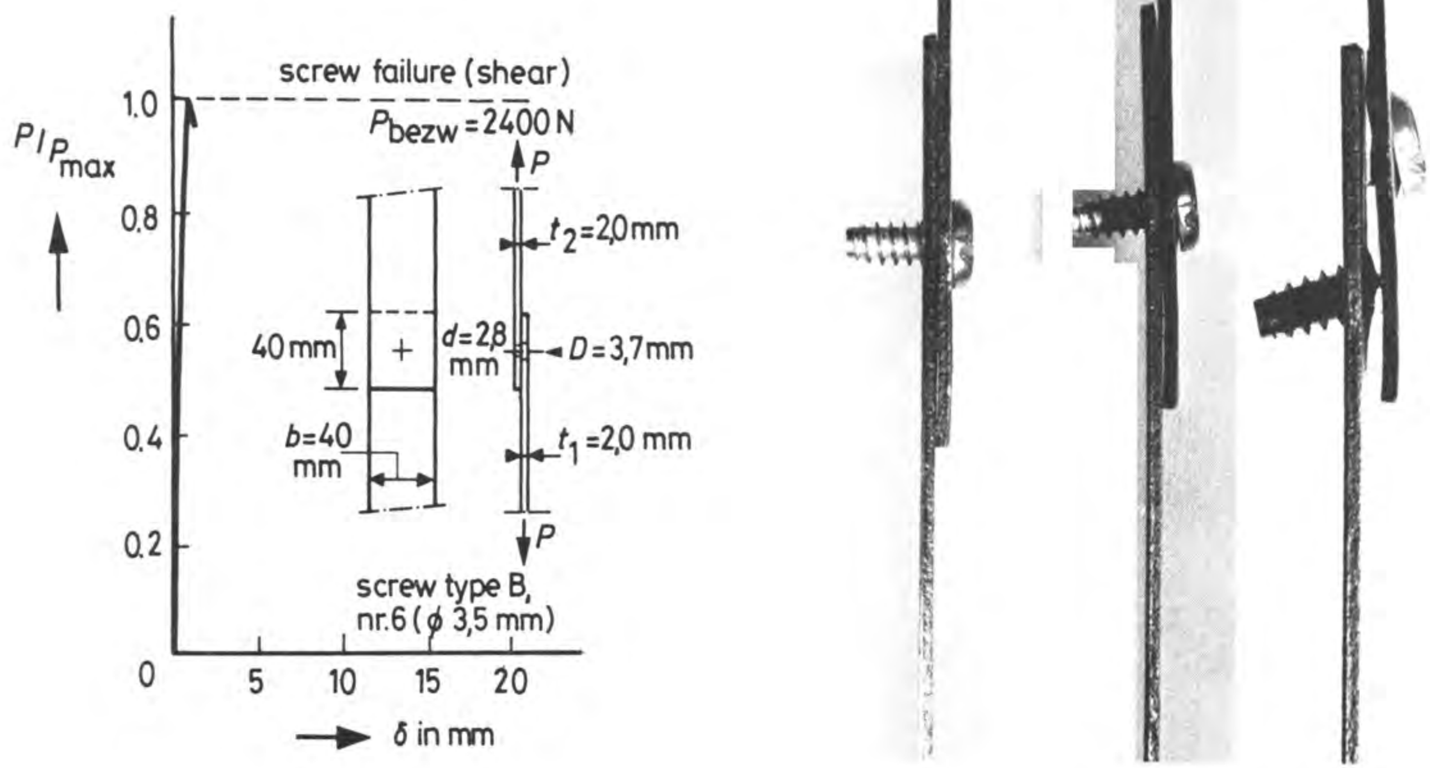
Test on a screwed connection
Failure mode: hole bearing

Fig. 9



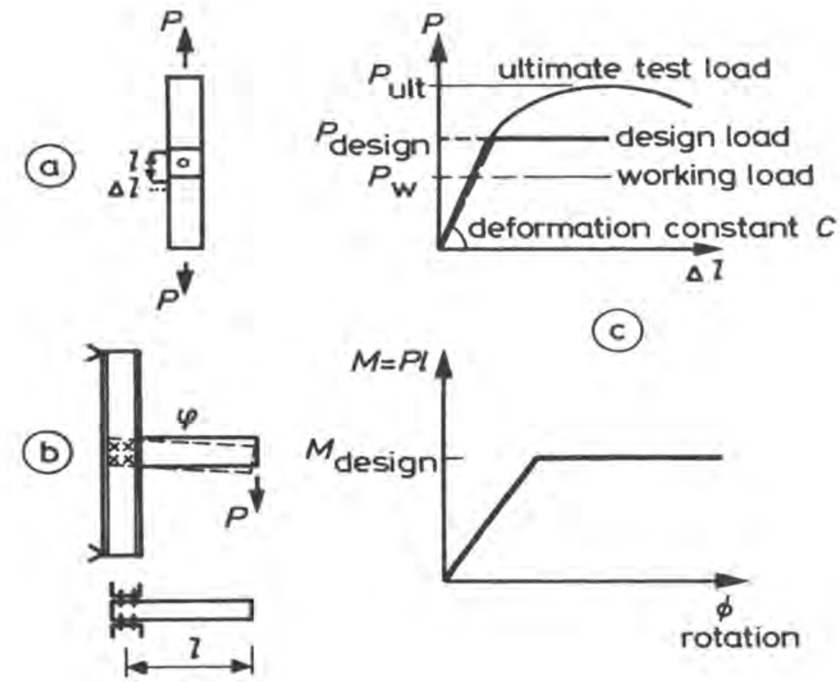
Test on a screwed connection
Failure mode: yielding of net section

Fig.10



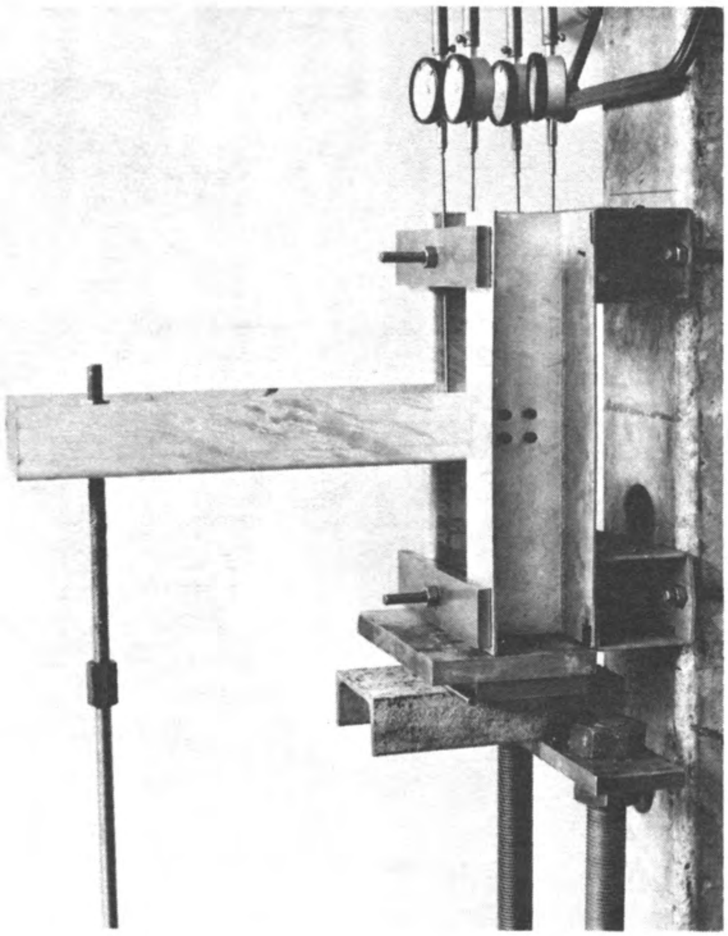
Test on a screwed connection
Failure mode: shear of fastener

Fig. 11



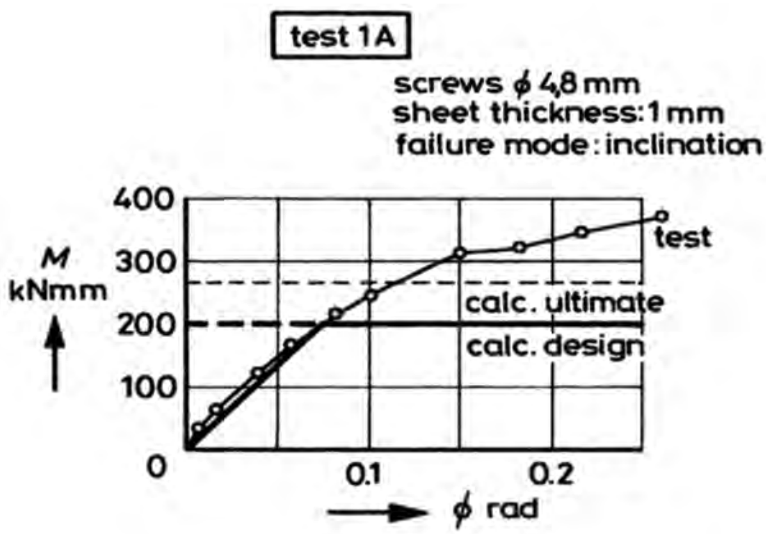
Relation between simple test and complete test

Fig. 12



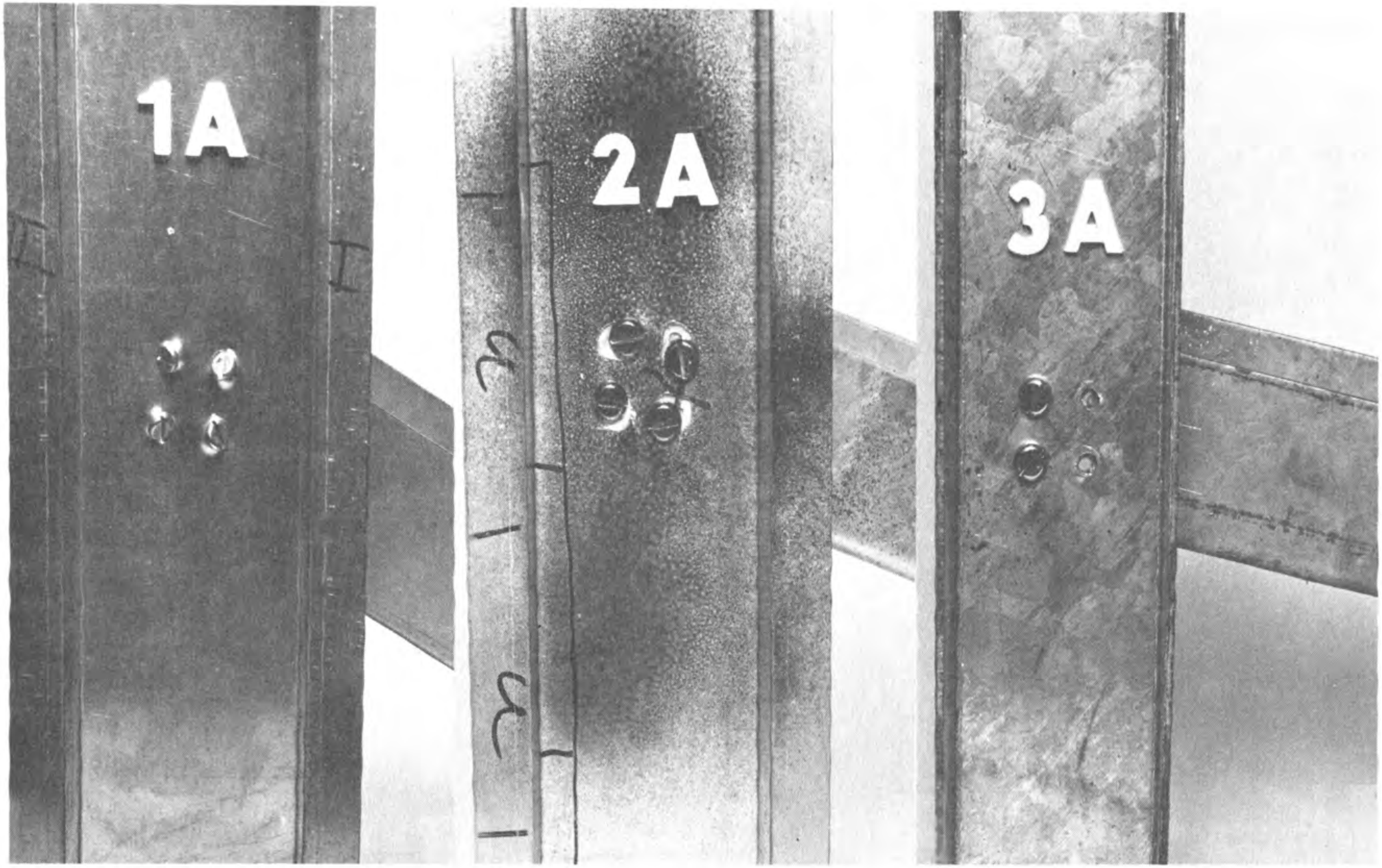
Test set-up of a complete connection

Fig. 13



Calculated and test results of a complete connection

fig. 14



a. inclination of screw　　b. hole bearing　　c. shear of screw

Some failure modes of complete connections with screws

Fig.15

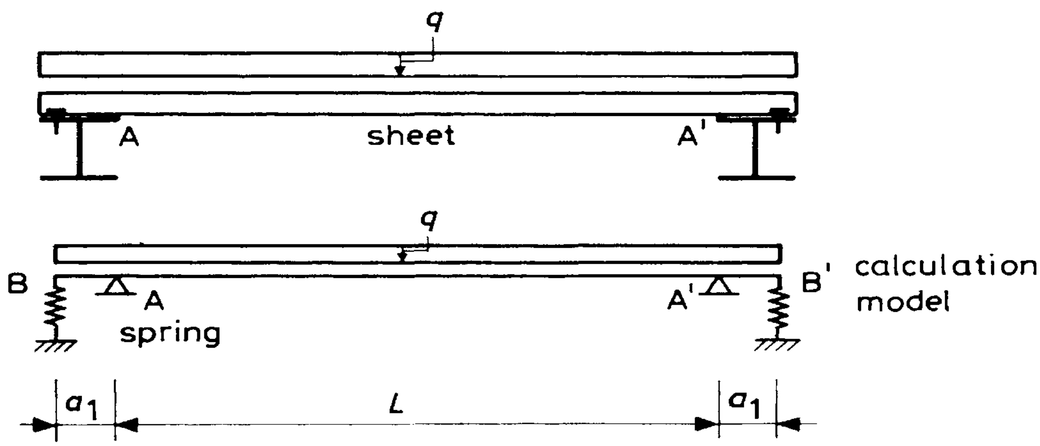


fig. 16 calculation model for the fastening system of a sheet supports under downward load

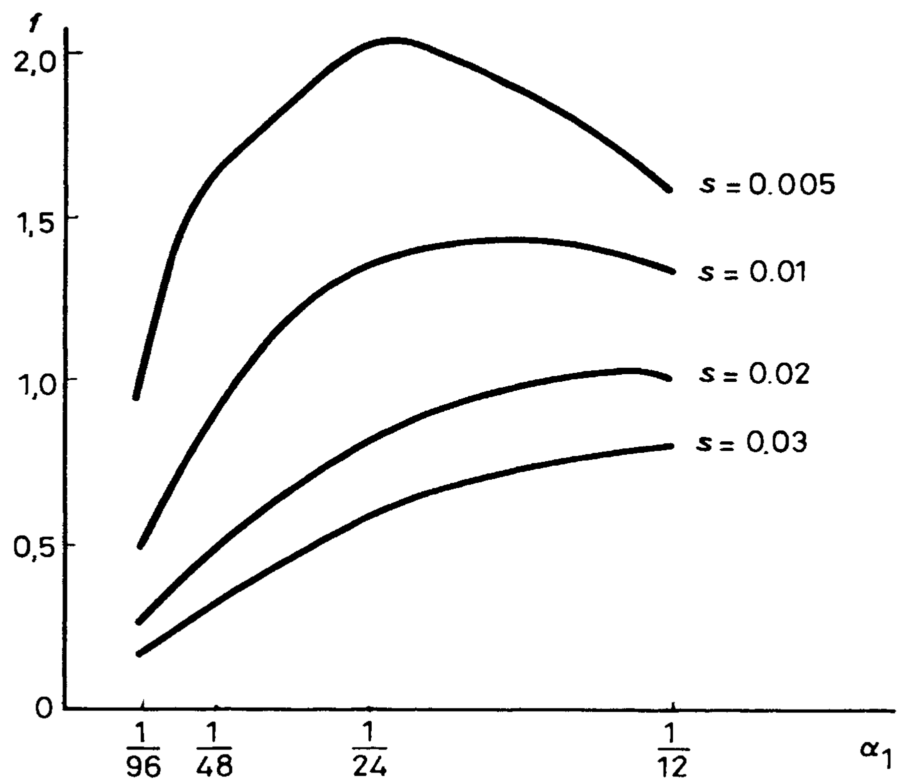
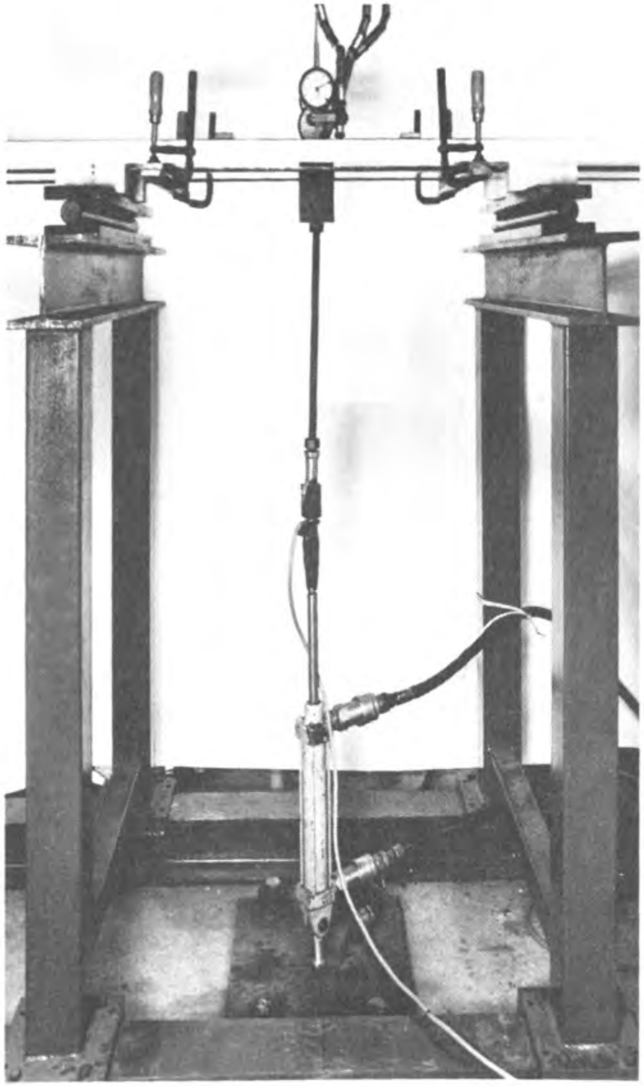
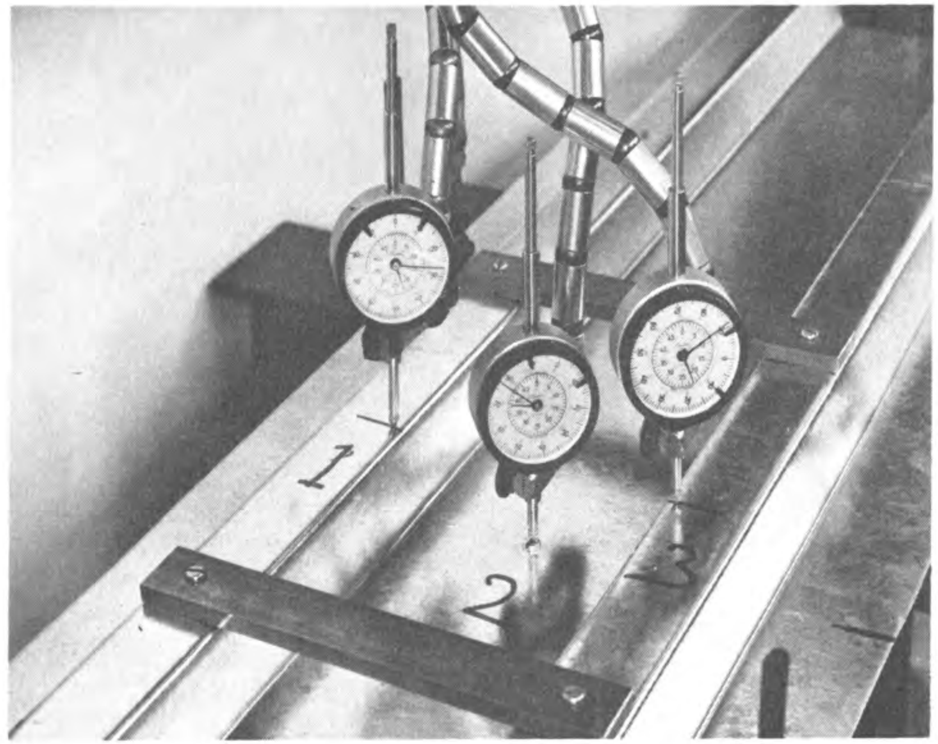


fig. 17 relation between f (prying - force factor) and α_1 ($\alpha_1 = \frac{a_1}{L}$) for a sheet on two supports according to equation (5.5)

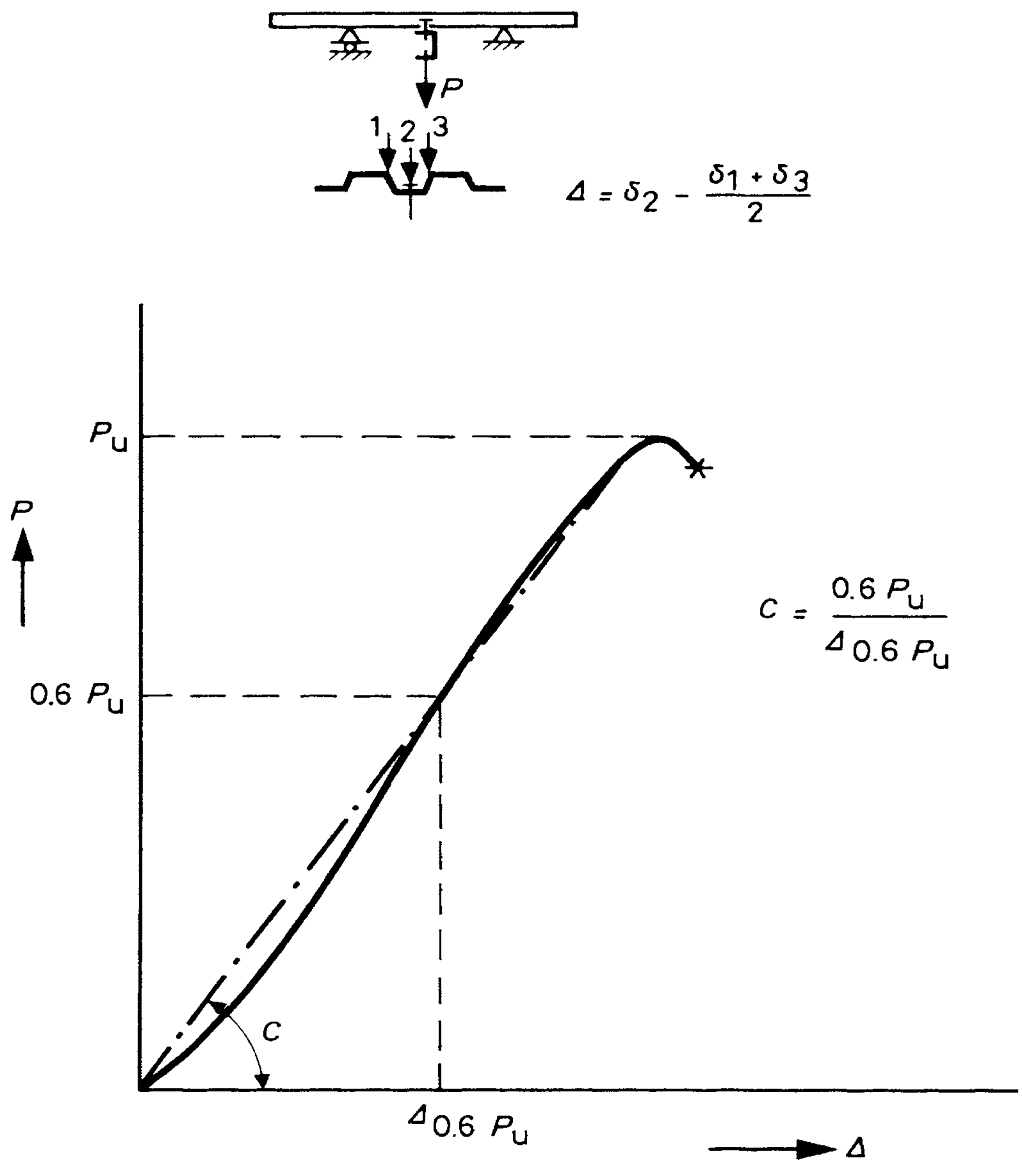


a. overall view of test set-up



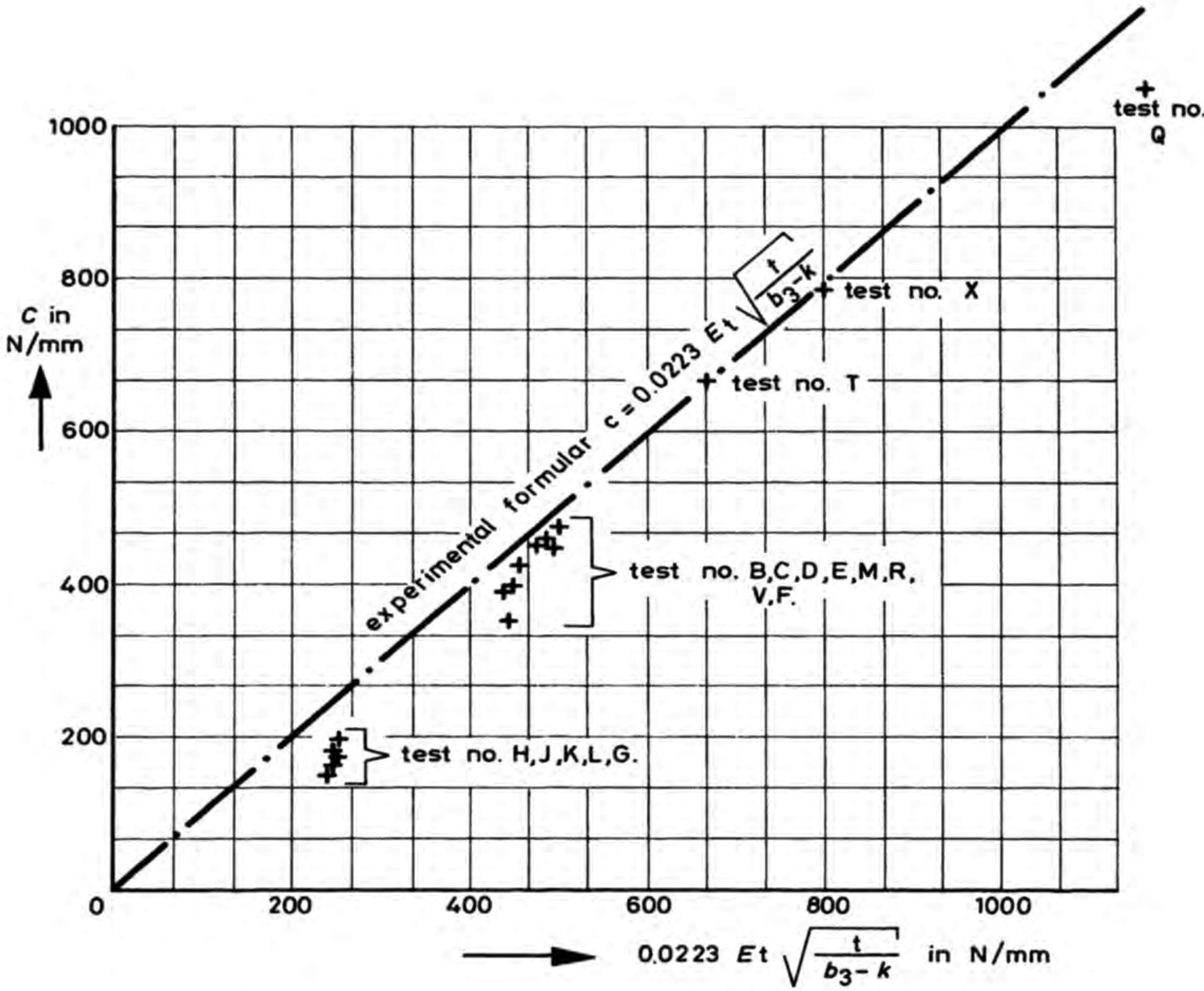
b. deflection measuring

Fig.18 Test set-up for detail tests to determine spring constant



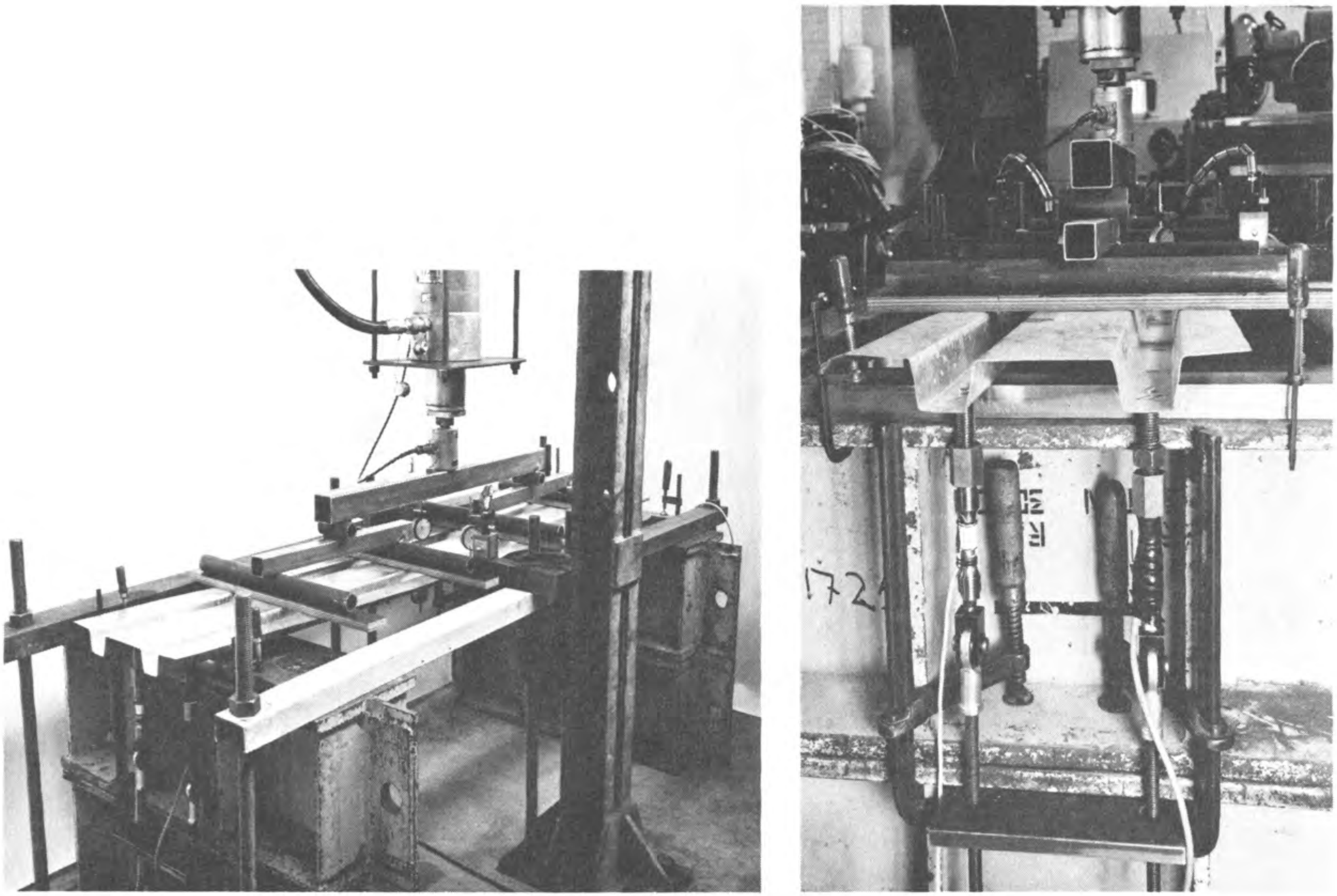
Qualitative load-deformation curve for detail tests according to Fig.18

Fig.19



Comparison of experimental formula for spring constant of connections with fasteners with a stiff head and test results

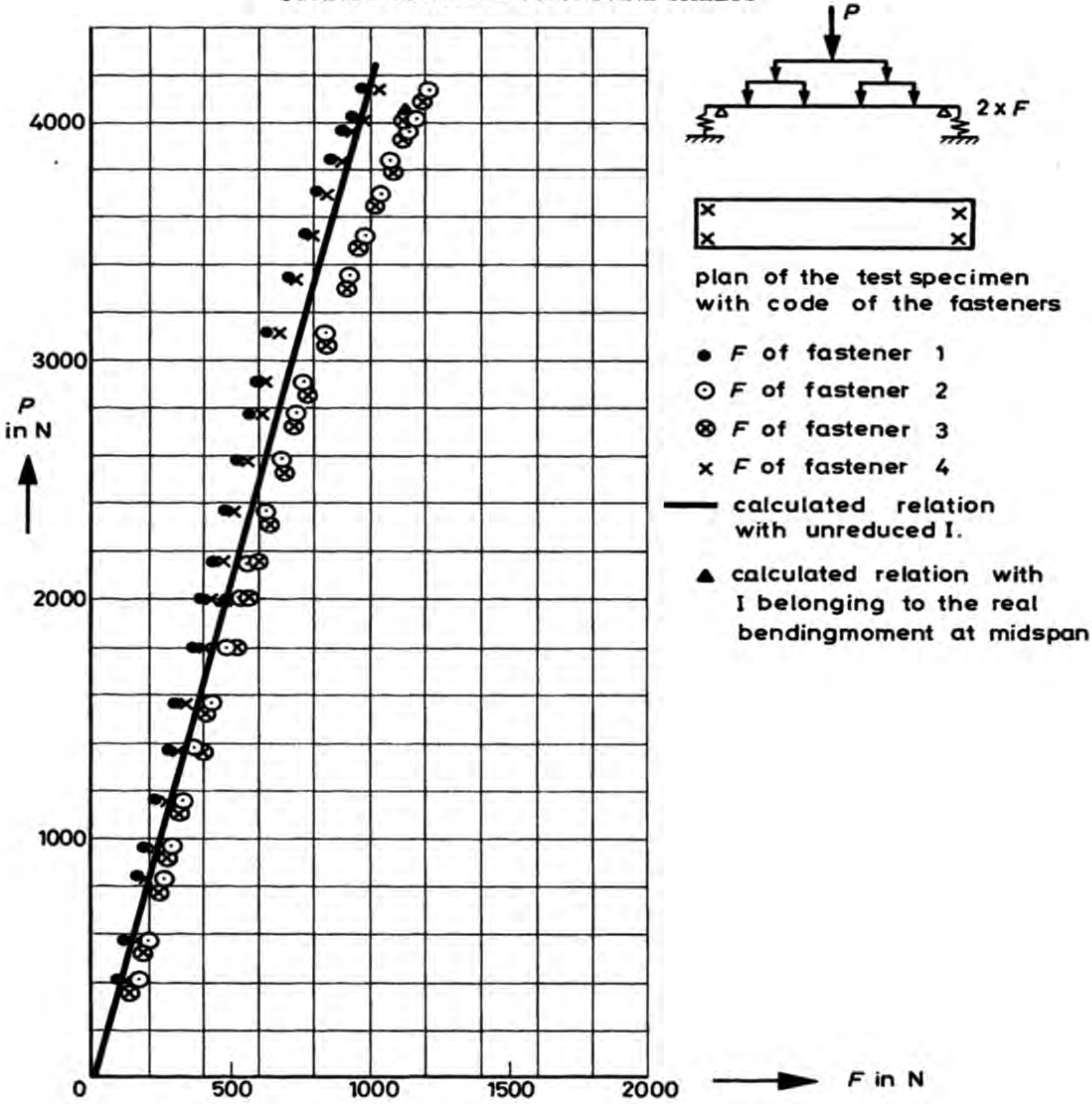
Fig. 20



a. overall view of the test set-up

b. detail of the measuring of the forces in the fasteners

Fig. 21 Test set-up to determine the forces in fasteners in a structure



Relation between the externally applied load, P, and the tension forces in fasteners F for downward load

Fig. 22