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THE CHARACTERISTICS AND DESIGN OF MECHANICAL CONNECTIONS IN PROFILED SHEET STEEL DIAPHRAGMS IN NEW ZEALAND

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PREFACE

This work forms the second part of a research programme undertaken by the Building Research Association of New Zealand to prepare design information for light gauge profiled steel diaphragms associated with local steel and timber framed constructions. The information from current overseas standards, codes of practice and design guides is either inappropriate or inadequate for use in New Zealand.

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NOTE

Trade names mentioned in this report are those of fixings used while testing the diaphragm connections which are its subject. Such mention does not imply exclusion of other products for these applications, nor specific endorsement by the Association.

This report is intended for structural engineers and other workers in the field of structural engineering research.

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ABSTRACT

Light gauge profiled steel claddings can act effectively as structural diaphragms and in so doing can enhance a building's performance. Much work has been done overseas to study the behaviour of these claddings, and data was collected regarding the characteristics of their mechanical connections which are essential for the design of this type of diaphragm. These data are not readily applicable in New Zealand because of differences in local products and practices.

This report summarises the results of an experimental investigation into the characteristics of mechanical connections and contains proposals for the design and testing of common connections in profiled sheet steel panels of local buildings. The purpose is to supply engineers and code-drafting authorities with advanced and comprehensive information.

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INTRODUCTION

General

In the 1950s (Godfrey and Bryan, 1959), tests on completely cladded steel framed structures revealed that the measured stresses and deflections were significantly less than those predicted by design which ignored the cladding contributions. Subsequent overseas investigations confirmed the practicality of using profiled steel claddings as diaphragms, leading to the development of valid design solutions (Davies and Bryan, 1982). The structural modes and necessary conditions for this type of diaphragm action and their design have been reviewed (Yiu, 1987). This work indicated possible benefits to New Zealand building design, at present inhibited by the lack of appropriate provisions in New Zealand Standards and comprehensive design data suitable for local use. This work forms the second part of a research programme undertaken by the Building Research Association of New Zealand to remedy this situation.

Over the years, work in this field has highlighted the prime requisite for cladding diaphragms: that the connections used to attach individual sheets to each other and to the framing, together with those throughout the supporting structure, be adequate for load transmission. Diaphragm action is dependent upon the ability of the cladding system to carry shear loads. It follows that the characteristics of the connections under shear are of fundamental importance. Moreover, there are situations where the cladding to purlin and shear connector connections are also subjected to wind suction (Hulst et al, 1977). Thus the combined effect of shear and tension may be significant.

This indicates that the need for sufficient fixings may increase the cost of the cladding system to the extent that it would cancel out the savings possible in the structural frame. However, for most cases, the fixings required to resist transverse wind loads have proved to be adequate to transmit the membrane forces as well. In fact, the cladding would strengthen the structure irrespective of whether diaphragm action had been accounted for in the design. By adopting a realistic design approach, not only can economies be ensured but the actual behaviour of the building can be understood; and it is also possible to avoid overloading the connections.

In order to realize the specific design of profiled sheet steel diaphragms of local structures, appropriate information for the designers and code-drafting authorities is necessary. This includes recommendations on design methods (various authors, see 1) and the availability of comprehensive design data, i.e., the characteristics of the connections, with recommended test or analytical methods for establishing them. The recommendations are universal and based on sound structural principles, while the design data needs to be related to local products and practices, rendering overseas data inappropriate.

The purpose of this work is to study the characteristics of and to generate design information on mechanical connections in profiled sheet steel diaphragms, with particular reference to the New Zealand situation. Because of the relatively recent promotion of this approach locally (Clifton, 1987; Yiu, 1987), some background information essential to the discussion in this report is presented. The local situation regarding cladding and fastening systems and practices is examined and the precise scope of the present work is defined. This is followed by a general review of diaphragm connection research and design. Experimental work has been carried out to obtain the necessary data, the results and relevant design aspects discussed, and the design data as well as proposals for design and testing of local connections are summarised.

Using this Report

The extensive review, details of tests, design data reduction together with the discussion included are for the benefit of those interested in the background and development of various recommendations in this report. Local engineers or research workers who are fully conversant with the design and testing of mechanical connections in profiled sheet steel diaphragms can directly refer to the sections on 'summary of design recommendations' and 'conclusions' for the necessary design information.

BACKGROUND INFORMATION

Terminology

The manner in which a series of diaphragms formed by the roof cladding and its supporting members in a flat-roofed building together with the arrangements of a panel (a sheeting and framing member assembly which forms part or the whole of the diaphragm) and its components are illustrated in Figures 1 and 2 respectively. The same principle and arrangements are equally applicable to timber framed structures.

Generally, diaphragms are divided into two basic types depending on whether the sheetings are fastened to the supporting structure on all four sides (direct shear transfer) or just at the end of the sheet (indirect shear transfer). The way of obtaining direct shear transfer involves the use of shear connectors as shown in Figure 2. These are usually purlin off-cuts and their use allows simplification of the sheeting to rafter connections.

Within a panel (Figure 2), there are three types of fixings essential to the structural action:

1. Primary fixings (for sheet-to-perpendicular member (purlin) or sheet-to-shear connector, if adopted) are fasteners which attach the sheeting to the structure and must withstand all the loadings applied to the sheeting. They may also have to permit sheeting movements due to thermal effects and on occasions provide electrical continuity for lightning protection. Both trough and crest fixings can be used (Figure 3). At present, diaphragm design associated with steel purlins has only been verified for trough fixed cladding.

2. Secondary fixings (for seams, Figure 3), which attach the sheetings together at side and end laps, are necessary to transfer loads between sheets and improve sealing of lap joints. They are normally not required in all profiles unless a diaphragm design is being undertaken.

3. Purlin-to-rafter or shear connector-to-rafter fixings which transmit the forces to the main structure.

In this report, the terms sheet-to-purlin connection and seam connection would be used to denote connections involving primary and secondary fixings respectively. A diaphragm connection is the combination of the fastener and the small region of attached materials in the immediate vicinity of the fastener.

Characteristics of Diaphragms as Related to Their Connections

The principal characteristics of a diaphragm in structural design are strength, ductility (deformation capacity before failure) and flexibility (Figure 4), all of which depend on the type, arrangement and integrity of the connections. Diaphragms with direct shear transfer are stronger and stiffer and hence more efficient than those with indirect shear transfer; therefore fastening on all four sides is recommended though not mandatory.

With regard to diaphragm strength, the design criterion is the least capacity of the panel which is governed by the local and overall shear buckling strength, the strength of the seam, sheet-to-perpendicular members, sheet-to-shear connector or purlin-to-rafter connections and the capacity of the edge members. For example, if there are sufficient numbers of strong connections in a thin diaphragm, the ultimate strength may be controlled by overall buckling (Yu, 1973). However, if the diaphragm has few connections, its strength can be below a general plate buckling load, and failure under shear can occur at the connections; or local shear buckling at panel corners.

A good principle to follow is to ensure ductility in routine diaphragm design, and ensure that the weakest link in the diaphragm is the strength of the seam connections (Bryan, 1987) so that the effect of combined loading on the sheet-to-purlin or sheet-to-shear connector connections is of no importance.

The ductility of a diaphragm-assisted structure is related to the ductility of the connections. While brittle failure of the sheeting or connection is undesirable and should be avoided, ductility can be ensured by designing the structure so that failure is associated with yield in bearing and tearing of the sheeting at the connections. Ductile connections permit a certain amount of redistribution of internal forces prior to failure and prevent sudden progressive collapse. With statically indeterminate structures (ECCS, 1984b), stresses due to differential settlements and temperature variations, faults in fabrication and assembly stresses cannot always be introduced into the design calculations. If the connections can be deformed adequately before failure, the effects of these stresses, which are superimposed on each other, can be disregarded.

The flexibility of the diaphragm dictates the rigidity and serviceability of the structure. It is calculated by summing a number of component flexibilities including sheet deformation (profile distortion and shear strain), deformation at various connections, i.e., seam, sheet-to-purlin and connections to rafters; and the axial strain in the edge members.

Besides using full scale tests, mathematical expressions (various authors, see 1) have been derived to determine the strength and flexibility of diaphragms. All of these require test data on the strength and flexibility of the connections.

Design examples using a local cladding product (Clifton, 1987) indicated that in most cases, the strength of the diaphragm is controlled by the seam strength while the connection flexibilities can amount to 9 to 56 per cent of the overall flexibility of the panel. The efficiency of primary fixings in every trough or alternate troughs is also illustrated.

New Zealand Claddings and Fastening Practices

An informal survey of local products and practices was conducted in 1987, involving forty-two major cladding manufacturers and suppliers. The result of the survey has been briefly described (Yiu, 1987). Findings relevant to this report are summarised as follows:

Yield Strength, Gauge and Finish of Steel Sheet

Common claddings have a nominal yield strength of 550 MPa with nominal thicknesses being 0.40 mm or 0.55 mm. Although claddings of 250 to 330 MPa yield strength, 0.75 mm and 0.95 mm thicknesses are also available, these are only adopted in special applications. Available finishes include plain galvanised and various coil-coated products, i.e., silicone polyester, polyester, polyvinylidene fluoride (PVF_2 or PVDF), polyvinyl fluoride or vinyl plastic barrier coatings (BRANZ, 1983).

Framing Members

Timber constructions are common in domestic buildings and low-rise commercial buildings while steel is more dominant in industrial applications. For timber structures, the purlins are mainly of *Pinus radiata* (Whiteside, 1984), whereas for steel structures (New Zealand Heavy Engineering Research Association (HERA), 1986), the purlins are normally of cold-formed sections of galvanised high yield strength (450 MPa) or black (280 to 340 MPa) steel with thickness of 1.6 mm, 1.9 mm or 2.5 mm.

Fastening Systems (Fixing Method and Fastener)

Besides 'rib and pan' profiles which use secret clip fixing devices, both trough and crest fixings have been used for sinusoidal and trapezoidal profiles. Crest fixings are more popular because of the wide belief that this practice reduces the possibility of leakage. Nevertheless, trough fixing is a common practice in wall construction.

Within this scope of trough and crest fixings, all the connections recommended are made with discrete fasteners such as screws, nails or rivets. Site welding of secondary framing and cladding (Departments of the Army et al, 1982) and the use of glue (White, 1986) are not practised in New Zealand. Seam fasteners recommended by manufacturers and suppliers include 4 mm and 4.8 mm monel rivets, 4.8 mm "Bulb-tite" rivets, galvanised gutter bolts with neoprene washers, "Tapits" and self-tapping screws; while the commonly adopted sheet-to-purlin fasteners are summarised in Table 1.

Table 1: **Commonly adopted sheet-to-purlin fasteners**

Purlin Material	Fixing method: Trough fixing	Fixing method: Crest fixing
Steel	12g x 20 to 25 mm "Teks" or "Steeltite" self-drilling self-tapping screws with hexagon heads and neoprene washers or 25 mm galvanised embossed washers	12g x 35 to 60 mm "Teks" or "Steeltite" self-drilling self-tapping screws with hexagon heads and neoprene washers or 14g x 45 to 95 mm "Teks" or "Steeltite" self-drilling self-tapping screws with hexagon heads and neoprene washers or rectangular galvanised embossed washers
Timber	12g or 14g x 25 to 40 mm "Type 17" or "Timbertite" self-drilling wood screws with hexagon heads and neoprene washers or 25 mm galvanised embossed washers	12g or 14g x 50 to 100 mm "Type 17" or "Timbertite" self-drilling wood screws with hexagon heads and neoprene washers or rectangular galvanised embossed washers or 60 or 75 mm "Weatherseal" or "Nu-Way" spiral shanked nails (4 mm diameter) with galvanised metal and neoprene or "Inseal" sealing washers

Note: Lead head plain shank nails, although used, are not recommended for timber because of the relatively poor withdrawal performance.

Choice of Fastening System

Structural considerations involve strength, flexibility (or stiffness) and ductility which have been discussed. Proper attention should be paid towards the suitability of fasteners for the particular type of loading, e.g., rivets are inferior to screws in a working environment involving vibration, fatigue or tension loadings.

Non-structural considerations include availability, function, economy, erection, service condition and exposure, compatibility, durability (corrosion, design life, maintenance regime) and resistance to weather penetration; most of which are interrelated.

One of the peculiarities of common New Zealand fastening systems is that crest fixings are extensively used because of the traditional belief that their weatherproofing performance is superior; though it is also known (Dixon, 1987) that crest fixings have the problem of corrosion due to condensation at the shank between the cladding and the purlin. In reality, by fixing at the trough rather than at the crest, it is possible to avoid spreading of the sheet and the problem of driving the fastener crookedly, as well as to fix much more tightly thus minimising the risk of leakage. However, this depends on the provision of permanent sealing around the pierced point and care taken not to obstruct the free flow of water. In these respects, it has been demonstrated (Bartak et al, 1977) that trough fasteners with special sealing grommets and special watertight fasteners have proved satisfactory. Trough fixings are also used extensively in the United Kingdom and Europe (Davies and Bryan, 1982; ECCS, 1984b) without any weatherproofing problem. This may be attributed to the fact that with neoprene washers and plastic coated sheets, the slight 'give' in these materials effectively seals the connection (CONSTRADO, 1973).

Although a degree of caution has to be given towards importing overseas technology which may introduce unnecessary problems locally, the potential advantages of trough fixings certainly warrant more attention in future New Zealand building practice.

The Need for and Scope of the Present Work

Because of the importance of connection characteristics in the design of diaphragms, this phase of the work has been arranged to study the characteristics and to generate comprehensive design data of mechanical connections suitable for local design. The strength and flexibility of diaphragms can then be easily computed for routine use (Bryan and Davies, 1981). This information also enables finite element analyses (various authors, see 2) to examine various aspects.

In New Zealand, there are some essential differences between local profiled metal systems and those in North America and Europe. These include yield strength and gauge of steel sheets as well as the fastening systems (Yiu, 1987).

From the structural efficiency point of view, trough fixings are more efficient than crest fixings and their benefits have been proven on steel framed structures (Davies and Bryan, 1982). Hence this phase of the work concentrates on trough fixed sheet-to-purlin or shear connector connections. Purlin-to-rafter or shear connector-to-rafter connections (Bryan and El-Dakhakhni, 1968a) are not included. However, seam connections (Table A.1) which are applicable to all the other situations, have been studied.

Within the context of trough fixed sheet-to-purlin and seam connections, the fasteners used in United Kingdom (Berry, 1976) are similar to those in New Zealand. It would be tempting to convert the relevant data collected overseas (Grimshaw, 1979) for local use because scaling regarding yield strength and gauge is permitted (Davies and Bryan, 1982). However, this extrapolation is only valid for sheeting of thickness up to one full test range above the thickest sheet tested, and down to one half the test range below the thinnest sheet tested; with similar limits for sheeting with different strength but of the same material (ECCS, 1984b). The limited range of ultimate strength of the sheeting from which these relevant data are obtained, i.e., 292 to 373 MPa, does not enable such extrapolation to be valid for the local materials, i.e., those with nominal yield of 550 MPa. Also there are no guidelines on the extrapolation of flexibilities. Hence there was no alternative but to carry out suitable tests to obtain the necessary information. This work identified the appropriate test methods, the suitable variables to be included, and the tests were performed acccordingly. The results of these are compared with local and overseas connection data. The validity of design expressions (various authors, see 3) proposed for connection strength and flexibility as well as overseas design philosophy of diaphragm connections based on tests are also examined.

A unique feature of the high strength cladding materials used locally is that it is associated with a very low ductility (Maricic, 1979). Thus it is interesting to examine the deformation capacity of the connections involving such materials which is not covered elsewhere.

Practical aspects regarding edge distances, end distances and spacings of the connections are examined. Although the test programme considered static shear loads only, the effects of other loading regimes, i.e., combination of shear and tension loads, fatigue as well as dynamic earthquake loads are also briefly discussed. The study of concentrated load effects (Davies, 1978) is not included.

GENERAL REVIEW

Diaphragm Connection Research

Development of Test Methods

Extensive diaphragm connection research began in the 1960s when the relationship between connection and diaphragm characteristics were put into perspective (Bryan and Jackson, 1968). Due to practical difficulties in developing analytical solutions for connections and the need for experimental verification, early work concentrated on shear testing of connections composed of overlapping strips of sheet metals in various forms (Baehre, 1975; Baehre and Berggren, 1971, 1973; Bryan, 1973; Bryan and El-Dakhakhni, 1968a, 1968b). A more sophisticated fixture (simulated diaphragm action test, Figure 5) designed to eliminate undesirable eccentricity and to restrain out-of-plane distortion, restricting the movement to that which is likely to occur at a connection in an actual diaphragm was also developed (Ammar and Nilson, 1972). These tests are derived to satisfy particular demand without regard to standardization.

Because of the diversity of the specimen dimensions and test methods, attention (Fraczek, 1974, 1976) was drawn towards drafting a standard connection test procedure. This led to the publication of the European recommendations on the design and testing of mechanical connections in steel sheeting and sections (ECCS, 1978, 1984b) which gives comprehensive test procedures and evaluation method to determine the design strength and deformation of the connections.

In view of the extensive reference to the ECCS (1984b) publication in this report, from here onwards it will simply be referred to as the European recommendations.

Other Aspects

The behaviour of sheeting in compression surrounding edge purlin connections, which arises from significant local forces induced by point loads applied within the length of a side of a diaphragm, was investigated experimentally using 6.1 mm self-drilling self-tapping screws and 0.57 to 0.66 mm sheets (Davies, 1978). This led to the derivation of a simple solution to the buckling problem, i.e., adopting a reduction factor for the connection capacity based on the slenderness of the fastened face of the sheet.

Relation to Finite Element Analyses

Extensive connection research has also been carried out to determine the load-deformation characteristics for finite element computer analysis.

In these analyses (Sved et al, 1972; Miller and Serag, 1978), the variety and complexity of diaphragm connections led investigators (Davies, 1974; Nilson and Ammar, 1974) to model them by spring elements in two orthogonal directions with the the spring characteristics obtained from shear tests. Sophisticated models for the two spring system have been used depending on the location of the connection (Atrek and Nilson, 1980).

Experimental evidence showed that excessive deformation, tearing of the sheeting around the fastener or tilting of the fastener are the only important sources of diaphragm non-linearity, with the limit of elastic response about forty per cent of the failure load (Nilson, 1973; Nilson and Ammar, 1974). Thus only the connections were modelled by nonlinear functions (Atrek and Nilson, 1981; Davies, 1980); both multi-linear approximations (Ha, 1979) and elastic-plastic idealizations (Chockalingam et al, 1979; Lawrence and Sved, 1972) have been used.

Shear Test Methods

European Recommendations

Shear test methods in the European recommendations are not unique (Berry, 1977). The standard shear test is essentially a single shear test using a simple specimen formed from two overlapping straps (with one or two fasteners) as shown in Figure 6, and the alternative shear test is the simulated diaphragm action test.

The standard shear test has the advantage of simplicity, with the single fastener arrangement particularly suitable for examining critical end distance of connections. The simulated diaphragm action test can cater for connections not in the plane of the sheeting, e.g., upstanding seams; and for the consideration of edge failure.

The dimensions of the test specimen are selected so as to prevent edge failure and transverse tension failure across the net section.

Comparison of Results from Different Shear Test Methods

Various workers (Berry, 1976; Fraczek, 1976; Huang and Luttrell, 1980) have performed tests to compare the results obtained from the different test methods as well as the real behaviour (Stol and Toma, 1978). Their results are summarised in Appendix B. It is noted that the agreement on strength results is satisfactory but the flexibility results are more variable.

Davies and Bryan (1982) pointed out that a test incorporating single fastener lap joints will give results almost identical to that using double fasteners for connection strength, but a considerable reduction in flexibility as a result of the jamming effect of distortions due to eccentricity within the connection. Fraczek (1976) suggested that the variation in results between single lap joints and simulated diaphragm action tests is caused by the smaller joint eccentricity in the latter (which can be reduced by packings in the former), and the larger overlap in the lap joint specimen providing greater restriction to the rotation of the fastener which deforms the material in the immediate vicinity of the fastener.

Therefore the choice of the shear test method is by no means conclusive.

Choice of Shear Test Method

Ideally, the perfect test set up should represent the real diaphragm connection behaviour and provide correct values for the shear stiffness and strength of the connection.

Although the simulated diaphragm action test can account for cases where the connection is not in the plane of the sheeting, a modified version of the standard shear test can generate satisfactory results for mechanical clinch connections not in the principal plane of shear (Chockalingam et al, 1978, 1979). Such arrangement is also not essential if edge distance is not the prime consideration. The simulated diaphragm action fixture is expensive to produce and difficult to use. The standard shear test is adequate in most cases.

With the standard shear test arrangement, the major objection to testing connections with multiple in-line fasteners is the complex load distribution across the connections in the elastic deformation range (Fraczek, 1976). However, this deficiency can be diminished by limiting the number of fasteners in the specimen to two, and noting that the elastic load distribution effect will be eliminated by yielding.

With double fastener standard shear test, the loads in the two connections are the same when the lapped sheets are of equal thickness. However, for specimens with two straps of different thickness, the load is not distributed equally between the connections. When the connection flexibility is small, the difference is more significant. Stol and Toma (1978) developed an analytical solution to determine the ratio of loads between the connections assuming the same flexibility for the connections. Using the data of common New Zealand sheetings and purlins, the maximum load difference between the connections is only 3.5%. Thus for practical purposes, this nonuniformity can be ignored and the load taken to be equally distributed for local test specimens.

In general, the double fastener test has distinct advantages over the single fastener test in that it produce lower stress concentration as well as smoothing out test variations by providing an average load per connection. In fact, the double fastener test is more representative of the actual connection particularly at seams. The single fastener test is unnecessary if corner and end distances are not critical.

The standard shear test using two fasteners is the most common test for mechanical connections, its simplicity and universal acceptance make it a natural choice as the recommended test method for diaphragm connections in New Zealand. Therefore it is adopted in the present work. In this report, the double fastener standard shear test in the European recommendations is simply referred to as the shear test.

Pull-Out Tests

In real structures, the sheet-to-purlin connections are complicated by the fact that various lap joint situations exist (Figure 7). Thus it is necessary to include the most unfavourable situation and the different failure modes in the design consideration. In Figure 7, connection type

(a) (which is covered by the shear test) gives the lowest strength for sheeting failure, while connection type (d) gives the smallest deflection at failure and also the lowest pull-out strength; both being important characteristics in the determination of deformation capacity and the lowest failure load. Detailed procedures to obtain these data (from the pull-out test which simulates the shear action of connection type (d) and shown in Figure 8) are included in the European recommendations.

Interpretation of Connection Characteristics from Tests

The characteristics of diaphragm connections can be obtained by testing to failure of connections in a suitable arrangement. Since the present work is based on shear and pull-out tests prescribed in the European recommendations, this section summarises the data interpretation, processing and design approaches (similar for both tests).

Characteristic Strength

Because of the statistical nature of connection characteristics, the characteristic strength of a connection type, i.e., P_k, is derived from test results based on Equation (1).

$$P_k = P_m - c.s \qquad ....(1)$$

where P_m = mean value of the maximum load reached in a deflection of 3 mm for a test series. With double fastener specimens, the load per connection is one-half the load recorded. The interpretation of the maximum load (P_{max}) within 3 mm deflection is illustrated in Figures 9(a) and (b).

c = a coefficient based on the number of test observations, a normal distribution, a chosen fractile part of 50% (mean value) and a confidence level of 95%.

s = standard deviation.

The maximum load within 3 mm deflection may not be the maximum load achieved in a test (Figure 9(a)) and this criterion was not in the ECCS (1978) recommendations. With reference to shear tests performed on sheet-to-purlin connections in the United Kingdom, employing self-drilling self-tapping screws with neoprene washers and mild steel sheets (Grimshaw, 1979), it is common for most connections to achieve the maximum load at deflections larger than 3 mm. Thus this '3 mm' criterion will always result in a conservative estimate for the maximum load.

The basis for this specification is not apparent from the literature though it is understood (Toma, 1988) that the '3 mm' criterion was introduced for the following reasons:

(1) It gives a well defined load level.

(2) For serviceability reasons.

(3) The '3 mm' value was arbitrarily chosen from a lot of test results which showed that the load increase after a deformation of 3 mm was only slight.

The last reason is not particularly convincing with reference to Grimshaw's results (1979).

The 95% confidence level recommended is the same as that commonly used for various characteristic strength computations for materials in New Zealand and therefore it is consistent with local practice.

Deformation Capacity

The objective of examining deformation capacity is to ensure that the connection deflection at failure is sufficient to enable force redistribution and to avoid considering secondary forces (see later). It is necessary to prevent brittle failure as well as to ensure a sound structure in the service condition.

The criteria for sufficient deformation capacity is 0.5 mm at failure for seam connections; and 3 mm at failure without excess hole deformation of the substructure for sheet-to-purlin connections. If these limits are not achieved, secondary forces, e.g., temperature effects, must be considered. The critical condition for sheet-to-purlin connections is for combined longitudinal and transverse lapped joints (Figure 7(d)) and the pull-out test (Figure 8) is appropriate to check this requirement.

In case failure can occur in different modes, it is desirable that a certain range be available between the characteristic strength belonging to a failure mode with insufficient deformation capacity (P_{ka}) and the characteristic strength belonging to a failure mode with sufficient deformation capacity (P_{kb}), i.e.,

$$P_{ka} \geq 1.3P_{kb} \qquad \ldots.(2)$$

The factor of 1.3 is to account for the variation in yield stress of the sheet material and the scatter in test results.

Toma (1978b) has commented that the deflection at the ultimate load level should be larger than 2 mm and a smaller deflection should be regarded as brittle failure without further elaboration. For mechanical connections involving mild steel sheeting, these deformation capacity criteria are easily satisfied (Berry, 1976; Grimshaw, 1979).

Design Strength

For limit state design, the design strength (P_d) is evaluated by adjusting the characteristic strength (P_k) with the appropriate material and correction factors, i.e.,

$$P_d = (K_t.K_o.P_k)/\gamma_m \qquad \ldots.(3)$$

where K_t = thickness correction factor, i.e., ratio of guaranteed minimum sheet thickness to thickness of the sheet used in tests, with a maximum value of 1.

K_o = yield correction factor, i.e., ratio of guaranteed minimum yield limit to yield limit of the sheet used in the tests, with a maximum value of 1.

γ_m = material factor, with a value of 1.1.

Furthermore, for sheet-to-purlin connections, if the maximum load is reached before a deflection of 3 mm (Figure 9(b)) then the remaining strength at 3 mm deflection (P_r) shall be at least the design strength, i.e., refer to Figure 10,

$$P_r \geq P_d \qquad \ldots.(4)$$

With reference to Equation (3), the yield correction factor may be grossly conservative in situations involving high yield steel sheets commonly used in New Zealand; i.e., nominal yield: 550 MPa, actual yield: 600 to 770 MPa and K_o: 0.71 to 0.92. This correction factor is also inappropriate for failure modes involving the shear of the fastener. For sheet-to-purlin connections, there is no correction factor for the deviations in purlin thickness and yield strength.

Flexibility

The shear flexibility of the connection (c_h), i.e., shear deformation per unit shear load, is determined from Equation (5).

$$c_h = (\Sigma\, a_h . \gamma_1)/(P_k . n) \qquad ....(5)$$

where a_h = the deflection or slip of the connection corresponds to a load of P_k/γ_1 obtained from the load-deflection curve for connection tests (Figure 11). For double fastener test arrangements, the load-deflection curve for an individual connection is the load-deflection curve for the test with the load value divided by two.

γ_1 = partial load factor times material factor and has a value of 1/0.6

n = number of tests in the series.

It should be noted that P_k/γ_1 is the maximum service load.

Design of Connections Based on Tests

Design Loads

The loadings on diaphragm connections may involve both primary and secondary forces. Primary forces include dead and constructional load, membrane forces, earthquake or other dynamic loads; while secondary forces may be induced by temperature variations or movements at supports.

Normally, if the connection has sufficient deformation capacity, only the effects of primary forces have to be accounted for. Otherwise, the combined effects of primary and secondary forces must be considered, e.g., one-third of the forces due to the maximum difference in temperature need to be included in the load combination (SISC, 1982).

For limit state design, the load factors are those specified in NZS 4203 (1984).

Strength Criteria

Diaphragm connections have to satisfy the strength criteria in Equation (6).

$$P_d \geq F_{de} \qquad ....(6)$$

where F_{de} = design load in the connection (sum of appropriate factored loads).

In some cases, additional requirements have to be satisfied to ensure an acceptable serviceability state (Davies and Bryan, 1982; ECCS, 1978, 1984b).

Serviceability Criteria

A satisfactory serviceability performance is where the recovery of deflection after loading up to service load (approximately 60 per cent of the characteristic strength) and removal of the loading does not lead to excessive deformation nor impair weatherproofing.

Temperature Effects

In practice, besides membrane forces, temperature variations of the sheet may also have a significant effect. It is known that they can give rise to annoying roof noise (Ellen et al, 1985).

Allowance for this should be made by suitable detailing at the design stage. Important considerations are the magnitude of temperature change, the erection and service temperature of the elements, and the design of the structure. Colour, insulation, ventilation and even the wind environment are also influential.

The shear force in the connection caused by variation of temperature can be calculated by taking into account the slip at the connection, strain of the sheets and deflection of the structure (Toma, 1978a). However, temperature effects can be neglected if the connection has sufficient deformation capacity. The loss of stiffness in the diaphragm caused by this effect can also be ignored if the design loads induced are smaller than the connection design strength. It should be noted that the contribution of sliding fixings is ignored in diaphragm design. Common practice is to limit the length of diaphragm based on experience (Thomson, 1987).

Design Expressions for Mechanical Connections

Design expressions for the shear characteristics of mechanical connections associated with profiled steel sheets have been proposed (various authors, see 3). They are related to the type of fastener and the failure mode. In general, irrespective of whether it is the ultimate or design strength of the connection under consideration, the critical strength is the least value obtained by comparing all the possible failure modes.

A summary of the design expressions available is given in Appendix C. They are based on empirical curve fittings or lower bound test results. Previous experience (Davies and Bryan, 1982) indicated that the expressions are generally very conservative in comparison with test results, implying a significant loss in economy.

TESTS

Planning of Tests and Selection of Test Variables

Although the local cladding and purlin materials comply with the relevant standards and the minimum yield strength guaranteed by the manufacturers, it is necessary to perform tensile tests in order to assess the properties of the materials used in the present tests. These data are also relevant in assessing the applicability of design expressions for mechanical connections.

The aim of the shear and pull-out tests is to cover as many varieties of common connections currently used in New Zealand as is possible within the framework of a practical test programme. Where similar results are expected, only those which would give upper and lower bound figures are included. The pull-out tests are only relevant for sheet-to-purlin connections.

For seam connections, the following variables are included:

(1) Cladding material: G550 steel, galvanised.
(2) Cladding thickness: 0.55 mm and 0.40 mm.
(3) Fastener: "Tapits", 4.8 mm and 4.0 mm monel rivets, 4.8 mm and 4.0 mm aluminium rivets, "Bulb-tite" rivets.

For sheet-to-purlin connections, the following variables are included:

(1) Cladding material: G550 steel, galvanised.
(2) Cladding thickness: 0.55 mm and 0.40 mm.
(3) Purlin material: G450 (galvanised) and G280 (shop coated with primer) steel.
(4) Purlin thickness: 2.5 mm, 1.9 mm (2.0 mm) and 1.6 mm
(5) Fastener (including washer): 12g neo, 12g emb and 14g neo.

Details of fasteners and the fastener abbreviation are given in Appendix A. In this report, the abbreviation 450/2.5 is used to denote G450 steel purlins of 2.5 mm thickness; similar abbreviations are also used for other types of purlin.

Tensile Tests

Test Specimens

The sampling and size of the tensile test specimens for sheeting and purlin materials are given in Table 2.

Table 2: Sampling and size of tensile test specimens

Material	Source	Sampling [1]	Size [2]
Cladding	galvanised steel sheets as received, flat, cut from coil (parent material of profiled cladding)	3 specimens for every 1 m^2 of sheeting	test piece type 1 (parallel sided strip)
Purlin	flanges of cold-formed steel purlin sections	5 specimens for every 6 m length of purlin	test piece type 2

Notes: 1. Tensile properties of light gauge steel can be determined in either the longitudinal or transverse direction depending on the quality and its end applications. The transverse test is used when ductility is important; while the longitudinal test tends to produce higher elongation value and lower yield stress (approximately 20 MPa) (John Lysaght Ltd, 1980a). For applications such as claddings and purlins where minimum yield stress is of prime interest, longitudinal test is appropriate.

Thus the test pieces are cut parallel to the direction of rolling. This is also in line with the requirements in NZS 3441 (1978).

2. In accordance with clause 5.2 of BS 18: Part 3 (1971).

Test Procedures

The tensile tests were performed in accordance with the provisions of BS 18: Part 3 (1971).

A relatively slow crosshead speed, i.e., 1 mm per minute, was used in the tests. It is well known (AS 1391, 1974; ASTM A370, 1977; ISO 86, 1974; Taraldsen, 1976) that an increase in the rate of straining would give an increase in yield strength, i.e., the result is strain rate sensitive and differences of two to ten per cent would be common (TELARC et al, 1977). However, strain rate control is impractical and it is sufficient to control the crosshead speed (Hamstad and Gillis, 1966). The speed used ensures that the maximum strain and stress rates specified for both the elastic and plastic deformations are not exceeded and this generally gives conservative results.

Shear and Pull-Out Tests

Forty-two series of shear tests and 10 series of pull-out tests were performed. All the fasteners and steel straps in the tests were nominally identical with regard to manufacturer or supplier, type, material and dimensions. Full particulars of the test series are shown in Tables 3 and 4.

Table 3: Seam connection shear test series

Fastener type	Cladding thickness 0.55 mm	0.40 mm
"Tapits"	X	X
4.8 mm monel rivets	X	X
4.0 mm monel rivets	X	X
4.8 mm aluminium rivets	X	X
4.0 mm aluminium rivets	X	X
"Bulb-tite" rivets	X	X

Table 4: Sheet-to-purlin connection shear and pull-out test series

	Cladding thickness											
	0.55 mm						0.40 mm					
Type of test:	Shear			Pull-out			Shear			Pull-out		
Fastener type [1]:	A	B	C	A	B	C	A	B	C	A	B	C
Purlin												
450/2.5	X	X	X		X		X	X	X		X	
450/1.9	X	X	X				X					
450/1.6	X	X	X		X		X	X	X			
280/2.5	X	X	X	X	X	X	X					
280/2.0	X	X	X				X					
280/1.6	X	X	X	X	X	X	X	X	X		X	

Note: 1. A = 12g neo; B = 12g emb; C = 14g neo.

Test Specimens

The dimensions of shear and pull-out test specimens are shown in Figures 6 and 8. The number of specimens of any one series was determined from clause C.2.6 of the European recommendations.

The source of the steel materials is the same as that in Table 2. The sheeting strips were cut by guillotine and the purlin strips sawn from their flanges. After marking out the fastener positions, the strips were rigidly clamped to a steel supporting structure and the fastener installed.

All fasteners were installed using procedures compatible with good site practice and in accordance with the manufacturers' recommendations. The objective was to produce a connection equal to the best that could be expected from favourable site conditions rather than to produce the best possible connection. Details of installation procedures employed are shown in Appendix A.

Test Procedures

The shear and pull-out tests were carried out in accordance with the European recommendations.

Loads were applied using a 100 kN servo-jack unit mounted in a "Dartec M1000/RE" straining frame. This system incorporated a 100 kN capacity resistance strain gauge load cell for measurement of loads to an accuracy within $\pm0.5\%$ of reading down to 4 kN and a LVDT for measurement of crosshead displacements. Relative movements at the connection were measured by two "Sakae Model 20LP100" linear-motion potentiometers (displacement transducers), with a maximum non-linearity of $\pm0.5\%$ full scale, placed one on either side of the specimen. Deflection was taken as the mean of the two readings.

Simultaneous reading of load, deflection and crosshead displacements were recorded, using a "Hewlett-Packard 3497A" data acquisition unit, at appropriate intervals until the failure of the test piece. Independent monitoring of load and crosshead displacement was also carried out using an x-y plotter connected to the test apparatus. This provided a back-up recording of the data in addition to an immediate graphic indication of connection behaviour during the test.

The behaviour of the connection during test and the failure mode were noted. In-house computer programs were used to plot the load per connection vs deflection automatically based on the data logged.

A specimen mounted on the testing machine with the transducers and undergoing shear testing is shown in Figure 12.

RESULTS AND STATISTICAL ANALYSIS

Steel Properties

The tensile test results are shown in Table D.1 in Appendix D. Typical load-deflection curves of the steel materials are shown in Figure D.1.

Failure of the G280 purlin materials is ductile but both the G450 purlin and G550 sheeting materials exhibit brittle fracture. Of particular interest is the relatively high strength and low ductility of the sheetings.

Shear and Pull-out Tests

Numerical and Graphical Results

The characteristic strength and shear flexibility of each series of connection tested have been evaluated in accordance with the European recommendations and the results presented in Appendix E. The loads sustained by sheet-to-purlin connections at 3 mm deflection (3 mm load) are also included.

While the behaviour and characteristics of rivet seam connections in shear tests and screw connections in pull-out tests show remarkable consistency, the results for screws (both seam and sheet-to-purlin connections) in shear tests are more variable. Nonetheless, in the sheet-to-purlin connection shear tests, the maximum loads obtained for each series are reasonably consistent, i.e., over 75% and 90% of the values fall within ±10% and ±15% of the mean results respectively. Deflections at lower load values, however, differed considerably from specimen to specimen in some series thus resulting in wider variation in the shear flexibility.

Typical load-deflection curves of each test series are included as Figures F.1 to F.6 in Appendix F. The variation of load-deflection characteristics in different tests for a particular series of sheet-to-purlin connection shear tests is shown in Figure F.7.

In order to facilitate the comparison of the behaviour of local connections and those in Europe (Grimshaw, 1979), the maximum load per connection for each test was plotted against the deflection at which it occurred. These are included as Appendix G.

General Behaviour and Failure Modes

Seam connections in shear tests

The failure modes of various seam connections in shear tests are given in Table E.1 in Appendix E.

For screw ("Tapit") connections, failure was by fastener inclination followed by tearing of the sheet and eventually pull-out (Figure H.1). This is a common failure mode for this type of fastener-sheet combination and is reasonably ductile, up to a certain deflection. Although it was stated (Davies and Bryan, 1982) that a sufficiently ductile mode of failure can only be achieved provided that the lower sheet has adequate thickness, i.e., 0.65 mm minimum and the thread cutting operation causes an adequate burr; both the 0.55 mm and 0.40 mm sheets performed satisfactorily.

The behaviour of the rivet connections can be characterised by the application of loads at different stages (Figure F.1). Generally, there was an almost linear elastic behaviour followed by elastic-plastic strain, giving an indication of the behaviour to be expected at failure. After maximum load was attained, the connections failed by shearing of the rivet, yielding and tearing of the sheeting or tilting of the rivet followed by its being pulled out. The actual failure mode is a function of the thicknesses of the connected parts in relation to the type of rivet. For the connections tested, there are mainly two types of failure.

Firstly, the monel rivet connections together with those formed by 4.8 mm aluminium rivets and 0.40 mm sheets, failed in tilting or bending of the rivet in combination with local yielding or tearing of the sheets followed by pull-out from the lower sheet (Figures H.2, H.3 and H.4). There is relatively little damage to the rivet shank after the test, illustrating that the strong and stiff monel rivets enforce the failure to be in the sheets, which also applies to the larger aluminium rivets in the thinner sheets. This mode of failure is very ductile up to the point of pull-out.

Secondly, all the aluminium rivet connections, except the ones involving 4.8 mm rivets and 0.40 mm sheets, failed in bending or tilting in combination with shear of the rivet (Figures H.4, H.5 and H.6). As the shear load was increased, the rivets bent or tilted, accompanied by a combination of shearing and crushing actions (because of the relatively thin sheets, it is difficult to distinguish between these actions) where the sheets 'cut' into the rivet shank. Eventually, the rivets failed in sudden brittle fracture as the sheets separated. With "Bulb-tite" rivets, the inbuilt neoprene washer also separated from the rivet head. Even after failure, the rivet holes in these connections hardly show any sign of deformation (Figure H.7) which indicates that the rivets are inadequate in mobilising the yield of the sheets and the failure is in the rivet alone.

In both the rivet and screw connections, there is little behaviour difference in connections involving 0.55 mm and 0.40 mm sheets, and the flexibility of the connections involving the thinner sheets is higher (Table E.1). Also the tilting of the rivet before pull-out, when it occurs, is more pronounced in the series with thicker sheets than those with thinner sheets.

Sheet-to-purlin connections in shear tests

In general, the way the sheet-to-purlin screw connections work is roughly characterised by four different stages of strain, which at moderate stress, depends on the transfer of load from the sheet to the threaded screw shank. This can be explained through an idealised load-deflection curve of the connection as shown in Figure 13.

At stage one, the load is transferred through the portion of the thread formed during driving; and to a lesser extent, through friction due to the interface forces brought into action by tightening of the screw. Because of the relatively thin sheets involved, the amount of thread formed in the sheet can vary between connections. This, together with the complication of slip, makes the behaviour of the connection at this stage highly irregular; though no particular attention is necessary apart from realising that such behaviour exists.

At the second stage, the load transfer gradually increases, the threads already in contact penetrate into the sheet material and other areas are brought into contact with it. At this point, the rate of growth of strain decreases and the reasonably linear initial stiffness resulting gives an indication of the flexibility at working load.

The third stage is characterised by a marked increase in strain, as a result of commencement of yield in bearing; the initiation of tearing in the sheet material; or the initiation of tilting of the screws. There may also be some consolidation as a result of tensile forces being brought into action in some screws owing to their inclination.

At stage four, failure eventually occurs with relatively large strain, as a result of yield in bearing, and tearing of the sheet material; possibly accompanied by marked tilting of the screw. During this final stage, the load either maintains a reasonably constant value or decreases slightly, then follows a fairly plastic behaviour (Figures F.2 to F.4). However, the post-maximum load behaviour can vary between tests in the same series (Figure F.7) and its precise course is less predictable.

Davies and Bryan (1982) pointed out that the behaviour of the connections is influenced by the thickness of the thicker strap. In the present tests (Figures H.8 to H.13), most of the screws remained reasonably perpendicular to the plane of the specimen until the third stage (Figure 13) when tilting became more noticeable. With the thicker (2.5 mm) and higher yield (450 MPa) purlins, there was relatively little tilting and failure was in tearing of the sheets alone; but with thinner straps (1.6 mm to 2.0 mm) and lower yield (280 MPa) materials, there was a more marked tendency for the fastener to tilt because of local yielding of the purlin materials around the fastener (the visible influence zone is about one screw diameter in the longitudinal direction from the edge of the screw hole). This tilting may produce some interlocking of the holes and deform the washer assembly against the sheet. It is also characterised by the less distinguishable behaviour between stage two and three for the various connections with 280/1.6 purlins (Figures F.2 to F.4) and the failure mode in this situation is in fact a combination of tearing of sheet with tilting and pull-out of the fastener. The contact of the hex head of the screw with the sheeting, which restrains the sheeting from displacing, may also contribute to the slightly higher maximum loads attained in this series of connections (those involving 280/1.6 purlins) in comparison with the others (Tables E.2 and E.3).

In practice, it is sufficient to state that the tested sheet-to-purlin connections failed in bearing with tearing of the thinner sheet, possibly accompanied by the tilting or pull-out of the fastener, without distinguishing the detailed mechanism between connections with different purlins.

The behaviour of connections with neoprene washers (12g and 14g screws) are largely similar, although those with the bigger screws exhibit a slight reduction in flexibility. On the other hand, with the same screw, i.e., 12g, the strength and flexibility of the connection can be increased by the use of embossed washers instead of the neoprene (Tables E.2 and E.3). Connections involving thinner sheets (0.40 mm) are more flexible than connections involving thicker sheets.

With reference to Figures H.8 to H.13, note that the washers, either neoprene or EPDM, displaced with the sheeting which suggests that the 'adhesion' between these materials and the sheeting is quite good.

Sheet-to-purlin connections in pull-out tests

In the pull-out tests, the first two stages of the overall connection behaviour are similar to those just described for the shear tests. However, the third stage is characterised by a markedly non-linear behaviour followed by a plastic or a reduction in loadbearing capacity and then plastic stage (Figures F.5 and F.6).

Generally, failure is very ductile and consists of tearing of the sheets together with tilting and pull-out of the fastener (Figures H.14 to H.17). This is accompanied by the bending of the top two dummy sheets; a common phenomenon in single fastener shear tests (Davies and Bryan, 1982). This bending is also due to the fact that the lower torn sheet materials 'piled' up thus inducing a prying action which aggravated the sheet bending and fastener pull-out actions. However, in the working load range, i.e., within 60 per cent of the characteristic strength as obtained from the shear tests, there is no noticeable tilting in any of the connections.

With reference to Figures H.14 to H.17, the behaviour of connections with different screw and washer types are similar, though those involving high yield and thicker purlins exhibit less fastener tilting than the others. The connections with thinner sheets (0.40 mm) also have more tearing and less fastener tilting.

In Tables E.4 and E.5, it is noted that for the same fastener type and sheet thickness, this type of lapped joint arrangement produces a lower flexibility for the connections than the single sheets in shear tests (Tables E.2 and E.3).

Statistical Analysis

Objective

For seam connections in shear tests, the mean maximum load together with the variation of twice the standard deviation are presented in Figure 14.

The shear and pull-out test result for sheet-to-purlin connections can be further examined by advanced statistical approaches to evaluate the influence on the connections by different types of sheeting, fastener and purlin. This work is initiated through the observation of the mean values of various connection characteristics with respect to purlin thickness times yield strength (Figures 15 and 16).

Analysis

Data for each combination of three fastener types (12g neo, 12g emb and 14g neo) and two sheeting thicknesses (0.55 mm and 0.40 mm) were analysed separately as a balanced factorial design on purlin type to assess differences in means where the identity of the purlin was determined by the purlin nominal yield (450 MPa or 280 MPa) and purlin nominal thickness (1.6 mm, 1.9 mm (2.0 mm) or 2.5 mm). Eight and five replicates of each combination of fastener, sheeting thickness and purlin were used in the shear tests and pull-out tests respectively. Shear and pull-out test results were analysed separately.

The three variables of interest were maximum load, 3 mm load and flexibility. A statistical test was used to check that standard deviation of results for a given fastener and sheeting thickness were the same across the purlin type. Analyses of variance tables (Tables I.1 and I.2) were produced to assess the effect of the purlins.

In Tables I.1 and I.2, the smaller the p-value, the less likely it is to observe an effect by chance when in fact the effect is zero. If the confidence level of the results is 95% (to be consistent with the European recommendations), the criterion for a particular variable being influential is when the p-value is less than 0.05.

If the purlin type or other variables are found to have no effect on the connection characteristics, then a characteristic value for the group can be calculated as

$$P = P_m - c_p . s_p \qquad .... (7)$$

with

$$s_p = \{(\Sigma\, v_i . s_i^2)/(\Sigma\, v_i)\}^{1/2} \qquad .... (8)$$

where

P = characteristic value of a characteristic of the group of connections from pooled results.

P_m = mean value of a characteristic of the group of connections.

c_p = coefficient taken from the student distribution tables based on a lower one-sided tolerance limit of 95% and the total number of samples.

s_p = pooled standard deviation.

s_i = standard deviation of the i^{th} group of result.

v_i = degrees of freedom of s_i (number of samples in the i^{th} group minus one).

Shear Test Results

With reference to Table I.1, the following deductions can be made:

(1) For maximum loads, purlin type (yield and thickness) in general has no effect except that with 0.40 mm sheeting and 12g neo fastener. A plot of the maximum load vs the purlin thickness for this group (Figure 17) indicates that the thin purlins (1.6 mm, both G450 and G280) can constitute a comparatively weak connection.

For the other cases, the mean values are practically the same and it is possible to pool together the cases where similar results occur. These are shown as characteristic strengths in Table I.3. It is evident that characteristic strength numbers 1 and 6, 2 and 5, 3 and 7 are very similar with differences being less than 3%. Thus, in order to simplify the design data, these similar cases are further combined to give the characteristic strengths of connections shown in Table I.4.

(2) For the 3 mm load, both the purlin and the fastener type have insignificant effects; the only important variable is the sheeting thickness. Thus the corresponding data are also pooled and the results presented in Table I.4.

(3) With regard to flexibility, although there may be purlin type influence, it is well known (Davies and Bryan, 1982; Grimshaw, 1979) that flexibilities of screw connections can vary between ± 40 per cent of their mean. The present variability is considered to be random variations rather than a definite influence by the purlin. Hence the flexibility values are given in Table I.4 without further processing.

Pull-out Test Results

Analysis of the pull-out test results is shown in Table I.2. Although the analysis suggests certain interaction of both sheeting and purlin, these are not pursued further as the objective of the pull-out test is to examine the strength and ductility of the connection in comparison with that from the shear test; though it is interesting to note that the types of fastener tested has relatively little effect on the maximum load and the 3 mm load for a particular sheeting thickness.

Design Expressions

The strength and flexibility of various connection types computed from the design expressions in Appendix C together with the test results are included as Tables J.1 to J.4 in Appendix J.

For failure in tilting and bearing of the rivets in Table J.1, which applies to monel rivets and 4.8 mm aluminium rivets in 0.40 mm sheets, the expressions generally give conservative results except those by Baehre et al (1973), Stark et al (1978) and ECCS (1984b) where they overestimated the connection strength in some cases.

In the same table, the test strength of the rivet connections with fastener shear failure, i.e., the aluminium rivets, show reasonable agreement with the shear strength of the fastener as supplied by the manufacturers though the present test strengths are somewhat lower. It should be realised that the thicknesses of the materials used in the manufacturers' tests are much higher than the thin sheets used in the present tests; and the 'knife-edge' effect which causes stress concentrated crack opening and crushing actions can result in a lower shear strength for the fasteners. This effect may be more severe for the "Bulb-tite" rivets than the aluminium rivets thus causing a greater discrepancy in the shear strength values.

In Table J.2, the flexibilities predicted by SISC (1982) are generally conservative for monel and aluminium rivets but not so for "Bulb-tite" rivets.

Some design expressions for the strength of screw connections (Table J.3) give reasonable estimates for the 4.8 mm screws but the results for 5.5 mm and 6.3 mm screws are gross overestimates of the test values. Failure by shearing of the screw is seldom critical.

With reference to Table J.4, all the design expressions provide underestimates of the flexibility of the screw connections except the case with 4.8 mm screws where SISC (1982) gives conservative results.

DISCUSSION

Design Philosophy of Local Connections Based on Tests

There are some essential differences between profiled sheet steel cladding systems in New Zealand and those in North America and Europe (Yiu, 1987); those pertinent to the structural characteristics of diaphragm connections include the gauge, yield strength and ductility of the steel sheets. The European recommendations need to be closely re-examined in light of the present tests; in particular the deformation capacity requirements and the deduction of characteristic and design loads from tests of local connections are discussed in detail below.

Background to Deformation Capacity Requirements

The background to the specific figures regarding deformation capacity in the European recommendations is not apparent from the literature. Without a certain amplitude of deformation before failure, connections can cause brittle fracture of a structure or its elements (Bakker and Stark, 1974). In this respect, the connection must be capable of plastic deformation. Connections in which the fastener fails through shear or other modes without appreciable plastic margin should be avoided. Preference should be given to failure in bearing, yielding of the sheets or inclination of the fastener, i.e., behaviour as shown in Figure 18(a) to (c) rather than that in 18(d) - (Figure 18 is reproduced from Stark and Toma (1978)).

Stark and Toma (1979) pointed out that a failure mode with little strain capacity has disadvantages because of its inability to redistribute the load in the structure.

Of special interest to the New Zealand situation is the European recommendations' requirements of minimum 0.5 mm and 3 mm deflections at failure for seam and sheet-to-purlin connections respectively. It is important to realise that failure is not when the maximum test load is reached but rather the total collapse of the connection (Toma, 1988). With sheet-to-purlin connections, it is the plastic behaviour which is important rather than the occurrence of a reduction in loadbearing capacity during the deformation process.

It is understood (Toma, 1988) that the 0.5 mm requirement for seam connections was judged by the European Committee to be sufficient because the deformations of different connections in a seam are almost equal; also the hole clearance is very small. The 3 mm requirement for sheet-to-purlin connections was based on known test results within the committee, with load-deflection diagrams in (a) and (b) of Figure 9 considered as acceptable but not that in 9 (c), i.e., failure prior to 3 mm with the failure mode being shear of the fastener itself.

Deformation Capacity and the Importance of the 3 mm Load for Local Connections

All the local seam connections satisfied the 0.5 mm minimum deflection requirement (Figure G.1). Therefore there is no further comment in this respect regarding these connections.

For the sheet-to-purlin connections, the general behaviour is characterised by a reasonably plastic behaviour irrespective of whether a peak strength is attained beforehand (Figures 13 and F.2 to F.7). However, regardless of the the connection type and fastened members, most connections have the maximum test load attained at deflections less than 3 mm (Figures G.2 to G.5).

With reference to the interpretation of the term 'failure' as discussed (Toma, 1988), local sheet-to-purlin connections also have sufficient deformation capacities in that plastic behaviour is at a satisfactory load capacity, i.e., greater than the design strength. Local diaphragm panel tests (Thomson, 1987) also confirmed this plastic behaviour. This enables great simplification of the design process, i.e., secondary stresses can be neglected.

Another interesting characteristic of the sheet-to-purlin connections tested is that the strength at 3 mm deflection (3 mm load) (Figures F.2 to F.6) is a reasonable representation of the plastic strength of the connection, up to a certain deformation. The consistency of this variable for different test series is illustrated in Tables E.2 to E.5; association with the 3 mm deflection also provides a tangible and consistent basis in its definition. However, this association may not hold in situations not included in the present study and deductions following similar lines should be applied with caution.

For the determination of characteristic loads, the 3 mm criterion in the European recommendations (Figure 9 (a) and (b)) may also be related to the fact that if the connections follow a non-linear behaviour and attain maximum load at say 10 mm or larger deflections, it would be inappropriate to reduce this load to obtain the design load. The actual deflection has to be sufficiently large to mobilise the working load computed in this manner. This may cause permanent damage, impairing the serviceability of the connection. Thus the specification of a deflection limit for deducing design capacity is a sound step in correlating the idealised design and the actual behaviour. In this case, the 3 mm figure seems a reasonable choice, with particular reference to the European experience (Grimshaw, 1979; Toma, 1988).

In New Zealand, there are two important strengths for the connections tested, i.e., the maximum load attained within 3 mm deflection (Figure 9 (a) and (b)) and the 'idealised plastic' load (3 mm load). In some situations (Figures F.2 to F.7), these figures are identical and especially in cases where the failure mode is associated with large tilting of the fasteners, e.g., with 0.55 mm sheets and 280/1.6 purlins (Figures F.2 to F.6), the 3 mm load is usually a conservative estimate of the 'plastic' load (as in Figure 9 (a)) with serviceability being more critical for such cases.

It is noted that the 3 mm load is independent of the washer type but is related to the sheeting and screw characteristics (see discussions later and Table I.4). For grossly different washer systems, e.g., steel washer only, different results can be expected. Nevertheless, the 3 mm load determined can be treated as a conservative estimate of the plastic strength for connections involving the same screw and sheeting but with stronger and stiffer washer systems.

The adoption of a plastic load (3 mm load) in diaphragm connection design is analogous to the use of critical state shear strength in geotechnical design whereby the uncertainties in density, bedding, orientation, strain localisation and strain path, which would affect typical design decisions, would often be best countered by a reliance solely on the critical state strength component.

Design Philosophy for the Strength of Local Connections

The fundamental design philosophy of diaphragm connections is to maintain a safety margin in strength for connection failure as well as to limit the deformations under working load. In practice, such rigid association between safety and functional requirements is often unnecessary and uneconomical; but is essential in order that a more liberal design and uniform test and evaluation methods be applied.

With the determination of the connection design strength based on the load-deflection diagram established experimentally, it has been pointed out (Strnad, 1979) that the character of the diagram, and mainly the development of the plastic deformations, determines whether the approach 'from above' or 'from below' should be used (Figure 19).

The approach 'from above' consists of establishing the design strength by dividing the maximum strength by a material factor. It is the recommended approach if plastic deformation increases substantially only in the final stage of loading. This design strength normally remains in the elasto-plastic range of the load-deflection diagram and leads to only small plastic deformations.

However, if there is considerable plastic deformation at the beginning of loading, the approach 'from below' provides a better determination of the design strength because the confinement of the design strength into the initial stage of the elasto-plastic range of the load-deflection diagram is more accurate.

With reference to the present results (Figure F.1 to F.7), although some of the seam and sheet-to-purlin connections exhibit early non-linear behaviour, within the working load range, i.e., about sixty per cent of the characteristic load (from the maximum load or the 3 mm load), the connection characteristics can be well represented by an idealised linear behaviour. The design loads deduced are also at the early elasto-plastic range with relatively small plastic deformation. Thus the 'from above' approach is suitable, which is in line with the European recommendations.

Material Factor and other Correction Factors

The European recommendations proposed a material factor of 1.1 based on the fact that a large number of connections are acting together in a diaphragm and every connection has sufficient deformation capacity, as well as on the statistical nature of the characteristic strength. Recognising the consistency of performance of the connections (as in the present case and also on previous tests (Thomson, 1987)) and provided that the degree of site supervision is sufficient to detect malpractice, the same figure has also been proposed for local use (Clifton, 1987).

With reference to Appendix D, the thickness and yield correction factors (Equation (3)) related to the present tests are shown in Tables 5 and 6 respectively.

Table 5: Thickness correction factors

Nominal sheeting thickness (mm)	Test sheeting thickness (mm)	Minimum sheeting thickness [1] (mm)	Thickness correction factor K_t
0.40	0.393	0.35	0.89
0.55	0.544	0.50	0.92

Note: 1. From Table 4 of NZS 3441 (1978)

Table 6: Yield correction factors

Guaranteed or tested yield strength (MPa)	Yield correction factor K_o
750	1.00
700	0.93
650	0.87
600	0.80
550	0.73

Notes: 1. The slight difference in tensile strength between 0.40 mm and 0.55 mm sheets in Appendix D is insignificant for design purposes.
2. Linear interpolation can be used for intermediate yield strength.

If these corrections are applied to the present results strictly in accordance with the European recommendations (Equation (3)), i.e., the minimum steel thickness coupled with the nominal yield of 550 MPa, the product of the correction factors is about 0.65 which is unduly conservative for practical design purposes.

For yield corrections, the factors 0.87 and 0.80 should be appropriate for 0.40 mm and 0.55 mm G550 materials respectively on the understanding that the strength of locally produced sheets rarely falls below 650 MPa and 600 MPa respectively. The thickness correction is unnecessary if the cladding thickness is not less than that in the present tests. The engineer should satisfy himself that the assumptions in his design regarding minimum thickness and yield are not violated in practice.

If the diaphragm strength is controlled by the seam capacity, the correct choice of the correction factors would have a direct bearing on the theoretical strength.

As mentioned before, one of the shortcomings of the European recommendations in specifying the thickness and yield corrections to the design strength (Equation (3)) is the failure to mention that such correction is related to the failure mode of the connection.

It is obvious that if the failure is by shearing of the fastener, then it is the fastener strength which is important, and the yield strength and thickness of the sheets, provided they do not fall below a certain limit, would not affect the failure mode and thus the failure load. Hence, applying the yield and thickness correction for such failure would in most cases be grossly conservative and illogical. As a general guide, provided that the fastener test strength in this failure mode is greater than 1.3 times the strength obtained from the design expressions in ECCS (1984b) for failure due to tilting and bearing, there is no need for yield and thickness corrections.

Design Philosophy for the Flexibility of Local Connections

In the European recommendations, the connection flexibility is determined on the basis of tests and a suitable factor γ_1 (Equation (5)). This assumes that the total deformation and elastic deformation are identical within the design range. This simplification leads to relatively simple calculations which in most cases are sufficiently accurate.

In the present tests, the mean flexibilities (Tables E.1 to E.5) correlate very well with the initial slope (measured with respect to the load axis) of the load-deflection graphs (Figures F.1 to F.7) within the working load range, which justifies this approach for local use.

In general, although only 'elastic' flexibility has been determined, its application to the deformation calculation of all the connections is fully justified because even if plastic deformation exists in the most highly loaded connection, most of the connections remain within the range where the 'elastic flexibility' is applicable. Hence this simplification is acceptable (Strnad, 1984).

The above assumptions are sufficiently accurate in case of single loading; for repeated loading it is better to determine the connection flexibility separately for the initial loading and subsequent repeated loads. This is particularly important if realistic data is required for theoretical repeated load analysis (Strnad, 1981).

Although connection flexibility is determined by static loading, it is satisfactory for critical deflection considerations because under unidirectional loadings, it has been demonstrated (Strnad, 1979) that after the first load application, subsequent response showed a stable hysteresis loop with a flexibility considerably lower than that obtained during the initial loading and the reloading behaviour remained stable over many cycles of loads. Thus the initial flexibility is critical as regards maximum deformation of a structure.

Design Philosophy for Local Tested Connections

The basic design philosophy in the European recommendations is largely suitable for local use.

For conventional design, the characteristic strength of the connection should be deduced from the maximum load attained within 3 mm deflection. The tested connections have sufficient deformation capacity, thus secondary stresses need not be taken into account in normal situations.

For sheet-to-purlin connections, the 3 mm load represents the plastic strength. The characteristic strength deduced from this variable should be used where large deformation is expected, e.g., earthquake situations, and also in situations where the use of plastic strength is appropriate. In cases where temperature or other secondary forces need to be designed for, e.g., when the diaphragm length exceeds the maximum recommended (see discussion later and Thomson, 1987), the design strength should be consistent with the deformation computed, i.e., if the deformation is greater than 3 mm, the characteristic strength based on the 3 mm load should be used.

It is also noted that the strength of a sheet-to-purlin connection is influenced by the type of washer and tightness of the assembly. On site, the assembly procedure should always be in accordance with the manufacturers' recommendations. For connections not included in the present study but consist of the same generic type screw and sheeting with stronger and stiffer washers, their characteristic strength can be conservatively deduced from the corresponding 3 mm load if test data are not available.

With reference to the present results (Table I.4), the use of the 3 mm load means about 7 to 21% reduction in conventional design strength. In this respect, it is noted that normally the diaphragm strength is controlled by the seam or sheet-to-shear connector connection strength (Clifton, 1987; Thomson, 1987). Hence the sheet-to-purlin connection strength is less critical than it would appear.

The determination of flexibility based on the conventional characteristic strength for the connections is satisfactory.

The most crucial decision by the engineer would be the proper choice of the yield and thickness correction factors, as they can affect both the seam and sheet-to-purlin connection strength drastically; consequently the economy of the design. The necessity for these corrections is related to the failure mode of the connection, and the engineer should decide whether such adjustment is appropriate.

Test Results

Behaviour of Seam Connections

The failure mode of seam connections is related to the relative strength and stiffness of sheeting and fastener.

For screw connections, Davies (1976) pointed out that this type of connection may be unsatisfactory at the thinner end of the range of sheet thickness, due to the tendency of the fasteners to tilt sideways as load increases followed by failure by the fastener pull out of the lower sheet. This is confirmed by the present tests. It was argued that in sheeting of the order of 0.5 mm thickness, an adequate thread cannot be formed, resulting in not only a lower failure load than might be anticipated but equally significantly a reduction in ductility. For the "Tapit" connections, the behaviour is satisfactory within the working load range, but the ductility issue needs proper attention, and the tilting of the washer in extreme conditions may also introduce weather penetration problems.

For rivet connections, shear failure of the fastener can be prevented by using strong fastener materials. The amount of tilting of the rivet can also be minimised by adopting a rivet with a large and stiff head, but the length of the rivet is not important if it suits the parts connected. Flexibility of the monel and "Bulb-tite" rivets is largely similar for the same fastener type irrespective of the thickness of the connection (Table E.1), suggesting that in these cases it is either the rivet or the sheeting material that deforms more.

Behaviour of Sheet-to-purlin Connections

For sheet-to-purlin connections under shear, plastic deformation normally prevails over elastic deformations from the early stage of the loading. This is brought about by (1) the uneven distribution of stresses and stress concentration at the vicinity of the fastener caused by the low sheeting thickness to screw diameter ratio, (2) the pressure of the screw thread on the threads unevenly cut by the screw in the thinner sheets and (3) the tilting of the practically undamaged screw during loading.

The behaviour of sheet-to-purlin connections is also influenced by the relative strength and thickness of the connected sheets. Strnad (1979) suggested that the failure mode of screw connections is related to the non-dimensional factor x as defined in Table C.2; which is the ratio of the thickness times ultimate strength for the thicker sheet to that of the thinner. Tilting and pull-out of the fastener requires x to be equal to 1. For tearing of the sheets, x should be greater than or equal to 3; and the mode is characterised by tension cracking at the side of the elongated hole in the thin member while the screw remains firm in the thick member. In cases where x is between 1 and 3, a combined mode of failure can occur.

With reference to the present tests, the factor x for various test combinations is shown in Table 7.

Table 7: Factor x for various tests combinations

Type of test	Sheeting thickness(mm)	Purlin type 450/2.5	450/1.9	450/1.6	280/2.5	280/2.0	280/1.6
Shear	0.55	3.4	3.1	2.5	2.5	1.9	1.6
	0.40	4.8	4.4	3.5	3.5	2.6	2.2
Pull-out	0.55	1.7	1.6	1.2	1.2	0.9	0.8
	0.40	2.4	2.2	1.7	1.7	1.3	1.1

Referring to the failure modes in Figures H.8 to H.17, Strnad's prediction of the failure mode using the factor x correlates well with the present results. For seam connections, x is always 1. Hence failure is by tilting and pull-out which ties in with the "Tapit" behaviour.

Characteristics of Seam Connections

In Table E.1, the strength of the seam connections increases with the sheet thickness and fastener diameter.

For failure modes involving failure in the sheeting, i.e., screw and monel rivet connections, the efficiency of the connection (characteristic load per unit thickness) generally increases with the sheet thickness. When failure is associated with the fastener, as in the cases of aluminium and "Bulb-tite" rivets, the connections with thinner sheets show higher efficiency.

The flexibility of seam connections generally decreases with increase in sheet thickness.

If a different washer system is used, the results with screw connections can be different (Davies and Bryan, 1982).

Characteristics of Sheet-to-purlin Connections

Effect of purlin type

With reference to Tables E.2 and E.3 and the statistical analysis presented, for the same fastener system, the effect of purlin yield and thickness included in the present study has practically no effect on the strength and flexibility of the connections, apart from influencing the failure modes as discussed earlier.

Effect of sheeting

In Table I.4, although the strength and efficiency of the connections increases with the sheeting thickness, the relationship is by no means linear. Also, as with the seam connections, the flexibility of the connections decrease with increase in sheeting thickness.

Effect of washer system

The effect of using a bigger and stronger washer with the same screw type (i.e., comparing 12g emb with 12g neo in Table I.4) is to increase both the characteristic strength (based on the maximum load within 3 mm deflection) and the flexibility.

However, the most important feature for these two fastening systems is that their characteristic strengths from the 3 mm load are practically the same, probably because when the eventual failure takes place in the sheet through bearing, yielding and tearing, the connection has attained the 'critical state' strength which is not influenced by the washer system and only dependent on the screw and sheeting characteristics.

Effects of screw size

The larger screws, i.e., 14g, have no advantage in terms of strength when compared with the smaller screws (12g), both with neoprene washers; although a reduction in flexibility is evident (Table I.4). This is true for both the conventional characteristic strength and the characteristic strength from the 3 mm load as well as for both sheet thicknesses.

For the failed samples (Figure 20), the visible areas of distress in the sheeting, caused by the two different screws, are very similar. A comparison of the characteristics of the samples in Figure 20 is shown in Table 8. The results of 12g emb connections are also included for information.

Table 8: Characteristics of connections

Sheeting thickness (mm)	Fastener type	Maximum load (kN)	Maximum load at deflection (mm)	3 mm load (kN)	Flexibility (mm/kN)
0.55	12g neo	4.03	2.39	3.75	0.22
	12g emb	4.18	3.17	4.12	0.34
	14g neo	4.05	1.25	3.47	0.23
0.40	12g neo	2.47	1.37	2.08	0.35
	12g emb	2.81	3.09	2.66	0.36
	14g neo	2.52	1.97	2.23	0.32

The similarity of strength may be due to the fact that the thread characteristics of the 12g and 14g screws are somewhat different (Figure 20). This difference may not cause noticable variation in connection characteristics when the sheet thickness is comparable with the screw size or the thread spacings, e.g., common U.K. sheets have thicknesses of 0.75 mm to 1.00 mm and the test results generally indicated a logical increase in strength with an increase in the screw diameter (Davies and Bryan, 1982). Although in one case, Davies and Bryan (1982) and Grimshaw (1979) did experience a higher strength with 10g screws in comparison with 12g self-drilling self-tapping screws with neoprene washers used in conjuction with similar sheeting and purlin arrangements.

In New Zealand, with the thin sheets used, the interaction between the sheet and the screw thread will have a profound influence on the characteristics of the connection. In this case, the different interaction between the 12g and 14g screws and the sheets resulted in similar maximum and 3 mm strengths.

The possibility of this complicated interaction also highlights that tests are absolutely vital in determining the characteristics of local connections and any theoretical proposal (e.g., the suggestion of strength being proportional to the screw diameter by various design expressions as discussed later) not confirmed by tests should be treated with extreme caution.

Application of Test Results

The tests included common sheetings, purlin and fastening systems currently employed in New Zealand.

Although the sheet-to-purlin connection test series (Table 4) does not include all the possible combinations of sheeting and purlin types, the test programme was designed to obtain upper and lower bound values for the combination under consideration. The results (Table I.4) are applicable to all the purlins covered in the series, and also to other cold-formed purlins within the yield and thickness limit covered in the present programme, i.e., nominal yield of 280 to 450 MPa and nominal thickness of 1.6 to 2.5 mm.

It has been demonstrated in the statistical analysis that, for practical design purposes, the variation of purlin type has little effect on the characteristics of sheet-to-purlin connections. This applies to cold-formed purlins commonly used in New Zealand. With hot-rolled purlins, the results may be different, but such purlins are not commonly used locally.

In the present tests, only galvanised sheets have been included; although coil-coated finishes are gaining popularity. With coil-coated finishes, the friction characteristic is largely dependent on the gross level of the coating. It is felt that a different finish of the sheeting has little effect on the characteristics of the connections and the results of other local tests (Thomson, 1987) as discussed later seem to support this view. Galvanised finishes provide a fairly smooth surface which may generate less friction than coil-coated finishes, thus producing conservative results. Therefore engineers can apply the data from the present results to other finishes.

Although the effect of insulation is not covered in this study, its inclusion would cause a significant increase in stiffness when the sheeting is fastened at alternate troughs but not when it is fastened in every trough, but in neither case does the insulation increase the strength of the diaphragm (Davies and Bryan, 1982). Hence the present results can also be used for insulated situations.

A Note on the Design of Local Connections

Diaphragm fasteners should not work loose in service, and neither pull-out nor fail in shear before causing tearing of the sheeting (various authors, see 4). The desirable failure mode is tearing in the sheeting which ensures relatively large deformations, so that load redistribution can take place. In practice, shear failure of screw fasteners is rare and can be readily checked using values from the manufacturer.

Due consideration should be given to the nature of the loading and other stiffness or ductility requirements. In this respect, the nature of the load-deflection relationship of the connection is important and any theoretical approach to obtain this characteristic is both impractical and unreliable. This further confirms the importance of tests.

Seam Connections

Both aluminium and "Bulb-tite" rivets normally failed in shear before causing distress in the sheeting, so they can be considered as unsuitable for diaphragm applications. Previous guidelines (Corrugated Steel Manufacturers' Association et al, 1981) recommended the minimum diameter of rivets to be 4 mm but also did not recommend the use of aluminium rivets.

In marine environments, the use of monel rivets may aggravate the possibility of accelerated corrosion in the steel sheeting due to dissimilar metals in contact (Thomson, 1987). Under such circumstance, aluminium rivets, which may only slightly or moderately increase the potential of corrosion, may provide a useful alternative. But their usage must be restricted to very lightly loaded structures and where ductility is not important.

Sheet-to-purlin Connections

Based on the comparison of shear and pull-out test results (Tables E.2 to E.5), the connections at lapped situations (Figure 7(d)) have a much higher strength and lower flexibility than single sheet connections (Figure 7(a)). The common practice overseas (ECCS, 1984b) of using only the data for single sheet connections in design for all lapping situations (Figure 7) is reasonable because the number of lapped connections is few compared with the number of single sheet connections. The common use of 'longrun' sheets in New Zealand also minimises the number of longitudinal laps.

Temperature Effects

In New Zealand, profiled steel sheeting can be supplied in long lengths as 'longrun' sheets. It is not uncommon for lengths of 26 m to be delivered and even longer lengths (30 m or more) may be obtained if required (Brookes, 1984). Regardless of the lengths of the sheets, the manufacturers or suppliers' recommendation for fixing to take account of temperature change is similar to that in U.K., i.e., for sheets over 8 m in length, non-rigid connections should always be used. Similar recommendations for use of sliding clips and sliding washers also exist (Corrugated Steel Manufacturers' Association et al, 1981).

In extreme cases, temperature stresses arising from temperature loading cannot be ignored in the design (Toma, 1978a), particularly if the sheet-to-purlin connections have insufficient deformation capacity. The stresses and deflection arising from temperature loading are readily calculable (Toma, 1978a) and should be considered with other relevant load cases. In this respect, the Swedish recommendation (Swedish Institute of Steel Construction (SISC), 1982) of incorporating one-third of the computed thermal stresses in the load combinations appears reasonable for local use.

Generally, the effects of temperature can be ignored if the diaphragm length is less than that recommended by Thomson (1987), i.e.,

(1) Light coloured roofs limited to 12 m, thereafter sliding washers for 8 m then sliding clips.

(2) Dark coloured roofs limited to 8 m, thereafter sliding washers for 4 m then sliding clips.

(3) If the insulation is hard up under roofing, deduct 4 m of positive fixing from the above.

(4) Galvanised roofs that are not post painted should be regarded as dark coloured due to the change in colour and surface over a number of years.

For longer diaphragm lengths, the effect of temperature should be designed for. In any case, the engineer should satisfy himself that temperature variations would not cause serviceability or other problems in the cladding.

Comparison with Previous Investigations

Thomson (1987) has performed some seam and sheet-to-purlin connection shear tests and his results are compared with the present work in Table 9.

Table 9: Comparison of local results

Connection	Fastener type	Characteristic strength (kN)		Flexibility (mm/kN)	
		Thomson	Present work	Thomson	Present work
Seam [2]	4.8 mm monel rivets	2.50	2.67	0.31	0.18
	4.8 mm aluminium rivets	1.80	2.18	0.23	0.11
	"Bulb-tite" rivets	1.60	1.61	0.49	0.42
Sheet-to-purlin [3]	12g emb	4.10	3.83	0.23	0.28

Notes: 1. Thomson's results are based on ten tests in a series while the present results are based on five and eight tests for seam and sheet-to-purlin connections respectively.
2. Sheeting thickness = 0.55 mm.
3. Sheeting thickness = 0.55 mm, purlin = 450/2.5.

In Thomson's tests, the sheets have a minimum yield of 650 MPa and they are coil-coated galvanised sheets whereas the present work uses plain galvanised sheets. But the difference in sheeting finishes is unlikely to have a major effect on the connection characteristics.

Agreement of the results in Table 9 is generally satisfactory and the failure modes reported have similarities; though the flexibilities of the monel and aluminium rivets are somewhat lower in the present work.

In the present work, the coefficient of variation for rivet connection results ranged from 0.5 to 4.0% for strength and 14 to 18% for flexibility respectively. For the sheet-to-purlin connections, the differences in characteristic strength may be due to the slight variation in tightening of the screws which is associated with the difference in flexibility. It is well known (Davies and Bryan, 1982; Grimshaw, 1979) that a tighter and hence less flexible washer assembly would result in stiffer connection, increasing the maximum load as well as providing more consistent results.

Design Expressions

Comparison of test and design expression results (Appendix J) indicates that some of the expressions are excessively conservative (particularly for rivet connections) and may even be unsafe for others (particularly for screw connections). The general implication from most of the expressions that the screw connection strength is proportional to the screw diameter is also invalid for local connections.

For the determination of strength, one of the shortcomings of design expressions is that although distinctions are made to various failure modes, possible situations involving combination of different modes are not catered for. Neither is distinction made to such important matters as fastener form and material nor to different types of screws, in particular their thread characteristics (which may be particularly important for thin sheets) and the influence of the washers.

With regard to flexibility, omission to give due consideration to the head dimensions, material and setting mechanism of the rivets and the influence of washers on screw connections also render the design expressions inadequate.

It is difficult to include all the relevant parameters in the design expressions because of the large number of proprietary components available, each having different characteristics (Davies and Bryan, 1982). Moreover, no expression is available for deformation capacity and ductility.

Since tests have to be carried out to confirm the calculated values and because the variation of local connections is small, there is no need for additional effort to derive generalised expressions for the present data. Connection characteristics should always be determined by testing which is economical and reliable.

Comparison with Overseas Data

The only comprehensive overseas connection data available is given by Davies and Bryan (1982) which are based on tests performed by Grimshaw (1979). Comparisons of the U.K. and New Zealand data are presented in Tables K.1 and K.2 of Appendix K.

These tables are based on actual test results without adjustment by the material factor or other corrections (Equation (3)). A similar comparison using the design strength may reveal a different picture. In Table K.1, although the failure mode descriptions are slightly different for the seam connections, the failure mechanism is largely similar for the same fastener.

With reference to Appendix K, for both seam and sheet-to-purlin connections, although the New Zealand sheets are thinner but of a much higher strength, the characteristic strength of similar connections are of the same order. Of special interest is that in all cases, the characteristic strength per unit thickness, which is an indication of the connection efficiency, is much higher for the New Zealand connections.

Apart from the local aluminium and "Bulb-tite" rivet connections which have inferior capacity (Table K.1), other seam connections shown comparable strength to the U.K. connections. Hence, if seam failure is the controlling mechanism, the New Zealand connections can be expected to give the same strength performance as their U.K. counterparts; although the problems of overall and local buckling (Davies, 1978) may be more critical for local sheetings.

The strength of local sheet-to-purlin connections with 0.55 mm sheets (Table K.2) is comparable to that with 0.76 mm sheets in U.K.. However, because of their thinner nature, local connections with 0.40 mm sheets have less capacity.

Despite the significant difference in the thickness, strength and ductility of the sheeting materials (Appendixes D and K), the flexibilities of the local connections are only marginally lower than that of the U.K.. This generally agrees with Davies and Bryan's view (1982) that flexibility is largely independent of the yield and tensile strength of the sheets. Moreover, the general trend of decreasing flexibility with increase in sheet thickness is also similar for connections in both countries. The higher ductility of the U.K. sheets may also be related to their sheet-to-purlin connections achieving the maximum strength at deflections larger than 3 mm.

Data for Simplified Design Tables

For the preparation of simplified design tables, it has been proposed (Bryan and Davies, 1981; Davies and Bryan, 1979) that the design values for the U.K. connections be those given in Table 10.

Table 10: U.K. data for simplified design tables

Connection	Characteristic strength per unit thickness (kN/mm)	Design strength [1] (kN)	Flexibility (mm/kN)
Seam	2.8	1.51	0.35
Sheet-to-purlin	6.6	3.56	0.35

Note: 1. Based on minimum sheet core steel thickness of 0.65 mm and the design strength is 0.83 times the characteristic strength, i.e., $\gamma_m = 1.20$.

No adjustment has been made to the design strength apart from the material factor (Equation (3)) but it was claimed that the strength and slip values are conservative for all types of fastener specified. However, the corrugated profiles included in the design tables (Bryan and Davies, 1981) are not similar to common local profiles. With reference to the New Zealand situation, based on the present work, it seems that the values given in Table 11 are appropriate for local simplified design tables.

Table 11: New Zealand data for simplified design tables

Connection	Characteristic strength per unit thickness (kN/mm)	Sheeting thickness (mm)	Design strength [1] (kN)	Flexibility (mm/kN)
Seam [2]	3.7	0.55	1.47 (1.83)	0.35
	3.6	0.40	1.13 (1.29)	0.35
Sheet-to-purlin [3]	5.5	0.55	2.18 (2.73)	0.35
	4.4	0.40	1.38 (1.59)	0.35

Notes:
1. The design strength is 0.9 times the characteristic strength, i.e., $\gamma_m = 1.1$; and the yield correction factors (Equation (3)) are 0.80 and 0.87 for 0.55 mm and 0.40 mm sheets respectively. The adjustment for sheet thickness is assumed unnecessary. Bracketed figures indicate design strength not adjusted for nominal yield, they are here for comparison only.
2. Seam fasteners should be monel rivets of 4.0 mm minimum diameter or 10g self-drilling self-tapping screws. Aluminium or "Bulb-tite" rivets are not included in this table.
3. Sheet-to-purlin fasteners should be 12g or 14g self-drilling self-tapping screws with neoprene washers.

In general, the values in Table 11 are more conservative than the U.K. values (Table 10), bearing in mind that the local sheetings are thinner and the design values have been adjusted for nominal yield whereas the U.K. values have not. The use of 6.6 kN/mm for the characteristic strength per unit thickness of sheet-to-purlin connections in U.K. is rather optimistic. With reference to Davies and Bryan (1982) and Grimshaw (1979), the value of 3.8 to 5.5 kN/mm which gives a design strength of 2.1 to 3.0 kN may be more appropriate. However, the suggestion (Bryan and Davies, 1981) of a common flexibility of 0.35 mm/kN for the U.K. connections seems reasonable for local use. In practice, the values recommended in Table 11 would generally lead to conservative design.

The data in Table 11 would be helpful at the preliminary design stage or in situations where no economy can be gained because the design is controlled by other factors, e.g., the wind suction loads may govern the number of connections required and these connections can also perform the shear function even using conservative design figures. However, the designer should be extremely cautious in using these data beyond the range of the fasteners of which they are applicable. In such circumstances, it is advisable to perform tests and base the design on the test data.

The responsibility of producing simplified design tables should rest with the manufacturers whose products would have dissimilar behaviours. This should be the case except for a common profile such as 'corrugated iron' (sinusoidal profile, NZS 3403 (1978)). Compiling design data in general tabular form involves the use of a number of conservative assumptions and as the number of variable is reduced, the design tables become simpler but also less economical.

Edge Distances, End Distances and Spacings of Mechanical Connections

A detailed discussion of the edge distance, end distance and spacing of mechanical connections as related to various recommendations, practical considerations and suitability for the New Zealand situation is presented in Appendix L.

Based on this appendix, it is concluded that the minimum edge and end distances for New Zealand fastenings should be 2d or 10 mm (minimum) and 3d or 20 mm (minimum) respectively. In any case either dimension should not be less than the washer diameter plus 2 mm; this is particularly critical for fastenings adopting embossed washers. The minimum pitch should be three times the fastener diameter while the maximum spacings should be in accordance with that recommended by Davies and Bryan (1982) as shown in Table L.2. Generally these criteria are already being complied with or can be achieved with ease in local constructions. Designers should be aware of the possibility of aggravating the risk of corrosion as described in Appendix L.

If the full connection strength can be attained when these criteria are satisfied, then the edge distance, end distance and spacing are not critical parameters that warrant further investigation.

Other Loading Regimes

Combined Shear and Tension Loads

In practice, some connections in a diaphragm can be subjected to shear and tension loads simultaneously, e.g., sheet-to-edge purlin connections in wind environments.

In the European recommendations (1984b), the consequence of combined loadings can be assessed by either linear or elliptic interaction formulae, whereas the Swedish Standard (SISC, 1982) only specified the elliptic interaction.

The linear interaction relationship was derived analytically (Grossberndt and Kniese, 1975). However, limited tests (Davies and Fisher, 1987) have been performed to support the elliptic relationship.

In practical design, except for the special case of diaphragms fastened on two sides only, forces in the sheet-to-purlin connections rarely govern the design (Davies and Bryan, 1982). Thus for the majority of diaphragms, this interaction may not be critical. It is also not important if seam failure controls and the diaphragm is fastened on all four sides. For New Zealand practices, it seems that the elliptic interaction is more appropriate than the linear.

In association with this, it should be noted that Davies and Fisher (1987) have recently recommended a safer design expression for the sheet-to-purlin connections (in comparison with that previously recommended in Davies and Bryan (1982) and ECCS (1977)), i.e.,

$$V = (0.6bF_p)/p \qquad \text{....(9)}$$

where V = permissible shear force
b = diaphragm width
F_p = design strength of an individual connection
p = pitch of the connection

The conventional expression of $V = (0.8bF_p)/p$, which implies an additional 25% reserve of safety to account for prying action and other effects, has been found to give a reasonable estimate of the failure load but is not necessarily conservative.

Fatigue Loading

Early work (Beck, 1974b) on simulated fatigue tests of shear panels, indicated that provided the fatigue loading is less than the yield load of the panel, there is little influence on the ultimate load behaviour and the flexibility. Subsequent analytical and experiment works by Strnad (1979, 1981, 1982) on individual screw connections show that under repeated reverse loadings the hysteresis loop is not stable and deflection tends to grow slowly. The entire process of loading is characterised by considerable plastic deformation. Tests on full-scale sheeted building and panels (Strnad et al, 1978) also revealed that neither the steel sheets nor the connections failed in fatigue caused by wind loads. Nevertheless, it is evident that the dynamic loads led to certain slip at the connection, without affecting any functions of the sheeting.

Klee et al (1979) performed repeated load tests and found that for shear loadings, the reduction in static strength of the connection is generally less than 20%, at maximum up to 40%. Safety factors against static strength were proposed which included the strength reduction by repeated loading.

Other workers (Baehre, 1969; Nissfolk, 1976, 1979) found that reduction in connection shear strength due to repeated loadings may occur, but it is not usually significant at less than ten thousand cycles. This work led to the recommendation (ECCS, 1984b) that the effect of repeated loading with a spectrum in accordance with wind loads can be neglected for design. As reversal of peak shear loads in diaphragms is rare, this effect may not cause any serviceability problem (Davies and Bryan, 1982).

Dynamic Earthquake Loads

The effects of dynamic earthquake loads on diaphragm connections have not been investigated seriously, though preliminary simulated dynamic action tests revealed that the failure modes (either in sheet tearing (Figure 21) or fastener shear failure) were similar to those obtained from the static tests.

This observation is also confirmed in a real structure, i.e., a warehouse for storing milk powder at Bay Milk Products Limited (a dairy factory situated on the outskirts of Edgecumbe) after an earthquake. On March 2, 1987, two earthquakes (one of magnitude (M_L) 5.2, the other magnitude 6.3) occurred near the township of Edgecumbe in the North Island, New Zealand (Pender and Robertson, 1987). The structure in question was a single storey building consisting of a long series of simple steel portals clad mainly with 'corrugated iron' (sinusoidal profile) with steel angle sheeting rails and mainly screw crest fastenings. After the earthquakes, this building had all the steel cross bracings broken, and was pushed sideways and tilted three degrees in its longitudinal direction (Figure 22) as a result of the shifting and toppling of bags of stored milk powder during the ground motion. It was only the claddings which maintained the building in a reasonable shape and prevented total collapse.

Detailed examination of the damaged structure revealed that the cladding and the sheeting rails suffered some buckling damage at various locations but most of the cladding had been torn at the sheet-to-sheet-to-purlin connections by as much as $\pm$35 mm (Figure 23). However, the fasteners were not damaged in the event (Figure 24). Similar though less severe damage was also observed in other steel-clad buildings in the area, with timber/steel sheeting rails and nail/screw and crest/trough fastenings. With nail crest fixed connections, some nails were pulled out (plain shank nails with insufficient withdrawal resistance) while others (spiral shanked nails) normally bent as a result of the sheeting displacements (Figure 24). In Figure 24, note the corrosion at the shank of the screw between sheeting and purlin.

The ductile behaviour of connections under dynamic earthquake loads was also observed in a pilot shear panel racking test (Figure 25) performed, with timber purlins and nail crest fastenings, under static loads. The fastening systems and purlin materials were different, but it can be inferred that if the fasteners do not fail or pull out, the ductile behaviour of the connections (by tearing of the sheets and shown in Figure 26) should be similar for both static panel test and the real structures under dynamic earthquake loads (Figure 23). Therefore, examination of the static failure mode for connections may be sufficient in deducing the behaviour in dynamic situations.

It is unrealistic to design for severe earthquake or other dynamic loads which may result in large displacements of the cladding at the connections. However, in moderate earthquake situations, ductile connection failure can be ensured through tearing of the sheeting while other brittle failure modes, e.g., fastener shear or pull-out, are avoided.

With reference to Figure F.1, the deformations at complete failure (i.e., shear of fastener or pull-out) for the seam connections considered, rarely exceed 3.5 mm. Thus seam connections are unreliable in diaphragm action that is mobilised by earthquake loadings or which involves large displacements, because such connections simply cannot cope with the relative movements between the claddings; even in moderate earthquake situations such as that at Edgecumbe. However, ductile modes of failure for sheet-to-purlin connections can be useful in preventing total collapse as illustrated; though it is vital to have sufficient edge and end distances for the expected displacements.

Under earthquake loads, mainly the self-weight of the cladded frames is involved. Thus the residual strength of the sheet-to-purlin connections in tearing may be sufficient to hold the structure in reasonable shape; but buckling and tension loads induced may also be critical. This also explains the relatively good performance of steel cladded roofs in domestic buildings in comparison with the heavy tiled roofs at Edgecumbe.

SUMMARY OF DESIGN RECOMMENDATIONS

Design Approach

Comprehensive design approach and equations suitable for New Zealand profiled sheet steel diaphragm design associated with steel framed buildings have been introduced by Clifton (1987), based on the work of Davies and Bryan (1982). In lieu of advanced development, the solutions proposed (Clifton, 1987) should be adopted in full except the following:

(1) The general requirement for sheet-to-perpendicular member (purlin) connections should be modified to

$$(0.6bF_p)/p \geq V^* \qquad \ldots.(10)$$

i.e., Equation (10) should replace Equation (9.7) in Clifton (1987) with the notation being the same. And Equation (9.18) (Clifton, 1987) should be replaced by Equation (11),

$$(0.6bF_p)/(p\alpha_3) \geq V^* \qquad \ldots.(11)$$

(2) In the design of sheet-to-purlin connections, the conventional characteristic strength (ECCS, 1984b) should be used. Secondary stresses need not be taken into account in normal design situations.

The characteristic strength based on the 3 mm loads should be used where large connection deformation is expected, e.g., earthquake situations, and also in situations where the use of plastic strength is appropriate.

For connections not included in the present study, which consist of the same screw type and sheeting with stronger and stiffer washer systems compared with those tested, their characteristic strength can be conservatively deduced from the corresponding 3 mm load if test data are not available.

(3) If the diaphragm length is longer than that recommended (Thomson, 1987), temperature effects have to be accounted for in the design.

(4) The thickness correction factor and yield correction factor should be included in the deduction of the connection design load, i.e., Equation (3). Its application is related to the failure mode of the connection.

The British Standards Institution is currently undertaking the drafting of the code of practice for stressed skin design (BS 5950: Part 9) under the chairmanship of Prof. Eric Bryan. The current design approach and equations should be reviewed when this document is available. The design data for local claddings presented in the next section should remain suitable unless there is a drastic change in the design philosophy, which is unlikely. Adjustments can easily be made because the basic information is also included in this report.

Design Data

For easy reference, the design data of mechanical connections in profiled sheet steel diaphragms recommended for New Zealand use are presented in Tables 12 and 13. They are based on tests in the present study. Thus Tables 12 and 13 should be used instead of Tables 9.10 and 9.9 respectively in Clifton (1987).

Table 12: Characteristics of seam connections

Fastener type [1]	Thickness of lapped sheets [2] (mm)	Characteristic strength (kN)	Flexibility (mm/kN)	Failure mode [4]
"Tapits"	0.55	2.87	0.14	1
	0.40	1.50	0.26	1
4.8 mm monel rivets	0.55	2.67	0.18	2
	0.40	1.81	0.20	2
4.0 mm monel rivets	0.55	2.04	0.20	2
	0.40	1.43	0.29	2
4.8 mm aluminium rivets	0.55	2.18	0.11	3
	0.40	1.68	0.16	2
4.0 mm aluminium rivets	0.55	1.28	0.11	3
	0.40	1.11	0.22	3
"Bulb-tite" rivets	0.55	1.61	0.42	3
	0.40	1.36	0.42	3

Notes:
1. Details of fastener types refer to Appendix A.
2. Sheets are G550 materials to NZS 3441 (1978).
3. Design strength is obtained by adjusting the characteristic strength with the yield and thickness correction factors as well as the material factor (γ_m = 1.1) (Equation (3)).
4. Failure modes: 1 = inclination of screw followed by tearing of the sheets and eventually pull-out; 2 = rivet inclination, local yielding of sheets followed by pull-out from the lower sheet; 3 = failure in bending or tilting in combination with shear of the fastener.

Table 13: Characteristics of sheet-to-purlin connections

Fastener type [1]	Sheeting thickness [2] (mm)	Purlin type [4]	Characteristic strength [5] (kN)	Characteristic strength from 3 mm load [5] (kN)	Flexibility (mm/kN)
12g neo	0.55	A	3.26	3.04	0.27
	0.40	B	2.23	1.77	0.38
		C	1.94	1.77	0.34
12g emb	0.55	A	3.70	3.04	0.30
	0.40	A	2.23	1.77	0.46
14g neo	0.55	A	3.26	3.04	0.22
	0.40	A	1.94	1.77	0.28

Notes: 1 to 3 same as in Table 12.

4. Purlin type: A - 450/2.5, 450/1.9, 450/1.6, 280/2.5, 280/2.0, 280/1.6.
   B - 450/2.5, 450/1.9, 280/2.5, 280/2.0.
   C - 450/1.6, 280/1.6.
   The characteristics given are also applicable to cold-formed purlins of intermediate yield strength or thickness.
5. The failure mode of sheet-to-purlin connections is by bearing and tearing in the thinner sheet, possibly accompanied by tilting or pull-out of the fastener.

Before using these tables, the engineer is strongly advised to become familiar with the development of the design principles, the basis of these data and other design aspects as presented in this report.

CONCLUSIONS

The principal conclusions of this study are as follows:

(1) The design of local diaphragm connections should be based on or confirmed by tests. Design expressions developed overseas should not be used.

(2) Based on the review of various established test methods, unless other boundary conditions dictate, the double fastener standard shear test and the pull-out test described in the European recommendations (ECCS, 1984b) should be used as standard test procedures for local connections.

(3) The deduction of design data based on tests described in the ECCS document is suitable for local use.

(4) Common local connections have sufficient deformation capacity, thus secondary forces need not be accounted for in normal design situations.

(5) For sheet-to-purlin connections, an alternative design strength, i.e., the characteristic strength based on the 3 mm load, is recommended for situations where large deformation is expected or the use of plastic strength is appropriate. This strength for a particular connection type can also be used, in the absence of test data, as a conservative estimate of the strength for similar connections with the same screw type and sheeting but with stronger and stiffer washer systems than those tested.

(6) In general, except for fastener failure, connection strength is a function of sheeting yield, stiffness and thickness; while connection flexibility is a function of sheeting thickness but independent of sheeting yield.

(7) For sheet-to-purlin connections, the purlin type has little influence on the connection characteristics. For the fastening systems included, the plastic strength of the connections is also independent of the washer system. The increase in screw diameter in local connections does not necessarily correspond to an increase in connection strength.

(8) The high yield and low ductility sheetings used locally resulted in some of the diaphragm connections behaving differently from those in Europe, although the strengths and flexibilities are comparable. This further confirms the applicability of profiled sheet steel cladding systems as diaphragms in local constructions.

(9) The correct choice of the yield and thickness correction factors in deducing the design load based on test results is critical to the economy of the design.

(10) Other aspects related to profiled sheet steel diaphragm connection design have been reviewed and discussed. Relevant recommendations for local use are also presented.

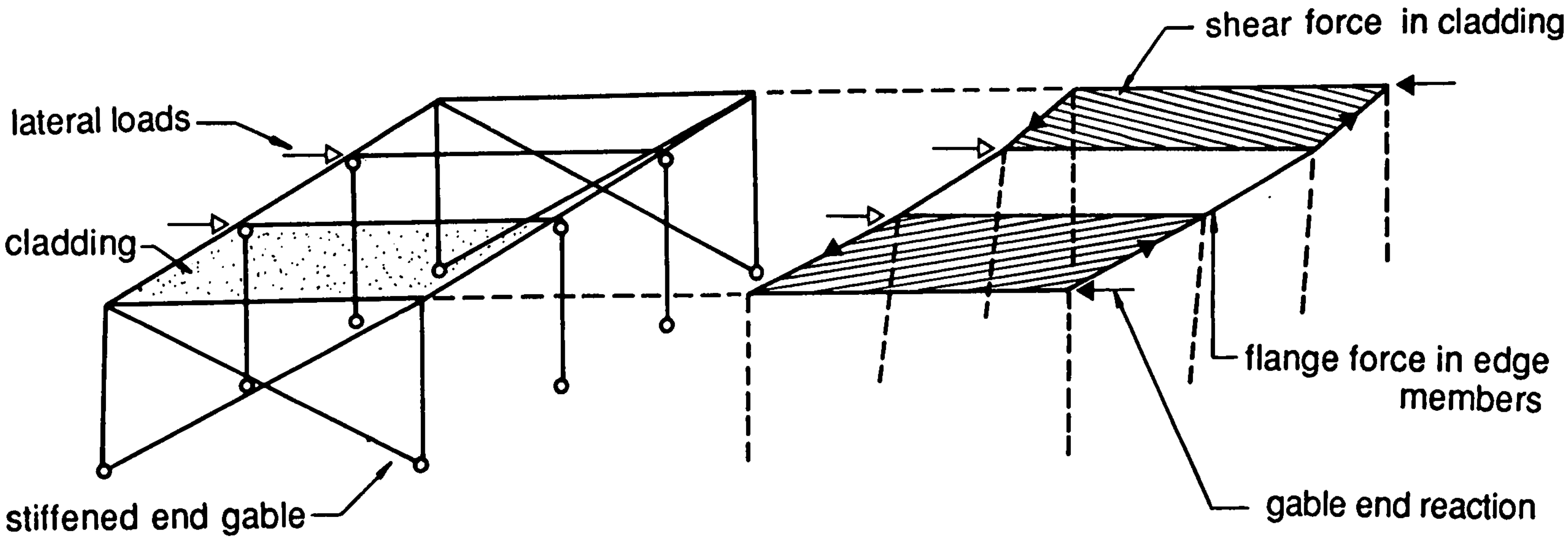


Figure 1 : Diaphragm action in a flat-roofed building with non-rigid frames.

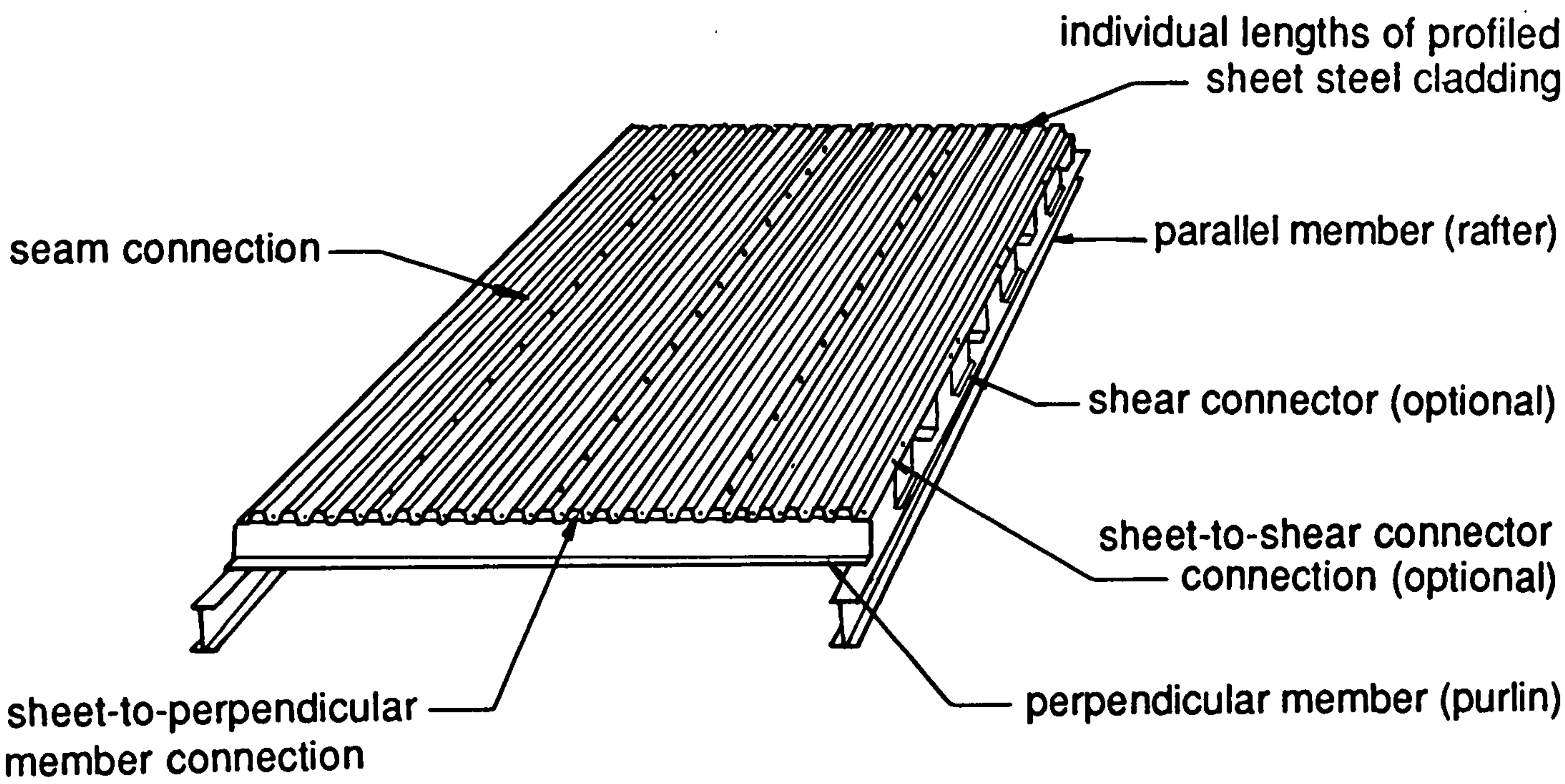


Figure 2 : Arrangement of individual panel and its components

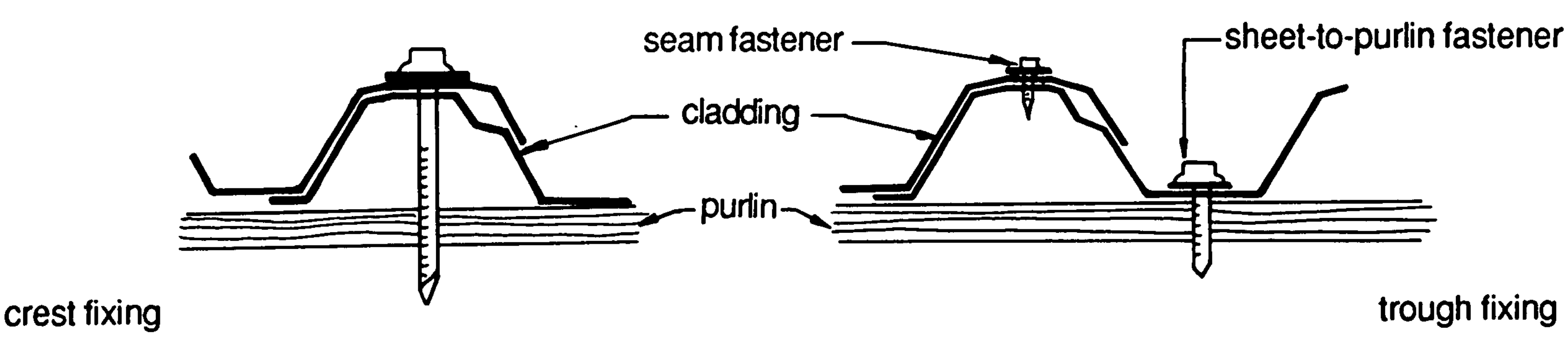


Figure 3 : General fixings

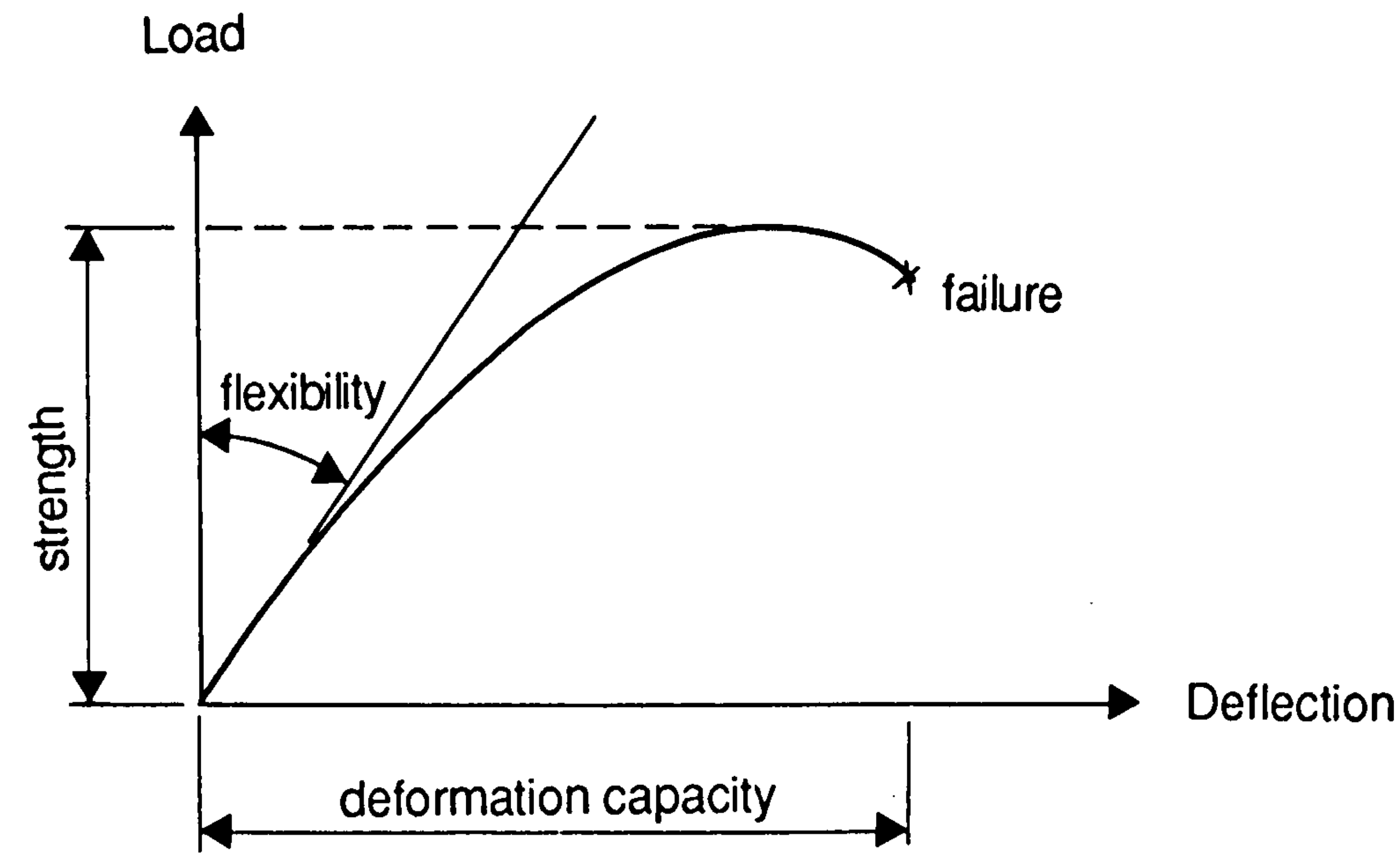


Figure 4 : Load-deflection curve of a diaphragm.

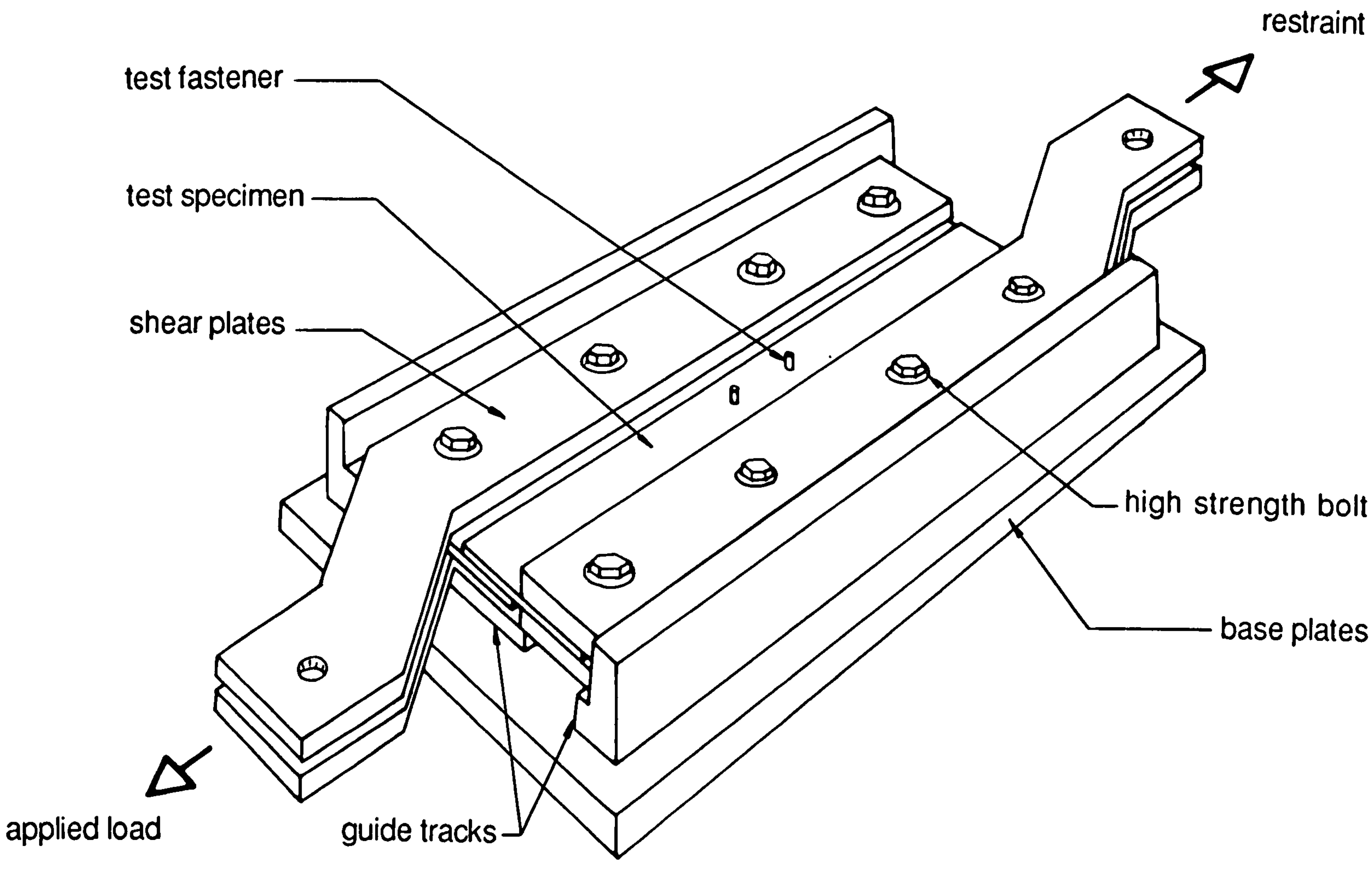
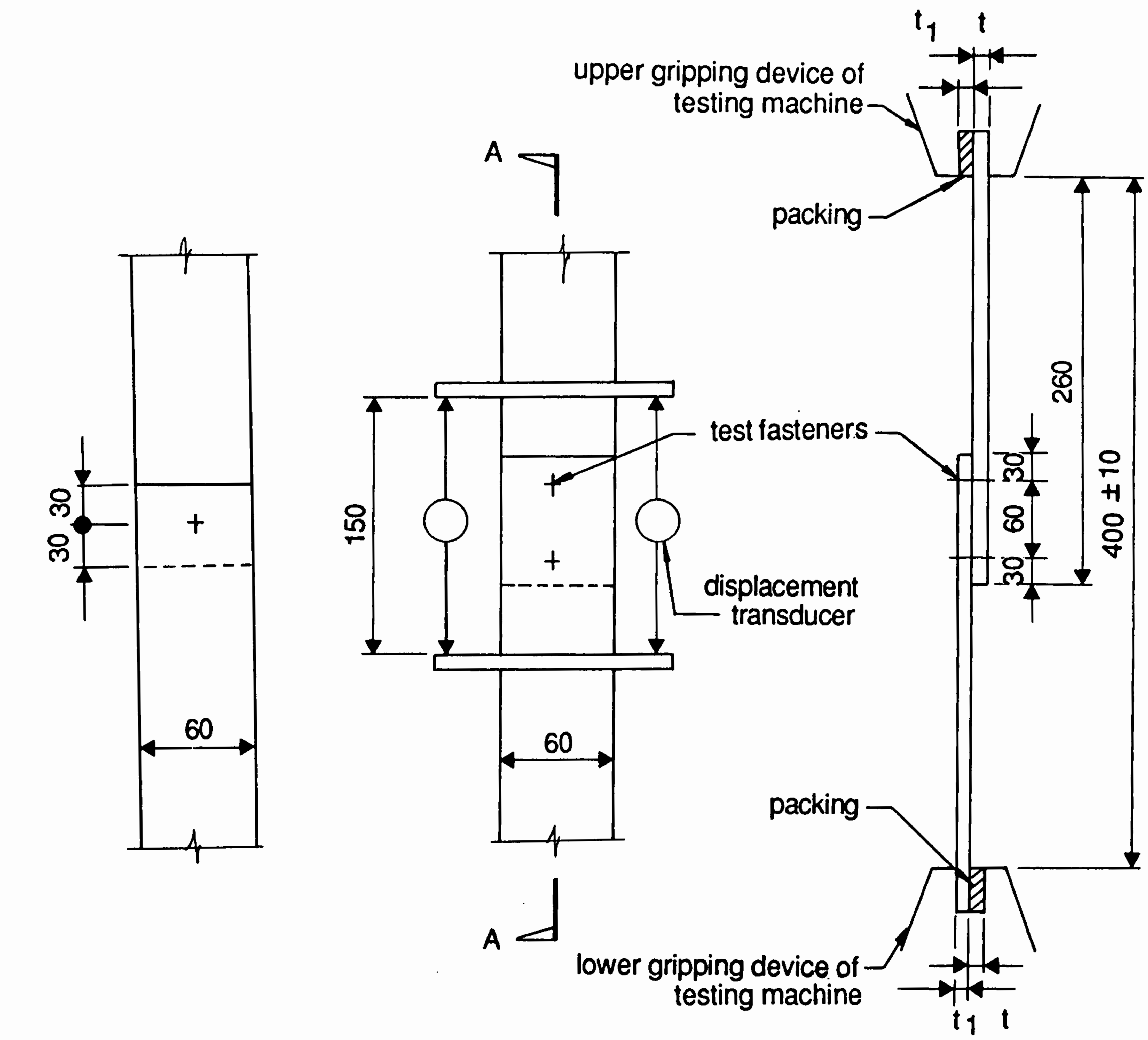


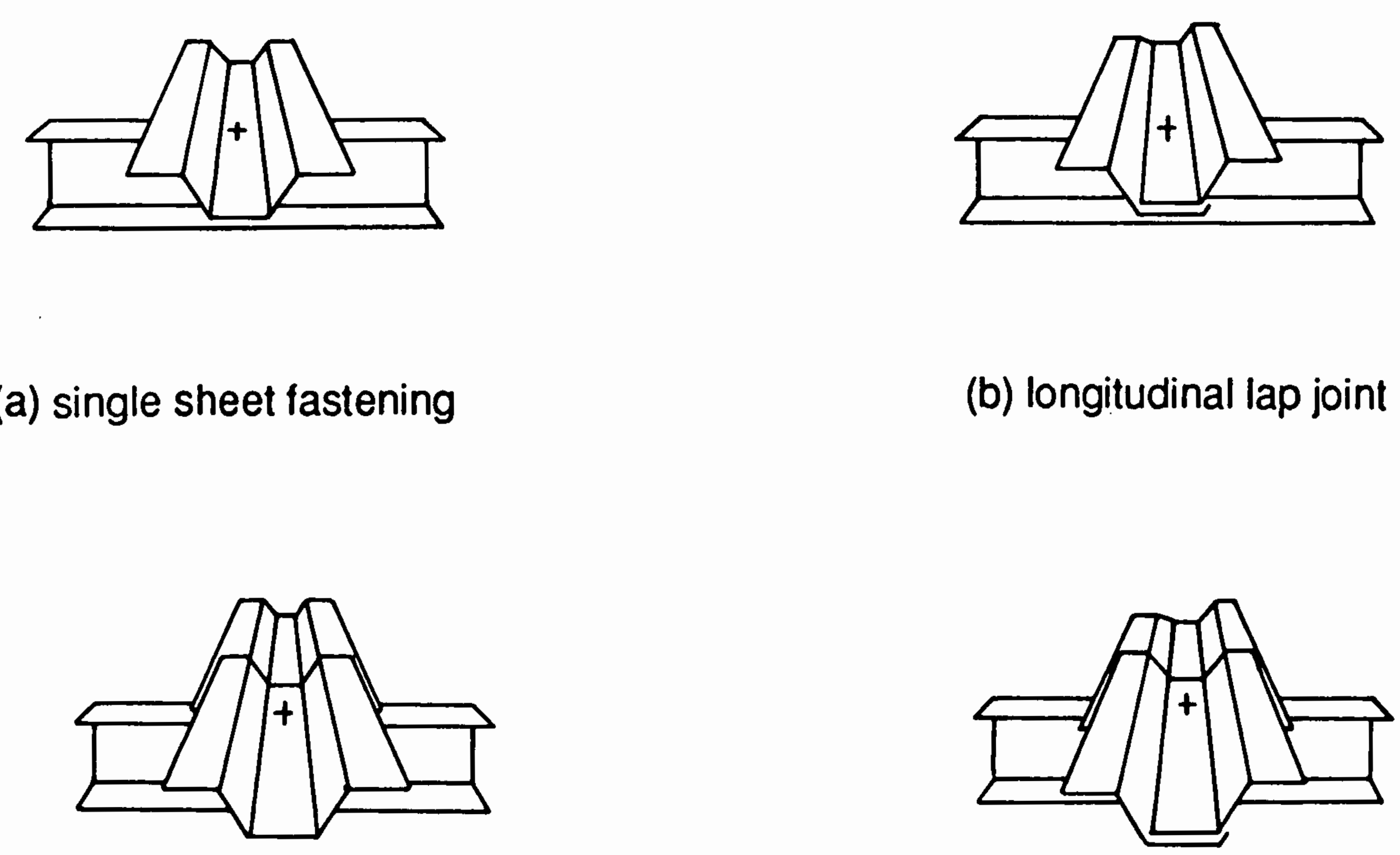
Figure 5 : Simulated diaphragm action apparatus.



(a) single fastener arrangement

(b) double fastener arrangement

Figure 6 : Similar shear test arrangements (single and double fasteners)



(a) single sheet fastening

(b) longitudinal lap joint

(c) transverse lap joint

(d) combined longitudinal and transverse lap joint

Figure 7 : Sheet to support member joints.

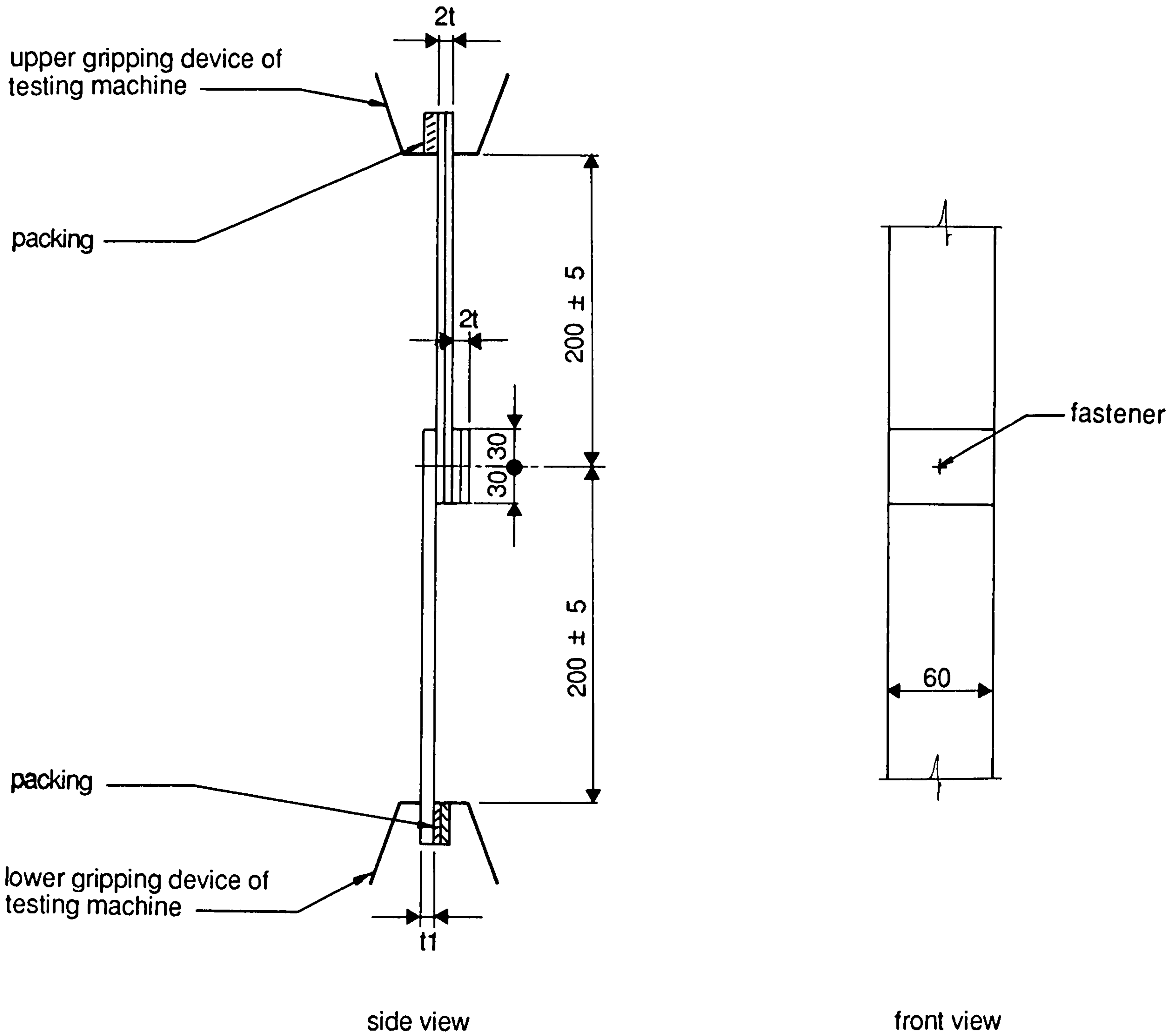


Figure 8 : Pull-out test arrangement.

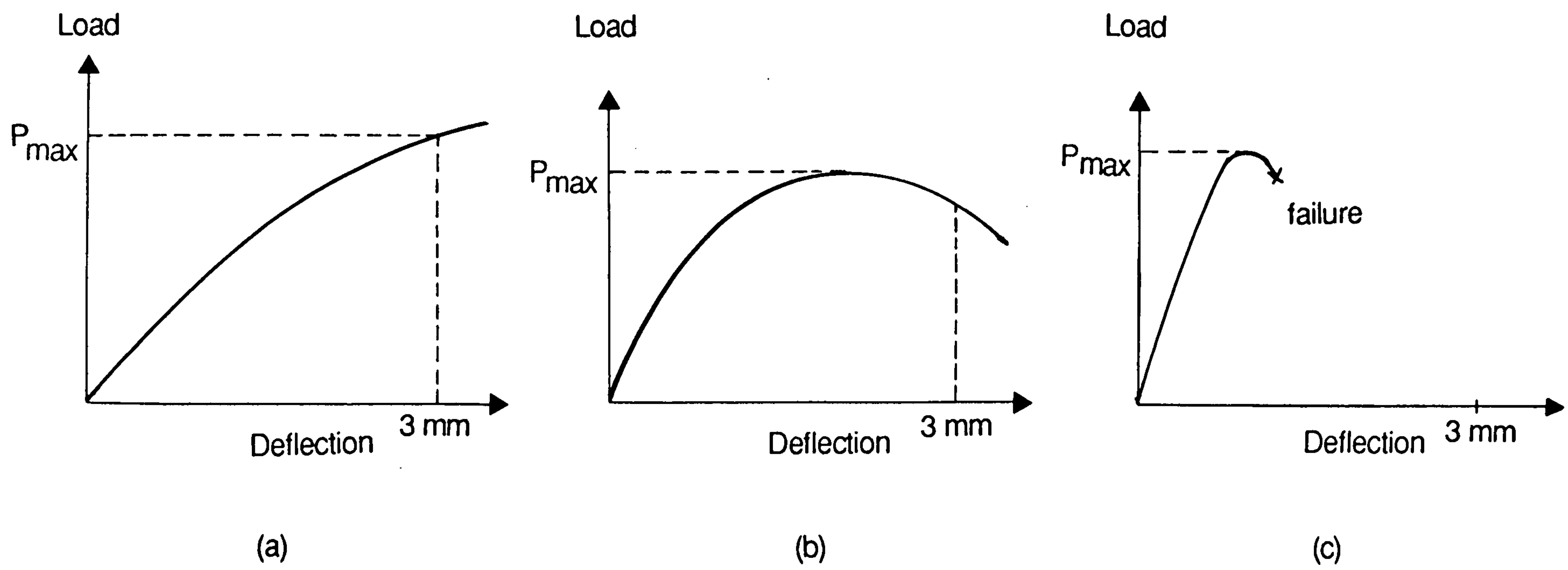


Figure 9 : Definition of maximum load.

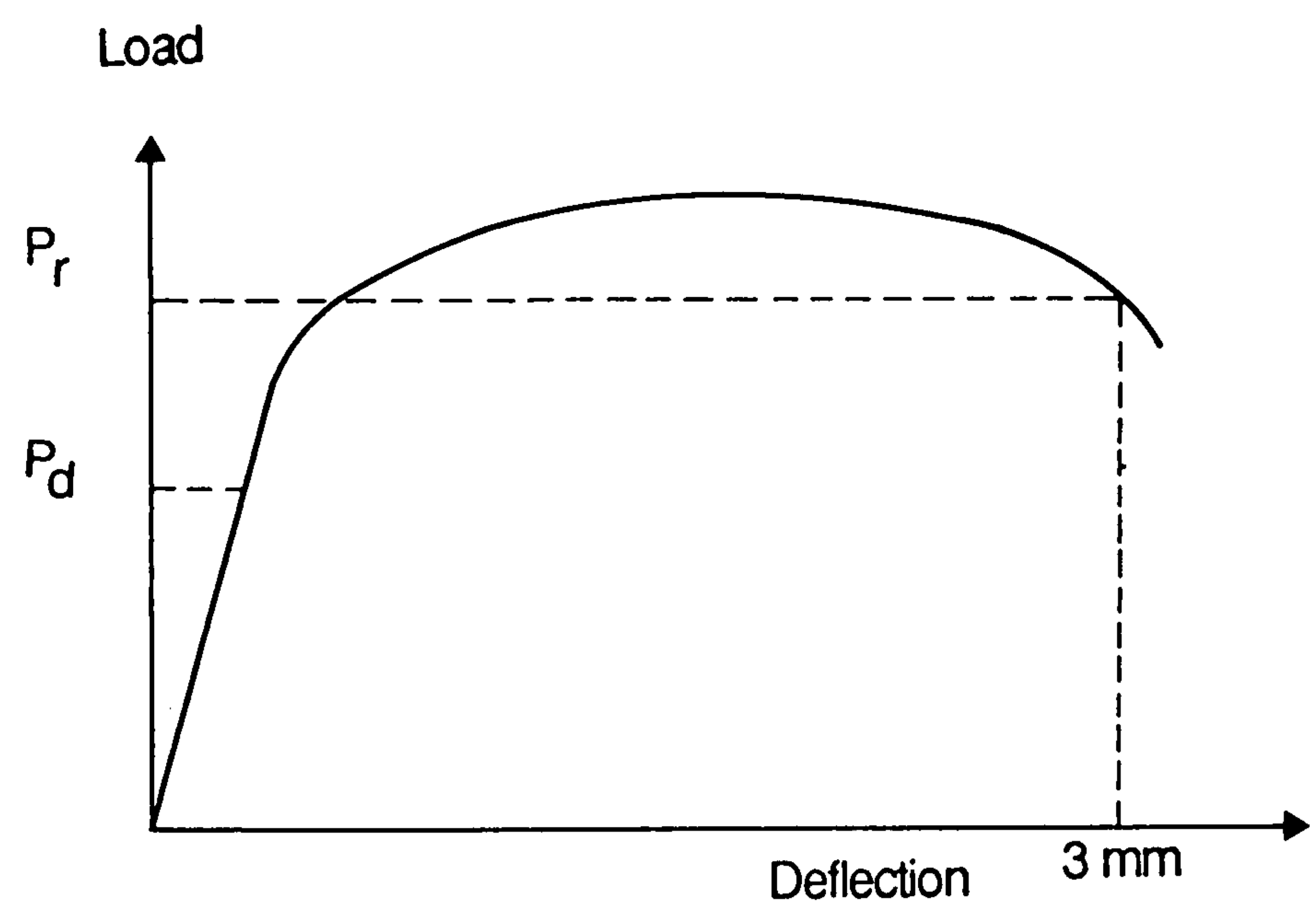


Figure 10 : Design strength requirement

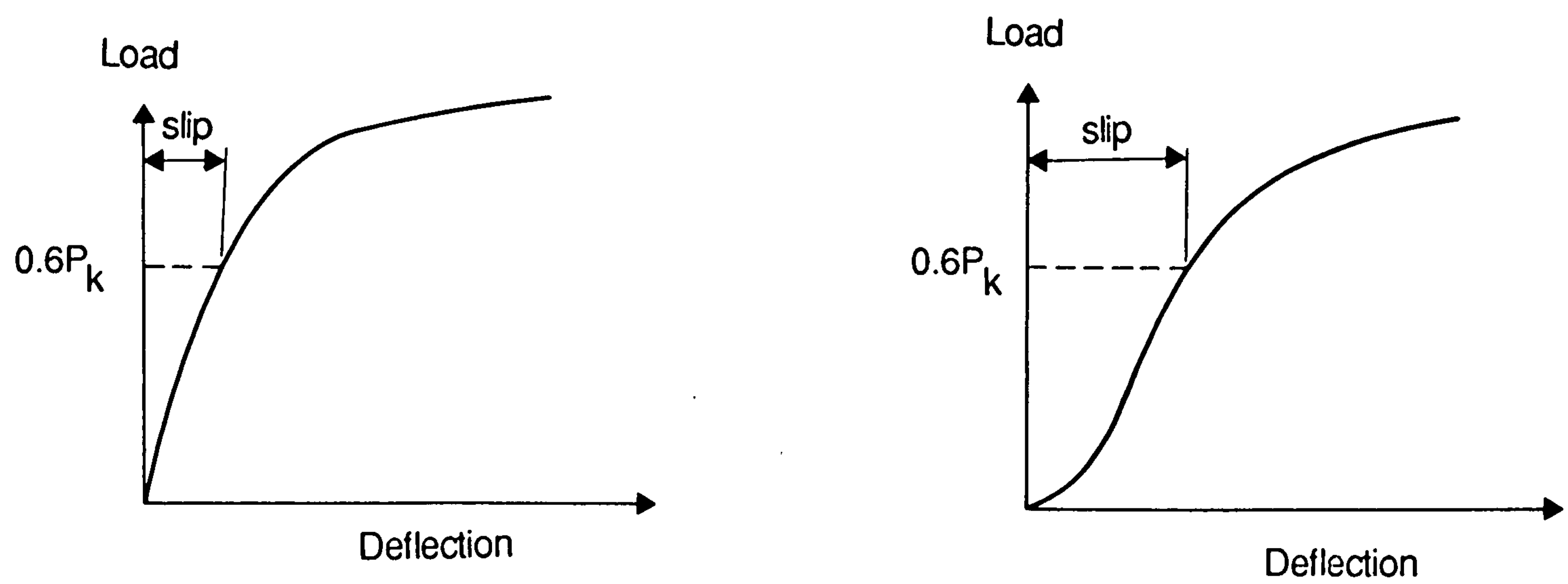


Figure 11 : Definition of slip.

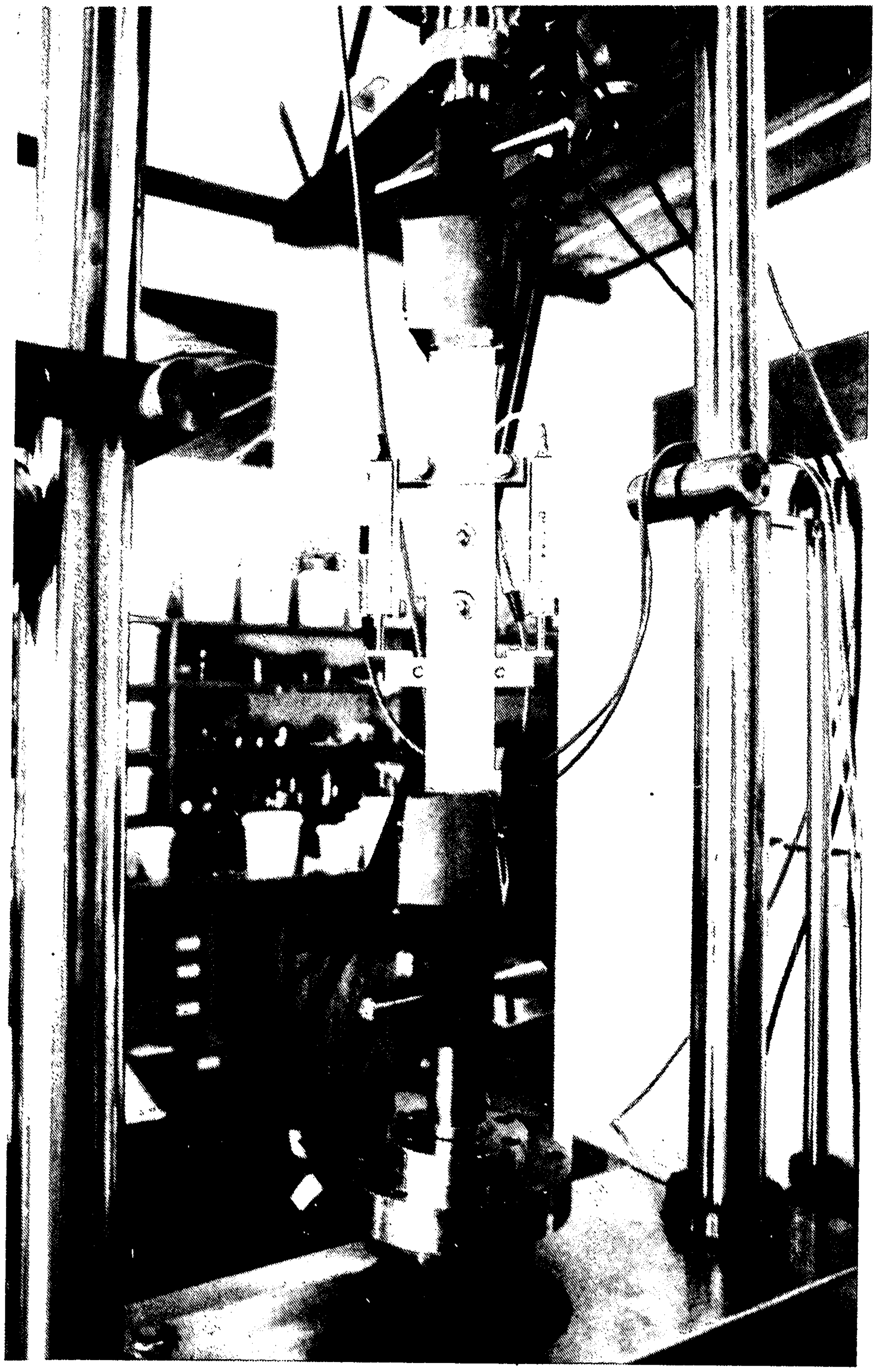


Figure 12 : Shear test in progress.

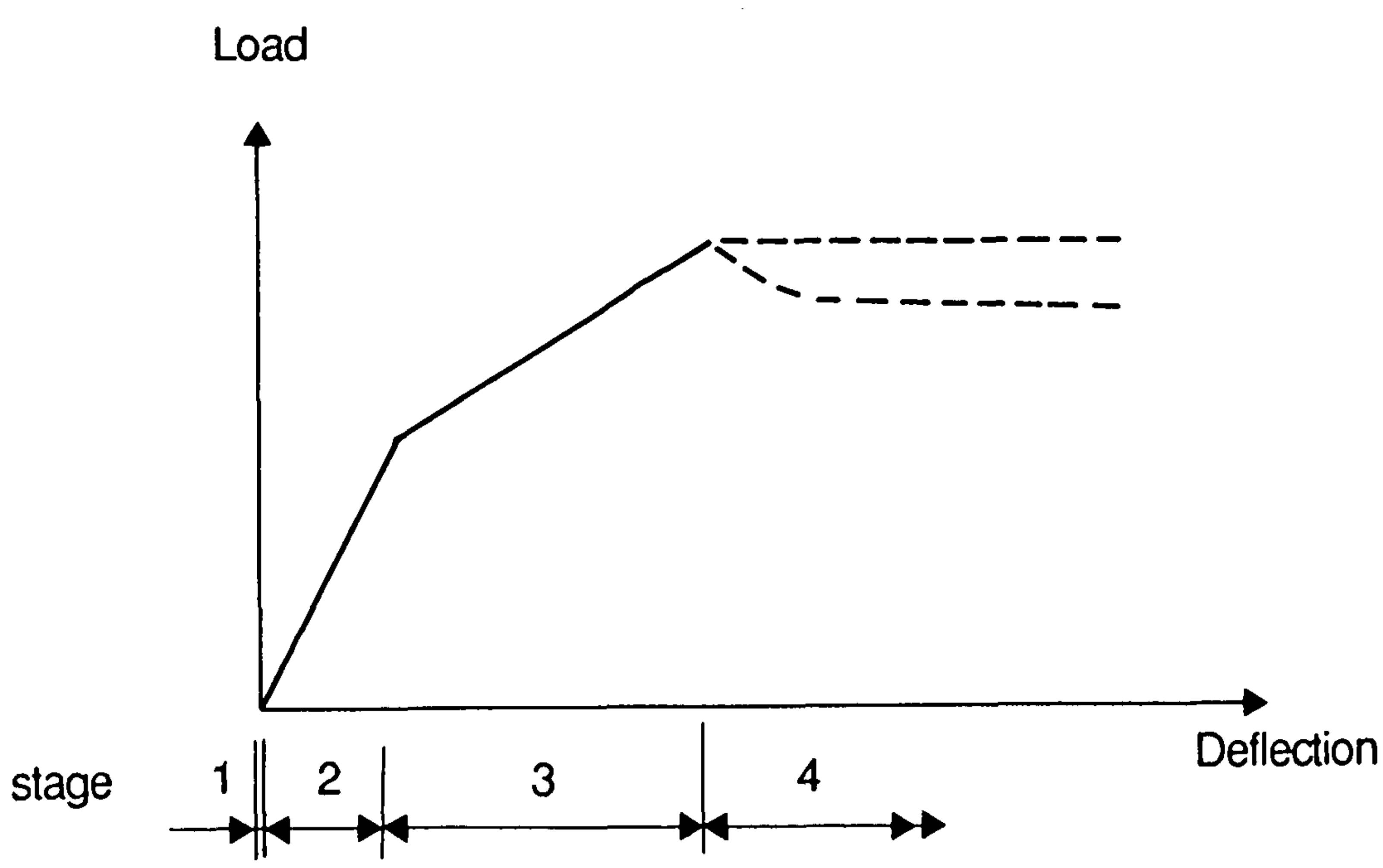


Figure 13 : Idealised behaviour of sheet-to-purlin connections in shear tests.

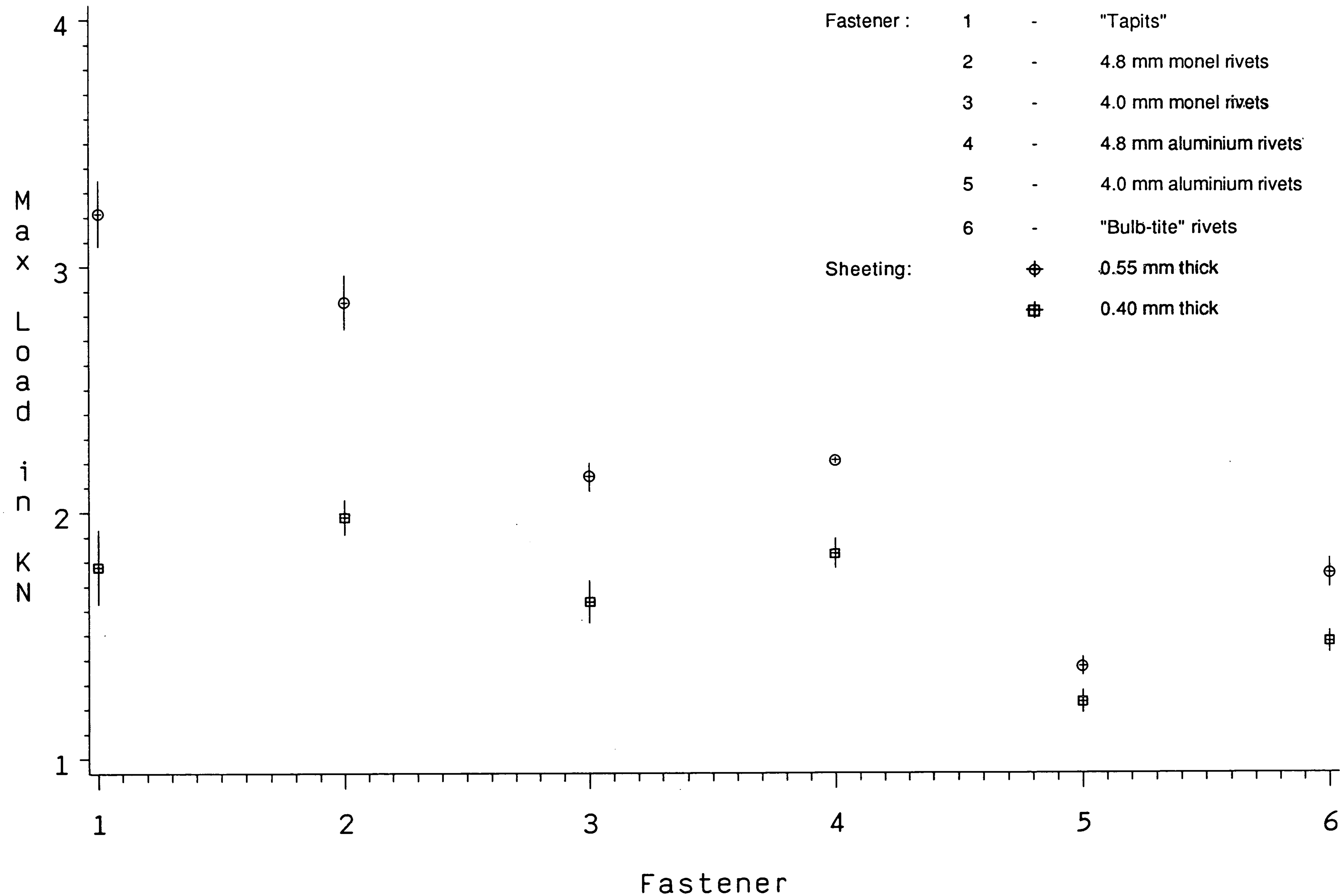
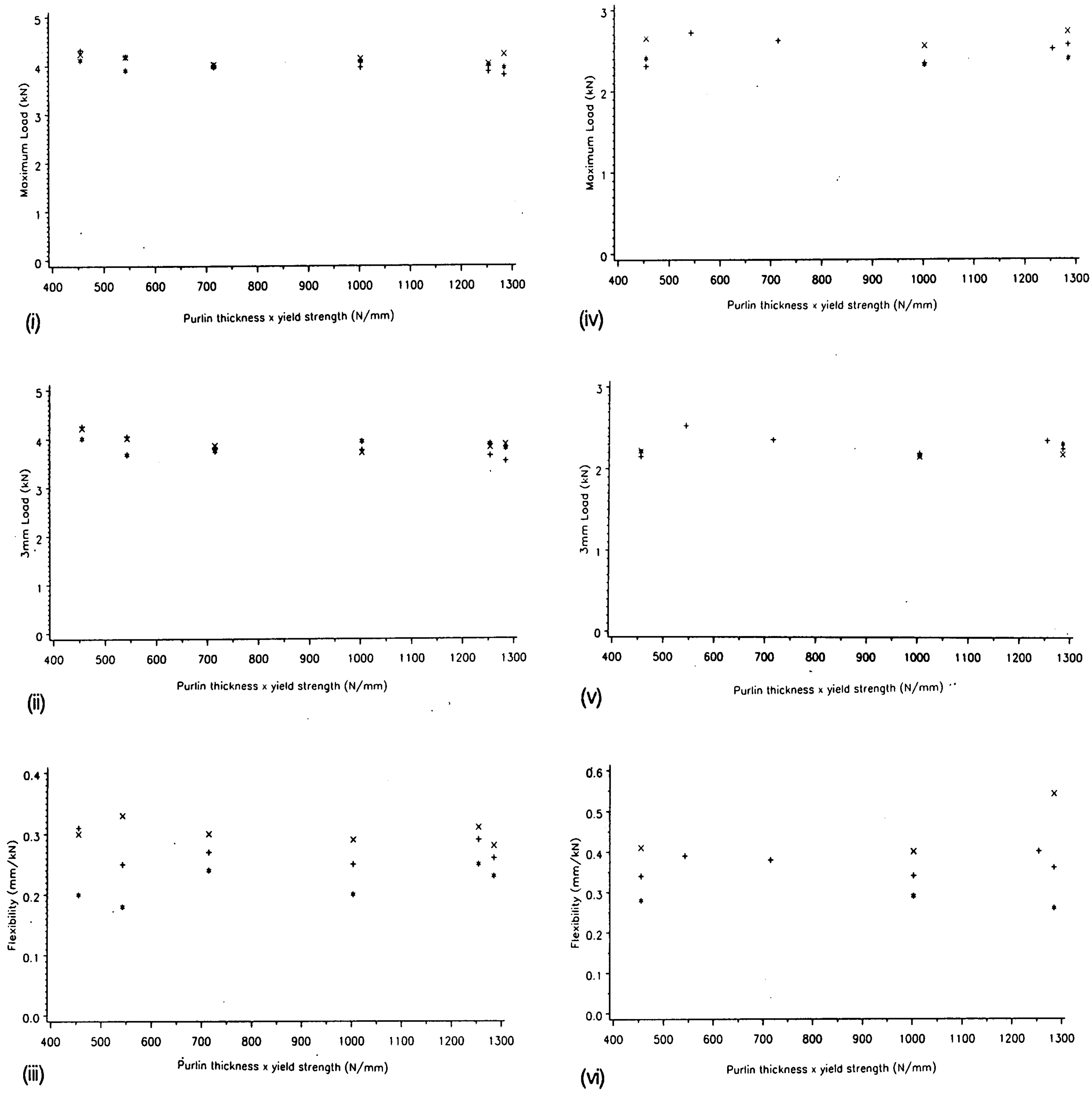


Figure 14 : Seam connection results.

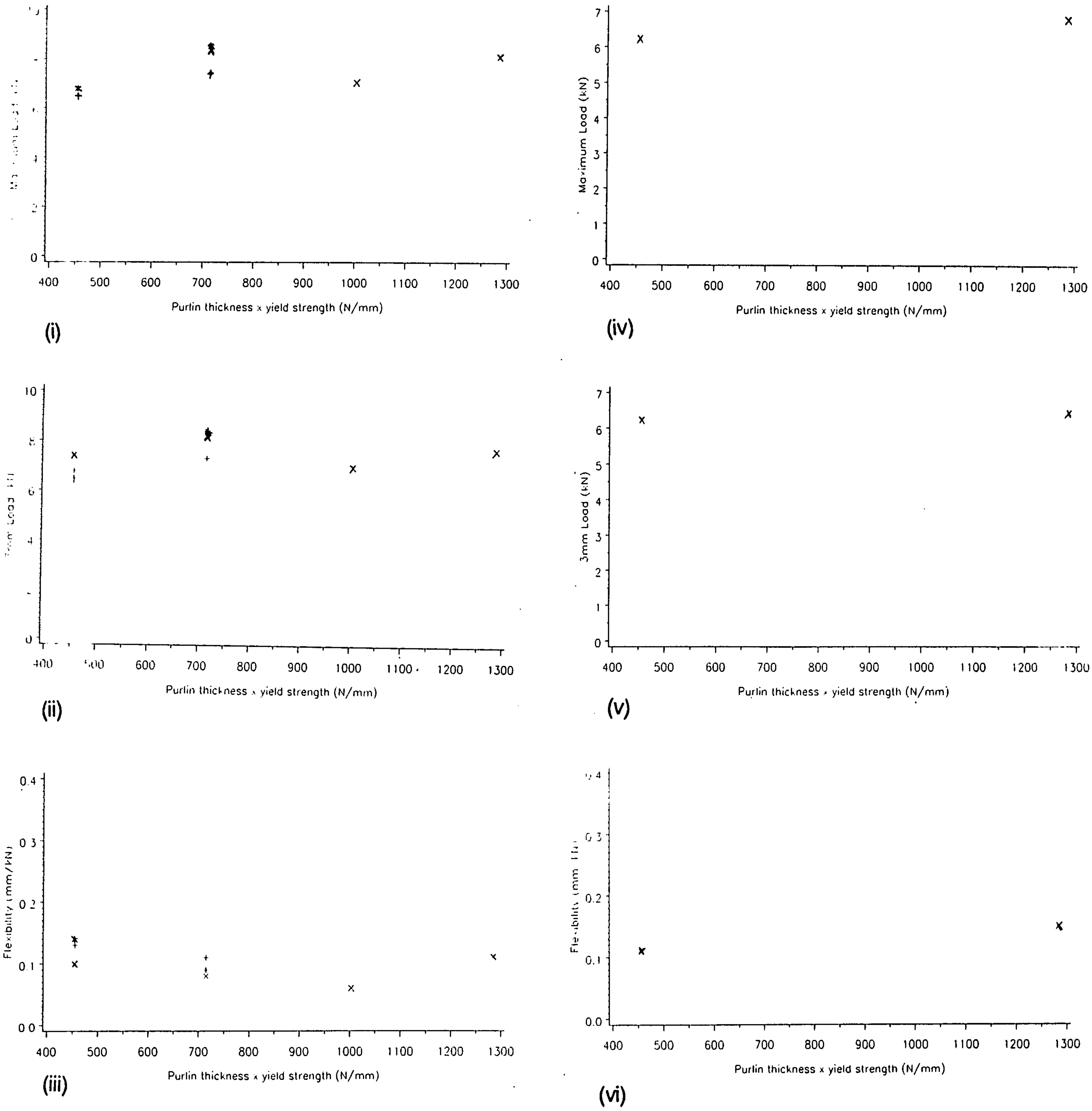


(a) sheeting thickness = 0.55 mm

(b) sheeting thickness = 0.40 mm

Notation:	+	-	12g neo
	X	-	12g emb
	*	-	14g neo

Figure 15 : Sheet-to-purlin connection characteristics from shear tests.



(a) sheeting thickness = 0.55 mm

(b) sheeting thickness = 0.40 mm

Notation:	+	-	12g neo
	X	-	12g emb
	*	-	14g neo

Figure 16 : Sheet-to-purlin connection characteristics from pull-out tests.

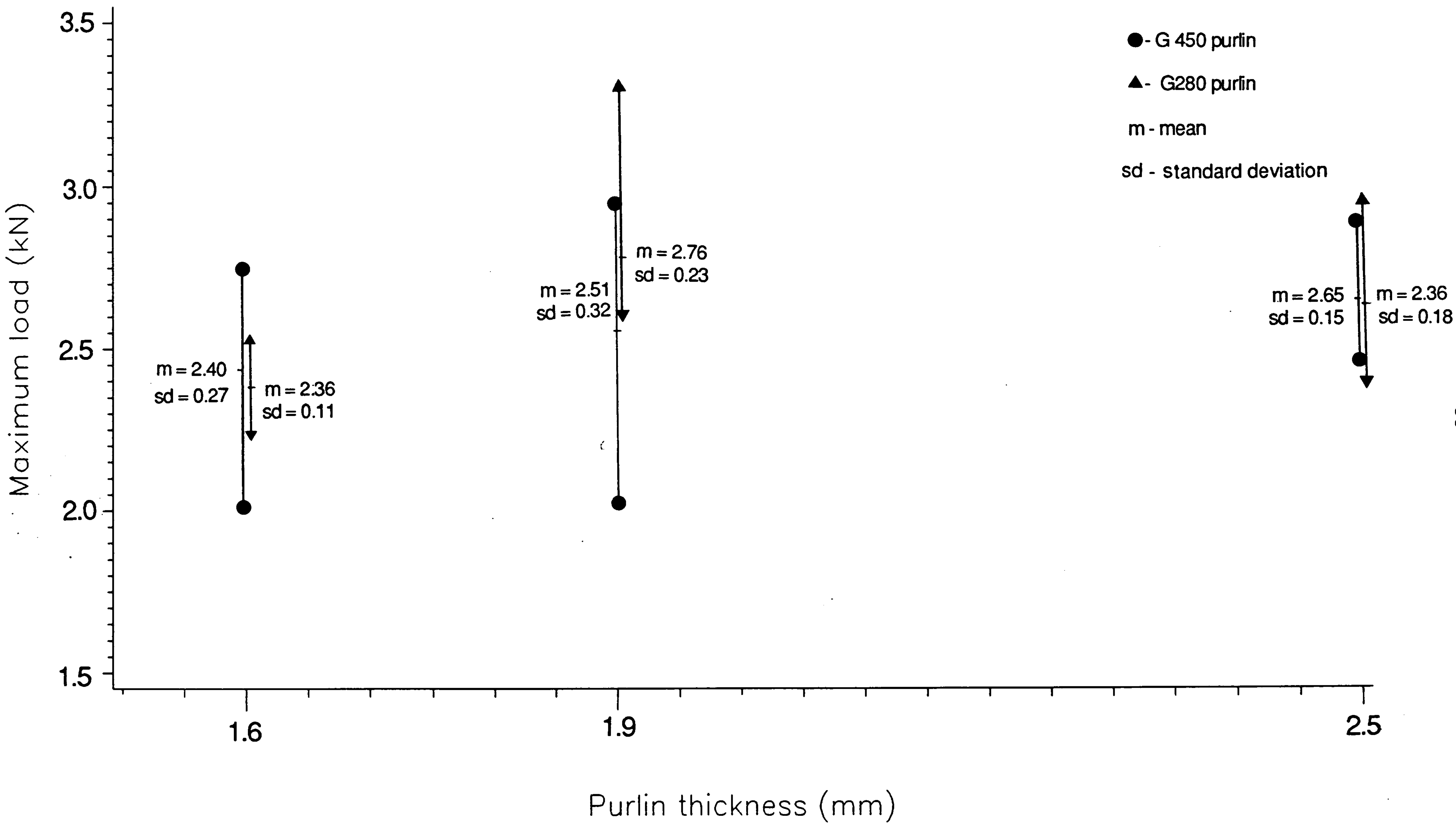
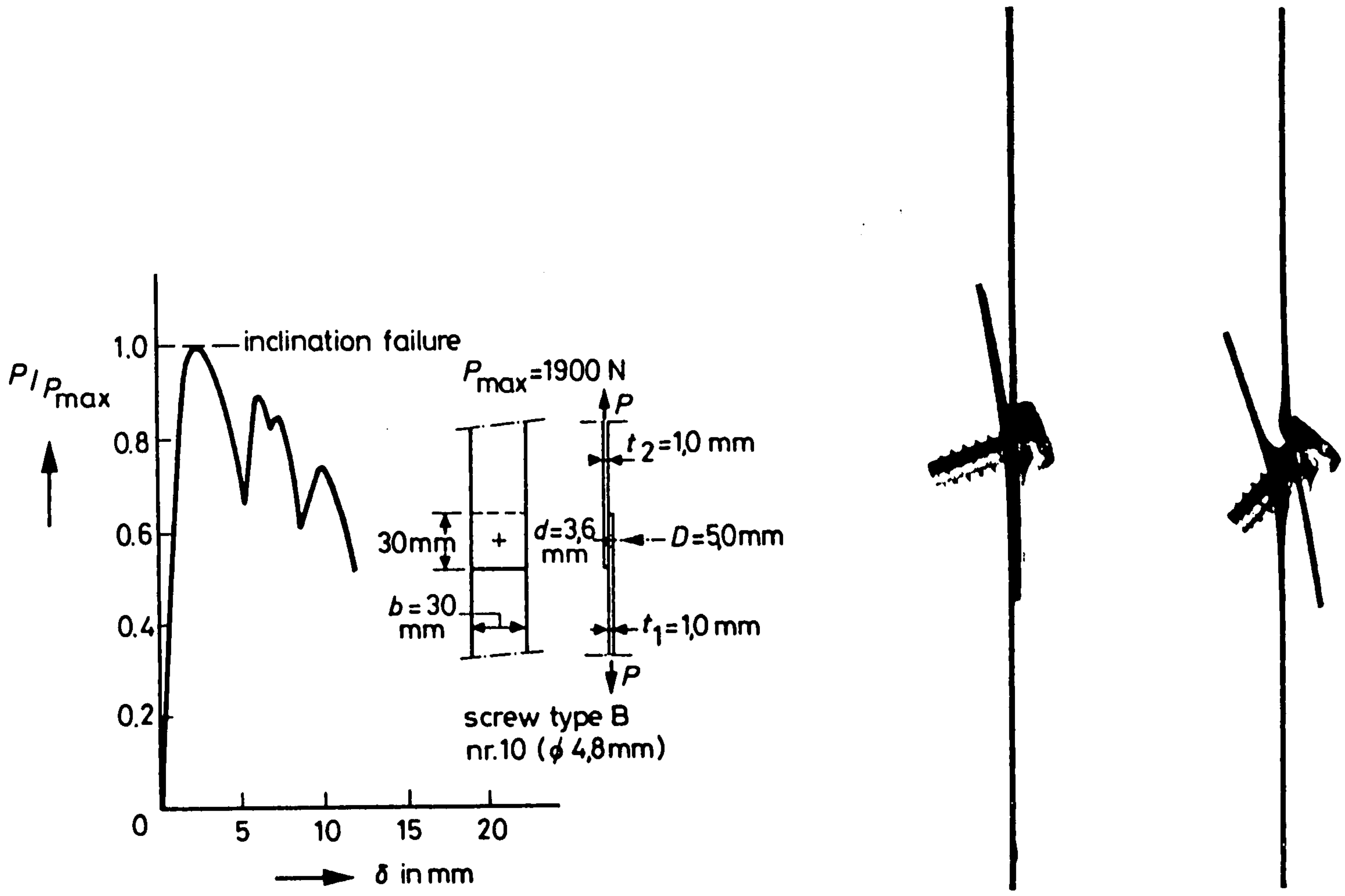
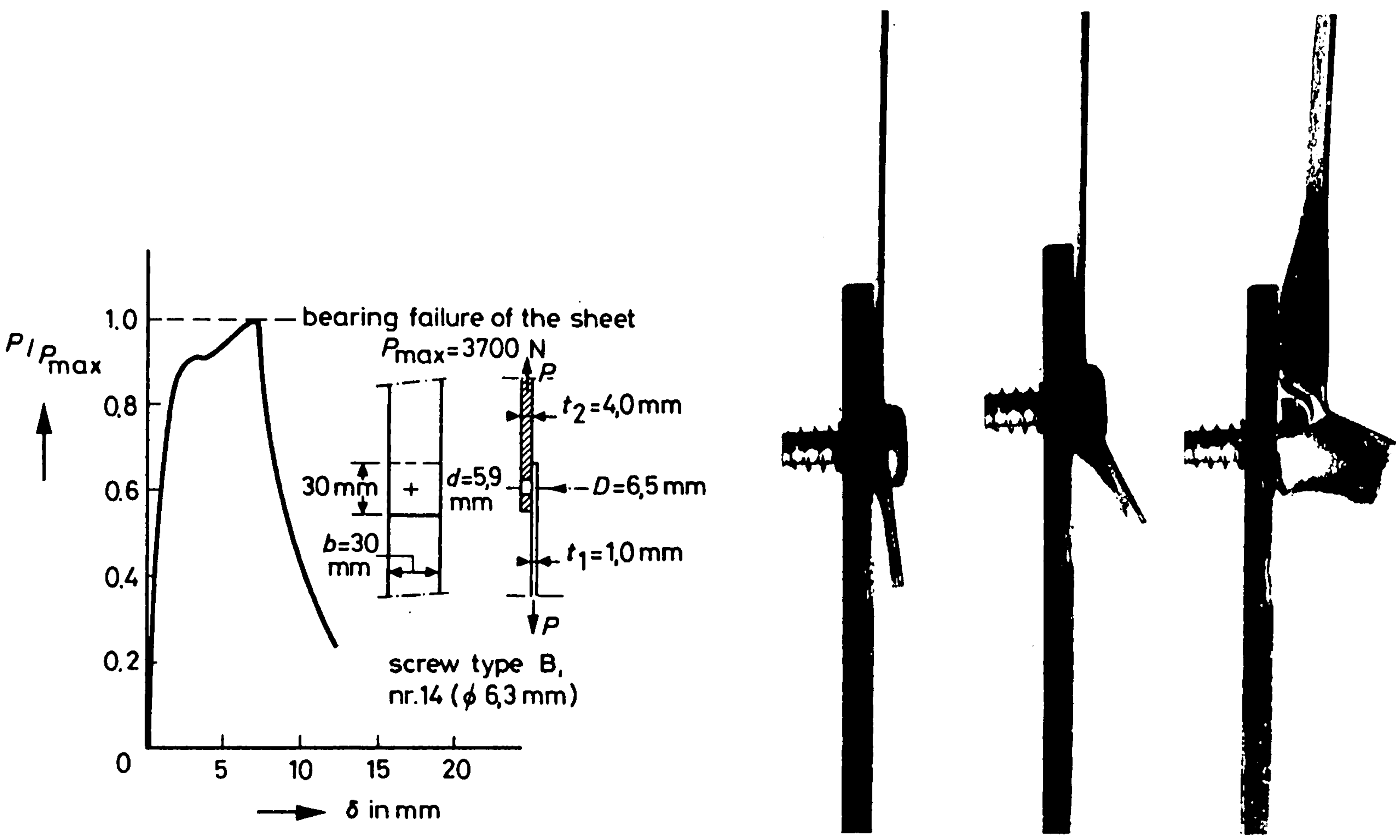


Figure 17 : Range of maximum load (8 tests per series) vs purlin thickness for sheet (0.4mm) to purlin connections (12g neo) in shear tests.



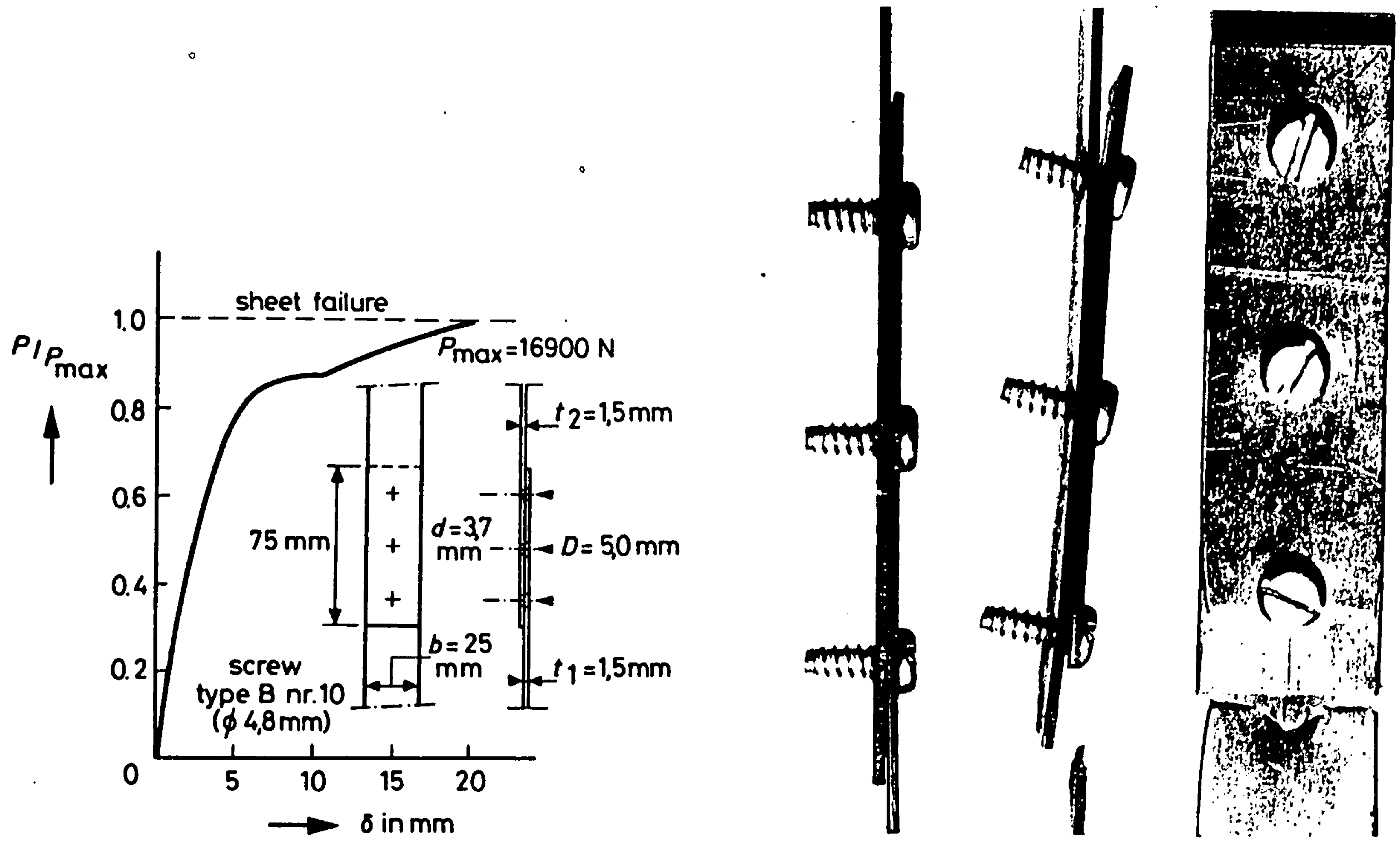
(a) **Failure mode : inclination of fastener**



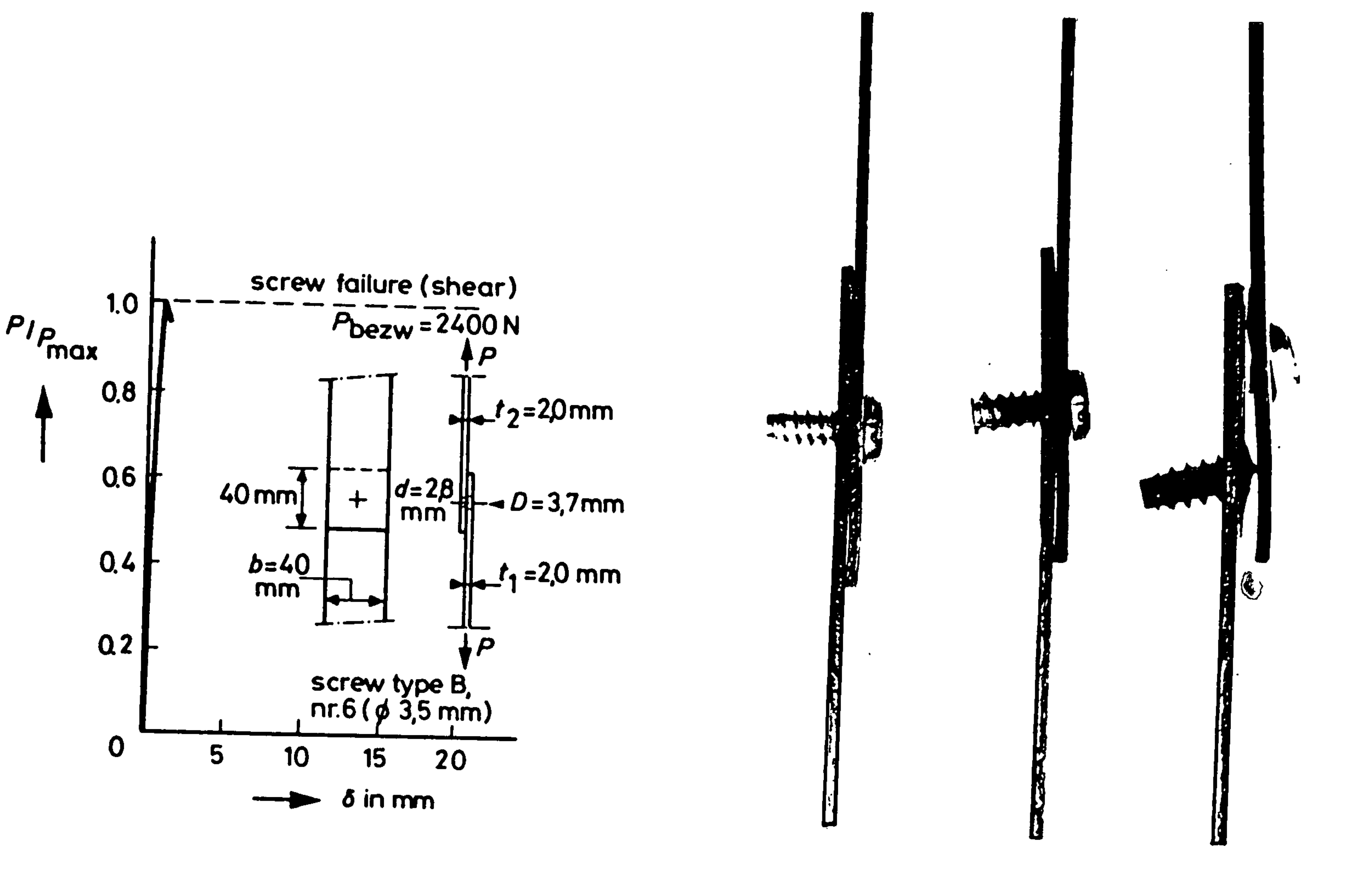
(b) **Failure mode : hole bearing**

From: Connections in Cold-formed Sections and
Steel Steets, by J.W.B.Stark and A.W.Toma;
Fourth Speciality Conference on Cold-formed
Steel Structures, University of Missouri-Rolla.

Figure 18 : Failure modes of screw connections



(c) **Failure mode : yielding of net section**



(d) **Failure mode : shear of fastener**

From: Connections in Cold-formed Sections and
Steel Steets, by J.W.B.Stark and A.W.Toma;
Fourth Speciality Conference on Cold-formed
Steel Structures, University of Missouri-Rolla.

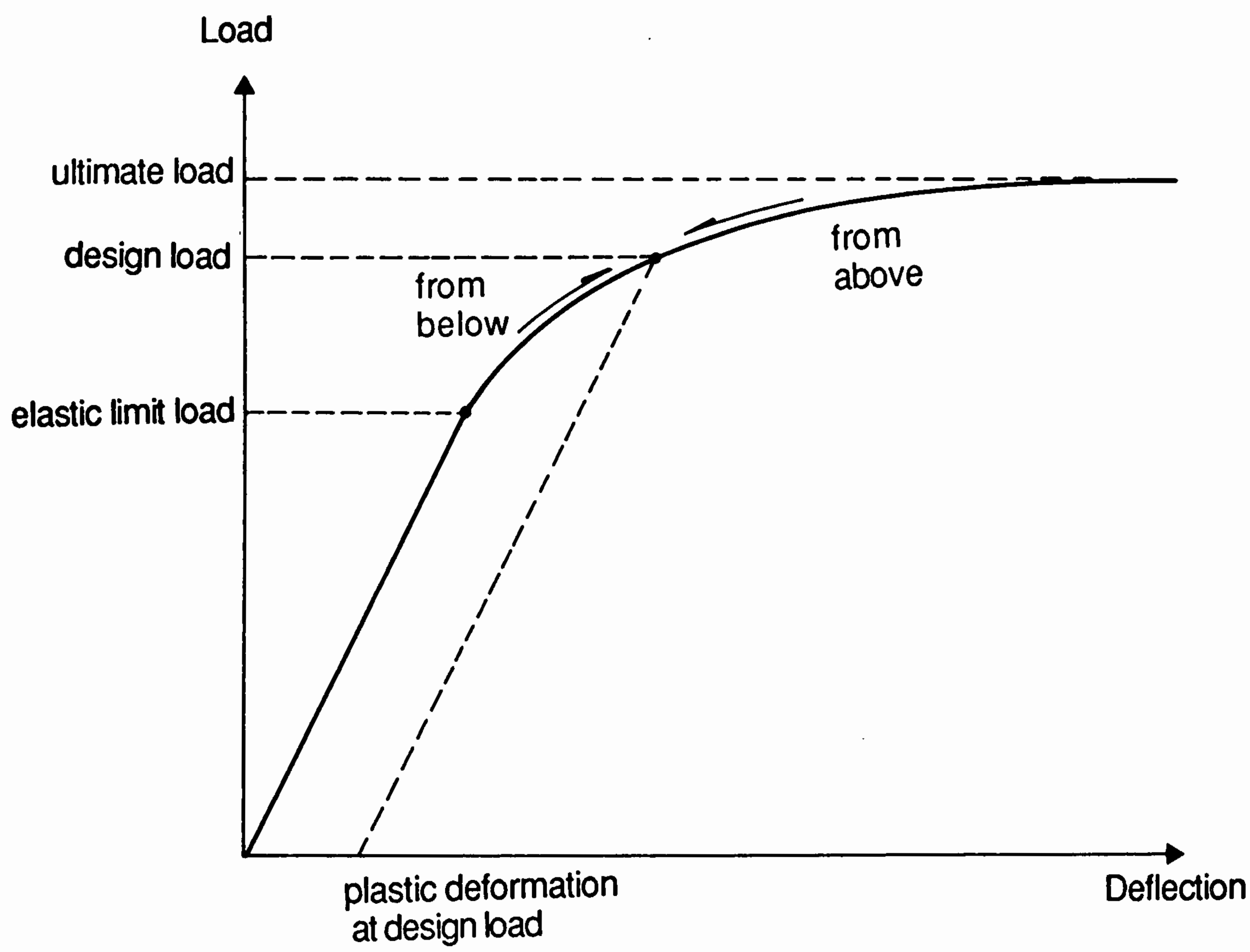
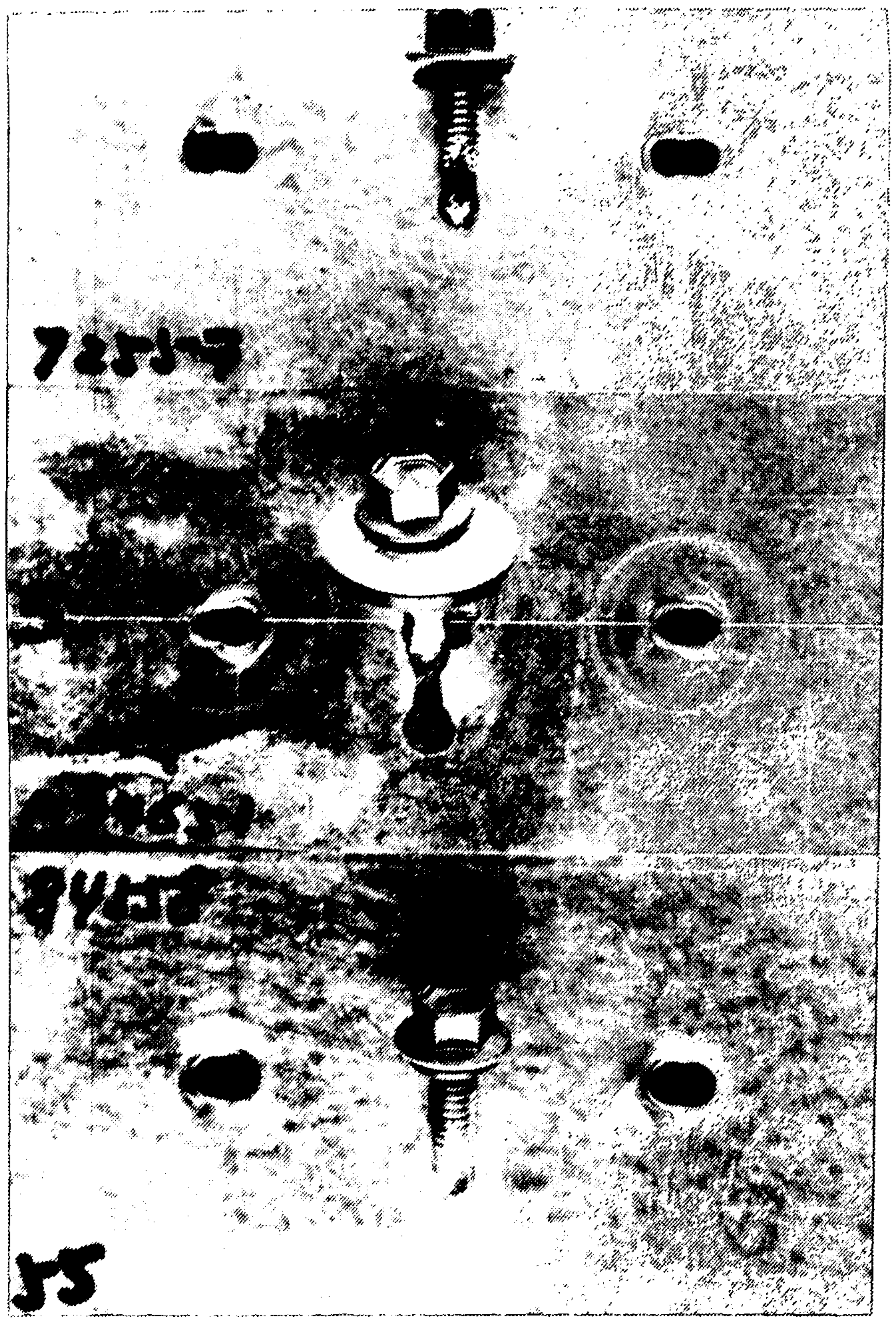


Figure 19 : Determination of design load.



12g neo

12g emb

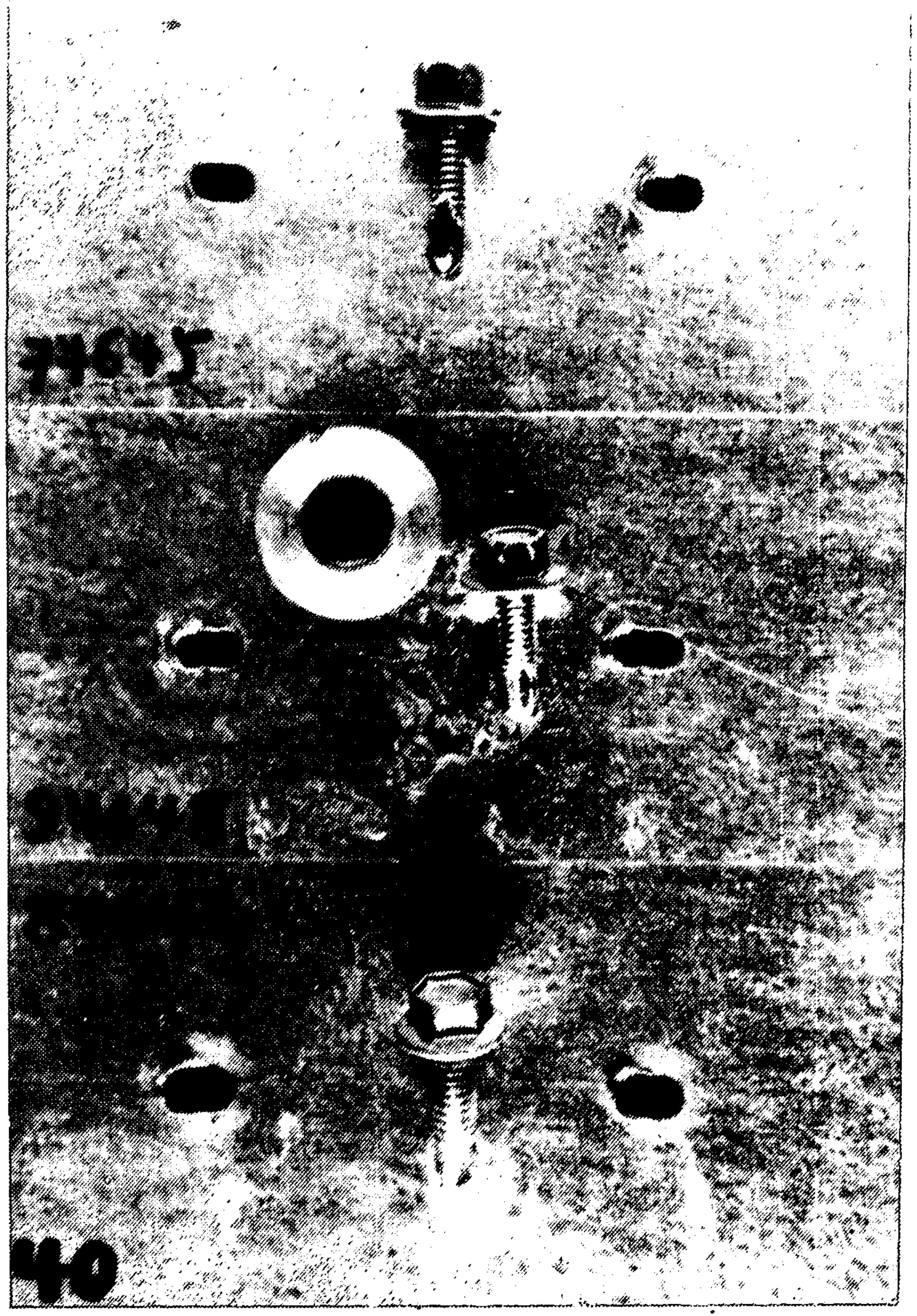
14g neo

(a) sheeting thickness = 0.55 mm

12g neo

12g emb

14g neo



(b) sheeting thickness = 0..40 mm

Figure 20 : Sheet-to-purlin connections.

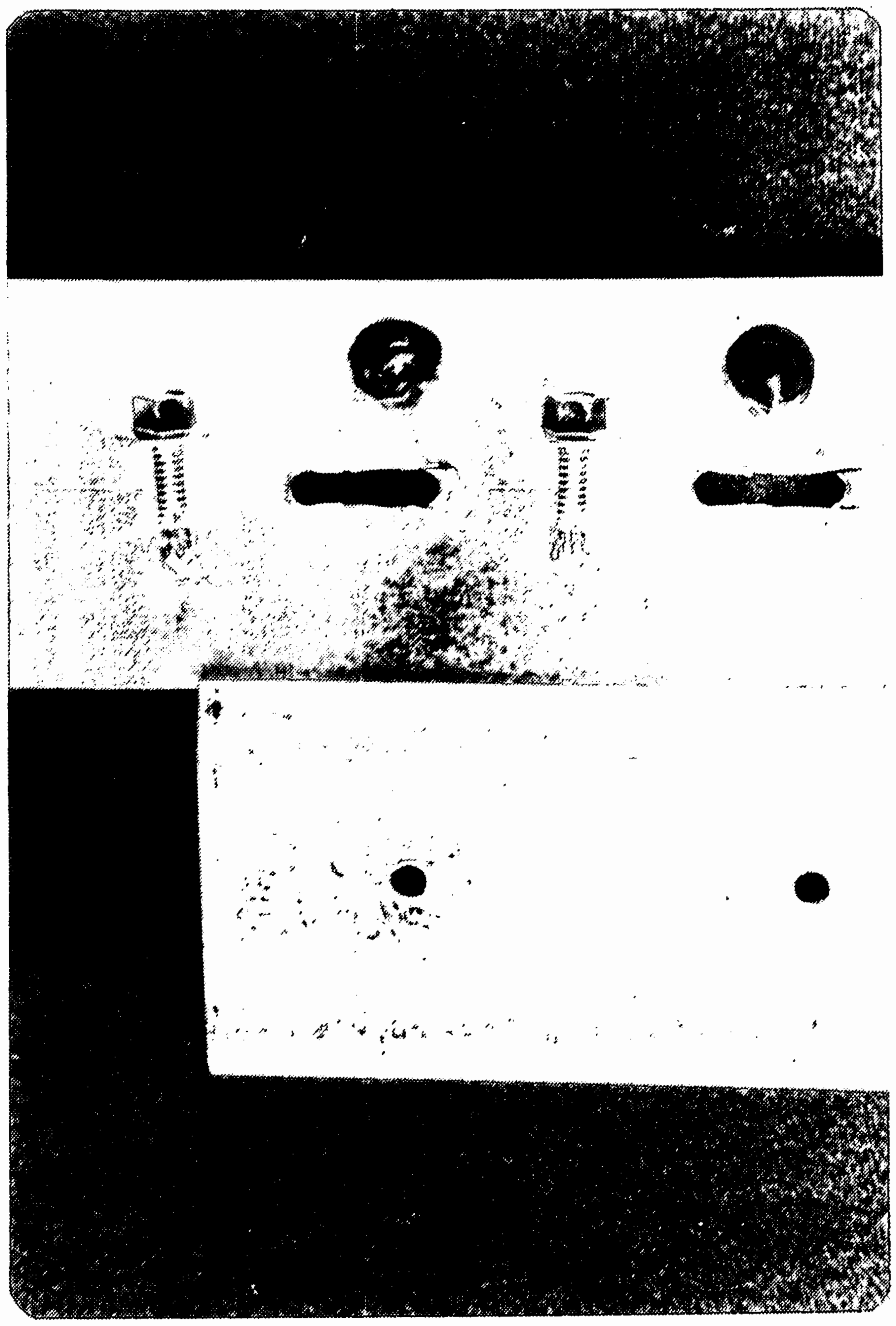
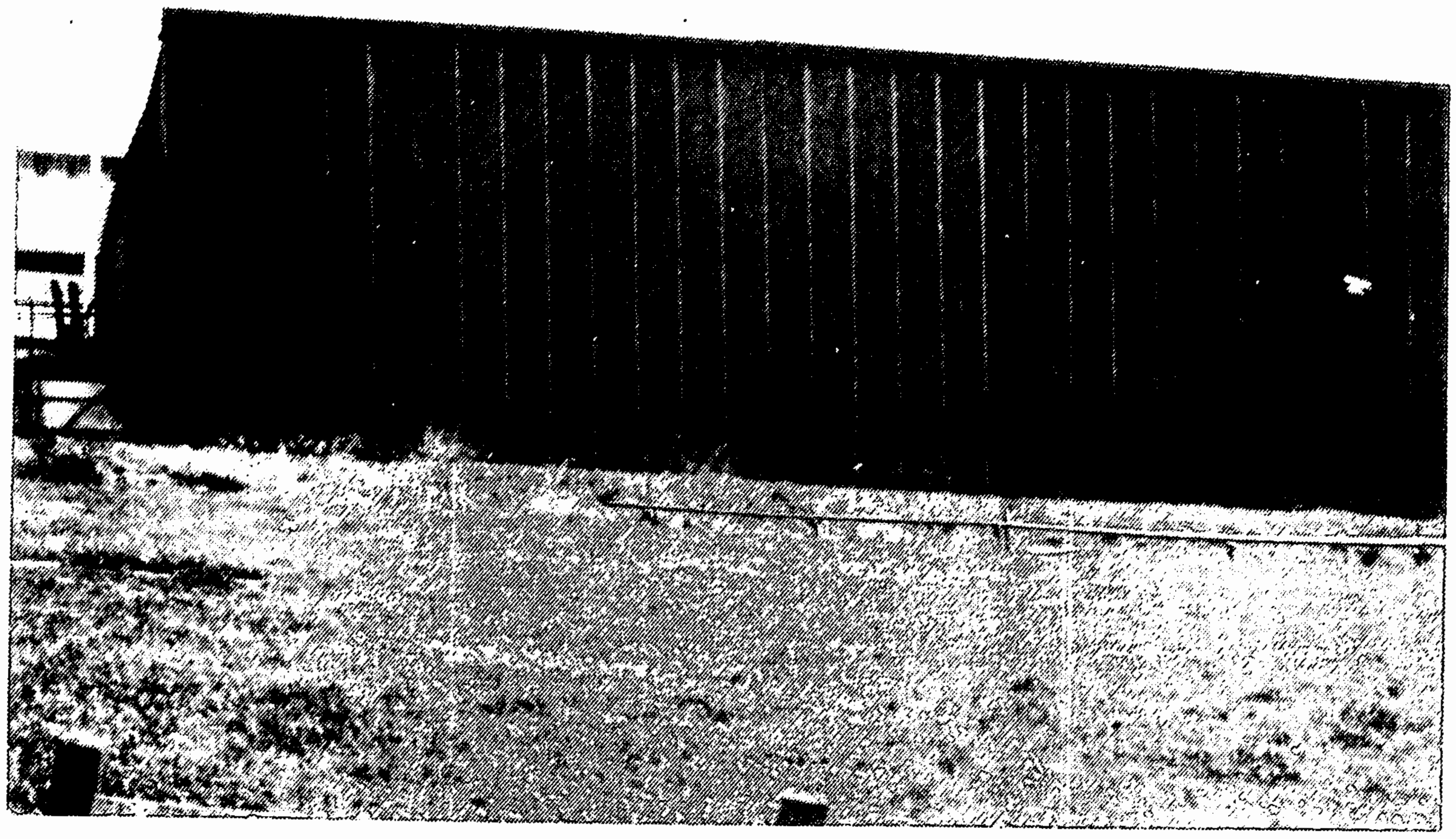
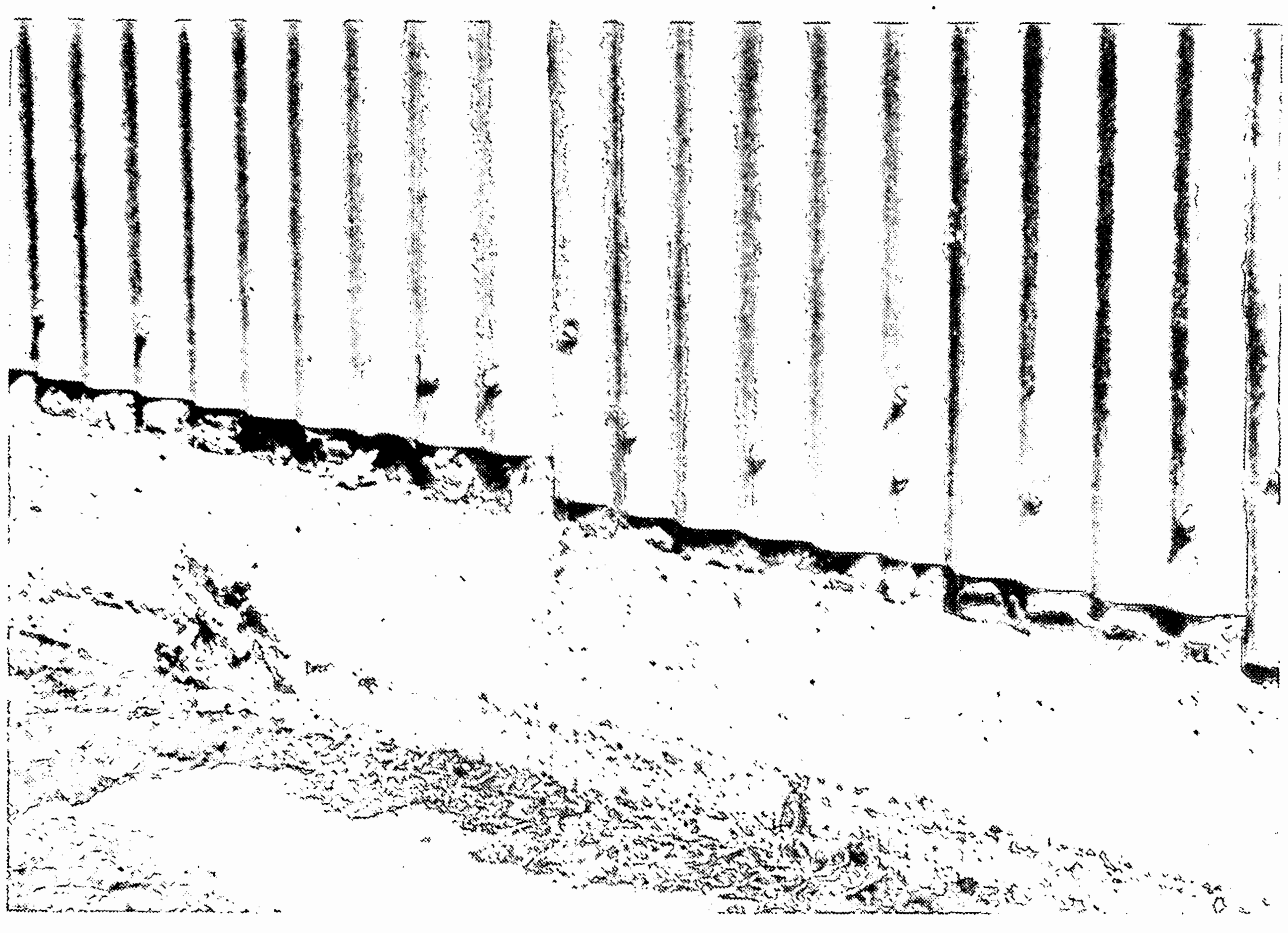


Figure 21 : Failure in 'dynamic' load.



(a)



(b)

Figure 22: Milk powder warehouse after earthquake.

Figure 23 : Connections after earthquake.

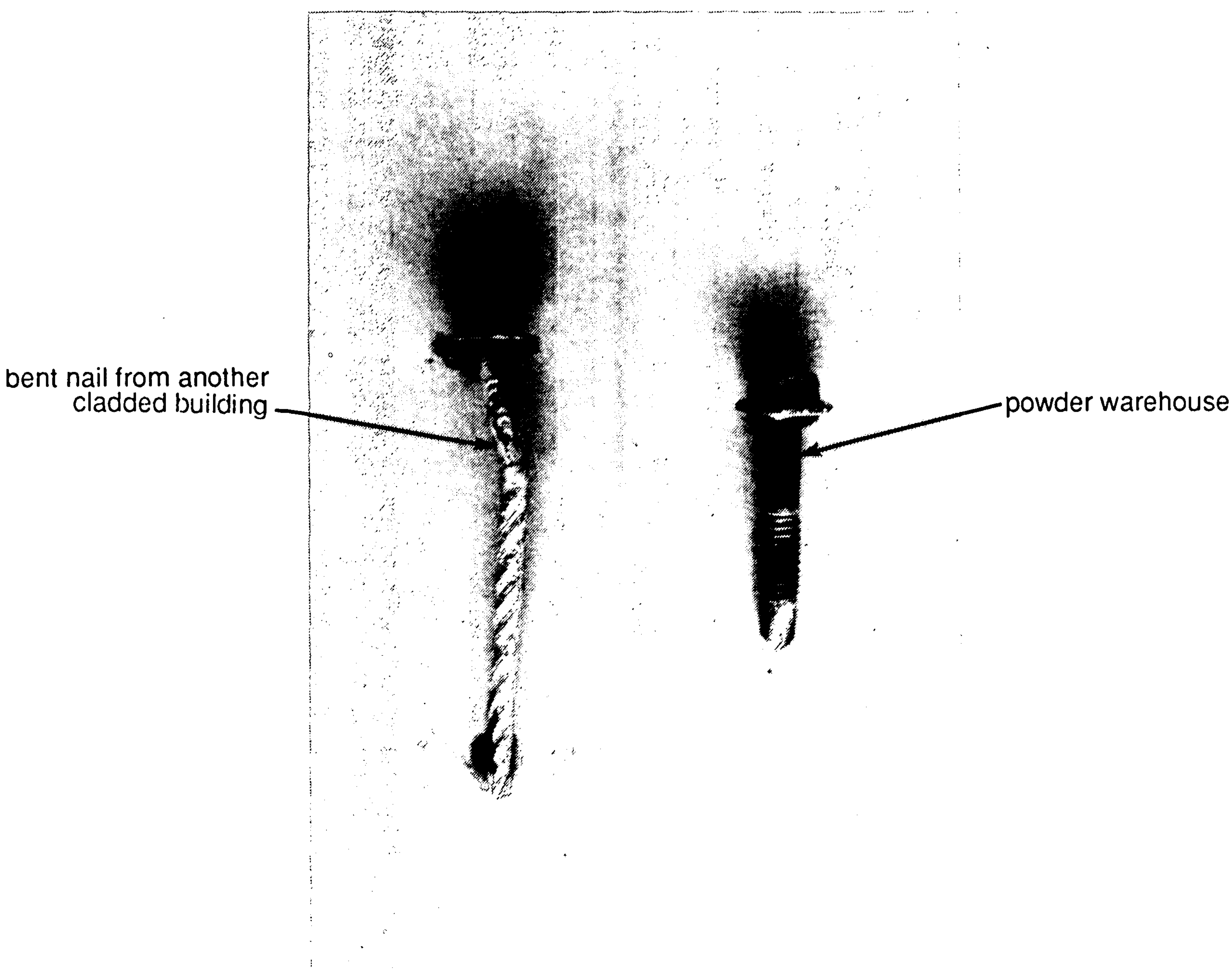


Figure 24 : Fasteners after earthquake.

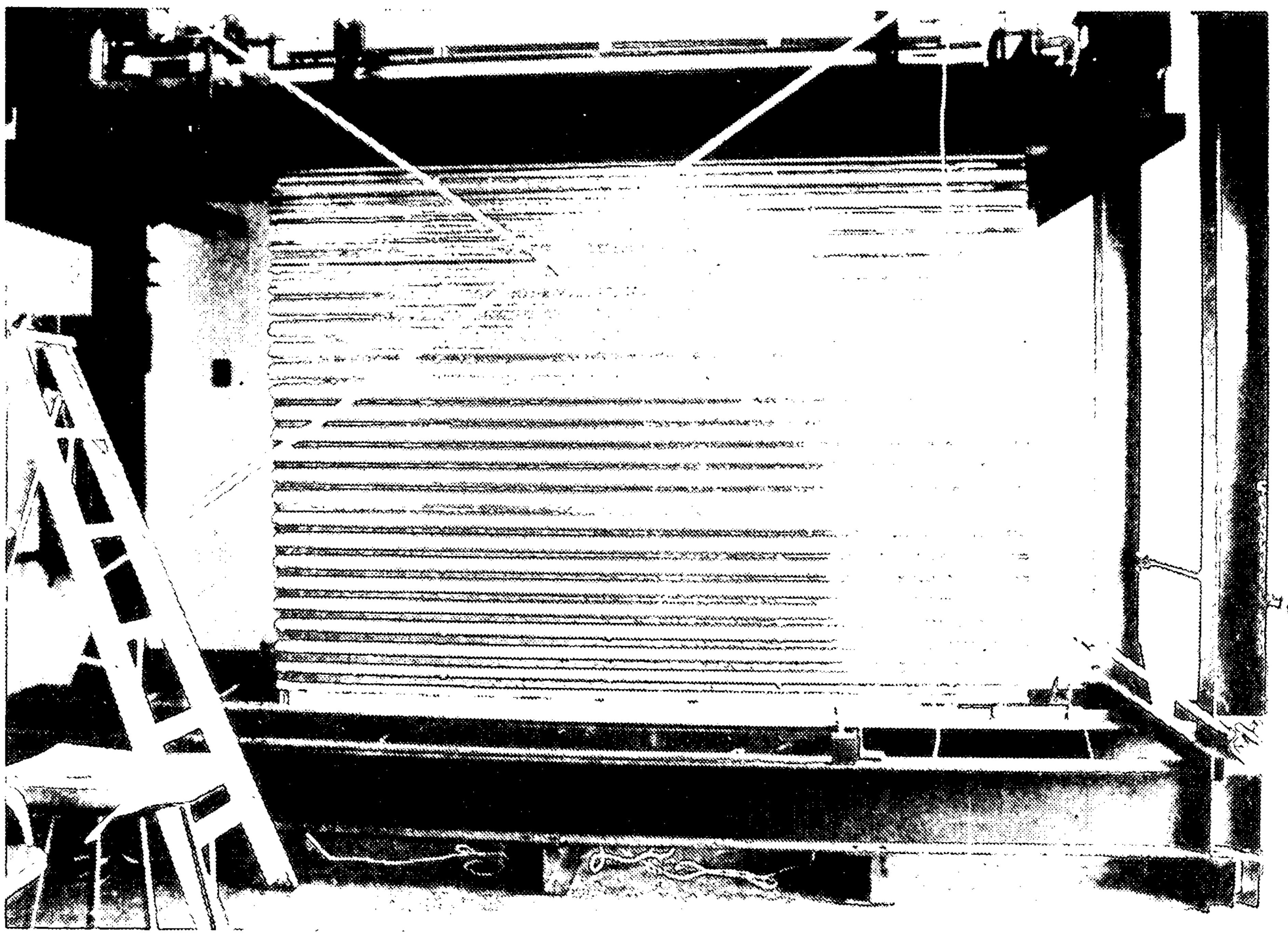


Figure 25 : Shear panel racking test.

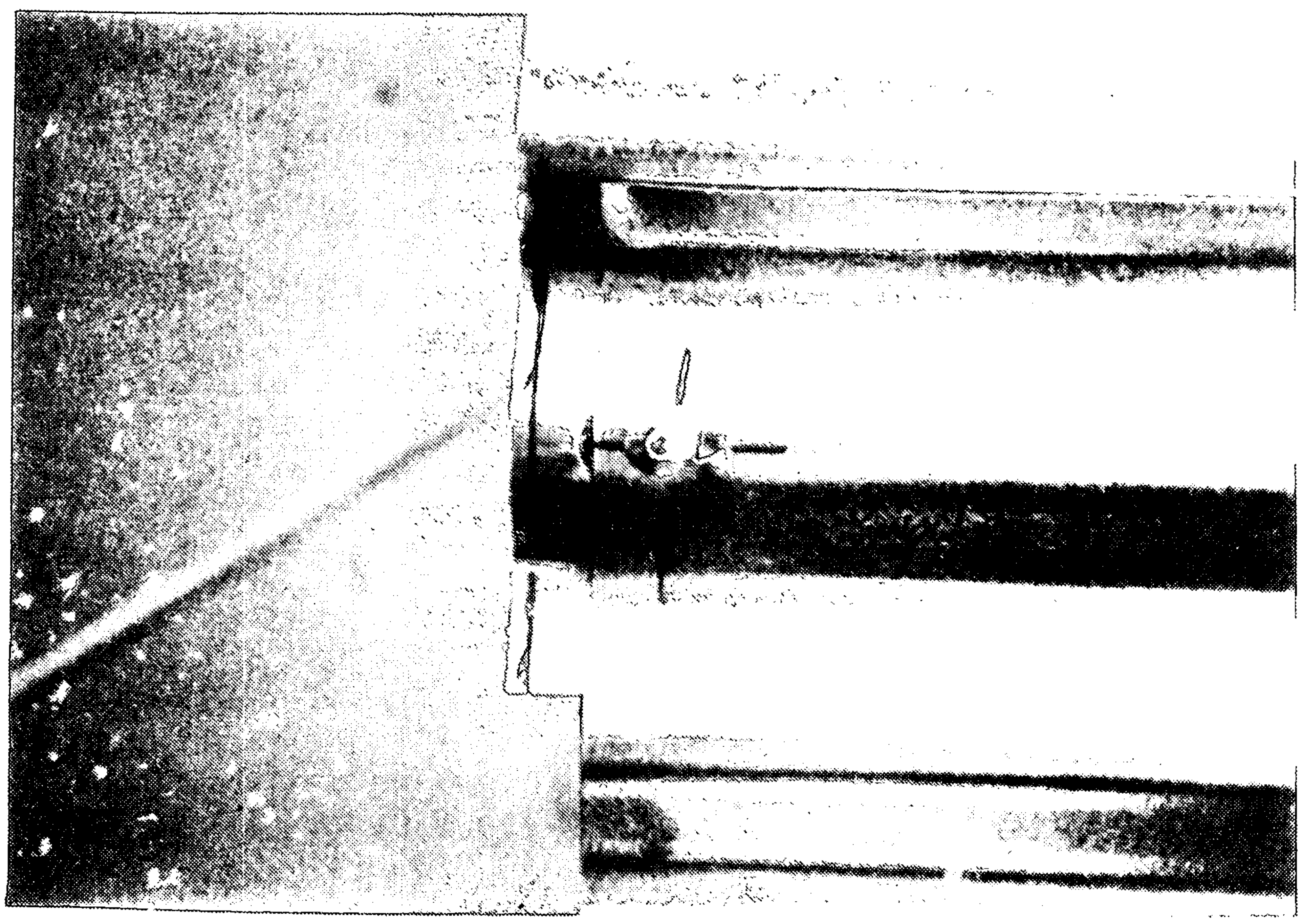


Figure 26 : Connection of shear panel after test.

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APPENDIXES

APPENDIX A: FASTENER AND INSTALLATION PROCEDURE

Table A.1: Seam fasteners

Fastener type	Designation	Actual diameter (mm) [1]	Shear strength (kN)	Installation details	Abbreviation in this report
10 gauge steel hexagon slotted, round washer head self-drilling self-tapping screws + 16mm dia. domed steel with pre-assembled neoprene washers	10x3/4 Hex Slot "Tapits" with bonded washers	4.85 (0.04)	6.24*	Installed with hand held battery powered drill, set until washer become less conical but not flat	"Tapits"
4.8 mm dia. monel metal pull break mandrel, open end, semi-filled core blind rivets	"TLP/D/639"	4.76 (0.01)	4.00^ 3.56#	Holes drilled with 4.90 mm hand held electric drill, set with air gun	4.8 mm monel rivets
4.0 mm dia. monel metal pull break mandrel, open end, semi-filled core blind rivets	"TLP/D/545"	3.95 (0.02)	2.45^ 2.40#	Holes drilled with 4.10 mm hand held electric drill, set with air gun	4.0 mm monel rivets
4.8 mm dia. aluminium alloy, pull break mandrel, closed end, semi-filled core blind rivets	"AD/612"	4.72 (0.01)	2.27^ 2.20#	Holes drilled with 4.90 mm hand held electric drill, set with air gun	4.8 mm aluminium rivets
4.0 mm dia. aluminium alloy, pull break mandrel, open end, semi-filled core blind rivets	"TAP/D/58"	4.01 (0.03)	1.33^ 1.16#	Holes drilled with 4.10 mm hand held electric drill, set with air gun	4.0 mm aluminium rivets

Table A.1 continued

Fastener type	Designation	Actual diameter (mm) [1]	Shear strength (kN)	Installation details	Abbreviation in this report
3/16" dia. aluminium alloy, slotted shank, break mandrel, semi-filled core blind rivets with pre-assembled neoprene washers	"Bulb-tite" "RV 6604-6-4S"	4.97 (0.01)	3.29'	Holes drilled with 5.30 mm hand held electric drill, set with air gun	"Bulb-tite" rivets

Notes:
1. Bracketed figures indicate standard deviation.
2. * -- Based on manufacturers' or suppliers' information.
3. ^ -- Based on manufacturers' or suppliers' tests.
4. # -- Based on ECCS (1984a), minimum acceptable ultimate shear strength for blind rivet tested in accordance with the procedures given in Part B of this document.
5. ' -- Based on manufacturers' or supplier's information. Shear strength tests were conducted in hardened alloy steel sheets with a thickness equal to the fastener diameter.

Table A.2: Cladding to purlin or shear connector fasteners

Fastener type	Designation	Actual diameter (mm) [1]	Shear strength (kN) [2]	Installation details [3]	Abbreviation in this report
12 gauge steel hexagon washer collar heads self-drilling self-tapping screws + neoprene washers	No. 12 x 20 mm HWF "Steeltite" screws with neoprene washers	5.49 (0.03)	8.58	Installed with electric screw gun, tightened sufficiently to form a weather-proof seal without damage to the sealing washer	12g neo
Ditto	No. 12 x 35 mm HWF "Steeltite" screws with neoprene washers	5.40 (0.01)	8.58	Ditto	12g neo
12 gauge steel hexagon washer collar heads self-drilling self-tapping screws + 25 mm dia. domed galvanised steel with bonded EPDM washers	No. 12 x 20mm HWF "Steeltite" screws with embossed washers	5.49 (0.03)	8.58	Ditto	12g emb
Ditto	No. 12 x 35 mm HWF "Steeltite" screws with embossed washers	5.40 (0.01)	8.58	Ditto	12g emb
14 gauge steel hexagon washer collar heads self-drilling self-tapping screws + neoprene washers	No. 14 x 22 mm HWF "Teks" screws with neoprene washers	6.24 (0.01)	11.30	Ditto	14g neo

Notes:
1. Bracketed figures indicate standard deviation.
2. Based on manufacturers' or suppliers' information.
3. Refer to Figure 30 of National Federation of Roofing Contractors (1982) and Josey (1986).

APPENDIX B: COMPARISON OF RESULTS FROM DIFFERENT SHEAR TEST METHODS

Table B.1: Comparison of results from different shear test methods

Reference	Fastened Material	Fastener type	Results		Notes
Berry (1976)			one vs two fastener lapped joints: thin-to-thin connections:		
			P_{k1}/P_{k2}	F_1/F_2	
	0.53 mm sheets	4.8 mm rivets	1.13	0.14	
	0.76 mm sheets		1.15	0.36	
	0.53 mm sheets	5.5 mm screws	0.83	0.24	
	0.76 mm sheets		0.72	0.16	
			thin-to-thick connections:		
			P_{k1}/P_{k2}	F_1/F_2	
	0.53 mm to 3.25 mm sheets	6.3 mm screws	1.13	0.65	
	0.76 mm to 3.25 mm sheets		1.00	0.42	
	thin sheets: thickness = 0.71-0.76 mm yield stress = 215-263 MPa thick sheets: thickness = 3.25 mm	"Teks" 12-24 screws diameter = 5.5 mm, with 12.5 mm diameter "Twinseal" washers	two fasteners single lap joint vs simulated diaphragm action tests: thin-to-thick connections: P_{us}/P_{u2} 1.0 to 1.07		1. studied strength only 2. two fasteners in the same line perpendicular to the direction of applied force 3. edge distance to diameter ratio = 1.82 4. failure in yield in bearing
Fraczek (1976)	combinations of 10, 16, 22 and 26 gage (3.12 mm, 1.55 mm, 0.76 mm and 0.53 mm) mild steel sheets between 18 gage (1.22 mm) galvanised steel sheets	#14x3/4" hex head Type A thread forming fasteners, assembled with 5/8" OD 20 gage washers of galvanised steel bonded to neoprene	double fastener lapped joint tests have mean 'yield' load ranged from -17% to +15% and mean ultimate load ranged from -21% to +25% compared with simulated diaphragm action tests lapped joint tests have mean 'yield' and ultimate loads 7% and 11% respectively greater than simulated diaphragm action tests		1. studied strength only 2. correlation reasonable, better for 'yield' load than ultimate load 3. failure by yield in bearing

Table B.1 continued

Reference	Fastened Material	Fastener type	Results	Notes
Stol and Toma (1978)	thin sheets: thickness = 0.77 mm yield stress = 303 MPa tensile stress = 410 MPa thick sheets: thickness = 8 mm		one vs two fasteners lapped joints: thin-to-thin connections: P_{k1}/P_{k2} F_1/F_2	
		4.8 mm aluminium rivets	0.89 1.22	
		4.2 mm self-drilling screws	1.09 1.97	
			thin-to-thick connections: P_{k1}/P_{k2} F_1/F_2	
		6.3 mm self-tapping screws	0.95 1.00	
		5.4 mm self-drilling screws	0.87 0.53	
			one fastener lapped joints vs frame tests, thin-to-thin connections: P_{kf}/P_{k1}	
		4.8 mm aluminium rivets	1.13	
		4.2 mm self-drilling screws	0.85	
Huang and Luttrell (1980)	sheeting: 0.6-1.8 mm plate: 6.4-9.5 mm	"Teks" 12g and 14g screws	connection strength increase roughly linearly with sheeting thickness for both sheet-to-sheet and sheet-to-plate connections, very little difference between results from one screw and two screw tests	1. studied strength only

Notation:
1. P_{k1}, P_{k2} and P_{kf} are the characteristic strengths from one fastener lapped joint test, two fasteners lapped joint test and frame test respectively.
2. F_1 and F_2 are the flexibilities from one and two fasteners lapped joint tests respectively.
3. P_{u2} and P_{us} are the ultimate load per fastener from the two fasteners single lap joint test and the simulated diaphragm action test respectively.

APPENDIX C: DESIGN EXPRESSIONS FOR MECHANICAL CONNECTIONS

Table C.1: Design expressions for the strength of rivet connections

Reference	Expression	Remarks
Baehre and Berggren (1973)	failure due to rivet inclination and yield in bearing: $F_u = k(d + 5)(t^2 + 0.22)p_u$ failure due to shear of fastener: $F_u = 0.8F_s$	$2.5\ mm < d < 6.5\ mm$ suggested safety factor=2.4 for steel sheets suggested safety factor=1.65
Bryan (1973)	$F_u = 2.5t$	4.8 mm monel pop rivets only, $0.43\ mm < t < 1.27\ mm$
Stark and Toma (1978)	failure by inclination of fastener: $F_d = k(d + 5)(t'^2 + 0.22)p_c$	
SISC (1982)	failure in bearing: $F_d = 2.8(t^3 d)^{1/2} p_{ty}$ but not greater than $1.6tdp_{ty}$	$2.6\ mm < d < 6.4\ mm$, $t'/t = 1$
ECCS (1984b)	failure by tilting and yield in tearing: $F_d = 3.6(t^3 d)^{1/2} p_e$ with a maximum of $2.1tdp_e$	$2.6\ mm < d < 6.4\ mm$, $t'/t = 1$
BS 5950: Part 5 (1987)	shear capacity in tilting and bearing: $F_u = 3.2(t^3 d)^{1/2} p_d$ or $= 2.1tdp_d$ whichever is the lesser shear capacity of fastener: $F_s > 1.25F_u$	$2.5\ mm < d < 7.5\ mm$, $t'/t = 1$

Notation:

- d = diameter of the fastener (mm)
- F_d = design strength of connection in shear (kN)
- F_u = ultimate strength of connection in shear (kN)
- F_s = ultimate strength of the fastener quoted and verified by manufacturer (kN)
- k = $0.111(t'/t - 1)^2 + 0.65$ $(0.65 < k < 0.90)$
- p_c = calculation value of yield stress of sheet material (kN/mm^2)
- p_d = design strength of member material, taken as the minimum yield strength but not greater than 0.84 times the minimum ultimate strength (kN/mm^2)
- p_e = design value of the yield stress, equal to the minimum guaranteed yield stress with a maximum of 0.7 times the ultimate tensile strength (kN/mm^2)
- p_{ty} = design value of yield stress; for the ultimate limit state, equal to minimum yield stress divided by 1.0, 1.1 or 1.2 depending on the safety class of the structure (kN/mm^2)
- p_u = ultimate tensile strength of the sheeting (kN/mm^2)
- t = thickness of the thinner sheet being joined (mm)
- t' = thickness of the thicker sheet being joined (mm)

Notes:

1. Edge failure of the connection is not included in these expressions.
2. BS 5950: Part 5 equations are primarily for the design of cold-formed section connections, they are included for comparison purposes only.

Table C.2: Design expressions for the strength of screw connections

Reference	Expression	Remarks
Baehre and Berggren (1973)	failure due to screw inclination and yield in bearing: $F_u = k'(d + 10)(t^2 + 0.22)p_u$	3.0 mm < d < 6.4 mm suggested safety factor=2.9 for steel sheets
	failure due to shear of fastener: $F_u = 0.8F_s$	suggested safety factor=1.65
Bryan (1973)	tearing failure: $F_u = 6t$	for "Teks" self-drilling self-tapping screws 1/4" (6.3 mm), 0.43 mm < t < 1.27 mm, 2.0 mm < t' < 3.2 mm
Grossberndt and Kniese (1975)	$F_u = 14.37t^2p_u + R$	for 6.3 mm screws only
Stark and Toma (1978)	failure by inclination of fastener[3]: $F_d = k'(d + 10)(t'^2 + 0.22)p_c$	for thickness of connected sheets being relatively small
	bearing failure of sheet material: $F_d = 2.1dtp_c$	this is with shear-bearing action along two distinctly inclined planes, causing the sheet material to pile up in front of the screw; failure load independent of the edge distance
Strnad (1979)	$F_u = K(d + 5)t"p_y"$	according to the European recommendations, the design stress is equal to the yield stress

Table C.2 continued

Reference	Expression	Remarks
SISC (1982)	failure by sheet tearing: $F_d = 2.8(t^3d)^{1/2}p_{ty}$ but not greater than $1.6tdp_{ty}$	3.0 mm $< d < 8.0$ mm, $t'/t = 1$
	$F_d = 1.6tdp_{ty}$	$t'/t \geq 2.5$
ECCS (1984b)	failure by tilting and yield in tearing: $F_d = 3.2(t^3d)^{1/2}p_e$ with a maximum of $2.1tdp_e$	3.0 mm $< d < 8.0$ mm, $t'/t = 1$
	$F_d = 2.1tdp_e$	$t'/t \geq 2.5$
BS 5950: Part 5 (1987)	shear capacity in tilting and bearing: $F_u = 3.2(t^3d)^{1/2}p_d$ or $= 2.1tdp_d$ whichever is the lesser	3.0mm $< d < 8.0$ mm, $t'/t = 1$
	$F_u = 2.1tdp_d$	$t'/t \geq 2.5$
	shear capacity of the fastener: $F_s > 1.25F_u$	

Notation: $k' = 0.156(t'/t - 1)^2 + 0.35$

$\leq$ 0.70, limiting value of k' based upon deflection limitation.

$K = 0.80 + 0.35(x - 1)$

p_u' = ultimate tensile strength of the thicker sheet (kN/mm^2)

p_y = design stress of thinner sheet being joined (kN/mm^2)

p_y' = design stress of thicker sheet being joined (kN/mm^2)

$p_y'' = 0.5[p_y(x - 1) + p_y'(3 - x)]$ (effective yield stress, KN/mm^2)

R = a constant, value depends on type of washer and tightening torque (kN)

$t'' = 0.5[t(x - 1) + t'(3 - x)]$ (effective thickness, mm)

$x = (t'p_u')/(tp_u)$

≤ 3

All other notation same as in Table C.1.

Notes: 1. Stark et al (1978) realised the small deformation capacity and brittle shear failure of common screws and recommended the design value for such failure be twenty five per cent larger than any other design strength of the connection in order to guard against such failure.

2. The expression for failure in bearing is essentially the same as those for the bearing strength of cold-formed steel in bolted connections (clause 4.5.4, AS 1538 (1974)) and for hot-rolled steel in bolted or riveted connections (clause 9.5.2, AS 1250 (1981)).

3. This expression is similar to that in Baehre et al except the replacements of F_u, t and p_u by F_d, t' and p_c respectively. It is suspected that t' in this expression should in fact be t as it would give grossly over-estimated results for sheet-to-purlin connections and the error may be due to mistyping in the original paper.

4. Other notes same as in Table C.1.

Table C.3: Design expressions for the flexibility of mechanical connections

Reference	Expression	Remarks
Strnad (1979)	$v = K/(3t''p_y''d)$	for screws, empirical formulae based on experiments carried out in Czechoslovakia
SISC (1982)	$v = 1/(k_2 dt^{1/2})$	for both screw and rivet, screw with a tightening torque of about 8 Nm
ECCS (1984b)	None	for both screw and rivet, value to be determined by testing

Notation: v = shear flexibility (mm/kN)

k_2 = a constant equal to 1 for t'/t = 1 and 1.5 for $t'/t \geq 2.5$

All other notation same as in Tables C.1 and C.2.

APPENDIX D: STEEL PROPERTIES

Table D.1: Steel properties

Material	Designation [1]	Thickness (mm)		Yield stress (MPa)		Tensile strength (MPa)		Elongation on 50 mm (%)
		Nominal	Actual [2]	Nominal	Characteristic [3]	Nominal	Characteristic [3]	
Cladding	G550	0.55	0.544 (0.002)	---	---	550~	752 (11)	2~
	G550	0.40	0.393 (0.003)	---	---	550~	743 (14)	2~
Purlin	G450	1.60	1.54 (0.01)	450~	651 (2)	500~	660 (2)	10
	G450	1.90	1.86 (0.01)	450~	674 (3)	500~	687 (2)	10
	G450	2.50	2.56 (0.01)	450~	502 (8)	500~	547 (2)	10
	G280	1.60	1.59 (0.01)	280	286 (5)	390*	406 (3)	34
	G280	2.00	1.90 (0.02)	280	286 (2)	390*	401 (4)	32
	G280	2.50	2.49 (0.02)	280	287 (8)	390*	405 (4)	38

Notes:
1. G550 materials are to NZS 3441 (1978) requirements. Bulk of G280 materials is to AS 1204 (1980) Grade G250 with coils having minimum yield of 280 MPa selected. G450 materials are either Grade D or E to ASTM A446M (1983) with a minimum yield of 450 MPa. All the G550 and G450 materials are galvanised while the G280 purlins are shop-coated with a primer.
2. For galvanised samples, the base metal thicknesses were obtained by stripping the zinc coatings using dilute hydrochloric acid (1 + 1), i.e., alternative standard method specified in ASTM A90-81 (1981). For painted purlin samples, the paint was removed by acetone.
3. Characteristic strengths were computed according to the European recommendations (ECCS, 1984b), i.e., based on the student distribution and a confidence level of 95%.
4. Bracketed figures indicate standard deviation.
5. For practical purposes, yield stress and tensile strength are the same for cladding materials.
6. ~ -- values from NZS 3441 (1978).
7. * -- values from BS 2989 (1975), for information only.
8. For information only (John Lysaght Ltd, 1980a), the range of yield stress and tensile strength for G450 materials are 450 to 550 MPa and 500 to 600 MPa respectively. The range of tensile strength for G550 materials is 670 to 770 MPa.

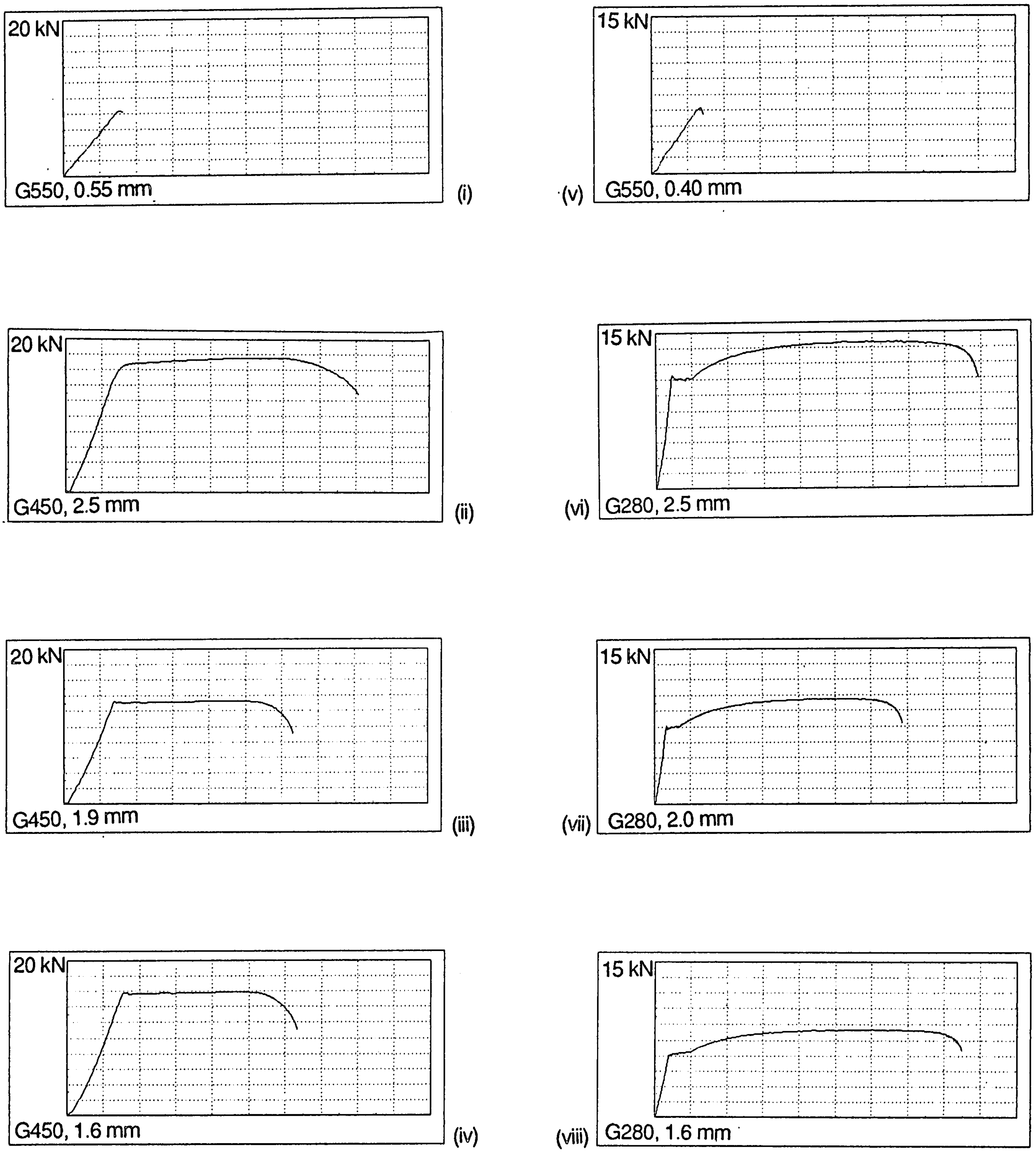


Figure D.1 : Typical load-deflection curves of cladding and purlin materials.

APPENDIX E: SHEAR AND PULL-OUT TEST RESULTS

Table E.1: Seam connection shear test results

Fastener type	Thickness of lapped sheets (mm)	Characteristic strength (kN)	Characteristic strength per unit thickness (kN/mm)	Average flexibility (mm/kN)	Mean load at 3mm deflection (kN)	Failure mode	Remark
"Tapits"	0.55	2.89 (0.15)	5.2	0.14 (0.09)	2.7 (0.2)	1	
	0.40	1.50 (0.18)	3.8	0.26 (0.10)	1.4 (0.2)	1	
4.8 mm monel rivets	0.55	2.67 (0.03)	4.9	0.18 (0.03)	2.8 (0.2)	2	
	0.40	1.81 (0.08)	4.5	0.20 (0.04)	1.3 (0.2)	2	
4.0 mm monel rivets	0.55	2.04 (0.06)	3.7	0.20 (0.04)	1.9 (0.3)	2	
	0.40	1.43 (0.10)	3.6	0.29 (0.08)	1.1 (0.1)	2	
4.8 mm aluminium rivets	0.55	2.18 (0.01)	4.0	0.11 (0.02)	1.4 (0.3)	3	rivets broke at 3.5 mm deflection
	0.40	1.68 (0.07)	4.2	0.16 (0.04)	1.4 (0.0)	2	
4.0 mm aluminium rivets	0.55	1.28 (0.04)	2.3	0.11 (0.02)	--	3	rivets broke at 1.9 mm deflection
	0.40	1.11 (0.06)	2.8	0.22 (0.04)	0.5 (0.1)	3	rivets broke at 3.1 mm deflection
"Bulb-tite" rivets	0.55	1.61 (0.07)	2.9	0.42 (0.06)	--	3	rivets broke at 2.8 mm deflection
	0.40	1.36 (0.05)	3.4	0.42 (0.08)	--	3	rivets broke at 2.9 mm deflection

Notes: 1. Bracketed figures indicate standard deviation.

2. Failure modes: 1 = inclination of screw followed by tearing of the sheets and eventually pull-out; 2 = rivet inclination, local yielding or tearing of sheets followed by pull-out from the lower sheet; 3 = failure in bending or tilting in combination with shear of the fastener.

Table E.2: Sheet(0.55 mm)-to-purlin connection shear test results

Fastener type	Purlin type	Characteristic strength (kN)	Characteristic strength per unit thickness (kN/mm)	Average flexibility (mm/kN)	Characteristic strength from 3 mm load (kN)	Characteristic strength from 3 mm load per unit thickness (kN/mm)	Failure mode
12g neo	450/2.5	3.21 (0.30)	5.8	0.26 (0.04)	2.78 (0.39)	5.1	1
	450/1.9	3.07 (0.41)	5.6	0.29 (0.04)	2.82 (0.43)	5.1	1
	450/1.6	2.75 (0.63)	5.0	0.25 (0.04)	2.38 (0.73)	4.3	1
	280/2.5	3.62 (0.20)	6.6	0.27 (0.03)	3.32 (0.26)	6.0	1
	280/2.0	3.30 (0.47)	6.0	0.25 (0.03)	2.86 (0.63)	5.2	1
	280/1.6	3.60 (0.37)	6.6	0.31 (0.03)	3.40 (0.45)	6.2	1
12g emb	450/2.5	3.83 (0.20)	7.0	0.28 (0.05)	2.78 (0.57)	5.1	1
	450/1.9	3.57 (0.23)	6.5	0.31 (0.08)	3.11 (0.37)	5.6	1
	450/1.6	3.43 (0.37)	6.2	0.29 (0.07)	2.76 (0.50)	5.0	1
	280/2.5	3.67 (0.18)	6.7	0.30 (0.07)	3.22 (0.34)	5.9	1
	280/2.0	3.67 (0.27)	6.7	0.33 (0.07)	3.24 (0.41)	5.9	1
	280/1.6	3.78 (0.24)	6.9	0.30 (0.06)	3.74 (0.25)	6.8	1
14g neo	450/2.5	3.32 (0.32)	6.0	0.23 (0.04)	3.26 (0.28)	5.9	1
	450/1.9	3.18 (0.42)	5.8	0.25 (0.04)	2.98 (0.47)	5.4	1
	450/1.6	2.88 (0.63)	5.2	0.20 (0.04)	2.52 (0.75)	4.6	1

Table E.2 continued

Fastener type	Purlin type	Characteristic strength (kN)	Characteristic strength per unit thickness (kN/mm)	Average flexibility (mm/kN)	Characteristic strength from 3 mm load (kN)	Characteristic strength from 3 mm load per unit thickness (kN/mm)	Failure mode
14g neo	280/2.5	3.29 (0.35)	6.0	0.24 (0.05)	2.78 (0.51)	5.0	1
	280/2.0	2.97 (0.49)	5.4	0.18 (0.05)	2.54 (0.60)	4.6	1
	280/1.6	3.06 (0.55)	5.6	0.20 (0.05)	2.88 (0.59)	5.2	1

Notes: 1. Bracketed figures indicate standard deviation.

2. Failure mode: 1 = bearing and tearing in the thinner sheet, possibly accompanied by tilting or pull-out of the fastener.

Table E.3: Sheet(0.40 mm)-to-purlin connection shear test results

Fastener type	Purlin type	Characteristic strength (kN)	Characteristic strength per unit thickness (kN/mm)	Average flexibility (mm/kN)	Characteristic strength from 3 mm load (kN)	Characteristic strength from 3 mm load per unit thickness (kN/mm)	Failure mode
12g neo	450/2.5	2.23 (0.17)	5.6	0.36 (0.05)	1.91 (0.19)	4.8	1
	450/1.9	1.91 (0.32)	4.8	0.40 (0.09)	1.68 (0.36)	4.2	1
	450/1.6	1.84 (0.26)	4.6	0.34 (0.10)	1.57 (0.33)	3.9	1
	280/2.5	2.28 (0.18)	5.7	0.38 (0.06)	1.79 (0.30)	4.5	1
	280/2.0	2.19 (0.28)	5.5	0.39 (0.13)	1.83 (0.37)	4.6	1
	280/1.6	2.05 (0.14)	5.1	0.34 (0.08)	1.96 (0.10)	4.9	1
12g emb	450/2.5	2.23 (0.26)	5.6	0.54 (0.11)	1.86 (0.17)	4.7	1
	450/1.6	2.14 (0.22)	5.3	0.40 (0.16)	1.44 (0.38)	3.9	1
	280/1.6	2.37 (0.15)	5.9	0.41 (0.13)	1.55 (0.35)	3.9	1
14g neo	450/2.5	1.85 (0.28)	4.6	0.26 (0.11)	1.74 (0.30)	4.4	1
	450/1.6	1.77 (0.29)	4.4	0.29 (0.06)	1.60 (0.31)	4.0	1
	280/1.6	2.00 (0.22)	5.0	0.28 (0.13)	1.68 (0.28)	4.2	1

Notes: 1. Bracketed figures indicate standard deviation.

2. Failure mode: 1 = bearing and tearing in the thinner sheet, possibly accompanied by tilting or pull-out of the fastener.

Table E.4: Sheet(0.55 mm)-to-purlin connection pull-out test results

Fastener type	Purlin type	Characteristic strength (kN)	Characteristic strength per unit thickness (kN/mm)	Average flexibility (mm/kN)	Characteristic strength from 3 mm load (kN)	Characteristic strength from 3 mm load per unit thickness (kN/mm)	Failure mode
12g neo	280/2.5	6.38 (0.50)	11.6	0.11 (0.02)	6.20 (0.53)	11.3	1
	280/1.6	4.78 (0.80)	8.7	0.13 (0.04)	4.74 (0.81)	8.6	1
12g emb	450/2.5	7.72 (0.38)	14.0	0.11 (0.02)	6.84 (0.65)	12.4	1
	450/1.6	6.79 (0.45)	12.3	0.06 (0.02)	5.62 (0.83)	10.2	1
	280/2.5	7.66 (0.37)	13.9	0.08 (0.03)	7.08 (0.59)	12.9	1
	280/1.6	6.30 (0.23)	11.5	0.10 (0.01)	6.30 (0.23)	11.5	1
14g neo	280/2.5	7.33 (0.36)	13.3	0.09 (0.02)	7.42 (0.13)	13.5	1
	280/1.6	6.57 (0.22)	11.9	0.14 (0.03)	6.55 (0.21)	11.9	1

Notes: 1. Bracketed figures indicate standard deviation.

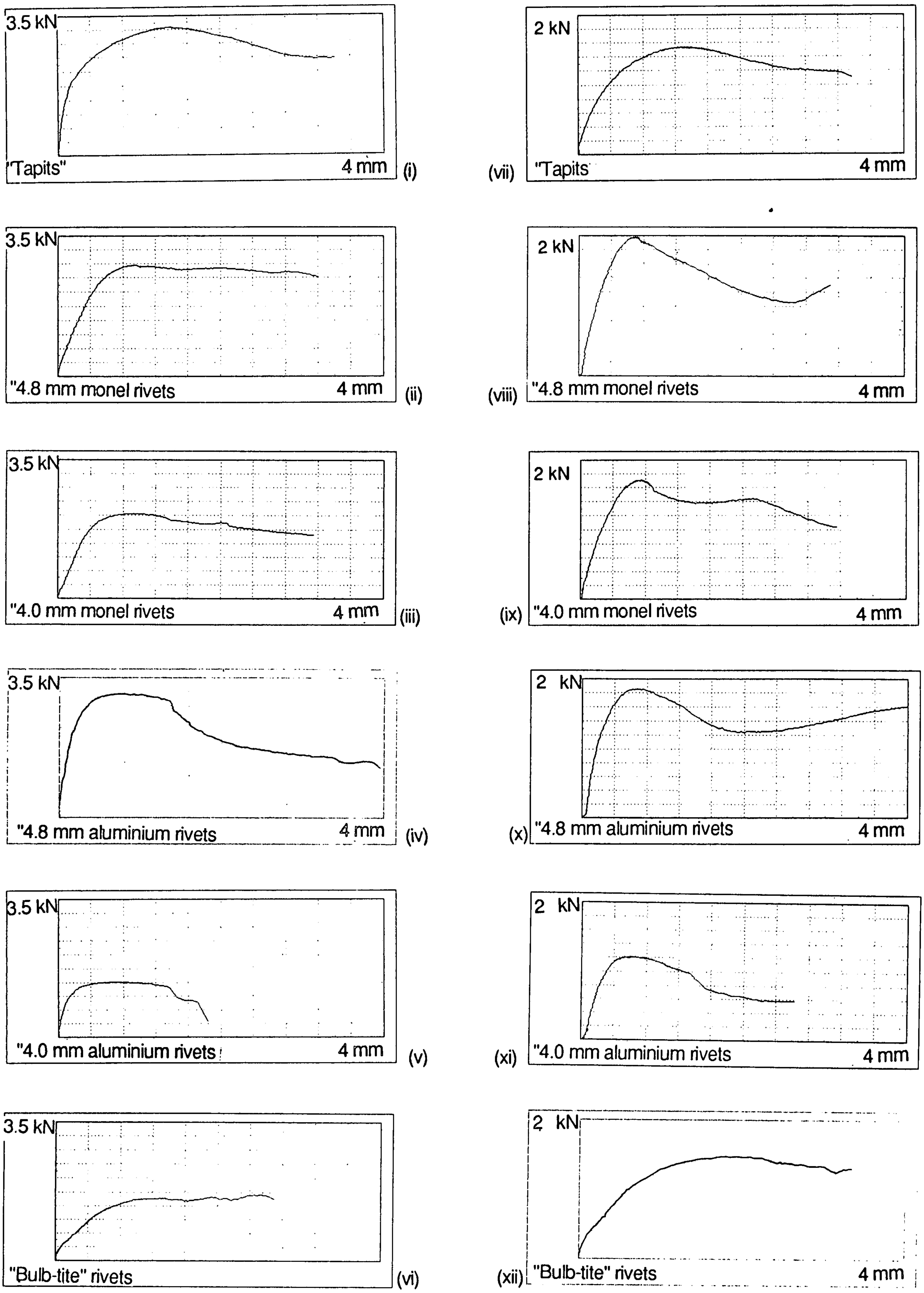
2. Failure mode: 1 = bearing and tearing in the thinner sheets, accompanied by tilting and pull-out of the fastener.

Table E.5: Sheet(0.40 mm)-to-purlin connection pull-out test results

Fastener type	Purlin type	Characteristic strength (kN)	Characteristic strength per unit thickness (kN/mm)	Average flexibility (mm/kN)	Characteristic strength from 3 mm load (kN)	Characteristic strength from 3 mm load per unit thickness (kN/mm)	Failure mode
12g emb	450/2.5	6.10 (0.32)	15.2	0.15 (0.06)	4.69 (0.81)	11.7	1
	280/1.6	5.78 (0.21)	14.4	0.11 (0.01)	5.75 (0.22)	14.4	1

Notes: 1. Bracketed figures indicate standard deviation.

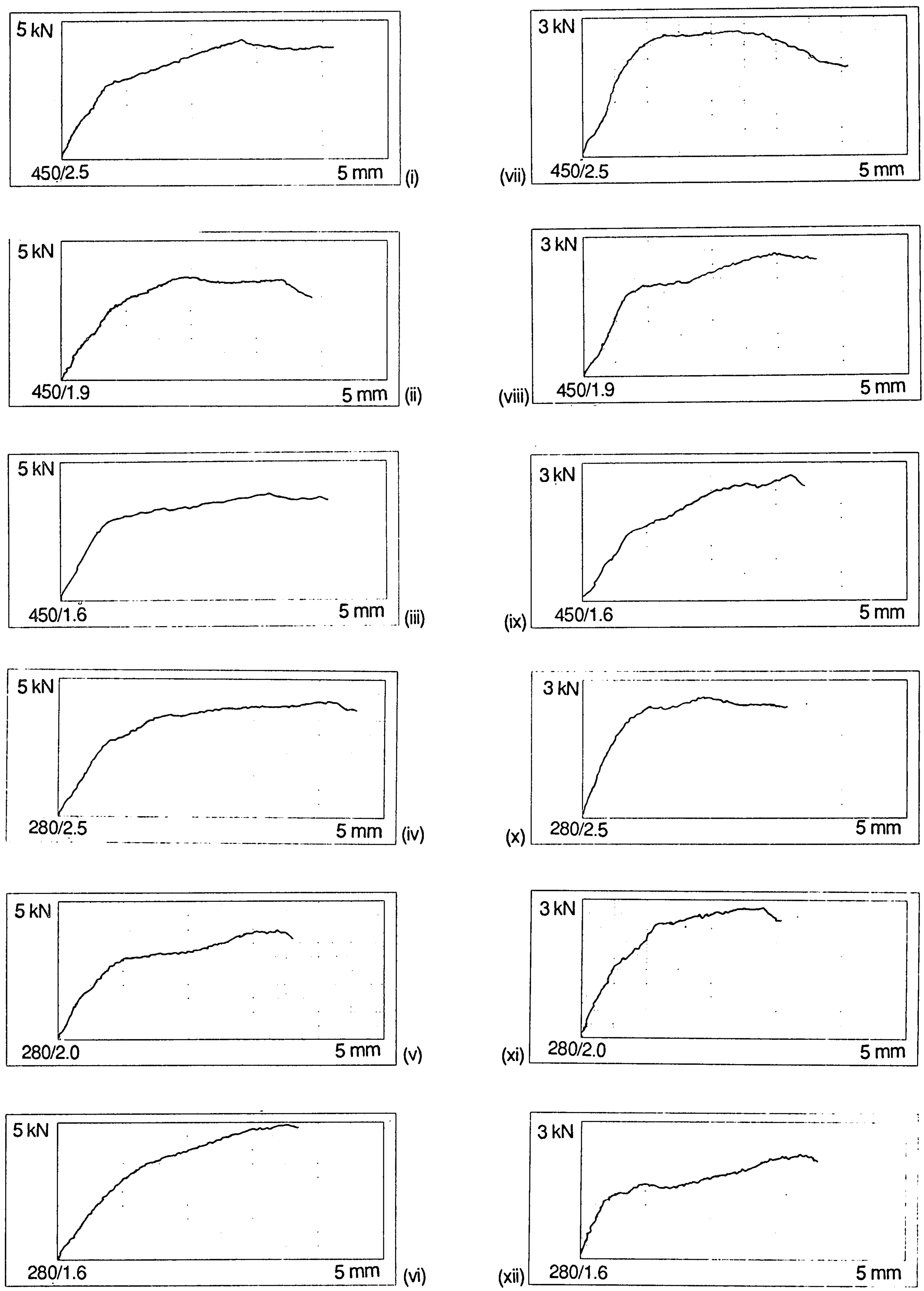
2. Failure mode: 1 = bearing and tearing in the thinner sheets, accompanied by tilting and pull-out of the fastener.



(a) sheeting thickness = 0.55 mm

(b) sheeting thickness = 0.40 mm

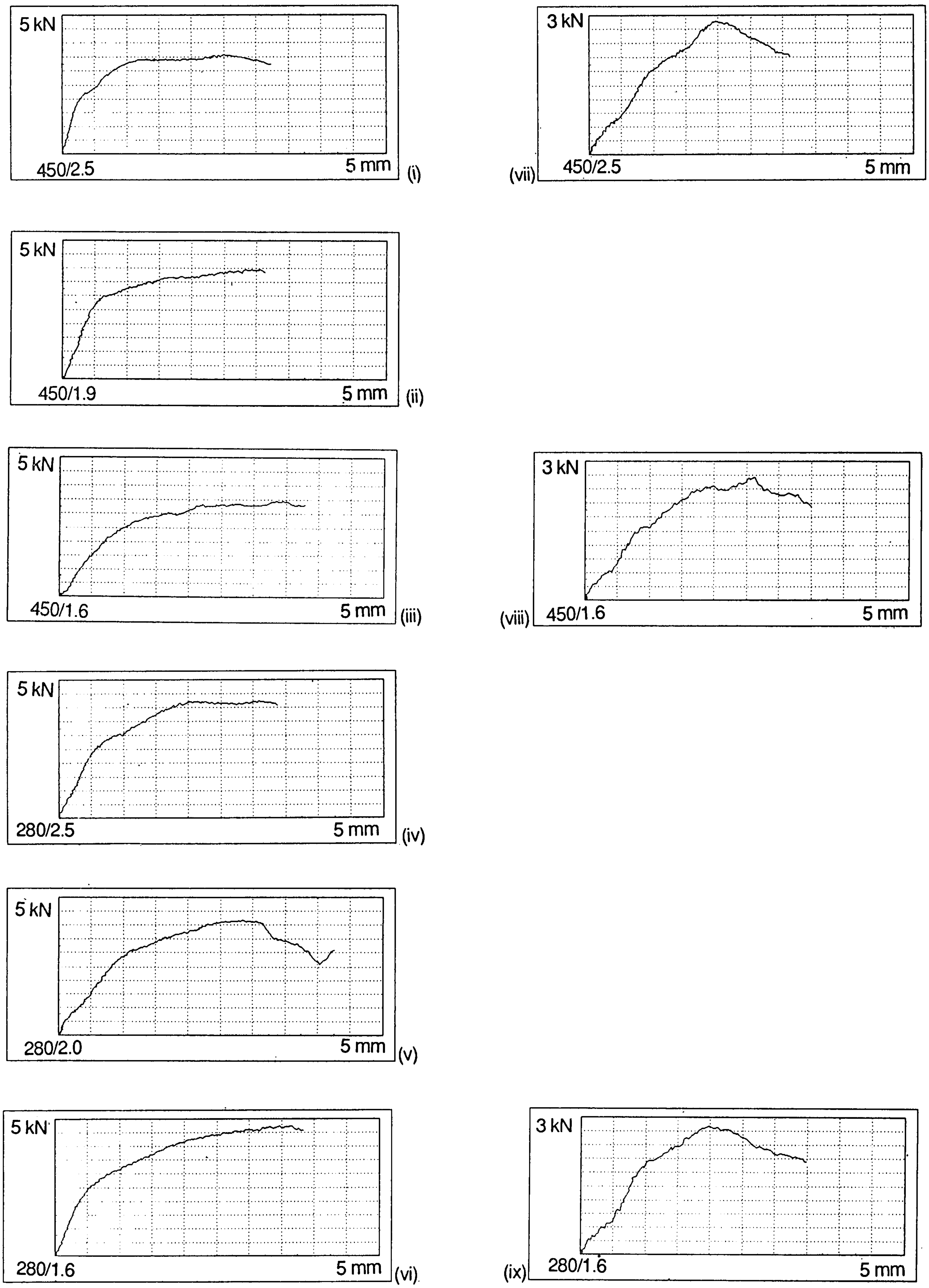
Figure F.1 : Load-deflection curves for seam connections.



(a) sheeting thickness = 0.55 mm

(b) sheeting thickness = 0.40 mm

Figure F.2 : Load-deflection curves for sheet-to-purlin connections (12g neo) in shear tests.



(a) sheeting thickness = 0.55 mm

(b) sheeting thickness = 0.40 mm

Figute F.3 : Load-deflection curves for sheet-to-purlin connections (12g emb) in shear tests.

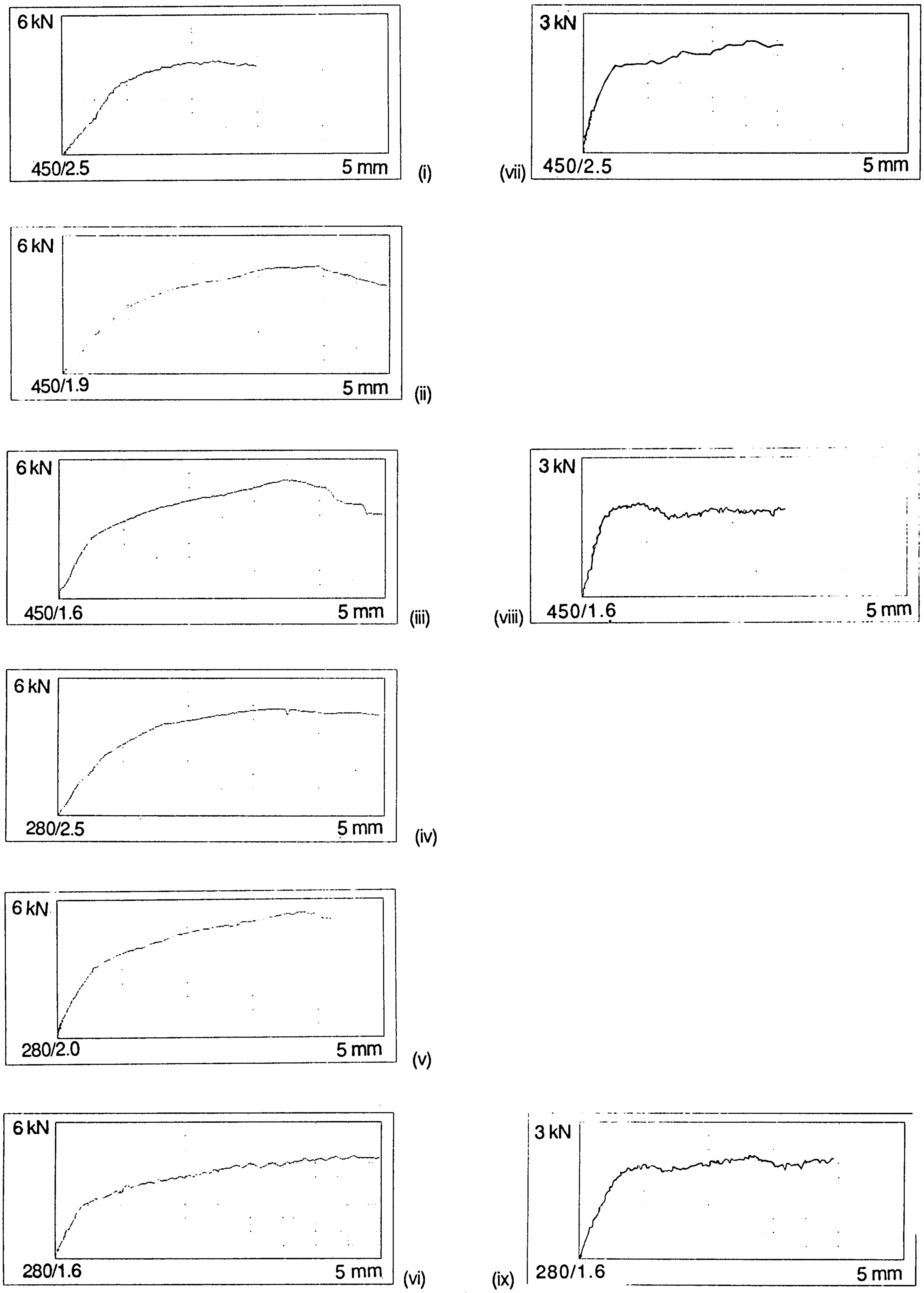
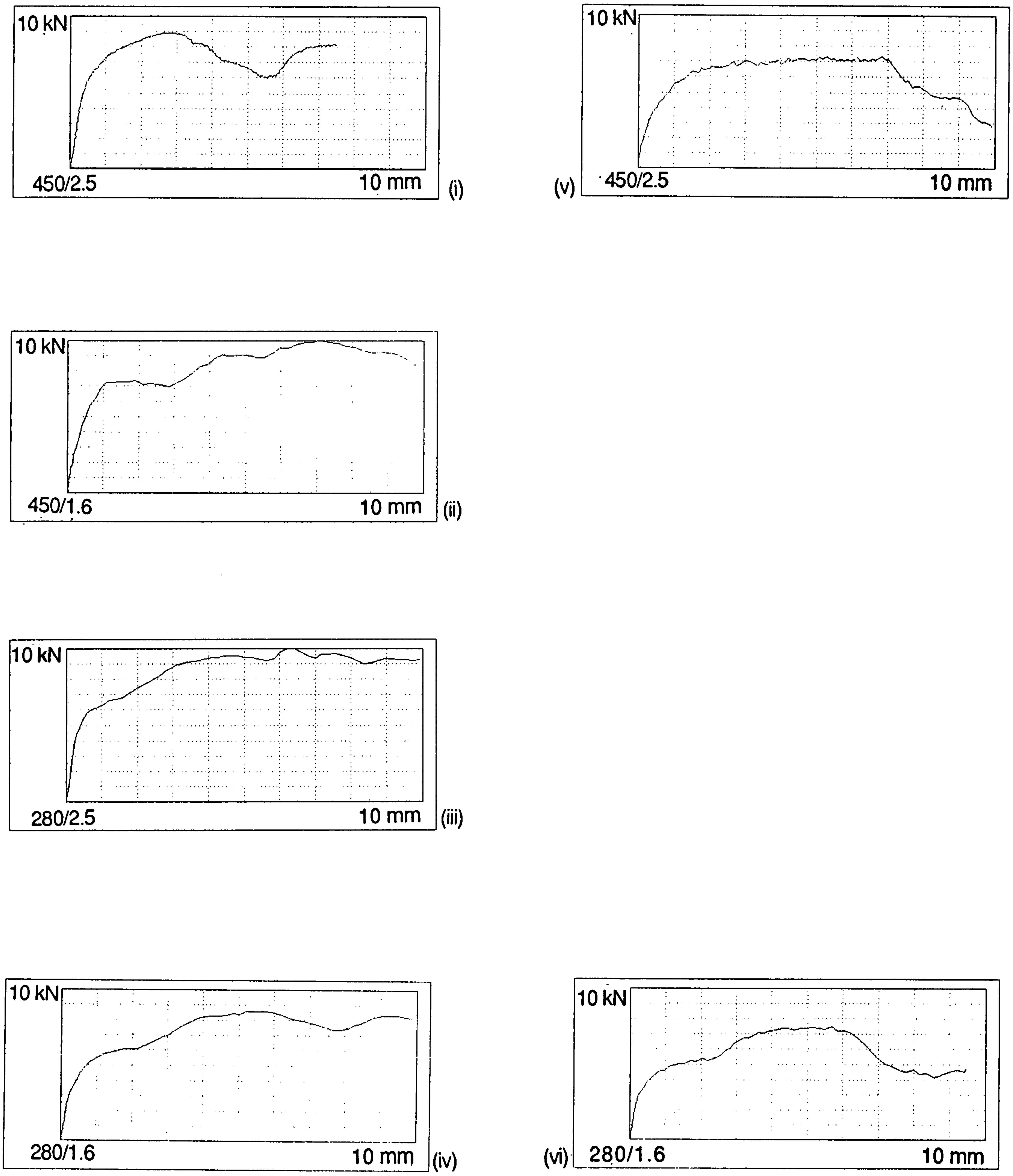


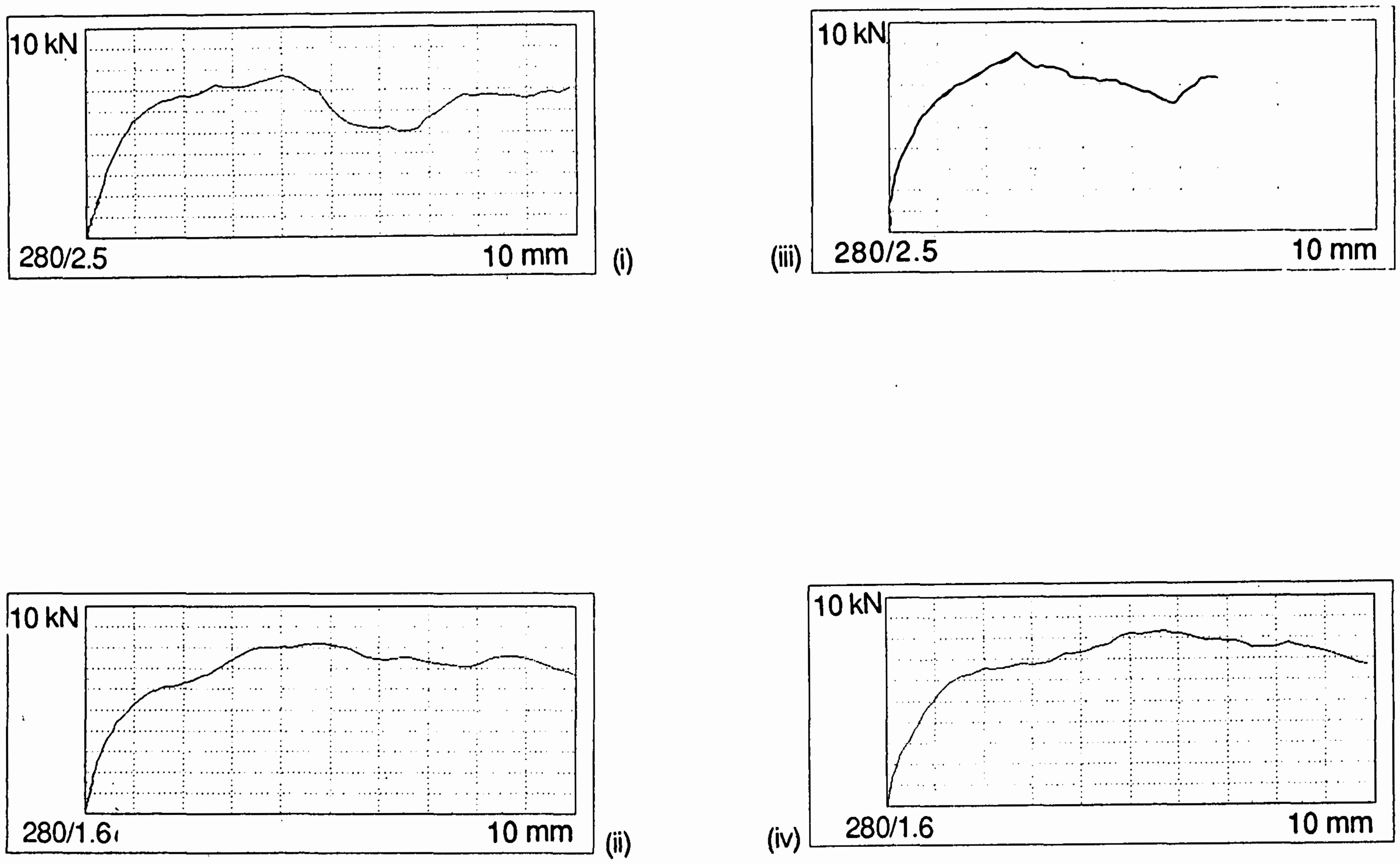
Figure F.4 : Load-deflection curves for sheet-to-purlin connections (14 g neo) in shear tests.



(a) sheeting thickness = 0.55 mm

(b) sheeting thickness = 0.40 mm

Figure F.5: Load-deflection curves for sheet-to-purlin connections (12g emb) in pull-out tests.



(a) fastener = 12g neo

(b) fastener = 14g neo

Figure F.6 : Load-deflection curves for sheet (0.55 mm)-to-purlin connections in pull-out tests.

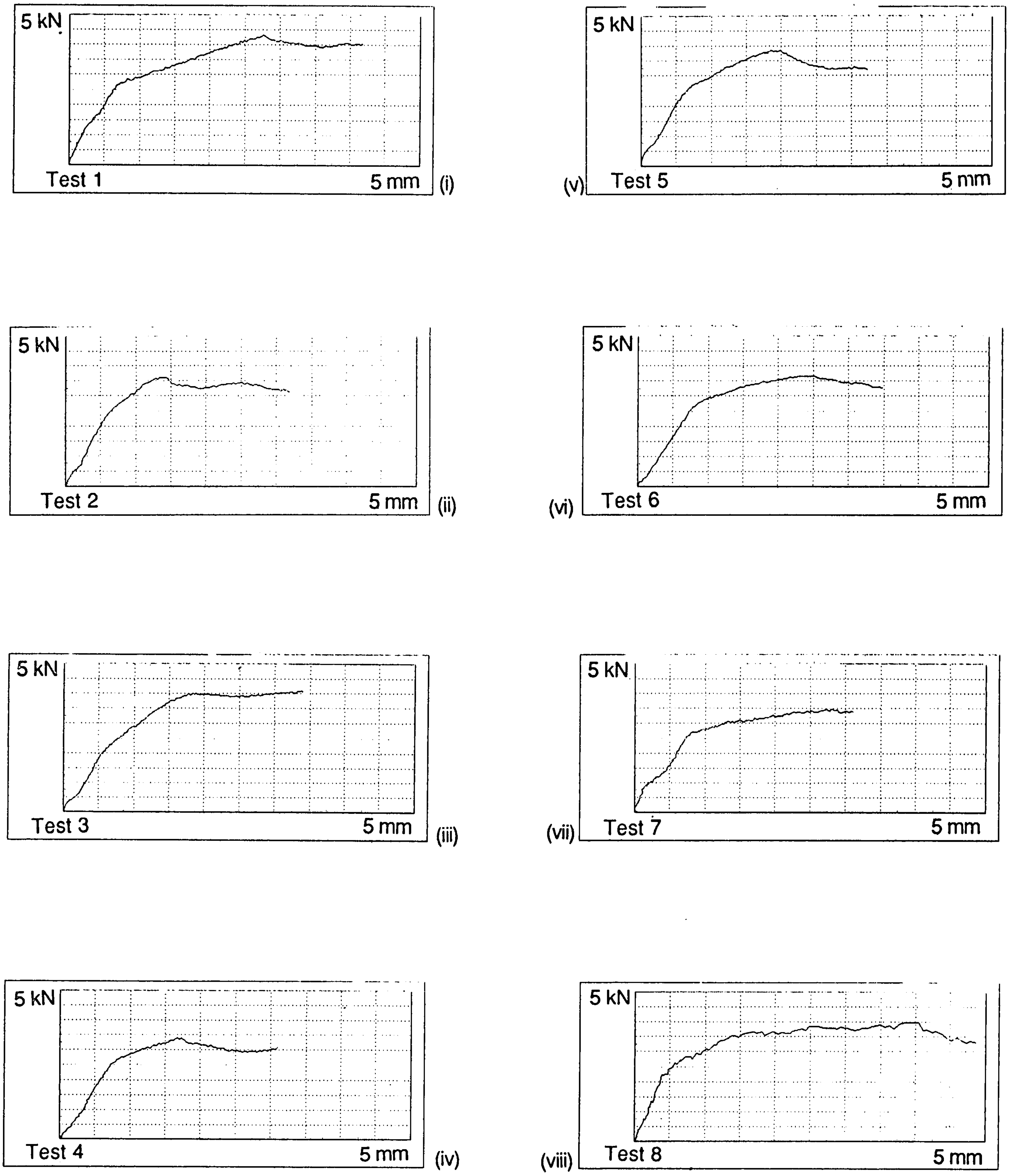
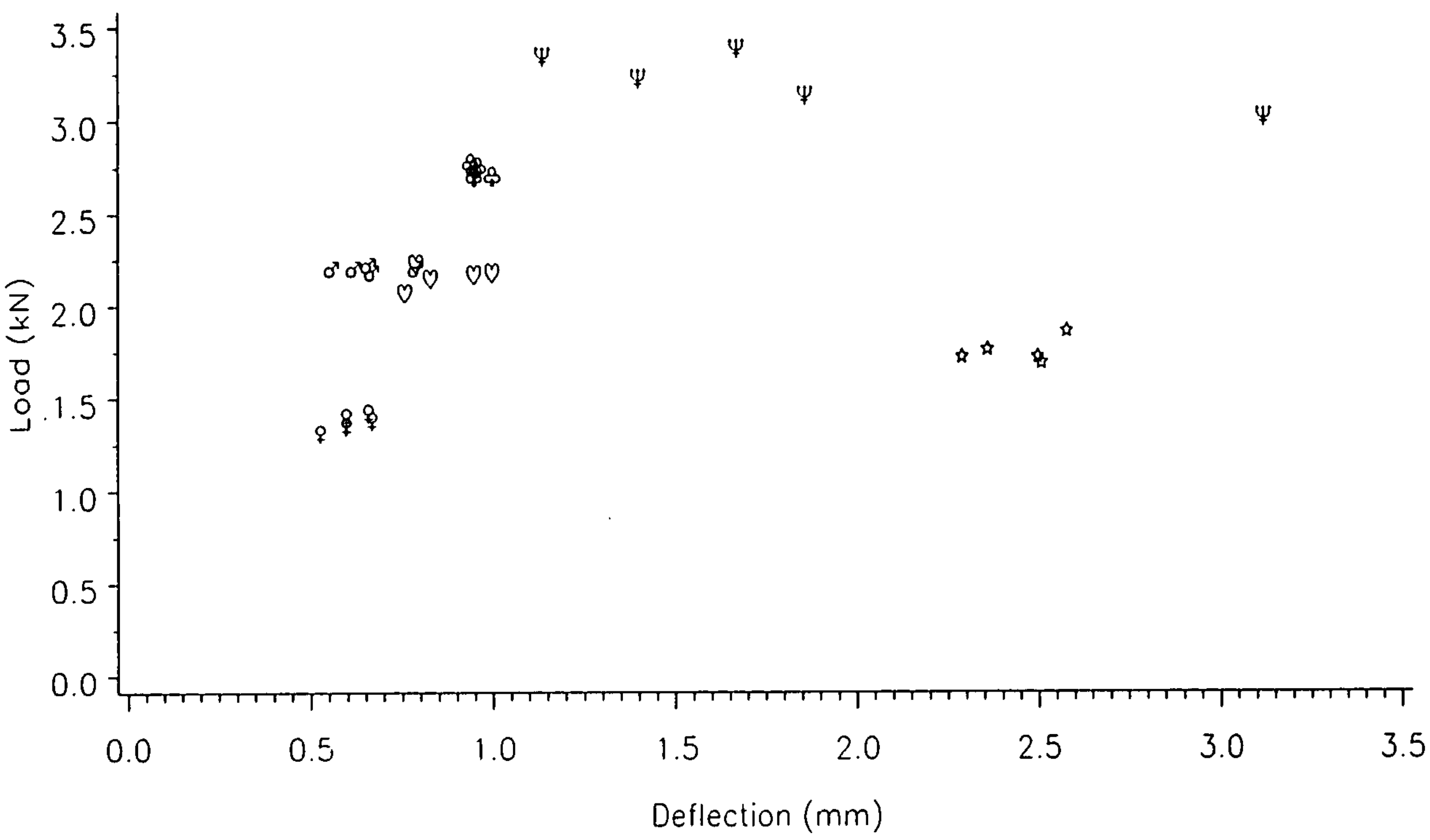
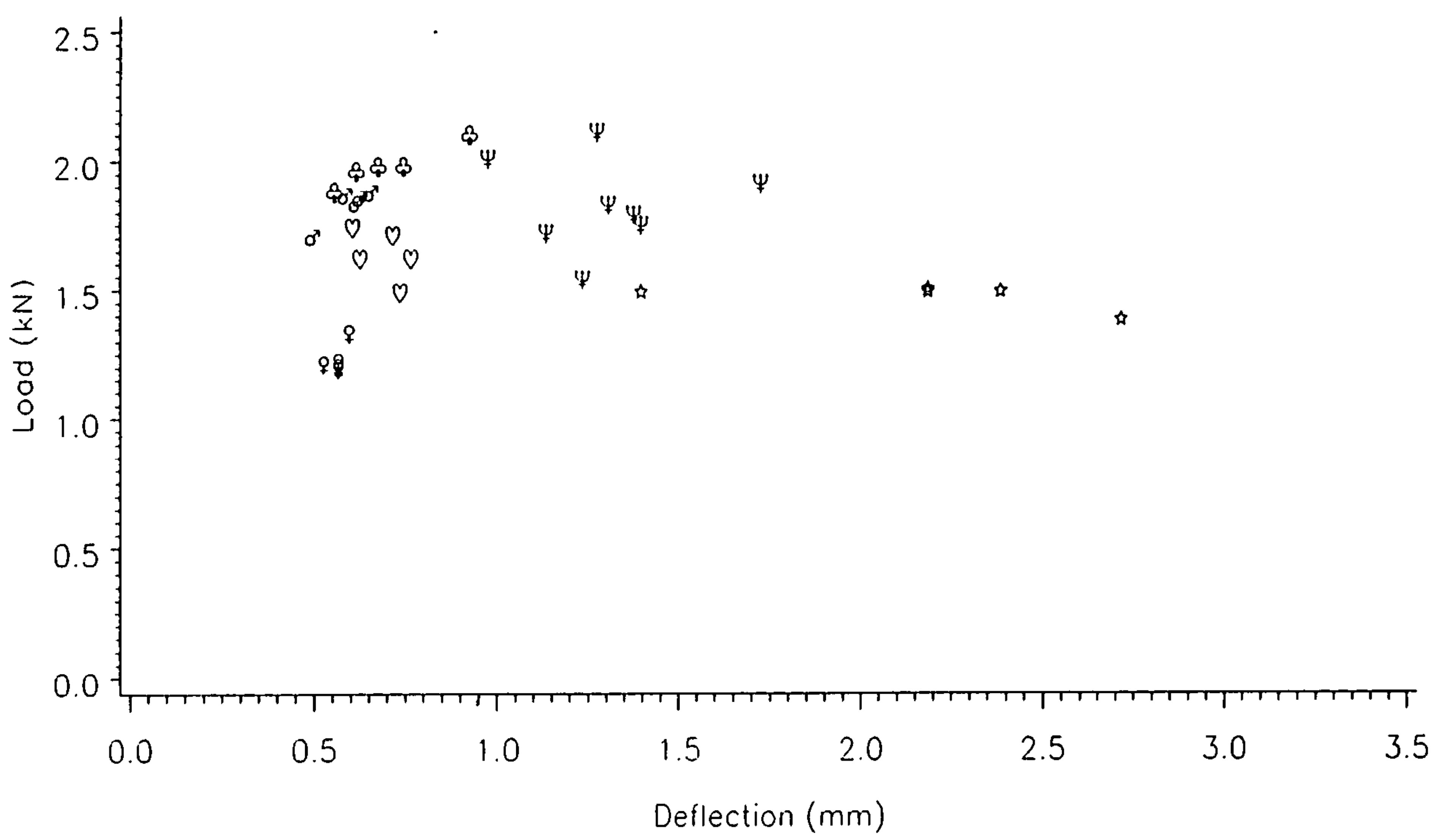


Figure F.7: Load-deflection curves for sheet (0.55 mm)-to-purlin connections (12g neo) in shear tests.

APPENDIX G: MAXIMUM LOAD VS DEFLECTION



(a) sheeting thickness = 0.55 mm



(b) sheeting thickness = 0.40 mm

Notation: fastener type:

- ♆ "Tapits"
- ♂ 4.8 mm aluminium rivets
- ☆ "bulb-tite" rivets
- ♧ 4.8 mm monel rivets
- ♀ 4.0 mm aluminium rivets
- ♡ 4.0 mm monel rivets

Figure G.1: Maximum load vs deflection for seam connections.

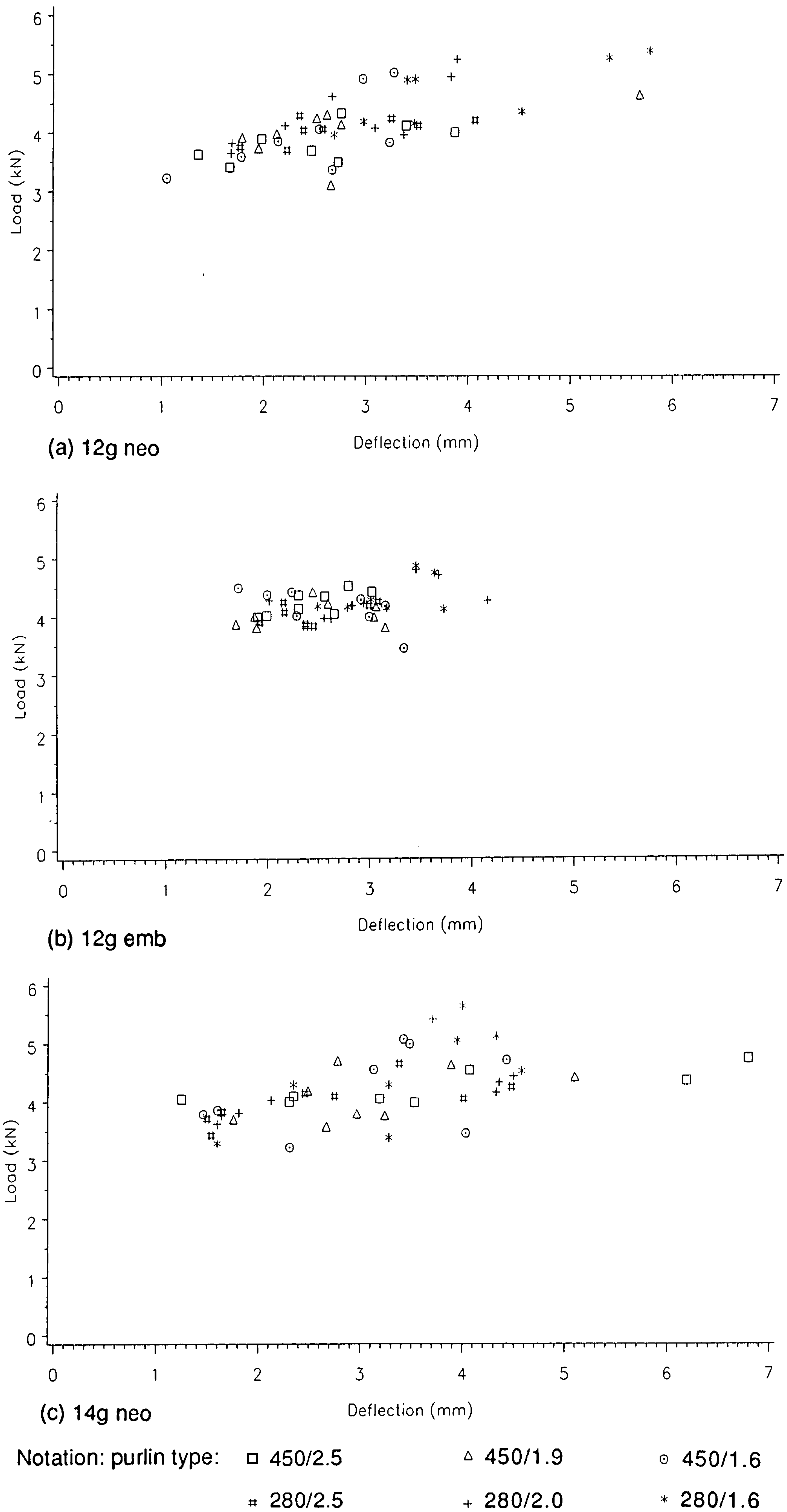
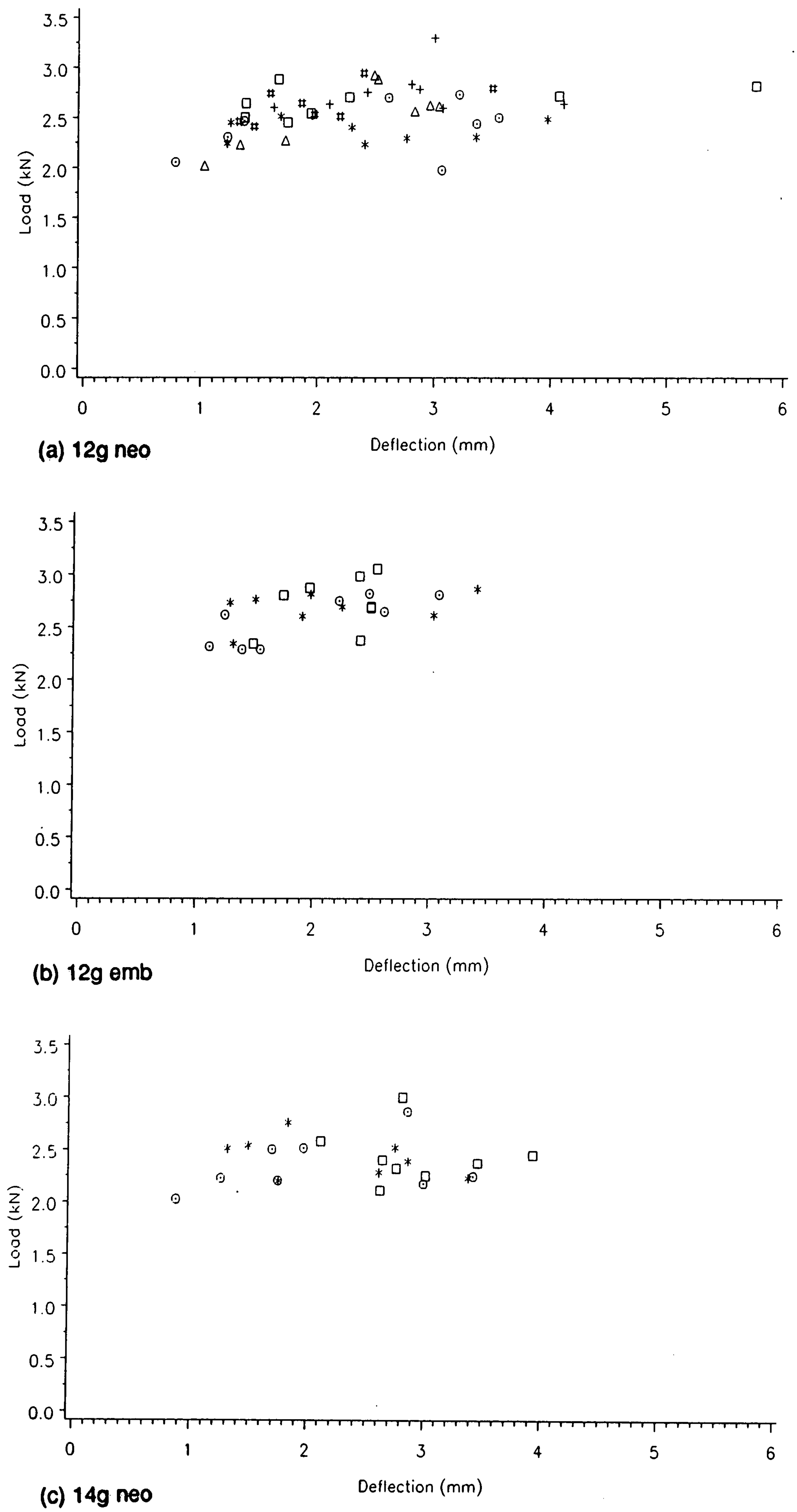


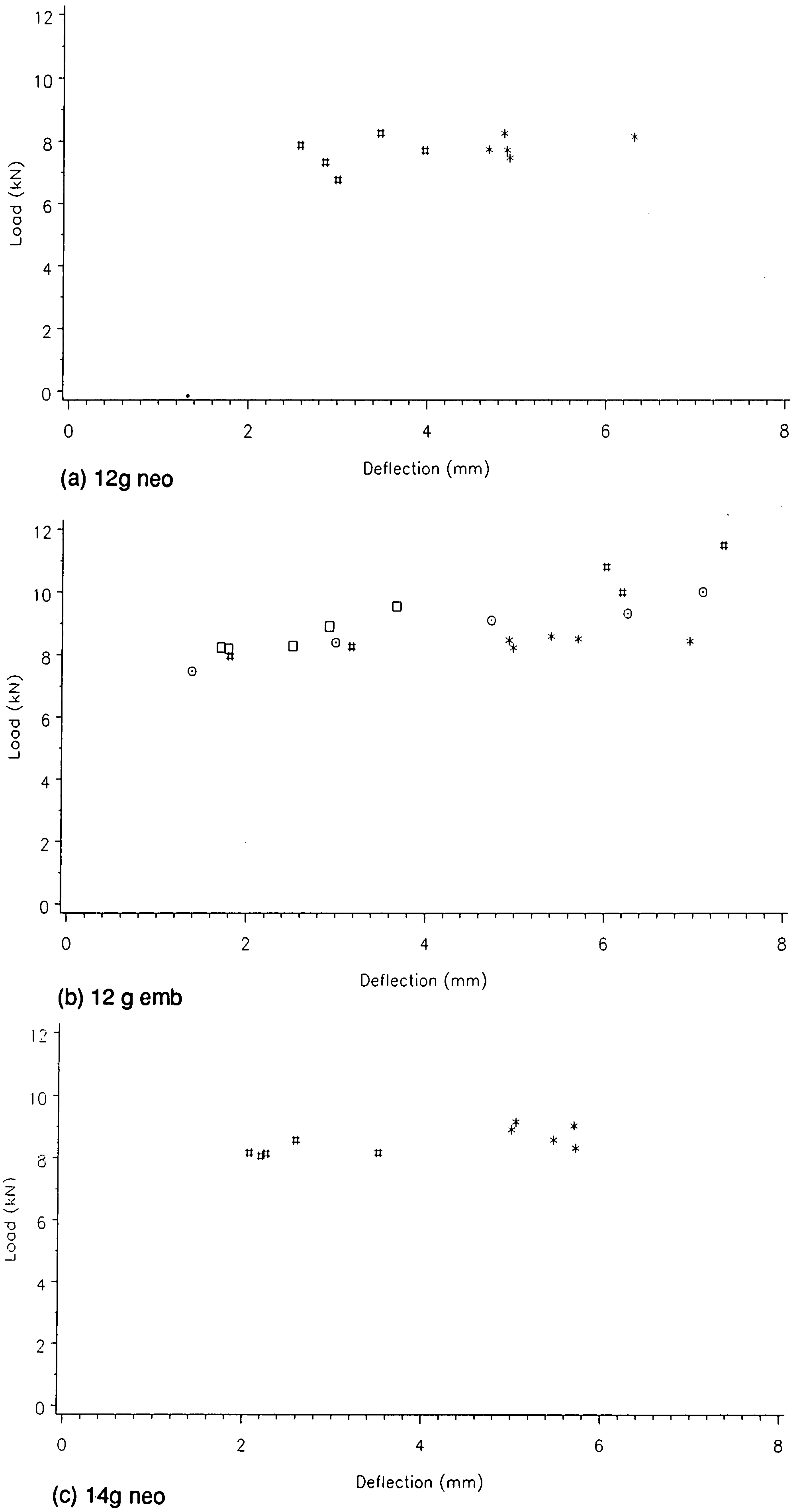
Figure G.2: Maximum load vs deflection for sheet (0.55 mm)-to-purlin connections in shear tests.



Notation: purlin type: □ 450/2.5 △ 450/1.9 ⊙ 450/1.6

\# 280/2.5 + 280/2.0 * 280/1.6

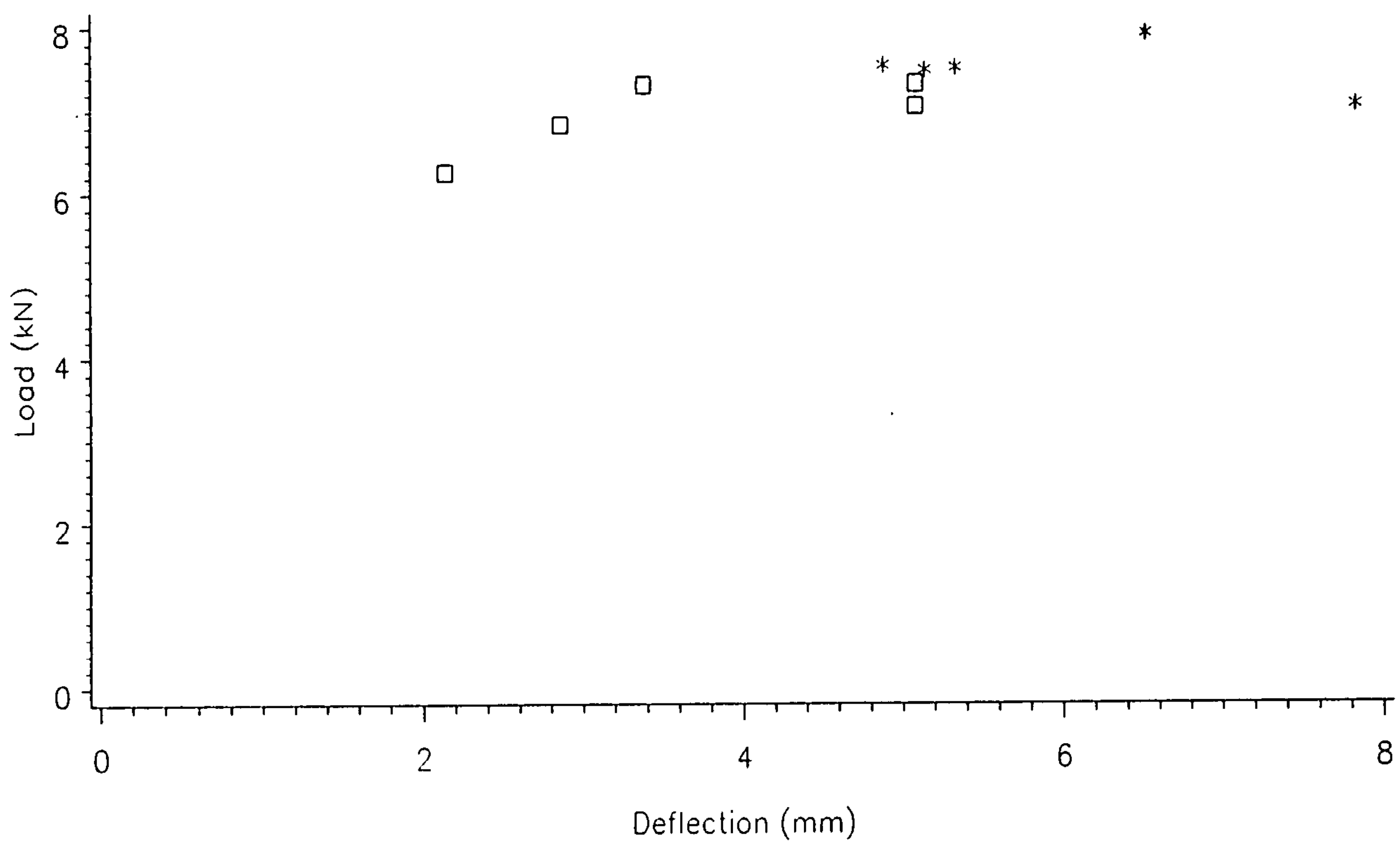
Figure G.3: Maximum load vs deflection for sheet (0.40 mm)-to-purlin connections in shear tests.



Notation: purlin type: □ 450/2.5 ⊙ 450/1.6

280/2.5 * 280/1.6

Figure G.4: Maximum load vs deflection for sheet (0.55 mm)-to-purlin connections in pull-out tests.



Notation: purlin type: □ 280/11.6

* 450/2.5

Figure G.5: Maximum load vs deflection for sheet (0.40 mm)-to-purlin connections (12g emb) in pull-out tests.

Appendix H : Failure Modes

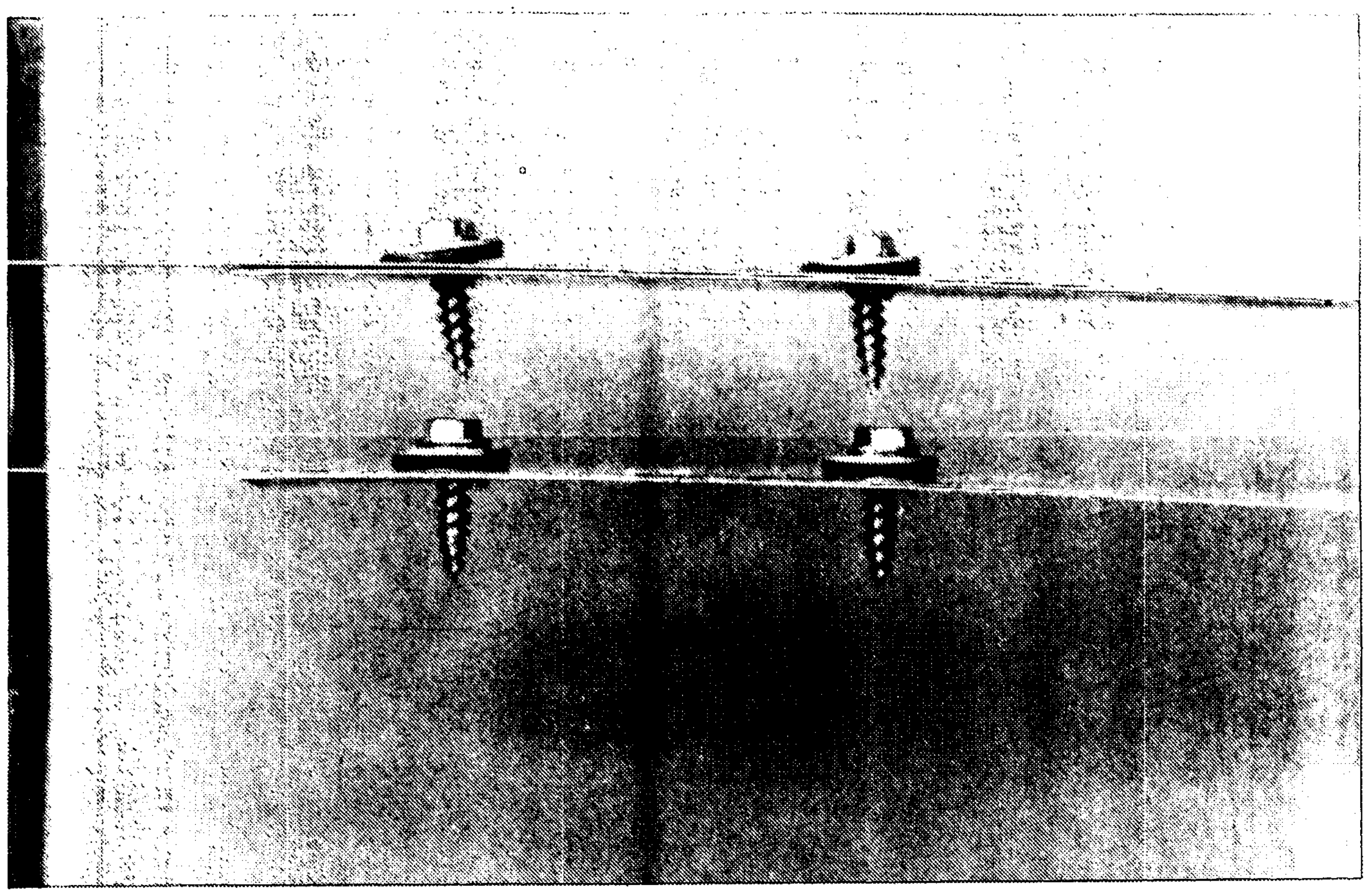


Figure H.1: "Tapit" connections.

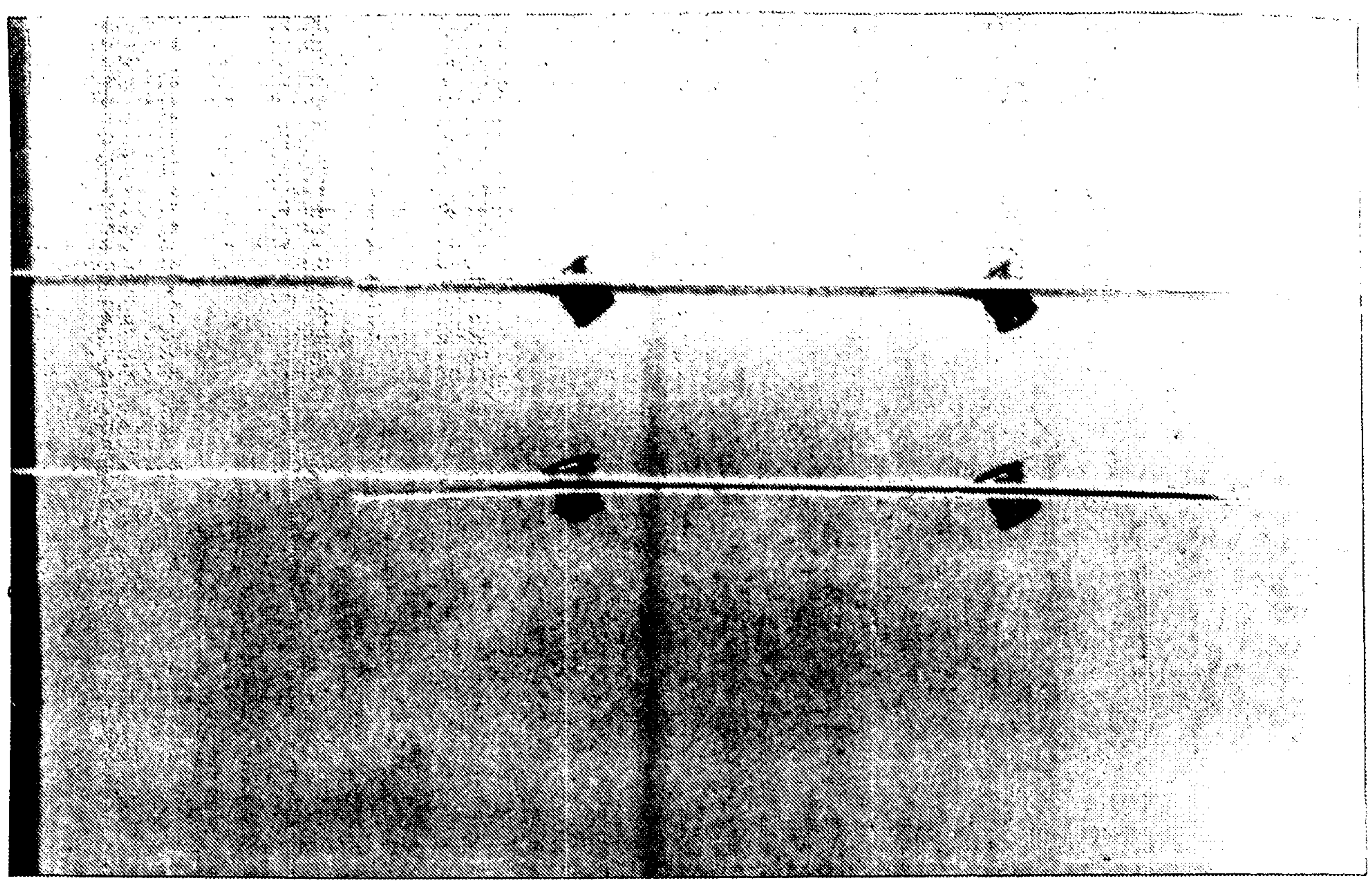


Figure H.2: 4.8 mm monel rivet connections.

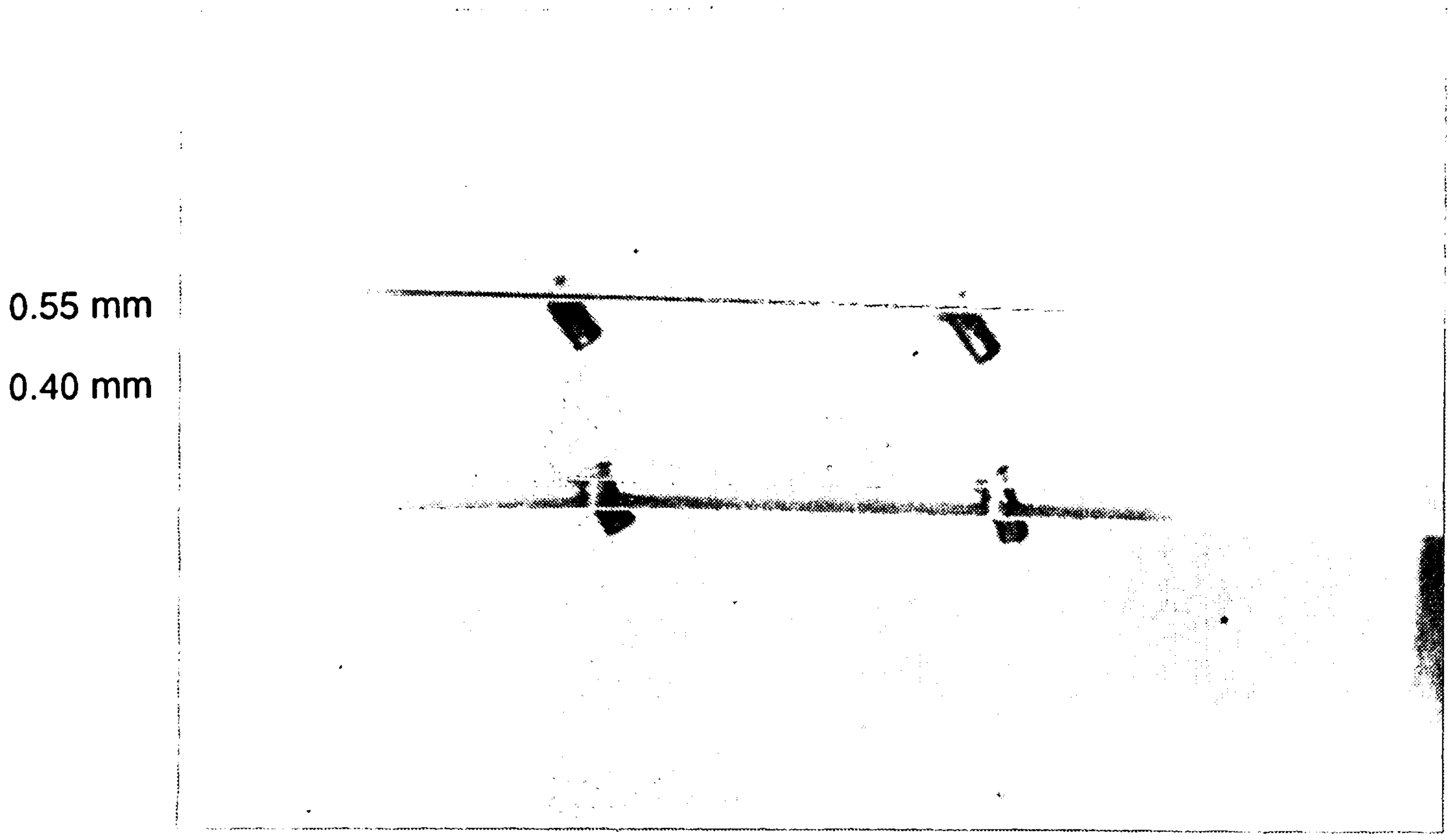


Figure H.3: 4.0 mm monel rivet connections.

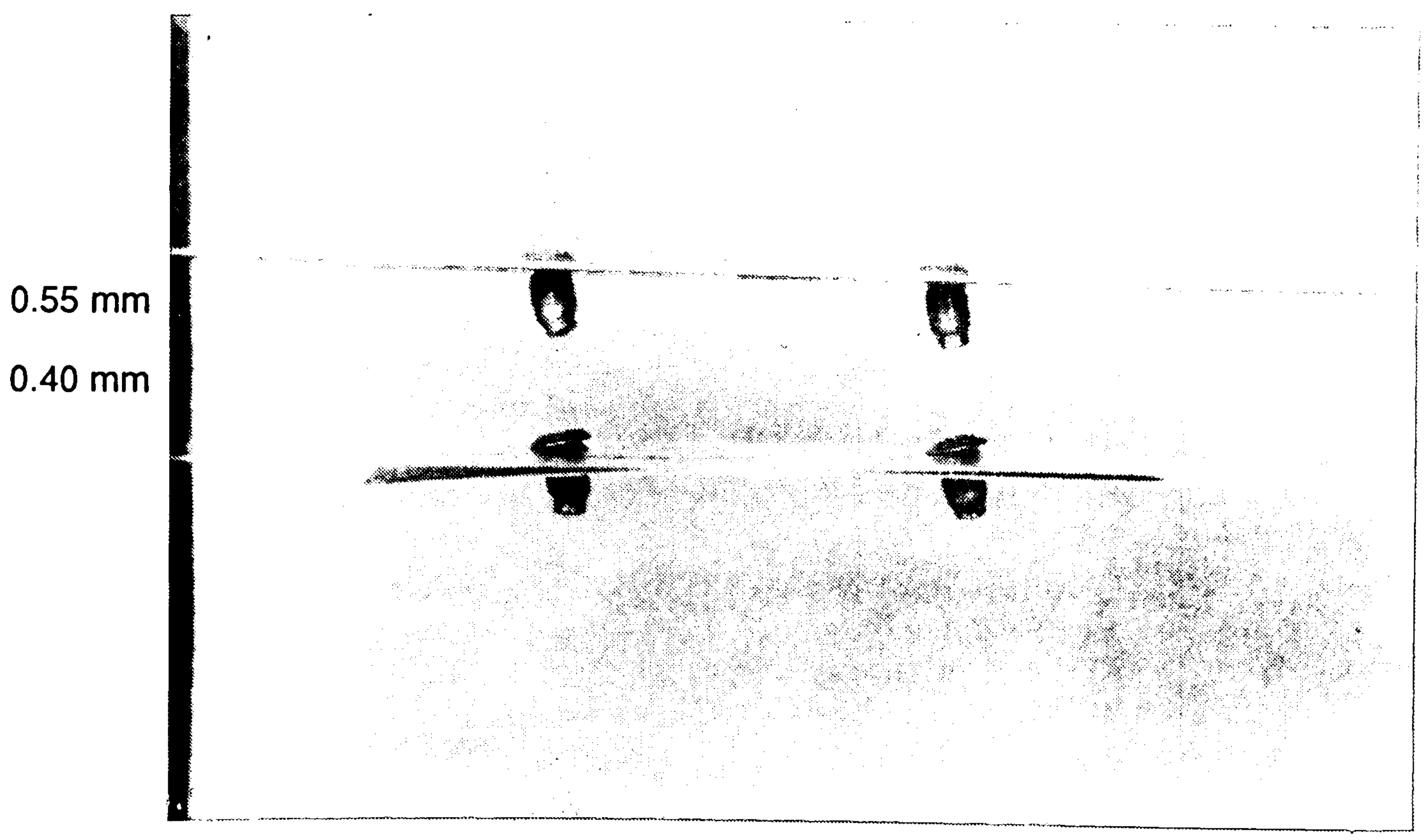


Figure H.4: 4.8 mm aluminium rivet connections.

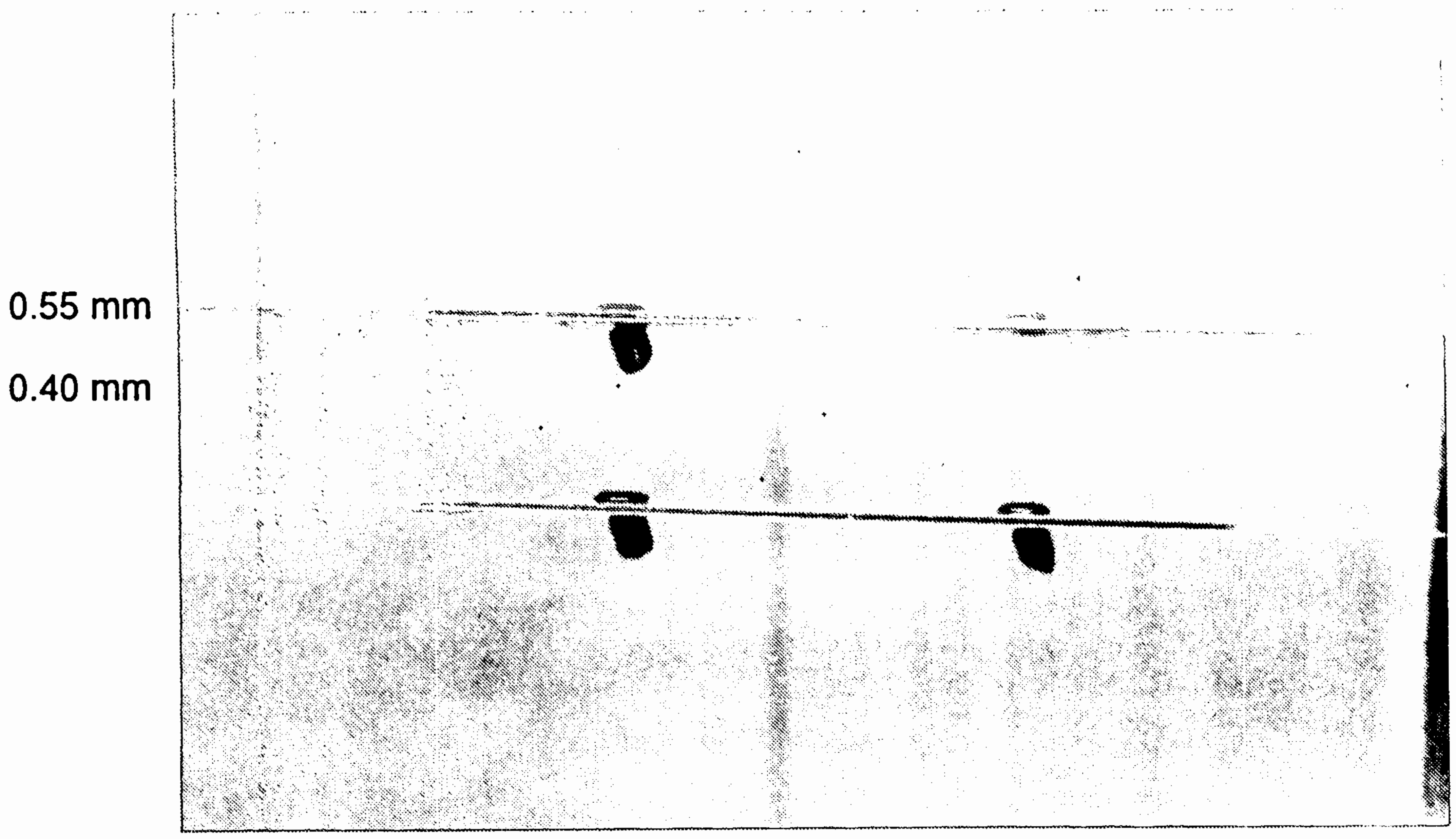


Figure H.5: 4.0 mm aluminium rivet connections.

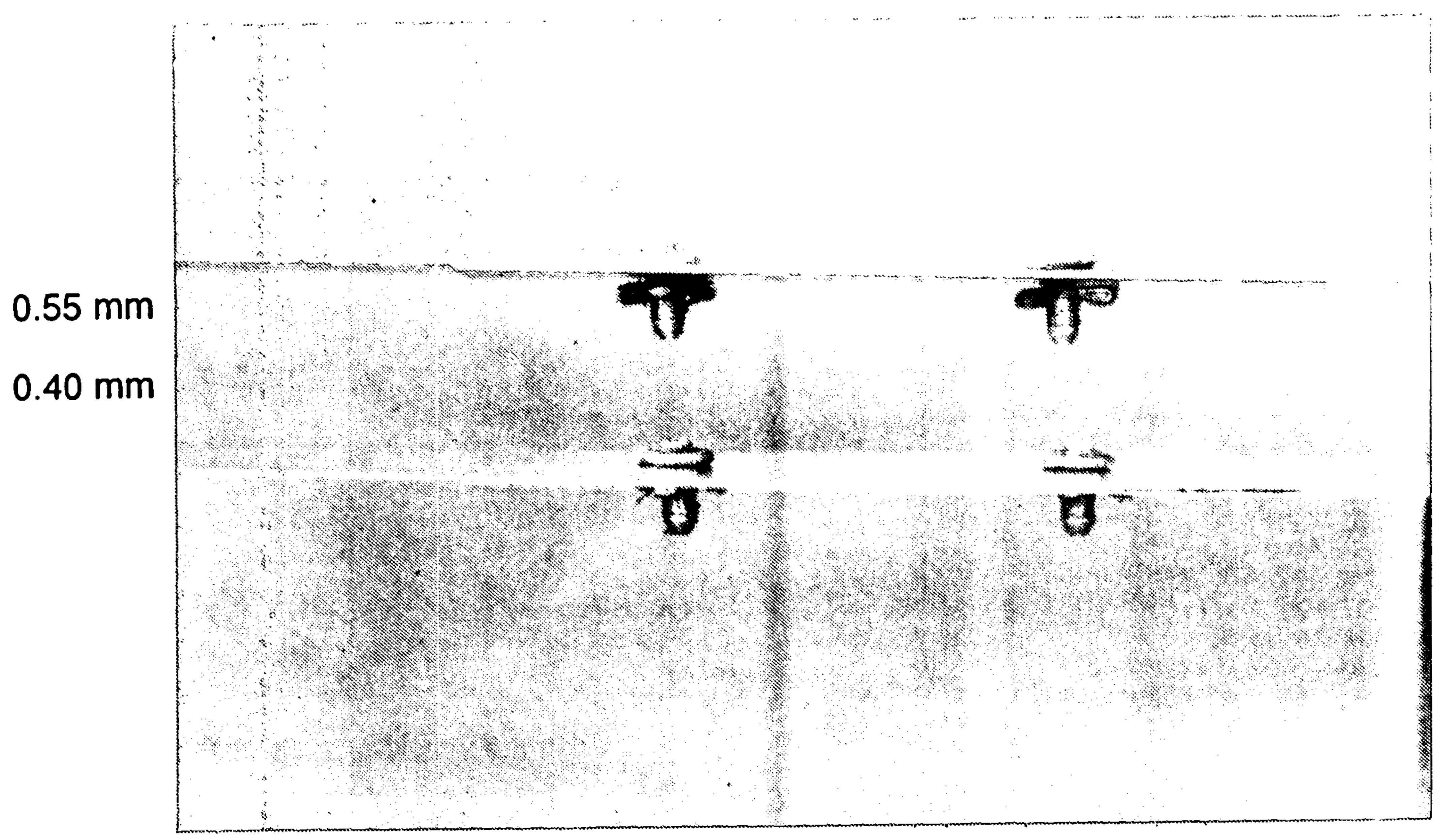
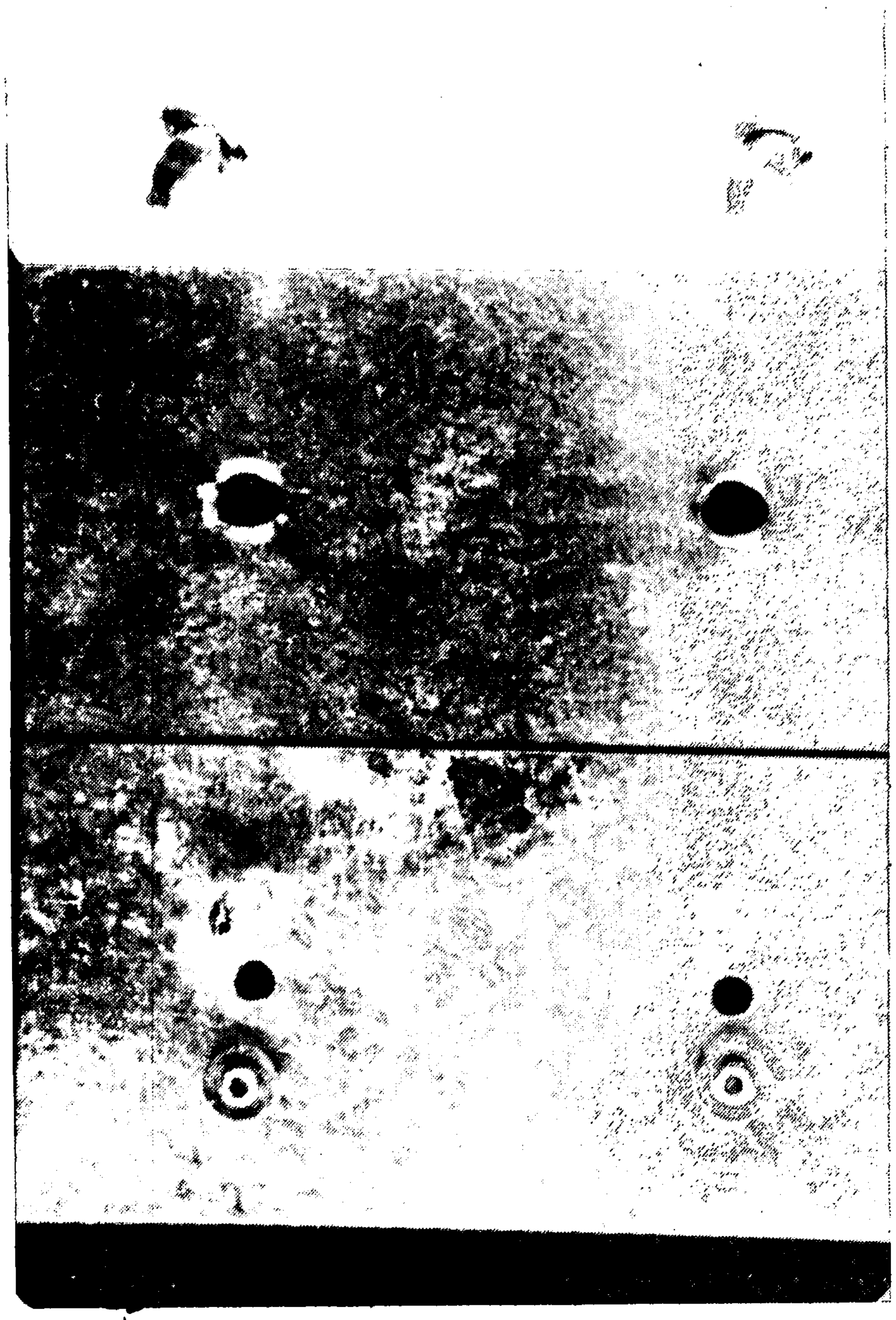


Figure H.6: "Bulb-tite" rivet connections.

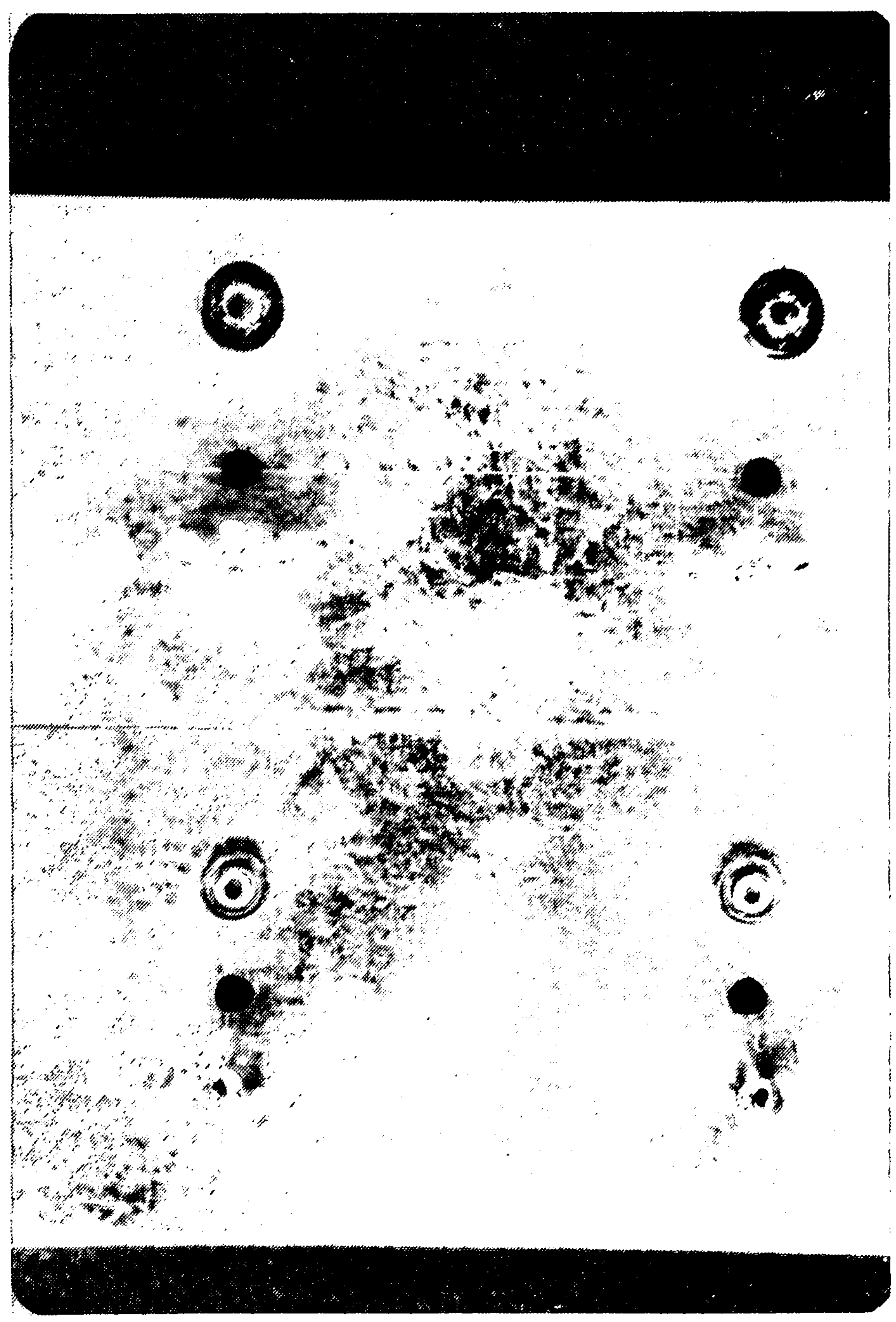


4.8 mm aluminium rivets

0.40 mm thick sheets

4.8 mm aluminium rivets

0.55 mm thick sheets



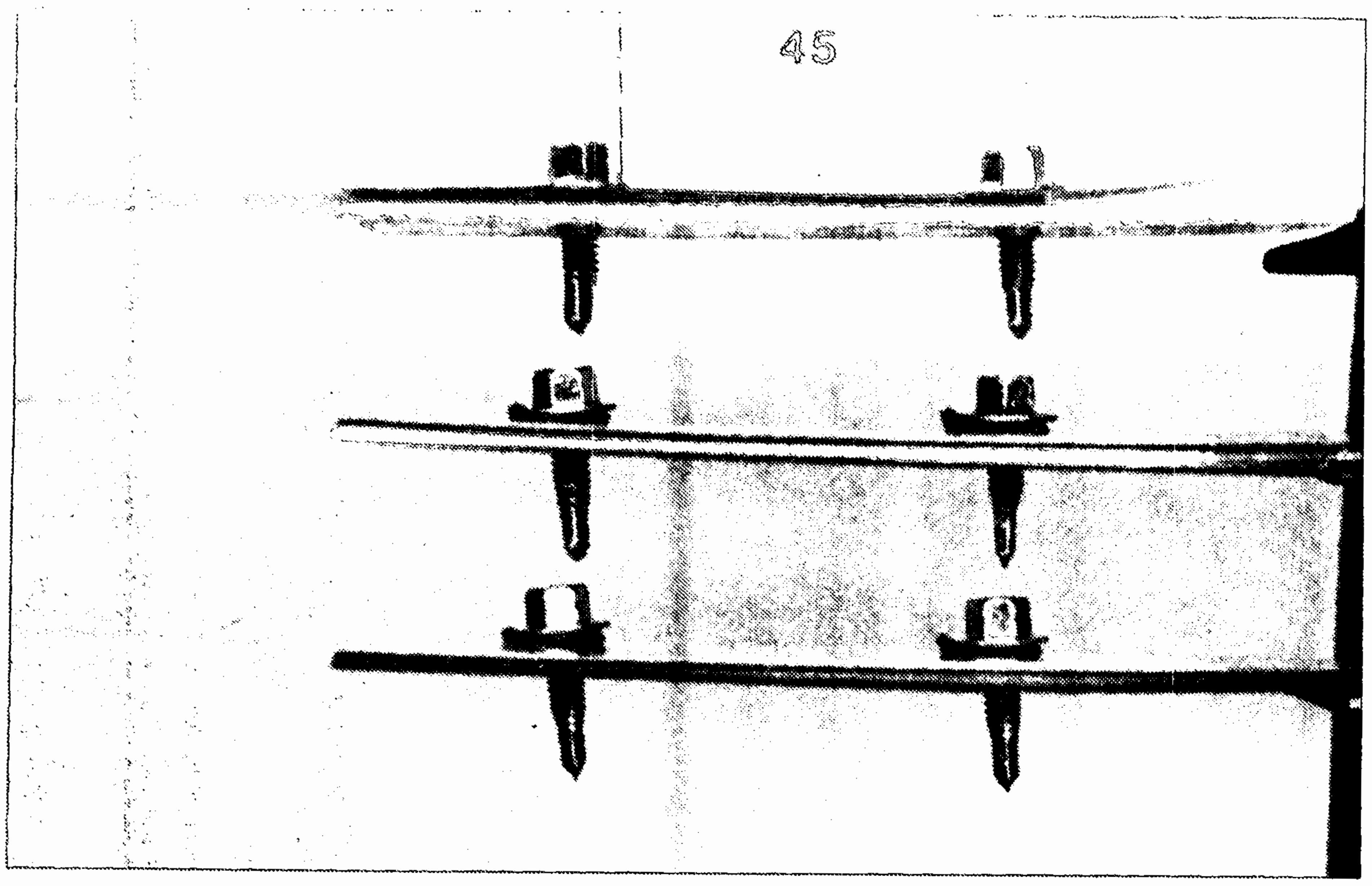
"Bulb-tite" rivets

0.40 mm thick sheets

4..0 mm aluminium rivets

0.40 mm thick sheets

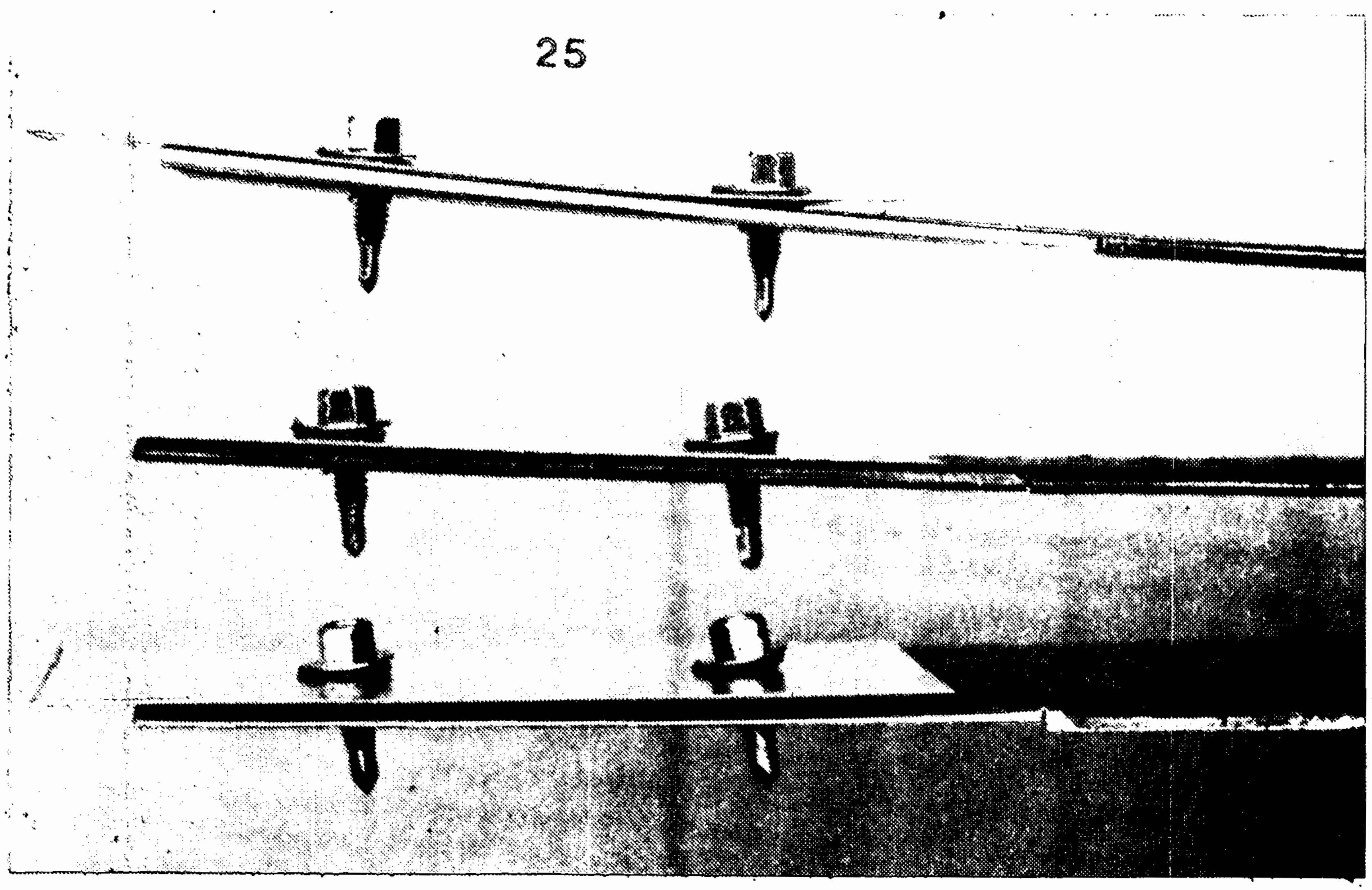
Figure H.7: Rivet holes after tests.



450/2.5

450/1.9

450/1.6

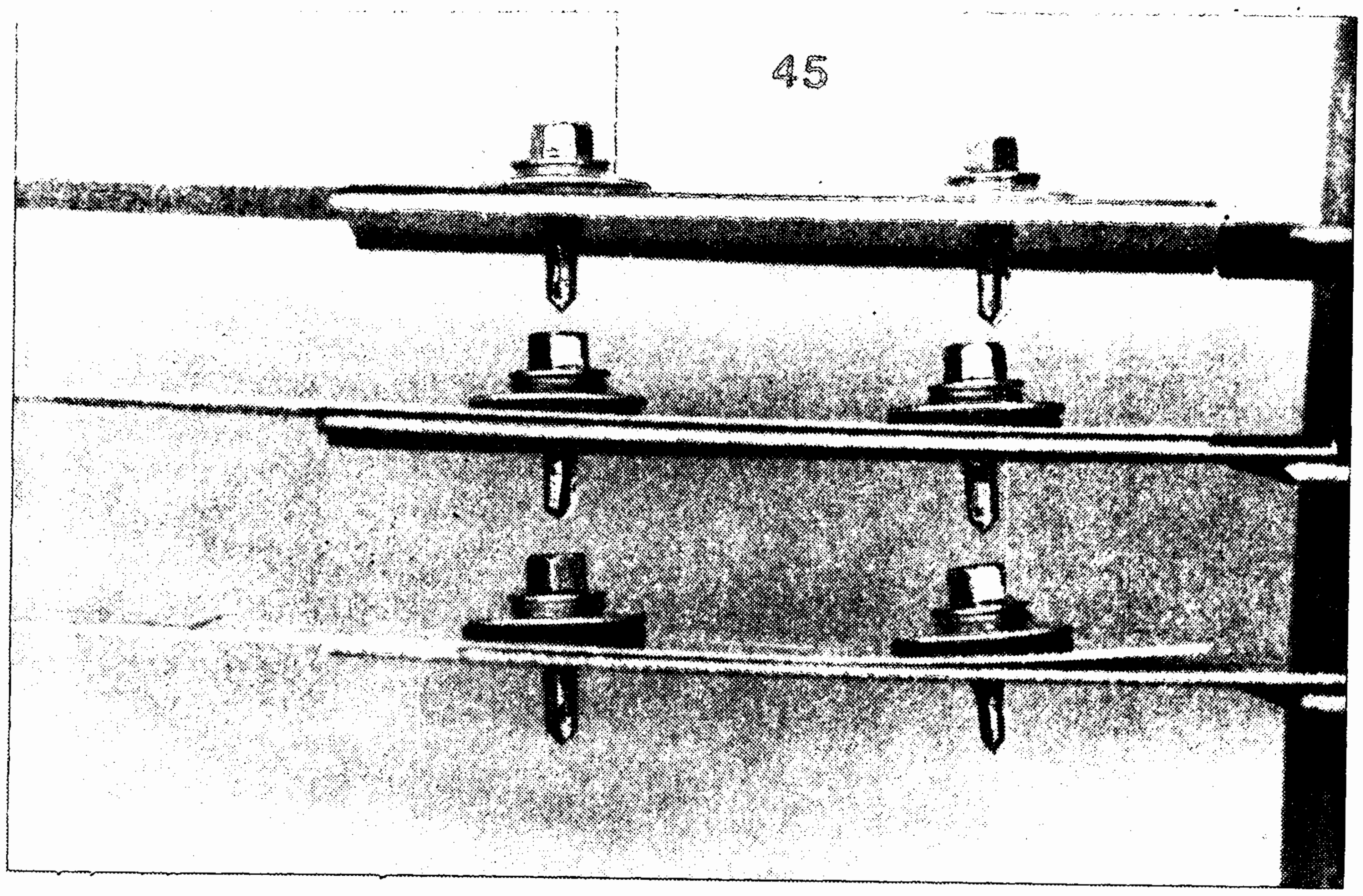


280/2.5

280/2.0

280/1.6

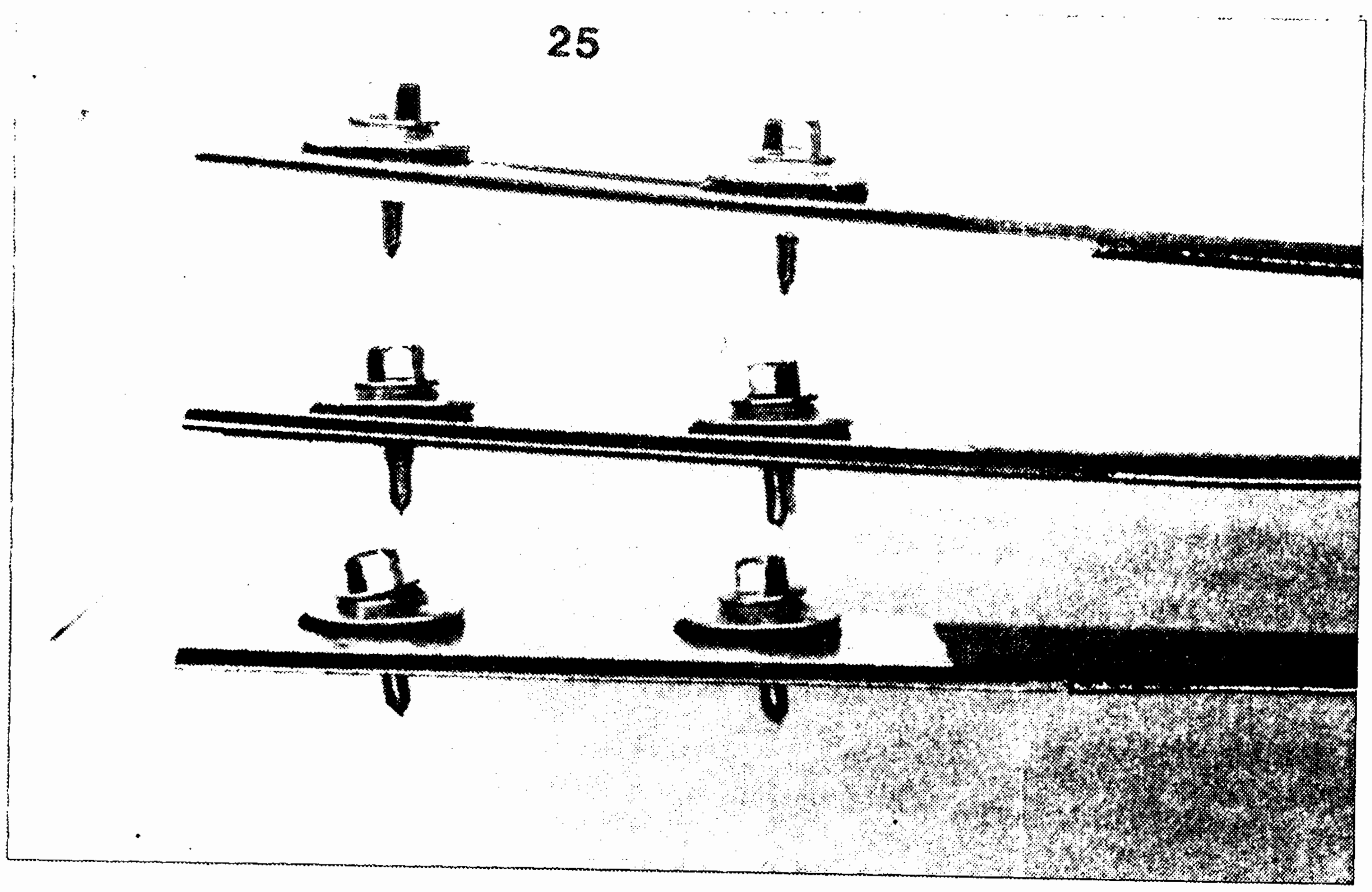
Figure H.8: Sheet (0.55 mm)-to-purlin connections (12g neo) after shear tests.



450/2.5

450/1.9

450/1.6

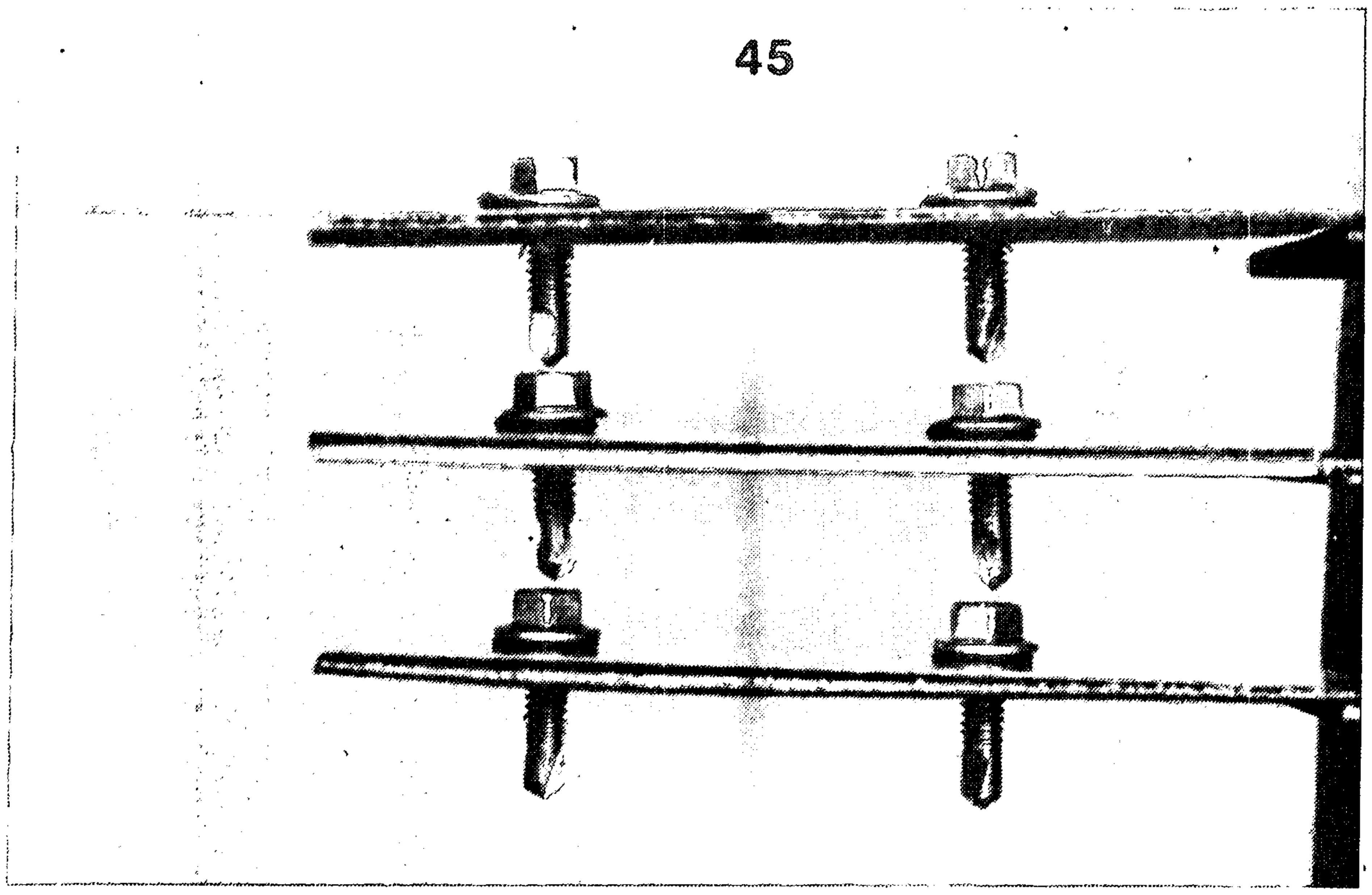


280/2.5

280/2.0

280/1.6

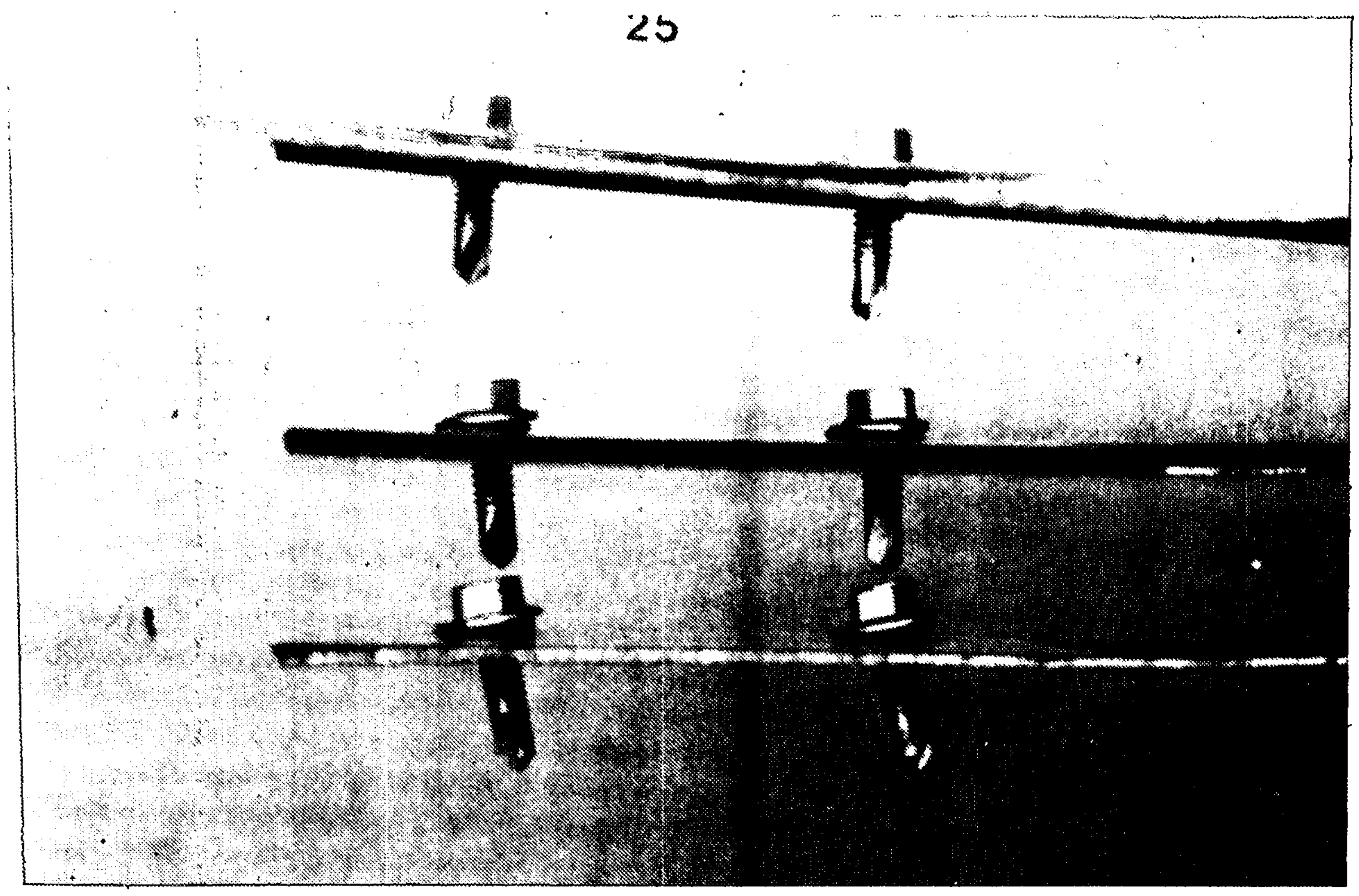
Figure H.9: Sheet (0.55 mm)-to-purlin connections (12g emb) after shear tests.



450/2.5

450/1.9

450/1.6

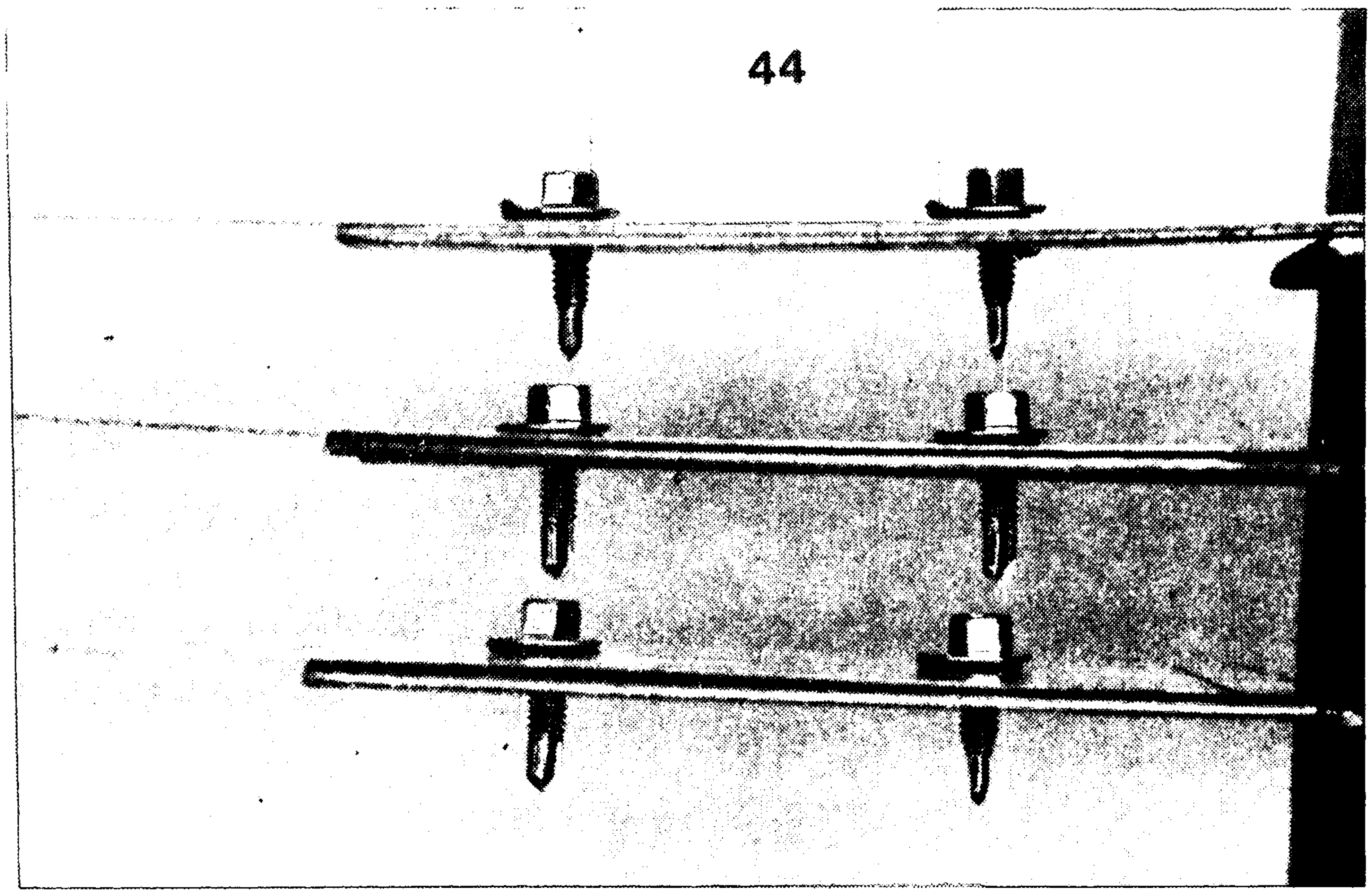


280/2.5

280/2.0

280/1.6

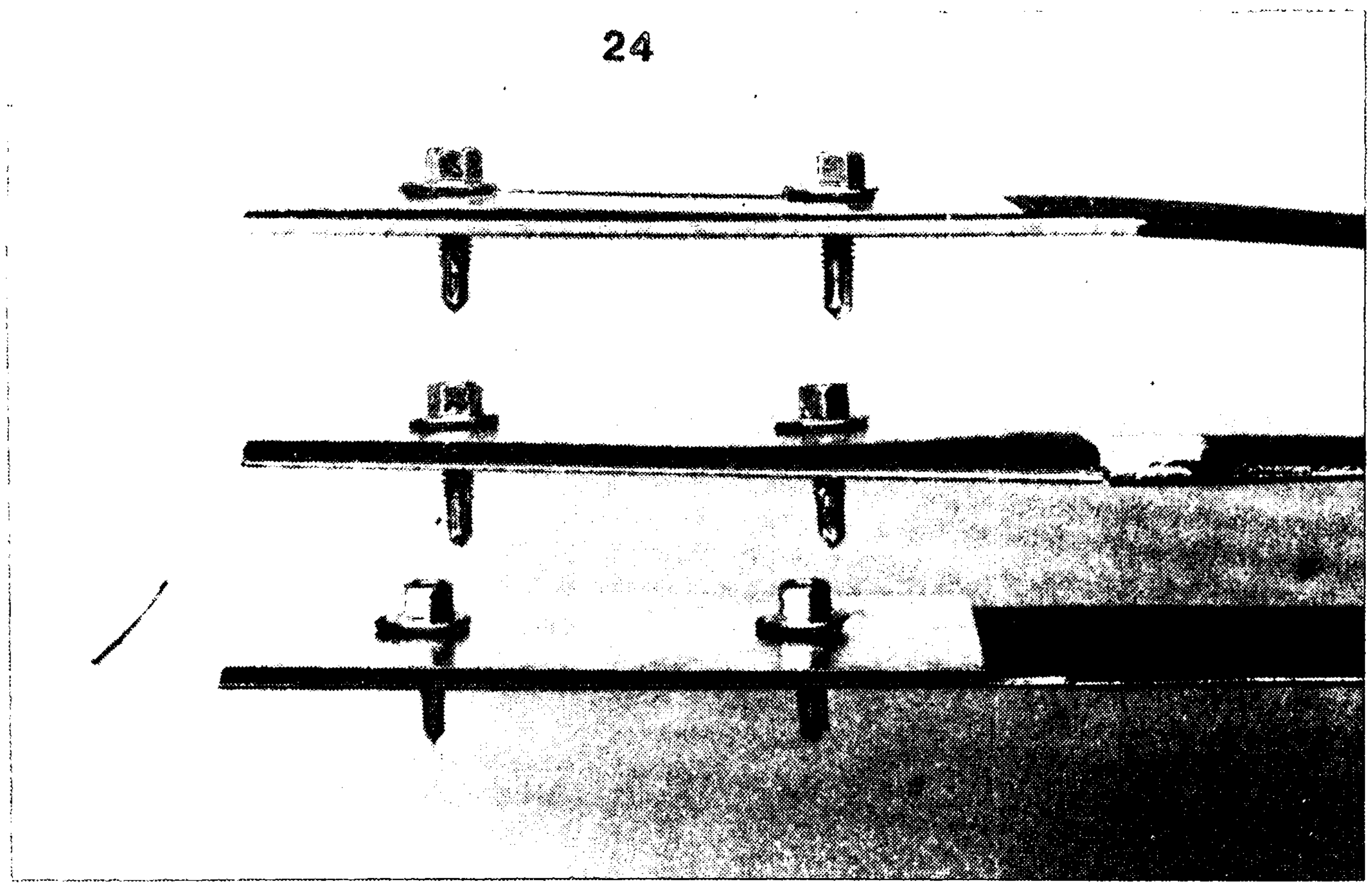
Figure H.10: Sheet (0.55 mm) -to-purlin connections (14g neo) after shear tests.



450/2.5

450/1.9

450/1.6

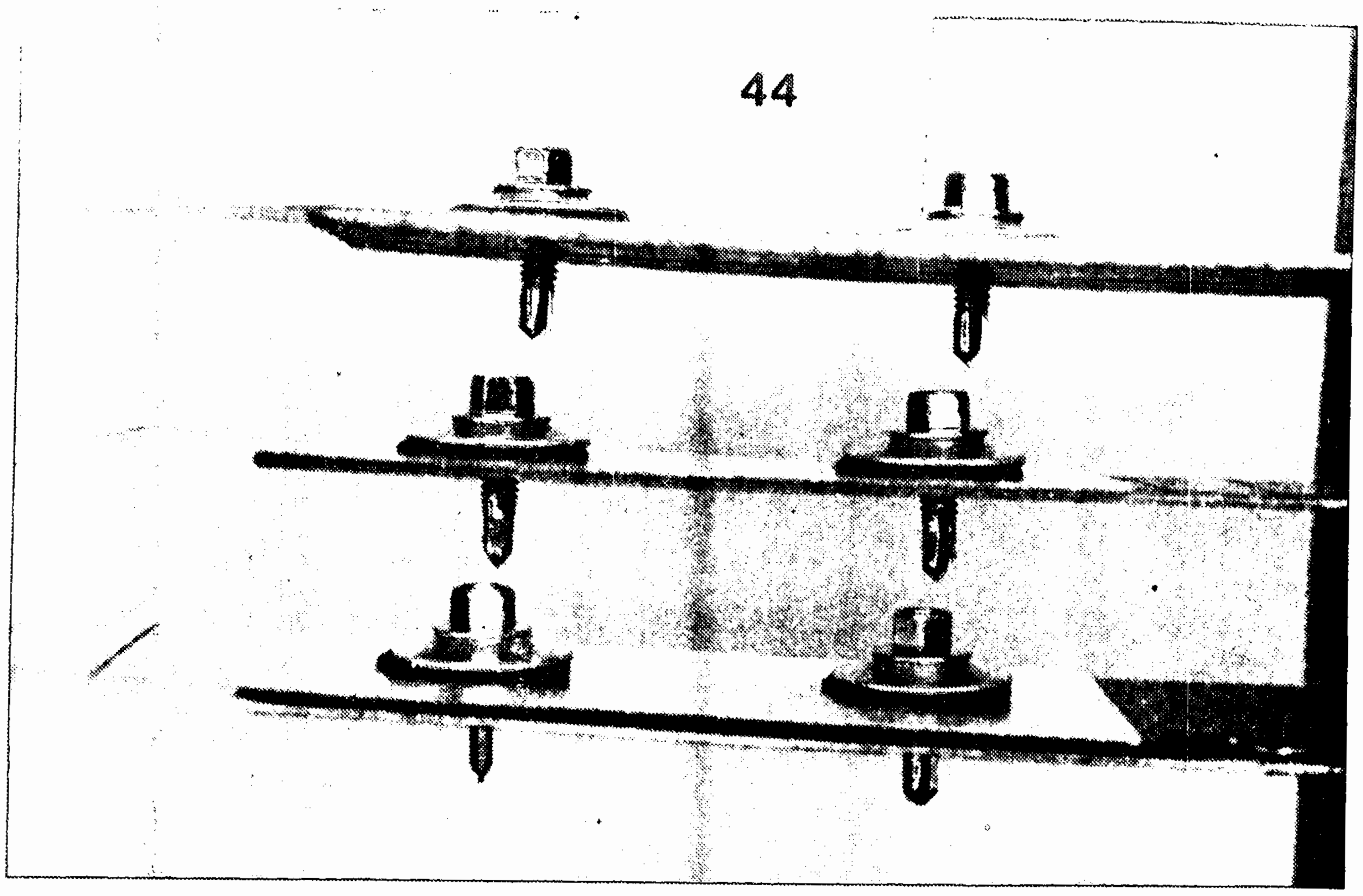


280/2.5

280/2.0

280/1.6

Figure H.11: Sheet (0.40 mm)-to-purlin connections (12g neo) after shear tests.

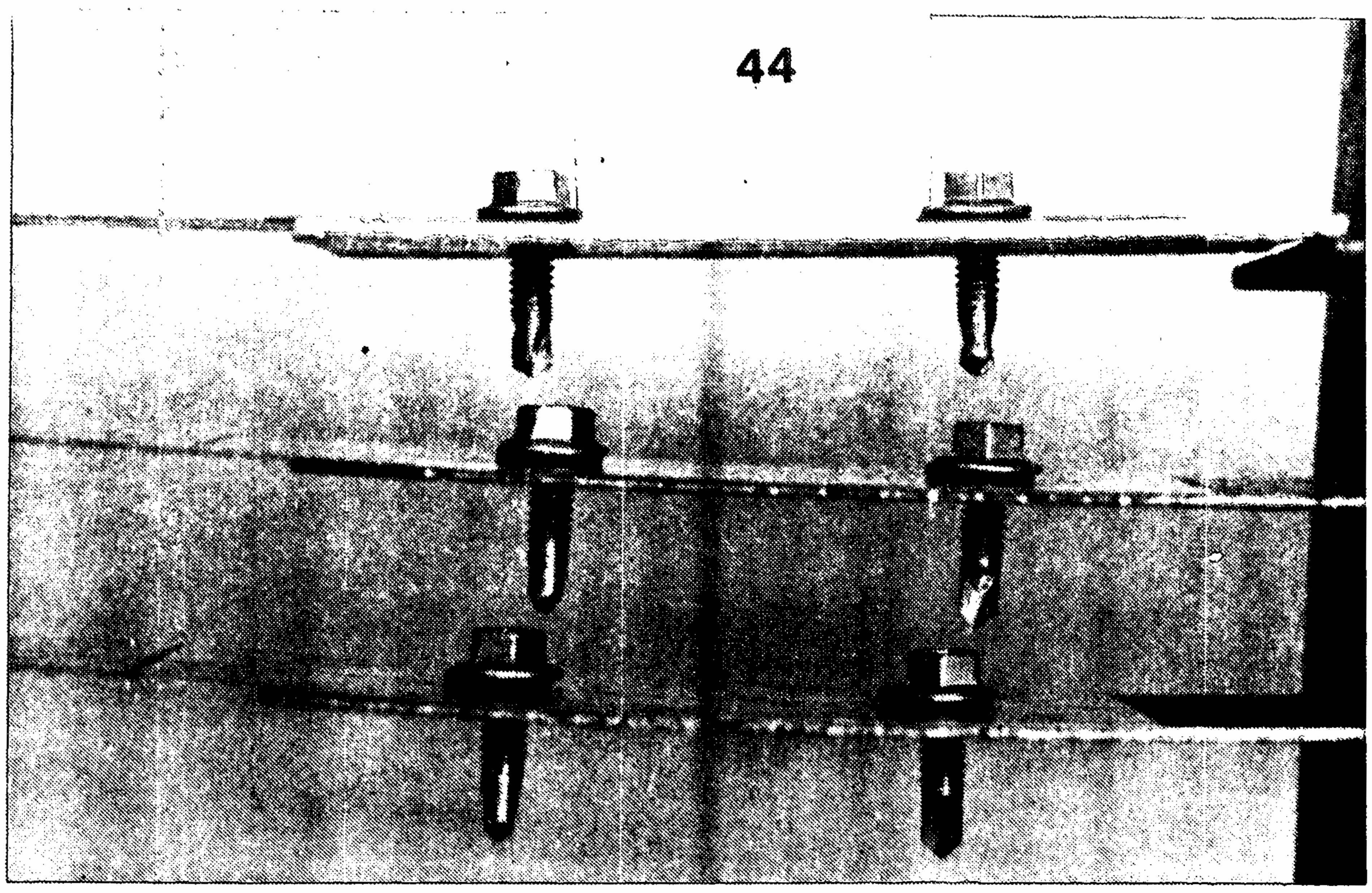


450/2.5

450/1.6

280/1.6

Figure H.12: Sheet (0.40 mm)-to-purlin connections (12g emb) after shear tests.



450/2.5

450/1.6

280/1.6

Figure H.13: Sheet (0.40 mm)-to-purlin connections (14g neo) after shear tests.

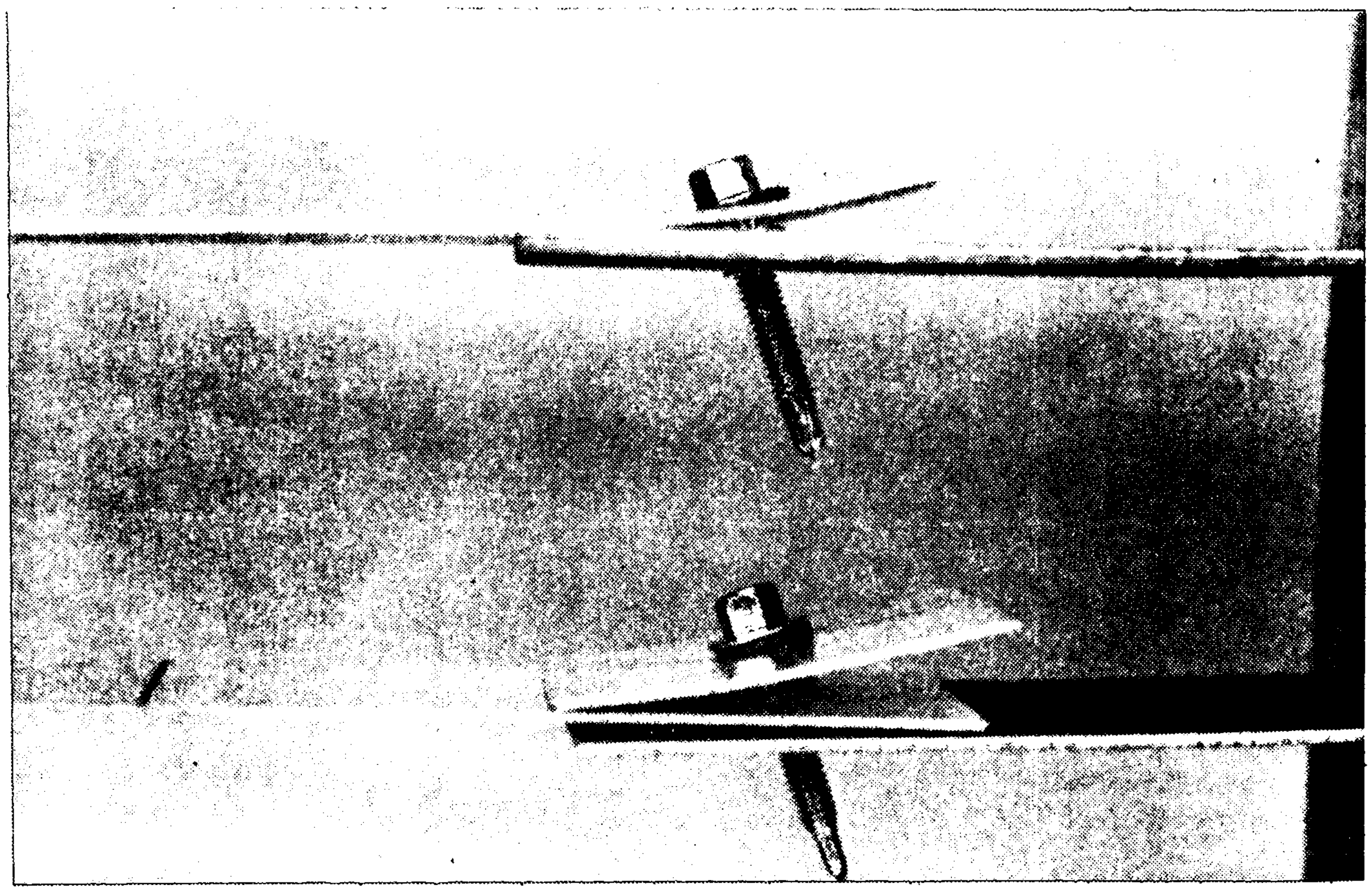


Figure H.14: Sheet (0.55 mm)-to-purlin connections (12g neo) after pull-out tests.

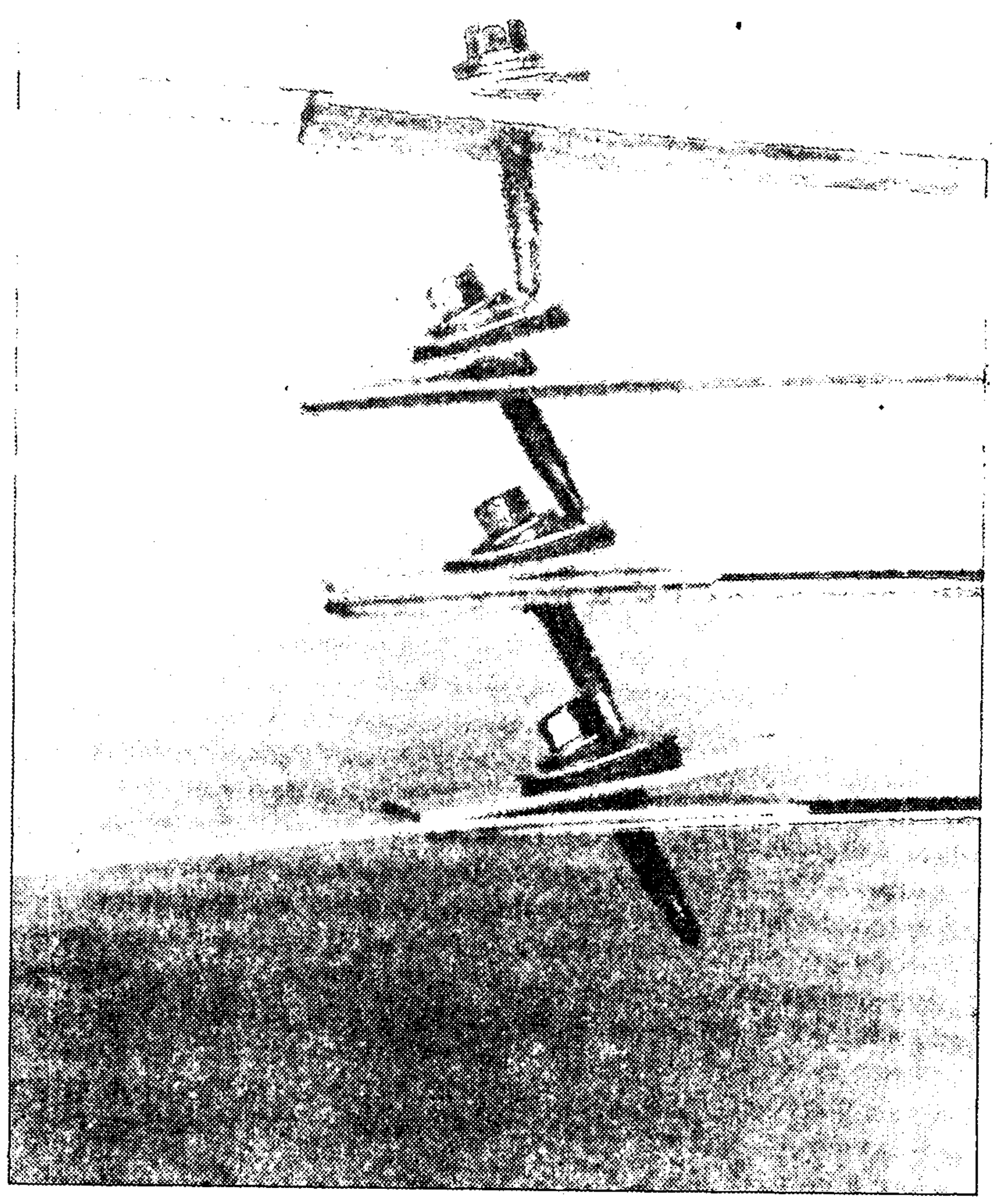


Figure H.15: Sheet (0.55 mm)-to-purlin connections (12g emb) after pull-out tests.

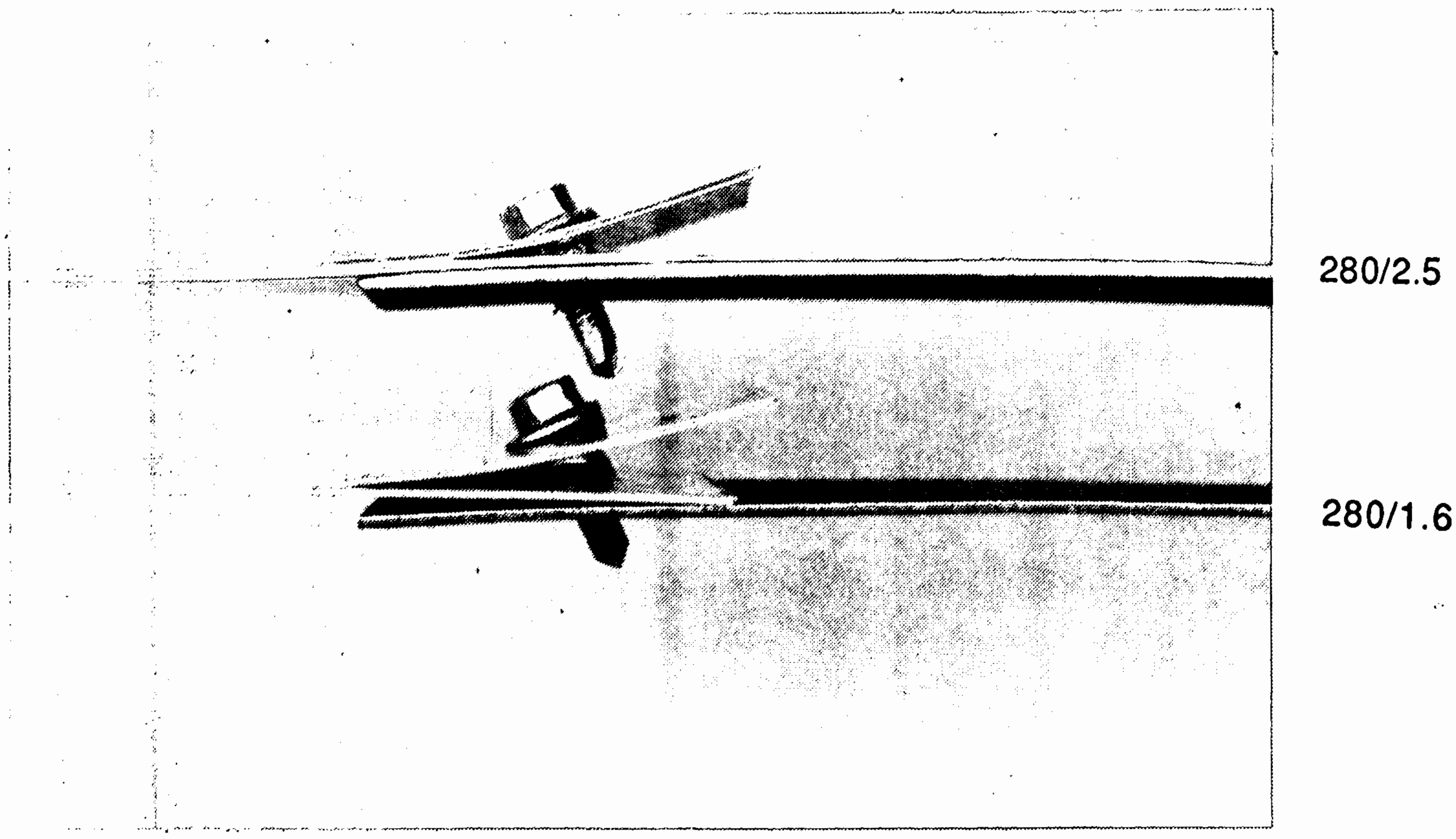


Figure H.16: Sheet (0.55 mm)-to-purlin connections (14g neo) after pull-out tests.

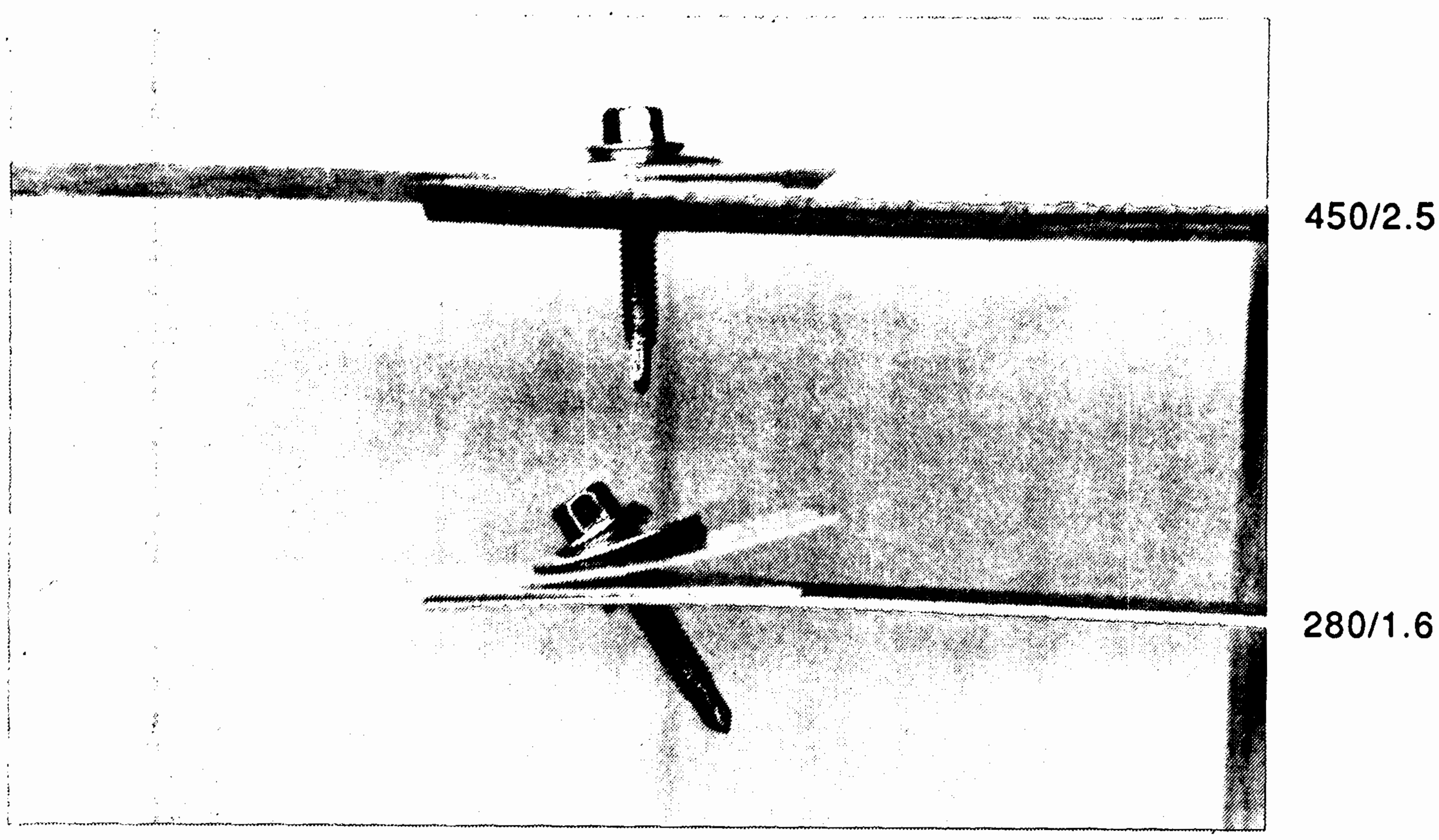


Figure H.17: Sheet (0.40 mm)-tò-purlin connections (12g emb) after pull-out tests.

APPENDIX I: STATISTICAL ANALYSIS

An analysis of variance was used to analyse the sheet-to-purlin connection shear and pull-out test results. Summaries of the analyses are presented in Tables I.1 and I.2.

Table I.1: Shear test results

Characteristics	Sheeting thickness (mm)	Fastener type	Mean	Standard deviation	Coefficient of variation (%)	p-value	Comments
Maximum load (kN)	0.55	12g neo	4.01	0.42	10.5	0.12	
		12g emb	4.13	0.26	6.2	0.32	
		14g neo	3.99	0.47	11.8	0.93	
	0.40	12g neo	2.51	0.23	9.3	0.006	thickness effect
		12g emb	2.64	0.21	8.1	0.29	
		14g neo	2.38	0.27	11.2	0.78	
3 mm load (kN)	0.55	12g neo	3.84	0.51	13.2	0.07	
		12g emb	3.91	0.42	10.7	0.22	
		14g neo	3.84	0.55	14.4	0.84	
	0.40	12g neo	2.31	0.29	12.6	0.14	
		12g emb	2.19	0.31	14.2	0.95	
		14g neo	2.24	0.30	13.3	0.62	

Table I.1 continued

Characteristics	Sheeting thickness (mm)	Fastener type	Mean	Standard deviation	Coefficient of variation (%)	p-value	Comments
Flexibility (mm/kN)	0.55	12g neo	0.27	0.03	12.0	0.003	effects of yield and thickness
		12g emb	0.30	0.07	22.3	0.71	
		14g neo	0.22	0.05	20.9	0.04	interaction of yield and thickness
	0.40	12g neo	0.37	0.09	24.1	0.70	
		12g emb	0.45	0.13	29.2	0.08	
		14g neo	0.28	0.10	36.1	0.81	

Table I.2: Pull-out test results

Characteristics	Sheeting thickness (mm)	Fastener type	Mean	Standard deviation	Coefficient of variation (%)	p-value	Comments
Maximum load (kN)	0.55	12g neo	6.96	0.66	9.6	0.05	thickness effect
		12g emb	7.88	0.37	4.6	0.0001	interaction of yield and thickness
		14g neo	7.60	0.30	4.0	0.0005	thickness effect
	0.40	12g emb	6.51	0.27	4.2	0.01	interaction of yield and thickness
3 mm load (kN)	0.55	12g neo	6.90	0.69	10.0	0.08	
		12g emb	7.68	0.61	8.0	0.003	thickness effect
		14g neo	7.35	0.17	2.4	0.0002	thickness effect
	0.40	12g emb	6.32	0.59	9.4	0.61	
Flexibility (mm/kN)	0.55	12g neo	0.12	0.03	25.3	0.28	
		12g emb	0.08	0.02	25.5	0.01	interaction of yield and thickness
		14g neo	0.12	0.03	21.8	0.007	thickness effect
	0.40	12g emb	0.13	0.04	31.7	0.12	

The results in Table I.1 are processed to obtain the characteristic strength of the connections, these are presented in Table I.3

Table I.3: Characteristic strength of sheet-to-purlin connections

Fastener type	Sheeting thickness (mm)	Purlin type [1]	Characteristic strength number	Characteristic strength (kN)	Characteristic strength per unit thickness (kN/mm)
12g neo	0.55	A	1	3.31 (0.42)	6.0
	0.40	B	2	2.19 (0.25)	5.5
		C	3	1.96 (0.21)	4.9
12g emb	0.55	A	4	3.70 (0.26)	6.7
	0.40	A	5	2.27 (0.21)	5.7
14g neo	0.55	A	6	3.20 (0.47)	5.8
	0.40	A	7	1.92 (0.27)	4.8

Notes: 1. Purlin type: A - 450/2.5, 450/1.9, 450/1.6, 280/2.5, 280/2.0, 280/1.6
B - 450/2.5, 450/1.9, 280/2.5, 280/2.0
C - 450/1.6, 280/1.6
2. Bracketed figures indicate standard deviation.

The characteristic strength of the connections are further pooled according to similar group and the results presented in Table I.4.

Table I.4: Sheet-to-purlin connection shear test pooled results

Fastener type	Sheeting thickness (mm)	Purlin type [1]	Characteristic strength (kN)	Characteristic strength per unit thickness (kN/mm)	Average flexibility (mm/kN) [2]	Characteristic strength from 3 mm load (kN)	Characteristic strength from 3 mm load per unit thickness (kN/mm)
12g neo	0.55	A	3.26 (0.45)	5.9	0.27 (0.04)	3.04 (0.50)	5.5
	0.40	B	2.23 (0.23)	5.6	0.38 (0.08)	1.77 (0.30)	4.4
		C	1.94 (0.24)	4.9	0.34 (0.09)	1.77 (0.30)	4.4
12g emb	0.55	A	3.70 (0.26)	6.7	0.30 (0.07)	3.04 (0.50)	5.5
	0.40	A	2.23 (0.23)	5.6	0.46 (0.15)	1.77 (0.30)	4.4
14g neo	0.55	A	3.26 (0.45)	5.9	0.22 (0.05)	3.04 (0.50)	5.5
	0.40	A	1.94 (0.24)	4.9	0.28 (0.10)	1.77 (0.30)	4.4

Notes: 1. Purlin type: A - 450/2.5, 450/1.9, 450/1.6, 280/2.5, 280/2.0, 280/1.6
B - 450/2.5, 450/1.9, 280/2.5, 280/2.0
C - 450/1.6, 280/1.6.
2. Based on the characteristic strength of the group.
3. Bracketed figures indicate standard deviation.

APPENDIX J: RESULTS OF DESIGN EXPRESSIONS

Table J.1: Strength of rivet connections

Reference	Rivet diameter 4.0 mm, Sheeting thickness 0.40 mm	Rivet diameter 4.0 mm, Sheeting thickness 0.55 mm	Rivet diameter 4.8 mm, Sheeting thickness 0.40 mm	Rivet diameter 4.8 mm, Sheeting thickness 0.55 mm
Design expressions:				
Failure due to tilting and bearing#:				
Baehre and Berggren (1973)	1.7	2.3	1.8	2.5
Bryan (1973)	---	---	1.0	1.4
Stark and Toma (1978)*	1.7 (1.2)	2.3 (1.7)	1.8 (1.3)	2.5 (1.8)
SISC (1982)*	1.1 (0.8)	1.7 (1.3)	1.2 (0.9)	1.9 (1.4)
ECCS (1984b)~	1.4 (1.0)	2.2 (1.6)	1.5 (1.0)	2.4 (1.7)
BS5950:Part5 (1987)~	1.2 (1.0)	2.0 (1.7)	1.3 (1.1)	2.2 (1.8)
Failure by crushing or shearing of fastener				
Baehre and Berggren (1973), BS5950:Part5 (1987)^:				
Monel rivets	2.0 (2.5)	2.0 (2.5)	3.2 (4.0)	3.2 (4.0)
Aluminium rivets	1.1 (1.3)	1.1 (1.3)	1.8 (2.3)	1.8 (2.3)
"Bulb-tite" rivets	---	---	2.6 (3.3)	2.6 (3.3)

Table J.1 continued

Reference	Rivet diameter 4.0 mm, Sheeting thickness 0.40 mm	Rivet diameter 4.0 mm, Sheeting thickness 0.55 mm	Rivet diameter 4.8 mm, Sheeting thickness 0.40 mm	Rivet diameter 4.8 mm, Sheeting thickness 0.55 mm
Characteristic strength from tests:				
Monel rivets	1.43	2.04	1.81	2.67
Aluminium rivets	1.11	1.28	1.68	2.18
"Bulb-tite" rivets	___	___	1.36	1.61

Notes:
1. All strength in kN.
2. # -- All computation based on the characteristic tensile strength of the sheetings as determined from tests.
3. * -- Bracketed figures indicate result using the nominal value of yield or tensile strength.
4. ~ -- Bracketed figures indicate result using 0.7 or 0.84 of the characteristic tensile strength.
5. ^ -- Bracketed figures are the shear strength of the fastener as quoted from manufacturers' tests or recommendations; unbracketed figures are 0.8 times the bracketed figures.

Table J.2: Flexibility of rivet connections

Reference	Rivet diameter 4.0 mm Sheeting thickness 0.40 mm	4.0 mm 0.55 mm	4.8 mm Sheeting thickness 0.40 mm	4.8 mm 0.55 mm
Design expressions:				
SISC (1982)	0.40	0.34	0.33	0.28
Test results: Monel rivets	0.29	0.20	0.20	0.18
Aluminium rivets	0.22	0.11	0.16	0.11
"Bulb-tite" rivets	___	___	0.42	0.42

Note: All flexibility in mm/kN.

Table J.3: Strength of screw connections

Reference	Screw diameter 4.8 mm Sheeting thickness 0.40 mm	4.8 mm 0.55 mm	5.5 mm Sheeting thickness 0.40 mm	5.5 mm 0.55 mm	6.3 mm Sheeting thickness 0.40 mm	6.3 mm 0.55 mm
Design expressions:						
Failure in tilting, yielding and tearing:						
Baehre and Berggren (1973)	1.5	2.0	3.1	4.3	3.2	4.5
Bryan (1973)	---	---	---	---	2.4	3.3
Stark and Toma (1978)*	1.5 (1.1)	2.0 (1.5)	3.4 (2.5)	4.8 (3.5)	3.9 (2.9)	5.5 (4.0)
Strnad (1979)*	2.3 (1.7)	3.2 (2.4)	4.7-6.6 (3.5-4.0)	5.7-9.3 (4.8-6.0)	5.0-7.1 (3.7-4.3)	6.2-10.0 (5.1-6.5)
SISC (1982)*	1.2 (0.9)	1.9 (1.4)	2.6 (1.9)	3.6 (2.7)	3.0 (2.2)	4.2 (3.1)
ECCS (1984b)~	1.3 (0.9)	2.2 (1.5)	3.4 (2.4)	4.8 (3.3)	3.9 (2.8)	5.5 (3.8)
BS5950:Part5 (1987)~	1.3 (1.1)	2.2 (1.8)	3.4 (2.9)	4.8 (4.0)	3.9 (3.3)	5.5 (4.6)
Failure by shearing of fastener:						
Baehre and Berggren (1973), BS5950:Part5 (1987)^:	5.0 (6.2)	5.0 (6.2)	6.9 (8.6)	6.9 (8.6)	9.0 (11.3)	9.0 (11.3)
Characteristic strength from tests:	1.50	2.87	1.94-2.23	3.26-3.70	1.94	3.26

Notes: 1. All strength in kN.
2. Other notation same as in Table J.1.

Table J.4: Flexibility of screw connections:

Reference	Screw diameter					
	4.8 mm Sheeting thickness		5.5 mm Sheeting thickness		6.3 mm Sheeting thickness	
	0.40 mm	0.55 mm	0.40 mm	0.55 mm	0.40 mm	0.55 mm
Design expressions:						
Strnad (1979)	0.19 (0.25)	0.13 (0.18)	0.15-0.31 (0.33-0.41)	0.10-0.22 (0.16-0.30)	0.13-0.27 (0.29-0.36)	0.09-0.19 (0.14-0.26)
SISC (1982)	0.33	0.28	0.19	0.16	0.17	0.14
Test results:	0.26	0.14	0.34-0.46	0.27-0.30	0.28	0.22

Notes: 1. All flexibility in mm/kN.
2. Notation same as in Table J.1.

APPENDIX K: COMPARISON OF U.K. AND NEW ZEALAND CONNECTION DATA

Table K.1: Seam connection data

Fastener type	Thickness of lapped sheets (mm)		Characteristic strength (kN)		Characteristic strength per unit thickness (kN/mm)		Failure mode [1]		Tensile strength of sheets (MPa) [2]		Average flexibility (mm/kN)	
	U.K.	N.Z.	U.K.	N.Z.	U.K.	N.Z.	U.K.	N.Z.	U.K.	N.Z.	U.K.	N.Z.
Self-drilling self-tapping screws with neoprene washers [3] diameter=4.8 mm (10g)	0.75		1.8		2.4		2		302		0.36	
		0.55		2.9		5.2		4		743		0.14
		0.40		1.5		3.8		4		752		0.26
4.8 mm monel rivets	1.01		3.1		3.1		3		373		0.17	
	0.75		2.1		2.8		3		302		0.28	
	0.51		1.4		2.7		3		292		0.35	
		0.55		2.7		4.9		5		752		0.18
		0.40		1.8		4.5		5		743		0.20
4.8 mm aluminium rivets	1.01		3.0		2.9		1		373		0.15	
	0.75		2.4		3.1		3		302		0.10	
		0.55		2.2		4.0		6		752		0.11
		0.40		1.7		4.2		5		743		0.16
"Bulb-tite" rivets	0.77		1.9		2.5		3		302		0.50	
		0.55		1.6		2.9		6		752		0.42
		0.40		1.4		3.4		6		743		0.42

Notes:
1. Failure modes: 1 = shear of fastener itself; 2 = inclination of screw followed by stripping of thread in formed sheet; 3 = crushing in combination with tilting and bearing; 4 = inclination of screw followed by tearing of the sheets and eventually pull-out; 5 = rivet inclination, local yielding or tearing of sheets followed by pull-out from the lower sheet; 6 = failure in bending or tilting in combination with shear of the fastener.
2. Mean tensile strengths for U.K. sheets and characteristic values for New Zealand sheets.
3. The New Zealand fastener includes a steel washer.
4. The U.K. sheets have an elongation of 21 to 34% and yield stress of 202 to 273 MPa.

Table K.2: Sheet-to-purlin connection data

Fastener type	Thickness of lapped straps (mm)		Characteristic strength [2] (kN)		Characteristic strength per unit thickness [2] (kN/mm)		Failure mode [1]		Tensile strength of sheets (MPa) [3]		Average flexibility (mm/kN)	
	U.K.	N.Z.	U.K.	N.Z.	U.K.	N.Z.	U.K.	N.Z.	U.K.	N.Z.	U.K.	N.Z.
Self-drilling self-tapping screws with neoprene washers diameter=6.3 mm (14g)	0.76-2.07		3.8		5.0		1		302		0.41	
		0.55-1.6, 1.9, 2.5		3.3 (3.0)		5.9 (5.5)		1		752		0.22
		0.40-1.6, 1.9, 2.5		1.9 (1.8)		4.9 (4.4)		1		743		0.28
diameter=5.5 mm (12g)	0.76-2.06		3.0		4.1		1		302		0.45	
		0.55-1.6, 1.9, 2.5		3.3 (3.0)		5.9 (5.5)		1		752		0.27
		0.40-1.9, 2.5		2.2 (1.8)		5.6 (4.4)		1		743		0.38
		0.40-1.6		1.9 (1.8)		4.9 (4.4)		1		743		0.34

Notes:
1. Failure mode: 1 = bearing and tearing in the thinner sheet, possibly accompanied by tilting of the fastener.
2. Bracketed figures indicate value from 3 mm loads.
3. Mean tensile strengths for U.K. sheets and characteristic values for New Zealand sheets.
4. The U.K. sheets have an elongation of 21 to 34% and yield stress of 202 to 273 MPa.

APPENDIX L: EDGE DISTANCE, END DISTANCE AND SPACING OF MECHANICAL CONNECTIONS

Recommendations

Overseas recommendations for edge and end distances of mechanical connections for profiled sheet steel claddings are summarised in Table L.1.

Table L.1: Overseas recommendations for edge and end distances

Reference	Edge distance e	End distance e'	Notes
Canadian Standards Association (1963) and AS 1538 (1974)	e<1.5d	--	for rivets
Baehre & Berggren (1973)	1.5d<e<3d	3d<e'<6d	for screws: 3.0 mm<d<6.4 mm and rivets: 2.5 mm<d<6.5 mm
ECCS (1978) and Davies & Bryan (1982)	e<1.5d (min. 8 mm)	e'<3d (min. 20 mm)	for seam and sheet-to-shear connector fasteners
	e<1.5d (min. 10 mm)	e'<3d (min. 20 mm)	for other fasteners
John Lysaght Ltd (1980b)	e<2d	--	to ensure connection bearing strength and to avoid buckling of the steel while clinching
SISC (1982) [3] and ECCS (1984b)	e<1.5d	e'<3d	for screws: 3.0 mm<d<8.0 mm and blind rivets 2.6 mm<d<6.4 mm
BS 5950: Part 5 (1987)	e<1.5d or 10 mm whichever is the smaller for applied force in one direction only, otherwise e<3d	e'<3d	for self-drilling screws: 3.0 mm<d<8.0 mm, and blind rivets: 2.5 mm<d<7.5 mm

Notes:
1. d is the diameter of the screw or rivet.
2. Edge distance is measured in a direction at right angles to the applied force while end distance is measured in the direction of the applied force.
3. For combined loadings, i.e., shear and tension, the minimum edge and end distances depend on the fastener location as well as the profile dimensions; refer to Table 33:142b of SISC (1982).

For the spacings, the minimum pitch recommended is three times the fastener diameter (BS 5950:Part 5, 1987; ECCS, 1984b; SISC, 1982). For rivet connections, smaller pitch can cause stress concentration in the riveted material and buckling at adjacent empty holes (John Lysaght Ltd, 1980b). The maximum spacings recommended are summarised in Table L.2.

Table L.2: **Maximum spacing of mechanical connections**

Reference	Maximum spacing p	Notes
CONSTRADO (1980)	p<450 mm for seam connections on roof p<500 mm for seam connections on wall	
Bryan and Davies (1981), Davies and Bryan (1982)	for seam connections: p<300 mm for diaphragms fastened on all four sides p<450 mm for diaphragms fastened on two sides only sheet-to-purlin connections shall be at every trough or alternative troughs; if shear buckling is critical, 'corrugated iron' should be fastened to the supporting structure at not more than 150 mm centres	
SISC (1982)	p<500 mm for longitudinal joints in side laps or for sheet-to-rafter connections along the profile of the sheeting; risk of local shear buckling also has to be considered	for both rivet and screw, other connections refer clause 32:726
Josey (1986)	p<450 mm for primary fixings p<500 mm for secondary fixings	
Thomson (1987) [1]	p<900 mm for monel rivet seam connections p<750 mm for monel rivet with washer in seam connections p<600 mm for "Bulb-tite" rivet seam connections primary connections in every pan	

Note: 1. These are recommended for "Steelspan 900".

Previous Work

Relatively little has been published to confirm the above recommendations.

Some tests has been carried out by Berry (1976) using "Teks" 12-24 screws with "Twinseal" washers in sheet-to-purlin (0.71 or 0.75 mm to 3.25 mm) connections adopting both the standard shear and the simulated diaphragm action arrangements. It was found that no edge failure occurred at an edge distance of 10 mm or more; in general the connection strength decreased with decreasing edge distance. Berry concluded that an edge distance of 10 mm or 2d, whilst giving slightly reduced strength, would be satisfactory.

Using an analytical approach, Soreide et al (1979) investigated the effects of transverse wind load on wall panels employing a finite element program which included material and geometrical non-linearities. Although the work lacks practical recommendations, it has been observed that transverse wind forces, membrane forces activated and fatigue all contribute to the rupture of the shèeting around the connection; the damage is particularly severe for connections with small edge distances.

Practical Considerations

Besides the strength consideration, the edge and end distances can be dictated by practical considerations. The appropriate distances should enable the fastener to be installed correctly without causing distress to the sheeting, and the fastener head or washer be prevented from protruding beyond the edge of the sheet.

For screws, Berry (1976) pointed out that driving tools normally require a minimum flat distance from the edge of the sheet of 20 to 25 mm for satisfactory trough fastening. Thus the minimum edge distance must necessarily be 10 to 12 mm.

In New Zealand, self-drilling self-tapping screws have washer head diameters of about 14 to 15 mm. Allowing for practical difficulties in driving screws to their intended positions on site and erection mishaps, as well as avoiding damage to the sealing washer by the edge of the sheet, a clearance of at least 2 mm is appropriate. Hence a minimum edge distance of 10 mm for screws is necessary.

On the other hand, due considerations should also be paid to the maximum edge distance and spacing. The suitable dimensions should ensure proper contact between the sheets. In this respect, large edge distance may give rise to deformations at the edge; and enhance the risk of corrosion in the gap between the sheets.

It is also noted that less weather penetration occurs due to capillary action than bad joints and sheets should be fitted as tight as possible at the laps. Useful means of improving watertightness include increasing the amount and sealing of the end and side laps, increases the number of seam connections as well as to eliminate or reduce the number of end laps by using long sheets.

New Zealand Situation

In order to satisfy overseas requirements for common fasteners used in New Zealand (refer to Appendix A), the following edge and end distances have to be provided.

Table L.3 Edge and end distances for common fasteners

Rivet or screw diameter d (mm)	Edge distance (mm)			End distance (mm)	
	1.5d	2d	minimum	3d	minimum
4.0	6.0	8.0	10.0	12.0	20.0
4.8	7.2	9.6	10.0	14.4	20.0
5.5	8.3	11.0	10.0	16.5	20.0
6.3	9.5	12.6	10.0	18.9	20.0

Notes: 1. Since overseas recommendations are similar, only the common criteria are adopted in this table.
2. The criteria chosen is appropriate for one principal loading direction.

With reference to Table L.3, the majority of the cases considered are controlled by the minimum dimensions specified; and there is neither economical nor practical reason to justify their reduction.

Some local cladding manufacturers are already recommending that all fixings should not be located less than 25 mm from sheet ends which well covers all overseas end distance requirements.

As regard spacings, the limits specified by Davies and Bryan (1982) are the most stringent and have proved to be satisfactory. Thus they should be adopted for New Zealand use.

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