Analysis and design of bolted connections in cold formed steel members

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To my parents who have devoted their lives to

the education of their children.

Acknowledgements

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Synopsis

The purpose of this thesis has been to investigate, both analytically and experimentally, the behaviour of bolted joints in cold formed steel members.

To this aim, the influences of all factors bearing a significant effect on behaviour of bolted connections in cold formed steel assemblies and structures have been investigated. Design expressions have been derived. With this information the moment capacity of bolted joints can be calculated with considerably more accuracy than the present existing expressions in the current codes of practice.

The findings of the thesis have already influenced the design expressions for bearing strength of bolted connections in the European code of practice for design of cold formed steel sections, i.e. Eurocode No 3, Annex A. It is hoped that the conclusions drawn are further incorporated in the above mentioned code, and replace the existing guidelines in the British Standard, BS 5950 Part 5, for the design of such sections.

Furthermore for the first time ever, as a result of the analyses and experiments carried out, designers are able to predict the moment-rotation relationship of such connections without having to resort to testing. It is intended to feed this information into design programmes to radically affect the elastic design of cold formed steel structures. It should then be possible to design optimum structures by specifying joints with appropriate characteristics.

Chapter One

Background and introduction to cold formed steel sections and sheeting

1. Background and introduction to cold formed steel sections and sheeting

Summary

This chapter is essentially in two parts. First, a general introduction to the subject is given. The history of cold formed steel and the relevant design codes and specifications are briefly described. The advantages (and the state of the art) of cold formed sections, their methods of manufacture and areas of potential growth in the future are illustrated. Second, the importance of load bearing bolted connections in cold formed steel sections and the inherent differences in their behaviour to that of hot rolled steel connections are pointed out. The modes of failure of such connections are then described and defined in detail.

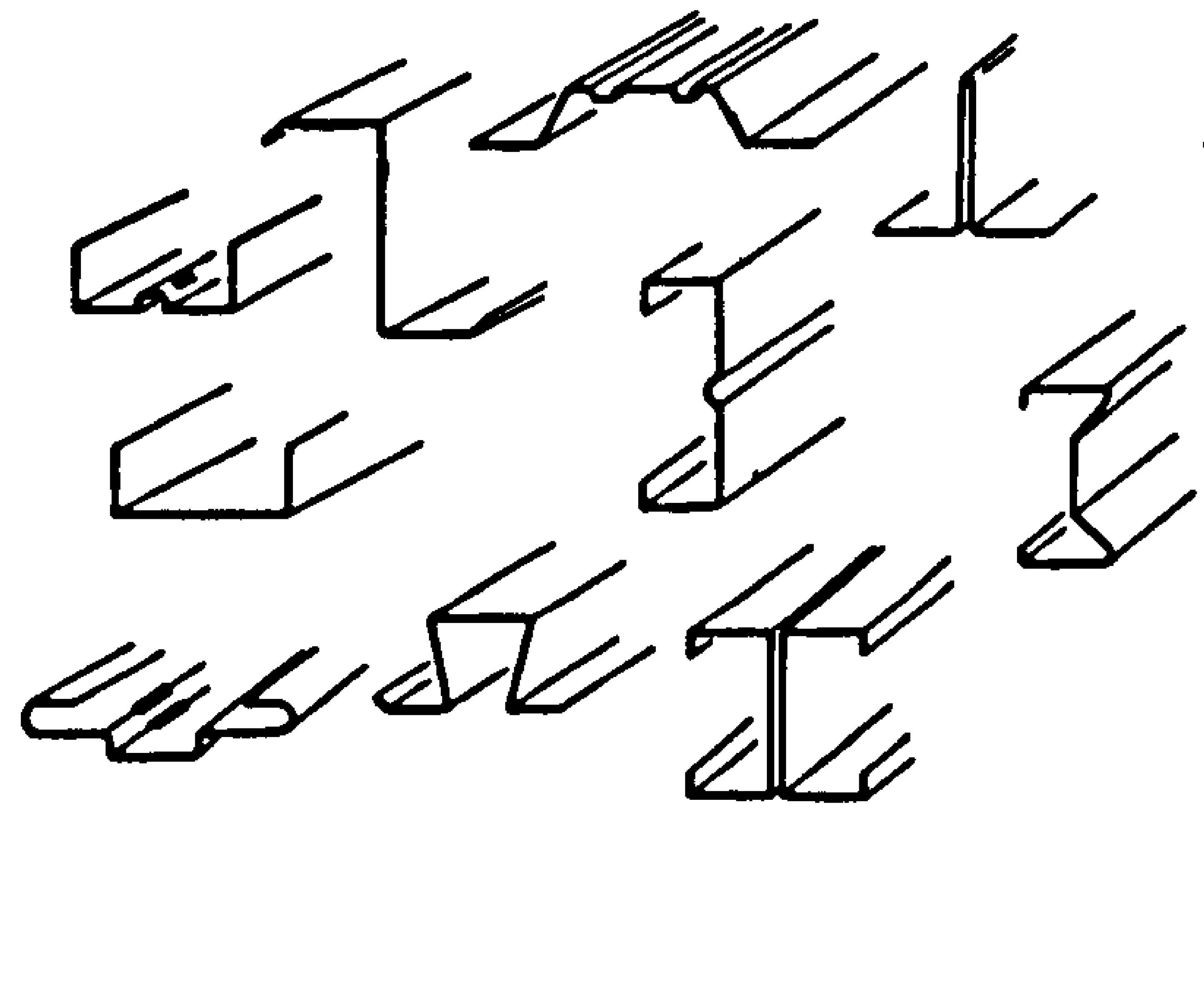
1.1. Introduction

Cold formed steel, as defined by BS 5950 Part 5[9], is primarily concerned with structural steel work of up to 8 mm thick.

Cold formed sections and sheeting - as their name implies - are fabricated at ambient temperatures (i.e cold) from steel strips. The strip is usually galvanized up to 3.2 mm thick. Ungalvanised strip is also available in a wider range of thicknesses. However, the main attraction of cold formed steel is that material is previously surface treated to form an attractive product, eliminating the need for additional protection against corrosion. Therefore cold forming ungalvanised steel will override this principle so its use is not common.

Steel sheet used in cold formed sections is typically 0.9 to 3.2 mm thick. Although thinner sheets (for some cases as thin as 0.5 mm) are used in decking and sheeting applications.

Cold formed steel sections - as laid out by BS 5950 pt.5 - may be either open or closed and should be made of flat elements bounded either by free edges or by bends with included angles not exceeding 135° and internal radii not exceeding 5t, where t is the material thickness.



Therefore sections can be formed to almost any required shape to obtain very strong, stiff, structural members of low weight and high efficiency. That is the material can be formed into the most efficient profile. Consequently, favourable strength-to-weight ratios can be obtained compared to hot rolled sections of the same thickness. (Fig. 1-1)

Cold formed sections usually nest together so that transport is easy, and they are light and easy to erect. As well as nesting, sections can interlock; features not open to hot rolled sections. Such factors bear structural benefits, as will be demonstrated in later chapters.

Load carrying sheeting (or cladding) made of cold formed steel not only withstand loads normal to their planes, but can be utilised to act as shear diaphragms to resist force in their own planes if they are adequately connected to each other and to the supporting members.

Another feature (although of cosmetic interest nevertheless demonstrates the scope for innovation in cold formed sections) incorporated in some members is to provide a concealed electrical trunking system within the section which accommodates electrical conduits, junction boxes, outlets and fixing brackets etc., thus eliminating the need for not so aesthetically pleasing independent electrical trunking systems and site roof drilling of purlins.

1.1.1. History of cold formed steel sections

The use of the cold formed structural members is just about as old as that of the hot-rolled variety. Both started their development around the middle of the last century. At that time not only there were the first small angles, channels and I sections hot rolled from wrought iron, but members were also cold-formed in the shape of angles and channels up to 32 mm thick and also corrugated sheets. The subsequent development of heavy steel construction was primarily characterised by the use of hot rolled shapes except for specialities such as corrugated sheets.

Earlier this century, with the invention of the aeroplane there came an overriding need for as light a structural form as possible. This was achieved by using thin metal sheets to which stiffeners (known as stringers) were riveted.

This phase was followed by the introduction of cold formed steel into the automobile industry. Where in the First World War techniques were developed to bend and shape light gauge steel, in its cold state, into body panels of cars and lorries. The techniques learned were subsequently applied to produce window frames and similar non-structural elements of a building.

The weight saving advantages of the cold formed steel were further realised and consolidated in the shortage of steel during the Second World War. Thus the potential economic gains of reducing the metal content of a structural member became obvious and have since been major factors dictating the development of structural forms to achieve as slender shapes as possible.

1.1.2. Methods of manufacture

Steel from which cold-formed sections are made, are hot rolled at the mills to produce strips of uniform thickness. The steel is then surface treated (galvanised or plastic coated) and delivered in coil form, typically 1 to 1.25 m wide, to the fabricator. The coil is then cut (slit) longitudinally to the correct width for the section being produced. It then can be cold formed by either rolling or by press braking.

1.1.2.1. Cold rolling

Cold rolling accounts for the great majority of commercially produced sections. Using this technique, the strip is fed through a series of roll formers. These rollers are set in pairs , act as male and female dies and move in a opposite sense so that the sheet is drawn through and formed progressively. The number of rolls needed to form the final section depends on the complexity of the section. With the more modern machinery the sections can be marked, punched and cut to length during the same process. The overall length of the forming machinery can be over 30 m. The length of the sections produced is usually governed by transport considerations.

Parts produced by roll forming are essentially uniform in cross-section and can be held within very close dimensional tolerances.

1.1.2.2. Press braking (or folding)

An alternative method of forming is by press braking. In this process, short lengths of strip are fed into the brake and pressed round shaped dies to form the final shape. Each bend is formed separately and the complexity of shape is limited to that into which die can fit. Therefore press braking can only be used for simple shapes. The length of the section being formed is restricted by the machine limitations, normally up to 3 m.

1.1.2.3. Economics of cold forming

Whether to roll or press brake depends largely on the economics.

In general, cold rolling is more suited to mass production, setting up costs of rolls are high, but once they are set up it is almost as easy to roll an intricate shape as a simple one. By using extra bends (or stiffeners) in a section, the average yield stress is enhanced and the stability of various flat elements are improved, resulting in an optimum profile. Adjustable rolls are often used which permit a rapid change of section depth or width.

Folding is less versatile so it is less widely used, its use is limited small quantities of relatively short sections.

It should be noted that in cold rolling section profiles a balance has to be struck between the efficiency of the section - in terms forming additional lips, stiffeners, yield strength of the steel and the sheet thickness - and the cost implications of forming and practical considerations during construction. The more complex a profile and the harder and thicker the steel the more difficult and costly it is to form the section. The constructional tolerances also become more sensitive in terms of lining of the sections and connecting them on site. Therefore, as often it is the case with all matters, a compromise of the theoretical and practical factors is necessary.

1.1.3. Advantages of cold formed steel over other structural materials

Compared with other materials such as timber and concrete the following qualities can be realised for cold-formed steel structural members.

- Fast and easy erection and installation, resulting in speedy construction, giving early return to developers investments.
- Lightness, therefore easy to manage on site.
- High stiffness and robustness to damage.
- Ease of fabrication and mass production, less dependence on skilled labour.
- Substantial elimination of delays due to weather conditions, i.e high productivity.
- More accurate detailing, usually done during fabrication.
- Non-shrinking and non-creeping at ambient temperatures.
- Formwork is not needed.
- Termite-proof and rot-proof.
- "Quality assured" in terms of material strength and dimensions (sections made to very close tolerances).
- Economy in transportation and handling, sections often nest and therefore pack much smaller.
- Non-combustibility.
- Longer spans are possible, offering versatile internal planning.

The combination of the above-mentioned advantages can often be cost saving during construction and result in a more suitable structure.

1.1.4. Specifications for design of cold formed steel sections

The first specification, covering the design analysis of cold-formed steel sections and structures was produced by the American Iron and Steel Institute[6] (AISI) in 1946. This was largely due to the pioneering research programme carried out by the late George Winter at the Cornell University, since 1939. The AISI has continually updated and improved this specification since then. The latest was published in 1986, together with a comprehensive Design Manual[7]. This has only been recently updated, in form of an addendum, in 1989 (first published at the beginning of this year). Due to the great inertia and hence slow momentum in the American construction industry this latest code is still on the basis of the permissible stress.

In United Kingdom the first cold formed steel specifications was produced in 1961[8]. This was replaced by BS 5950 Part 5[9], in 1987. In fact, the design aspects of cold formed steel products for use in structural form is not confined to Part 5 alone. Out of nine parts of BS 5950[9], produced to comprehensively deal with all facets of the steel construction industry, four deal specifically with cold formed steel and two with interaction of cold formed steel in conjunction with other materials. The latest of these BS 5950 Part. 9 "Code of practice for stressed skin design" was approved in its final form only in a matter of a few weeks ago. Out of these however, BS 5950 pt.5 gives the basic rules and procedures to be followed in the design analysis of cold formed steel sections. This document is essentially "equation orientated", based on limit state design. It therefore fits in well with today's desk top computer design philosophy. Overall BS 5950 standards are a significant advance on the previously existing codes.

Over the continent, throughout the last decade or so the steering force has been the European Convention for Constructional Steelwork, ECCS, set up to coordinate the efforts of all the participating members and integrate their contributions into a single European Code for design of steel structures. A series of documents, in form of recommendations, covering many aspects of steel design have been produced. These are now mainly superseded by the publication of Eurocode No. 3[10] (EC3) last year. ECCS committee TC7 was commissioned to draw guidelines for design of light gauge steel structures. A number of European Recommendations were published followed by Annex A to EC3 , now in final draft sent to appropriate authorities and liaison engineers for final comments. This document is directly comparable to BS 5950 pt.5 and were both under drafting concurrently with the latest AISI specification. No doubt that a unified code within the whole European community is a potent design tool and will have far reaching benefits.

Section 8 of EC3, Annex A deals with the strength of connections in cold formed steel sections. Like BS 5950 pt.5 no reference has been made to stiffness of such connections. For the part of this research programme, Salford has been instrumental in shaping and defining the design strength of such connections in the Annex A. Although it is believed that there is still significant room for improvements. This will be elaborated upon in more detail in the following chapters. It is hoped that the findings of this work will further permeate through to Annex A of EC3 , and replace the existing guidelines in BS 5950 pt.5 in the near future.

In cold formed steel, because of the thinness of the material, design tends to be dominated by considerations of sheet bearing against the fasteners. It is interesting to note that the available codes of practice for cold formed and hot rolled tend to have substantially different expressions for bearing strength of bolted connections. This gives rise to a most irrational step function between the design bearing strengths of the two forms of steel. The main reason for this difference lies in the limit state chosen to assess the capacities against. In cold formed steel the ultimate limit strength is considered, while in hot rolled steel the main concern has been limiting the deformation of the connected components, i.e elastic limit behaviour is used. This matter will be considered in more detail in the later chapters.

1.1.5. Application of cold formed steel sections

In this section first the important factors contributing to the recent demand in cold formed steel are noted, followed by a brief description of the major existing and growing markets in the field of structures.

1.1.5.1. Factors influencing the market for cold formed steel

It has been described that the use of cold formed steel sections in structures and the drafting of specifications covering their design began almost 50 years ago. However, the real impression on the construction industry has been made over the last decade. This delay in recognition has possibly been due to a lack of understanding of the phenomena involved, always initially associated with any "new structural form". As with much other structural development, increase in practical application has been accompanied by a corresponding evolution in design.

In recent years, while steel production as a whole has fallen, there has been an increase in the production of coated steel strip. Specification of composite profiled steel decking, for instance, has increased by ten folds in the last seven to eight years. A similar pattern of production growth has taken place in many other European countries. Cold formed steel is now recognised as the fastest growing application of steelwork.

There are a number of factors which have contributed to this increase in application. These will be described here.

1.1.5.1.1. Economics of construction

With ever increasing material, labour and land costs in the construction industry, there is a constant need, especially in the context of urban and housing problems, for more radical economical construction. This can not be achieved without substantial industrialisation of the construction processes. Cold formed steel lends itself exceptionally well to industrialisation.

1.1.5.1.2. Aesthetics

With modern manufacturing techniques, cold formed steel strips are produced with a range of plastic colour coatings applied prior to the fabrication process. (A good example of industrialisation) The aesthetic appeals of such surfacing is well appreciated by the architect, and it is not solely confined to the colour. Different profiles in different orientations can be used to create texture as well as colour. So, both dramatic and subtle effects are possible. In fact this is now a common architectural concept in many small scale office and/or industrial buildings. It is hard not to notice the significant increase in the proportion of buildings clad in steel, particularly in the industrial market, which is a clear indication of the industry recognising the architectural potentials of such sections.

1.1.5.1.3. Durability

Perhaps it may be argued that the much of the success of cold formed products in recent years is a consequence of modern protective coatings. The fact that cold-forming can be carried out with steels already finished with a variety of surface coatings, such as zinc, aluminium or a coated plastic finish, without damaging the surface or impairing the benefits of such treatments is a vast improvement with regard to corrosion protection over ordinary painting. It has caused the "iron rusty" image often associated with the old corrugated sheets to disappear. Nowadays corrosion is no more of a problem with cold formed than it is with hot rolled steel. Life expectancy of 25 years up to the first maintenance is often the norm.

1.1.5.1.4. Interaction with other forms of structural material

In the past decade or so the construction industry has not only come to recognise the individual features and merits of cold formed steel, but even more so its value as a mutually supplementary and complementary material to other forms of structural material. Composite floors, faced panels, sandwich panels, etc. are prime rapidly expanding, examples of this in the industry.

1.1.5.1.5. Building physics

An industrial building is no longer regarded as merely a watertight roof over machinery, etc. With the energy crisis in the seventies there came a need for higher levels of thermal insulation. In addition buildings are expected to have a better sound insulation, deter condensation, give adequate fire protection and in general a higher overall performance with regards to their physics. Thus contributing to lower overhead and maintenance costs.

It is important that the steel skin should provide maximum resistance to moisture penetration. Profiled steel sheeting is used as the outer waterproof skin, with the insulation placed inside this skin (known as "cold roof" construction). The water tightness of roofs has increased greatly in recent years with innovations such as :-

- Profiles with troughs to increase run-off rates. (Fig. 1-2)
- Trough fasteners with efficient sealing washers. (Usually in form of self-drilling, self-taping screws with neoprene washers.)

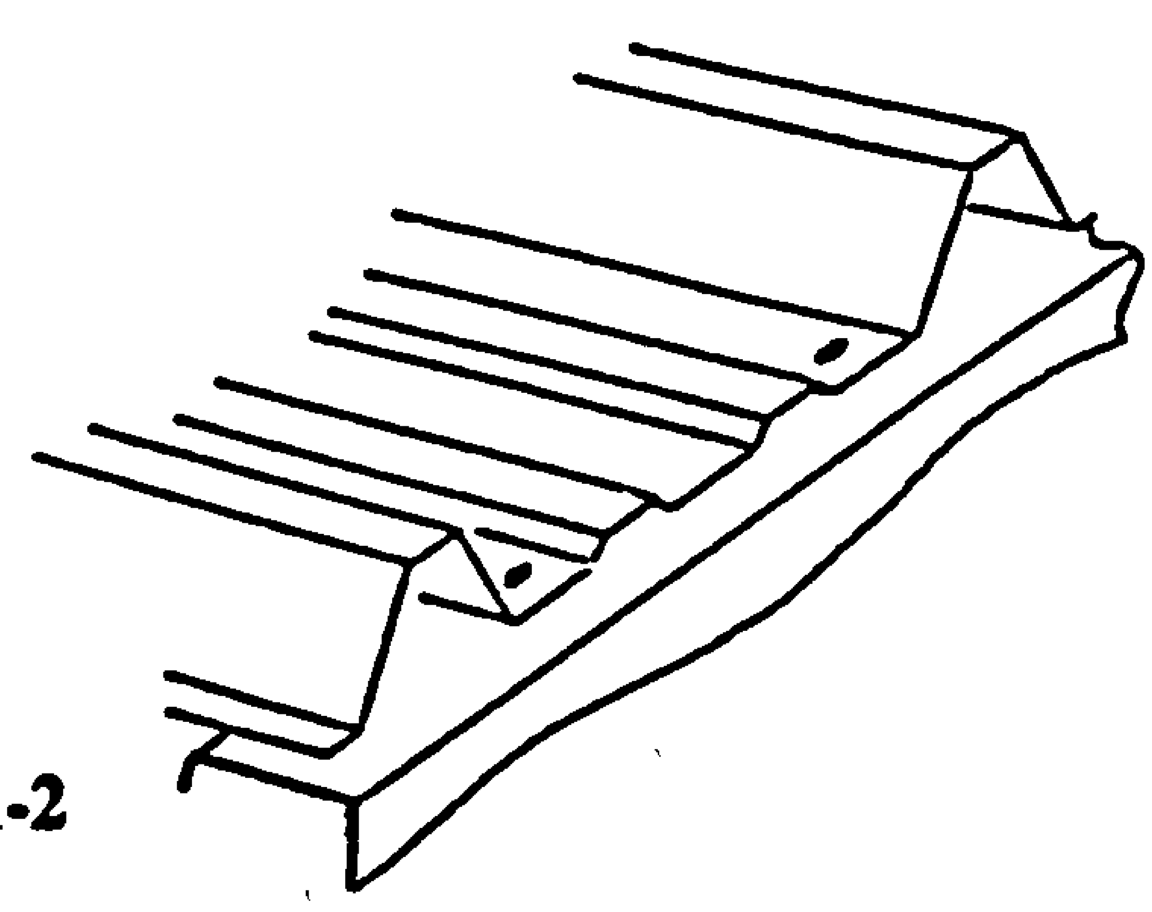


Fig. 1-2

- Concealed fix and standing seam profiles. (Fig. 1-3) The danger of rainwater penetration can be further reduced by anti-syphoning grooves and sealants.

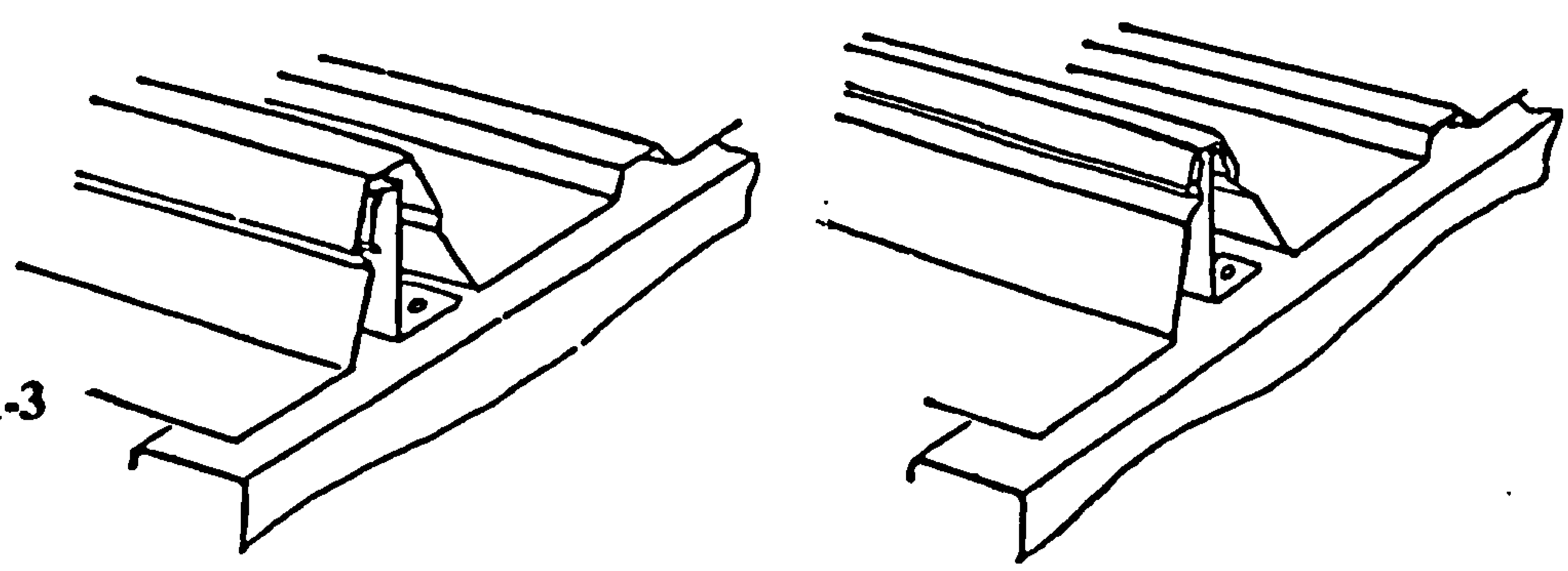


Fig. 1-3

Site damage is a problem often identified with such sections. To prevent this, the material thickness is kept to a minimum of 0.7 mm and low web slopes are avoided - or alternatively - a higher yield stress steel and more complex profiles are used to make the sections stiff enough so that they would not dent easily.

With regard to thermal insulation and condensation, sandwich panels have been developed to provide a "complete cladding system" both in terms of an excellent and durable heat insulation and; an absolute water and vapour barrier.

Sandwich panels are often used as wall or roof units and consist of two metal skins formed of flat or profiled sheets with a relatively low-strength, low-density core material with suitable insulating and stiffness properties (as an integral part of the whole unit) sandwiched in between them.

The advantages of such panels over conventional applications (where two skins are kept apart by spacing members and the insulation in loose) is that for loose insulation it is possible for quilts to slip down, particularly in walls, resulting in a loss of efficiency in the insulation and condensation in the space between the two metals, in turn spurring an increased risk of corrosion.

The concept of sandwich panels is more advanced and widely used in the Scandinavian countries, notably Sweden and Finland. There is also a trend towards increasing thickness of the insulation. In the forementioned countries, insulation as thick as 200 mm is not uncommon.

Such panels also posses good sound insulation compared to homogeneous wall or roof elements of the same weight.

Sandwich panels using a rigid foam core do not reach a notable fire resistance time. To achieve some fire code classification mineral fibre mats must be used or an additional cladding provided for the original facings. New materials, such as mineral Rock Wool or inorganic pregnated Honeycomb cores are under investigation to give them a better fire performance.

1.1.5.1.6. A total design concept

It is interesting to note that in a typical steel-framed, steel-clad industrial building the cost of cold formed steel components significantly exceeds the cost of the frames.

The costs breakdown of the elements of a typical industrial building - as produced by a fabricator - are as follows[11]:

A) Hot rolled steel	Main frames Gable framing Tubular bracing	31%
B) Cold formed steel	Purlins and sheeting rails Roof sheeting Side sheeting	42%
C) Other	Gutters and downpipes insulation to roof and walls	27%
		100%

Up until recent years, all the design efforts was directed at the frames (category A), which only constitutes less than a third of the cost of the whole building. Not enough design consideration was given to the rest of the building elements. The rest of the building was simply put together by consulting manufacturers safe load tables. It may even be true to say this is still the case with the majority of designs being carried out at the present. However, there is increasing evidence that a significant change in the design conception of these buildings is taking place.

The industry has realised that further efforts to refine frame design are unlikely to prove as fruitful as efforts to optimise the design of complete building package to integrate the components of this package (main frames, sheeting, purlins, rails, liner trays, etc.) in a complete interacting system.

In recent years major manufacturers of cold formed steel sections have identified the scope for innovation in cold formed steel and have sought to explore the potentials of cold formed sections that can nest and interlock. These are benefits, not open to the hot rolled variety.

One such example is the "Swagebeam", developed to enhance the moment resistance of bolted connections of adjoining sections.

One application of this section has been to develop standardised portal frames, a typical example is shown in Fig. 1-4[12].

Such frames employ double Swagebeams (two swaged channels back to back) for the main frame columns and rafters, and single Swagebeams for purlins and side rails. The joints are made with bolted swaged gussets, between the main frame members, and swaged web cleats for purlins and side rails. Thus the sections lock into each other which provides additional stiffness to the connections and to the frame as a whole. This leads to significant savings in the size of the sections used.

Fig. 1-4 : Swagebeam building frames.

All the sections (main and secondary members) are of a uniform size. This allows the purlins and side rails to be connected within the depth of the main frame members to give flush inside and outside faces for easy cladding and lining. The uniformity of section sizes are no doubt a considerable attraction at the time of design specification and erection.

The cladding is directly attached to the mainframe and secondary members, by mechanical fasteners capable of transmitting the horizontal wind loads from cladding, in shear, to the mainframe. Therefore taking account of the stiffness of the cladding in "stressed skin" action. This virtually eliminates the need for traditional wind bracing. Furthermore by taking account of the stressed skin action, frame stresses and deflections (compared to bare frame calculations) can based on more representative values of how elements of a building actually interact in-situ.

This yields an ideal solution for industrial and agricultural buildings with 9 to 15 m span.

Making use of the adaptability of cold formed steel to stream-lining and industrialisation results in further benefits listed below.

Sections arrive on site marked, cut to length and punched at the factory to design specifications.

Light weight sections of uniform size, results in a significant increase in the speed of construction.

Light weight construction requires smaller foundations.

No especial skills and minimum lifting gear are required for erecting.

The frame sections are supplied with complete connectors and brackets, eliminating the need for complex connection detailing.

Custom made cladding and linear can be specified.

Future extension or relocation, due to the light and adaptable nature of the structure, is easily possible.

Therefore significant savings are made on traditional construction, together with the attraction of "one stop shopping" for the client.

This a prime example of a total design concept, approaching an optimum structural and economical performance, for a particular use.

Therefore with the above factors, contributing to an increase in application and versatility of cold formed steel, the challenge for the engineer is to strive for fresh ideas, making use of the potentials offered by cold formed steel.

1.1.5.2. Existing and expanding markets for structural cold formed steel sections

From what has already been mentioned it is evident that the structural application of cold formed steel can be classed into two broad categories, as described below.

1.1.5.2.1. Sheeting and decking

Sheeting and decking are not strictly considered as "sections", but as described already, are well established and make up for a large proportion of the cold formed steel market. Their design is largely governed by the consideration of local buckling. A distinction is drawn between sheeting (cold roof) and decking (warm roof). For sheeting the primary consideration is watertightness of the skin and rapid dispersal of water. Whereas in decking the load bearing capabilities of such panels are of rudimentary concern. This has resulted in complex, highly stiffened profiles, capable of spanning over 10 meters.

Examples of their structural use (sheeting and decking) are as follows:

Composite floor decking, One of the most widely used types of flooring in high rise office buildings, designed to act in conjunction with in-situ concrete floors.

Particular design attention is given to the shear connector of such sections, and hence the level of interaction obtained between the steel deck and its concrete cover.

Roof and wall members, perhaps the major market for cold formed steel at the present, used as cladding for industrial buildings.

Prefabricated building units, transportable prefabricated building units are also a common application of the use of the cold formed steel.

Frameless steel buildings, Steel folded plate roofs by relying partly on stressed skin action, obtain long spans (typically designed for a column grid of 10 m) and a clean internal and external appearance. A commercial development of this has been marketed as "Pyradome". This system has been used for a wide range of industrial and commercial purposes. It is already well established in the United States and its market is likely to increase in the UK.

1.1.5.2.2. Individual structural framing members.

The other principal application of cold formed steel is in individual structural framing members.

Housing, there are already moves to introduce cold formed steel sections in the housing market, i.e competing with the traditional brick and timber construction. In modern houses however, the cost of basic shell (excluding finishes and services) may only account for 15 to 20% of the selling price of the house. The cost of the material component may only one be half this figure. Therefore, despite the competitive nature of the housing market the cost of the building is relatively insensitive to the cost of material used in construction. The decisive factor in success of cold formed steel under such circumstances may therefore be speed of construction associated with almost every form of steel structure.

Storage racking, industrial storage racking is a major market for cold formed steel. They usually consist of slotted columns and cold formed steel beams. The whole field of storage racking is a specialised one. The use of perforated members is not covered in BS 5950. The storage equipment manufacturers have set up their own Codes of practice to deal with the design of Racking and Shelving components.

Industrial and commercial steel frames, it has already been mentioned that cold formed steel members are being increasingly used in short and medium span frames. They have proved particularly successful in light commercial and industrial buildings. Until recently their use was limited to secondary members. The impetus for their wider use has been the publication of BS 5950 pt.5. For the first time the designer has been given sufficient information to design main frames of structures in cold formed steel. One such development has previously been referred to (§ 1.1.5.1.6). However, the key to the successful use of cold formed steel members is the design of their connections. This principle is illustrated in the second part of this chapter, where the theme of the thesis is clearly set out.

1.2. Connections in cold formed steel

The state of the art terminology of mechanical fasteners for use in cold formed steel components are:

Connection	:	a group of (1 or more) fastenings.
Fastening	:	fastener interacting with its surrounding material.
Fastener	:	the connecting element in a fastening.

The common types of fastening between cold formed steel components are :

	Type	Usual application(s)
a)	Bolts	Connecting cold formed members.
b)	Self tapping screws	Fixing (1) sheeting to members (<6mm thick) (2) sheeting to sheeting at sidelaps.
c)	Blind rivets	Fixing sheeting to sheeting at sidelaps.
d)	Powder activated pins	Fixing sheeting to members (>6mm thick)
e)	Spot welding	Factory jointing of thin steel.
f)	Other	eg. lock seaming, crimping and bottom punching, "twist grip" nails, etc. for more particular applications.

For cold rolled sections, where holes are punched during forming, bolts are by far the most common type of fasteners used in practice. Their ease of application and economy (not requiring any special skills or expensive tools) has a particular attraction, in all types of steel structures.

Fasteners used to join steel are required to be at least as corrosion resistant as the steel parts to be joined. Therefore bolts are always galvanised when used to connect cold formed steel sections.

Thickness of the sections, used in bolted cold formed steel connections, almost always lie in the range of 1.2 to 3.2 mm.

Connections are usually arranged so that the bolts are in shear, and because of the thinness of the material the design tends to be dominated by the bearing capacity of the thinner steel sheet rather than the shear of the bolt.

1.2.1. Objectives

The great momentum in the development of cold formed steel has already been outlined. This brings about a constant need for modified and reliable design information that not only improve the economy of design, but also the versatility of cold formed steel construction. Connections in cold formed steel was one these matters that needed to be worked on in detail from research to specification in practice.

Since the 1940's numerous tests on bolted lapped joints in cold formed steel, have been carried out by many investigators. To date, the main point of focus in all their work had been the strength of such fastenings, little was known about the stiffness of such connections.

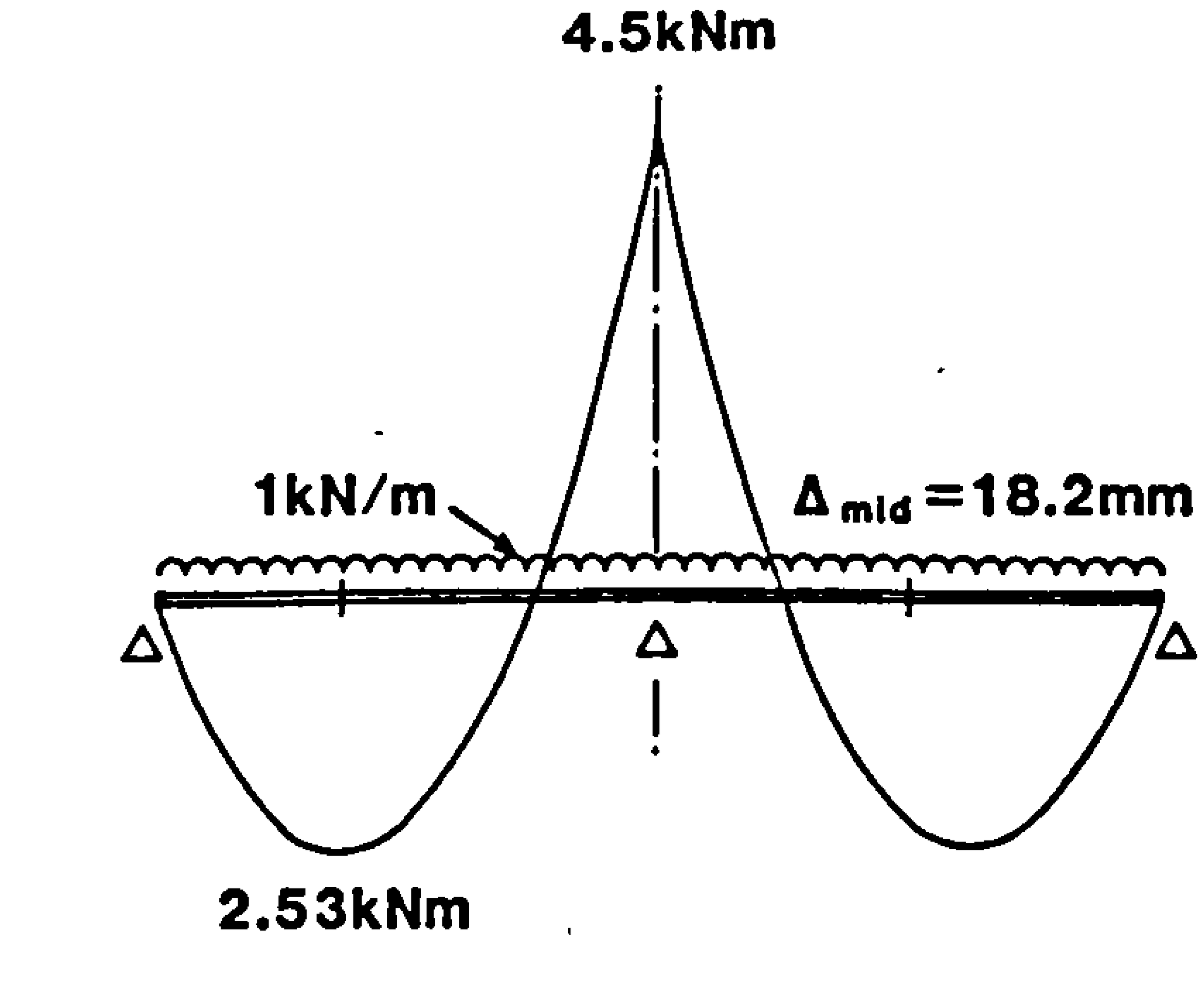
At present this is counterbalanced by basing the design of standard sections such as purlins and sheeting rails upon testing. Test results form the essence of manufacturers safe load design tables for particular spans and loadings.

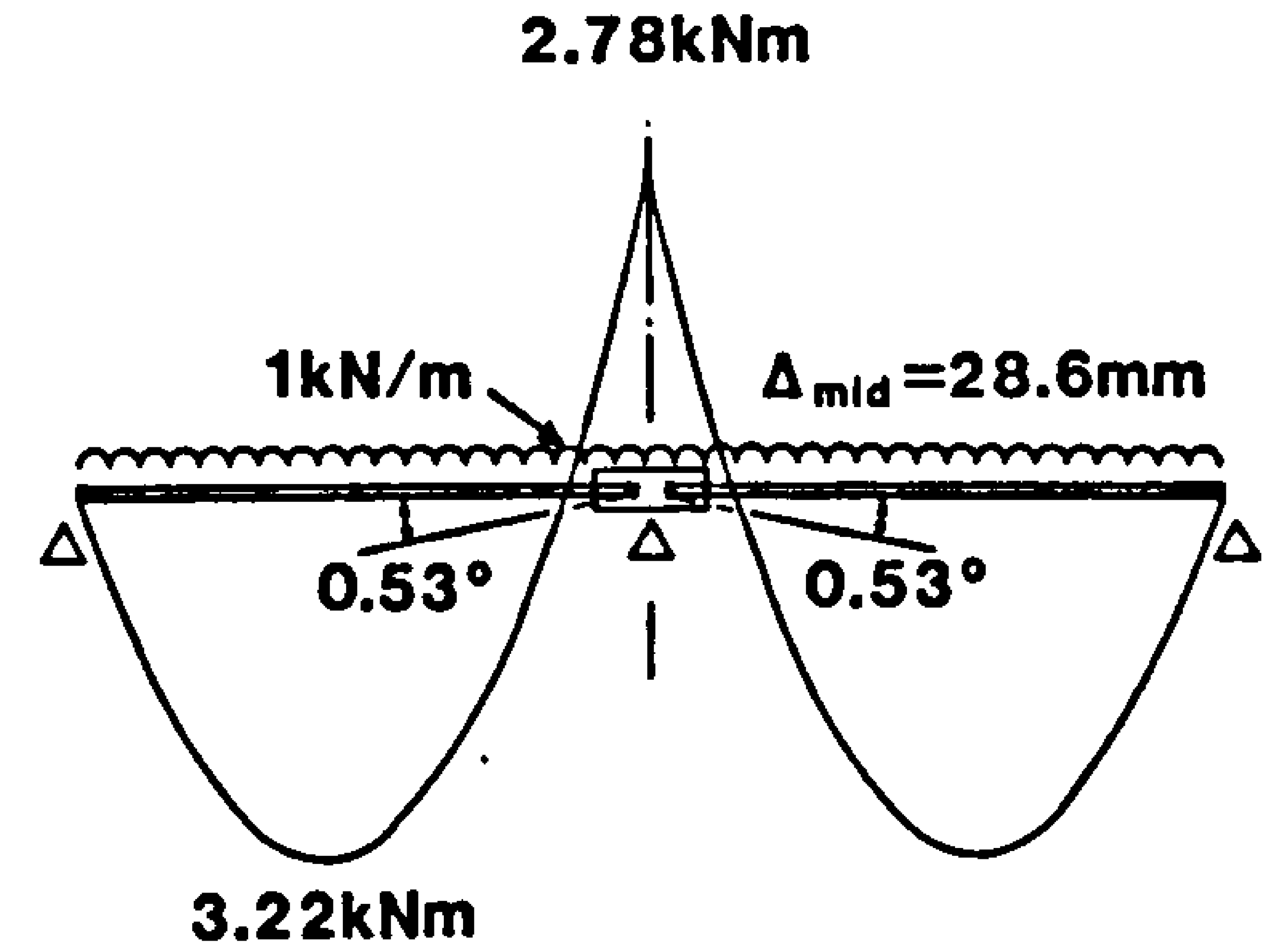
With the introduction of cold formed sections in the main frame members joint characteristics assume even greater importance for economic design. There is therefore a need for improved, more realistic and better design methods.

If the moment-rotation of such joints could be adequately predicted, then it not only eliminates the need for costly and time consuming test procedures but also gives the designer the ability to design for optimum moment distribution within the sections. As mentioned above, this not only improves the economy but also the versatility and the scope for innovation in cold formed steel structures.

The importance of the stiffness of connections in cold formed steel and their moment-rotation characteristics is illustrated in Fig. 1-5[17]. It is seen that the flexibility of the supports allow a limited redistribution of the applied bending moment in order to more closely match the resistance moment of the section.

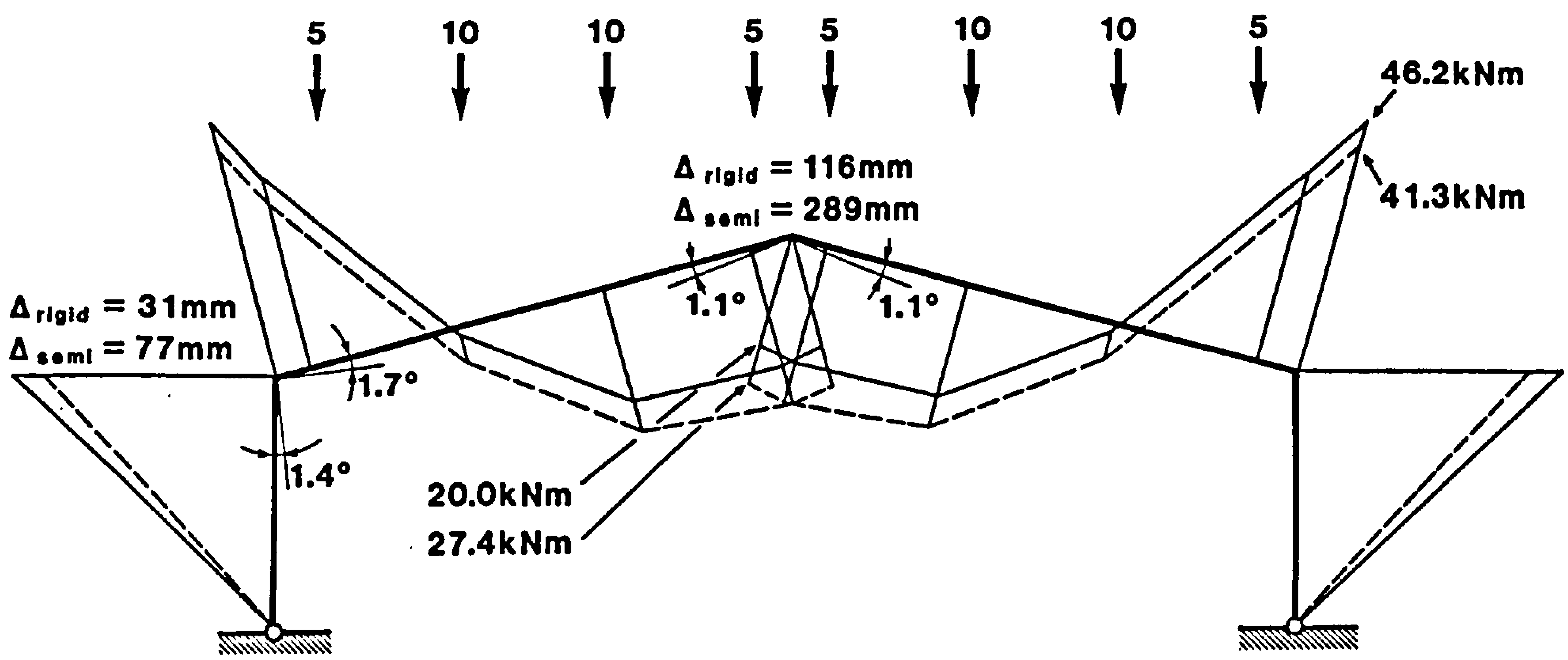
Therefore, the objective of this thesis is to shed light on the behaviour of bolted joints in cold formed steel assemblies or structures. The aim will be to predict the strength and rigidity of common types of joint, so that this information can be fed into design programmes to radically affect the elastic design of cold formed structures. It should then be possible to design optimum structures by specifying joints with the appropriate characteristics.





(1) DOUBLE 6m SPAN BEAM WITH RIGID JOINT

(2) DOUBLE 6m SPAN BEAM WITH SEMI-RIGID JOINT (k=300 kNm/rad)



KEY

——— Frame with rigid joints

- - - - - Frame with semi-rigid joints (k = 1400kNm/rad)

BENDING MOMENT DIAGRAMS
FOR 12m SPAN PORTAL FRAME WITH PINNED BASE

Fig. 1-5 : Effect of joint rigidity in typical cold formed steel assemblies / structures.

1.2.2. Previous and related work

The European Convention for Constructional Steelwork, ECCS working group TC7, have carried out some tests on the stiffness of connections in profiled sheets as wall and roof panels, namely seam connections and connections of the sheet ends to the supporting structure[18][19][20]. These connections usually consists of the lighter fastener types, mentioned in § 1.2. Their main structural importance is in diaphragm stiffened structures, where diaphragm action is often relied upon for structural stability. The stiffness of such connections together with that of the connected sheets, will govern the stiffness of the whole diaphragm.

Extensive work has been carried out at Sheffield University on the stiffness of connections, with hot rolled steel as the parent material, to develop an understanding of semi-rigid connections[21]. This work had a direct bearing on the initiation of the research work leading to this thesis. However, the information available on the behaviour of bolted connections in conventional steel structures is not directly applicable to light gauge cold formed construction mainly because:

i) In hot rolled steel construction the bolt diameter is roughly of the same order of magnitude as the thickness of the connected elements, while in cold formed steel the thickness is usually a fraction of the bolt diameter. This gives rise to phenomena such as bolt tilting or sheet curling out of plane, not seen in conventional bolted connections.

ii) Cold formed steel often receives some kind of "surface treatment" which effectively reduces its coefficient of friction compared to hot rolled steel.

iii) In general in conventional steelwork one or two grades of steel are used. In contrast in cold formed steel, a large variety of steel ranging from a yield stress of 200 N/mm^2 to 350 and even 550 N/mm^2 for sheeting (above this value steel becomes too hard to be cold formed) are used. The most common strength of cold formed steel used nowadays has a nominal yield stress of 280 N/mm^2 (actual ranging from 300 to 320 N/mm^2). Although there is an increasing trend to move to nominal 350 N/mm^2 (actual 370 to 390 N/mm^2) yield steel.

iv) The restraint offered by a connection to cross section warping is a secondary effect in hot rolled construction, but with cold formed sections warping has a dramatic effect on the strength of member.

Other work carried out on bolted connections in cold formed steel, with a direct relevance to the thesis, will be described and the results obtained discussed in later chapters.

1.2.3. Behaviour of bolted connections in cold formed steel

As mentioned above, the structural behaviour of bolted connections in cold formed steel is somewhat different from that in hot rolled heavy construction, mainly consequent upon the thinness of the connected parts. Before the behaviour of complete connections can be considered it is essential that the failure modes and load/slip characteristics of simple lap joints are fully understood. From numerous tests conducted on lap joints since the 1940's four distinct failure modes have arisen.

1.2.3.1. Modes of failure of bolted connections in cold formed steel

The potential modes of failure of bolted connections in cold formed steel are described and their mechanical models are defined from first principles in this section.

Sheet tearing mode of failure

For relatively short end distances in the direction of the applied force the connection may fail by longitudinal shearing of the sheet along two nearly parallel lines, a distance equal to the bolt diameter apart. (Fig. 1-6).

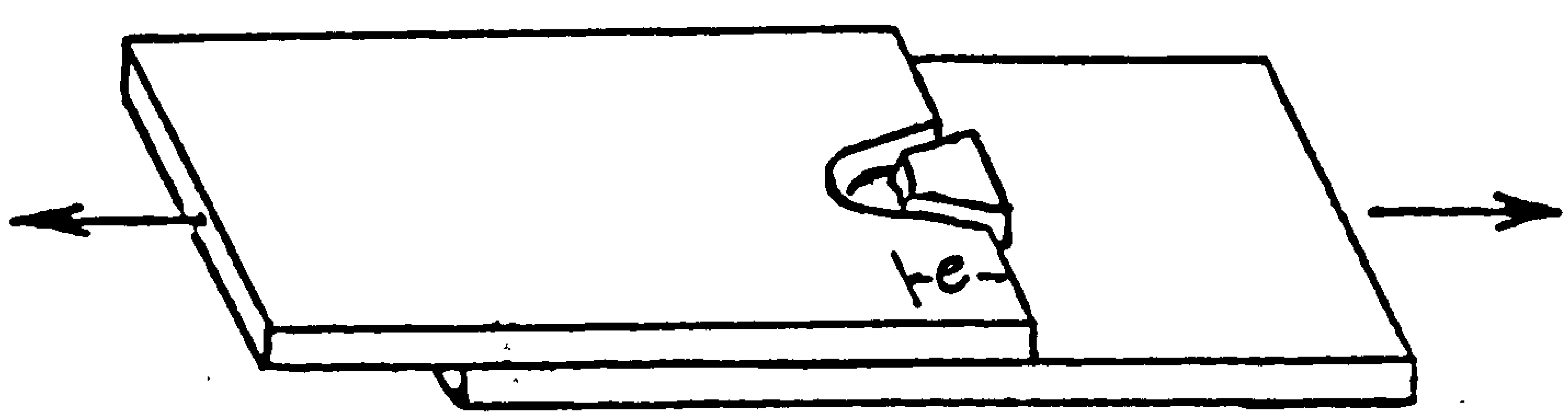


Fig. 1-6 : Sheet tearing mode of failure.

Upon examination of failed specimens it was apparent that the longitudinal shearing distance of the sheets could be approximated to e (= end distance). The actual shearing occurs over a length slightly greater than (e - d/2), say $[(e - d/2) + \xi]$, depending on the bolt diameter. Moreover the actual shear lines are not parallel but at a small angle ß (say) to the line of stress. Therefore the exact shearing takes place over a length of $[(e - d/2) + \xi] / \cos \beta$ which can be approximated to the end distance ($\approx e$).

Therefore, from first principles the ultimate load is defined as :

$$P_t = 2e \cdot t \cdot \tau_u \qquad (\tau_u = \text{Ultimate shear stress})$$

Substituting 0.6 σ_u in place of τ_u :

$$P_t = 1.2\ e.t.\sigma_u \qquad (\sigma_u = \text{Ultimate tensile stress})$$

Sheet bearing mode of failure

For sufficiently large bolt end distances the connection may fail by bearing or piling up of steel sheet in front of the bolt. (Fig. 1-7)

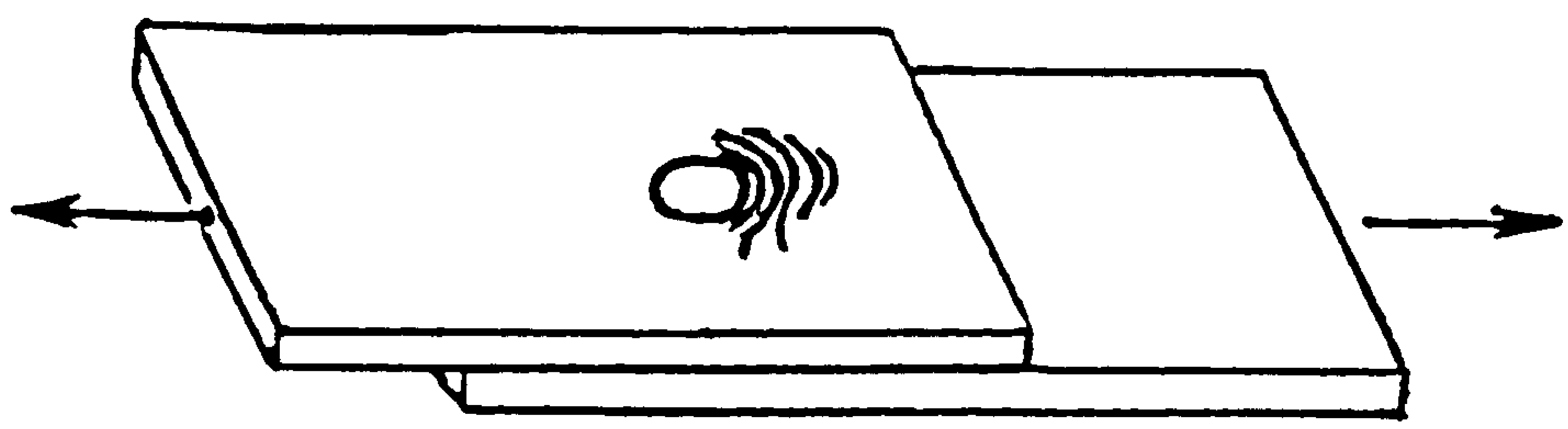


Fig. 1-7 : Sheet bearing mode of failure.

The bearing strength of a bolted connection is equal to the tensile area of the bearing strength times a factor α, determined from test results.

$$P_b = \alpha\ d.t.\sigma_u \qquad (d = \text{bolt diameter})$$

The factor α (= P_b / $dt\sigma_u$) depends on several parameters, namely :

- Thickness of connected parts.
- End distance in the line of stress.
- Diameter of the bolt.
- Ultimate (or yield stress) of connected parts.
- Use and type of washers.
- Ultimate/yield strength ratio.
- Bolt tilting.

There is a trend to integrate the two above mentioned modes of failure (i.e Sheet tearing and Sheet bearing) into a more general Bolt bearing mode of failure.

The sheet tearing mode of failure is implicitly defined in the factor α, and the minimum (end distance / bolt dia.) ratio (e/d) is kept to a minimum of 1.5.

The end distance at which the mode of failure changes from sheet tearing into sheet bearing may therefore be obtained by equating the two mechanical models:

$$
\begin{aligned}
P_b &= P_t \\
\alpha\, d.t.\sigma_{ult} &= 1.2\ e.t.\sigma_{ult} \\
&= 1.2\ (e/d)\ d.t.\sigma_{ult} \\
\therefore \quad \alpha &= 1.2\ (e/d)
\end{aligned}
$$

Note that the end distance in line of stress is the governing factor on whether failure occurs by sheet tearing or sheet bearing.

The more general bolt bearing mode of failure is illustrated in Fig. 1-8.

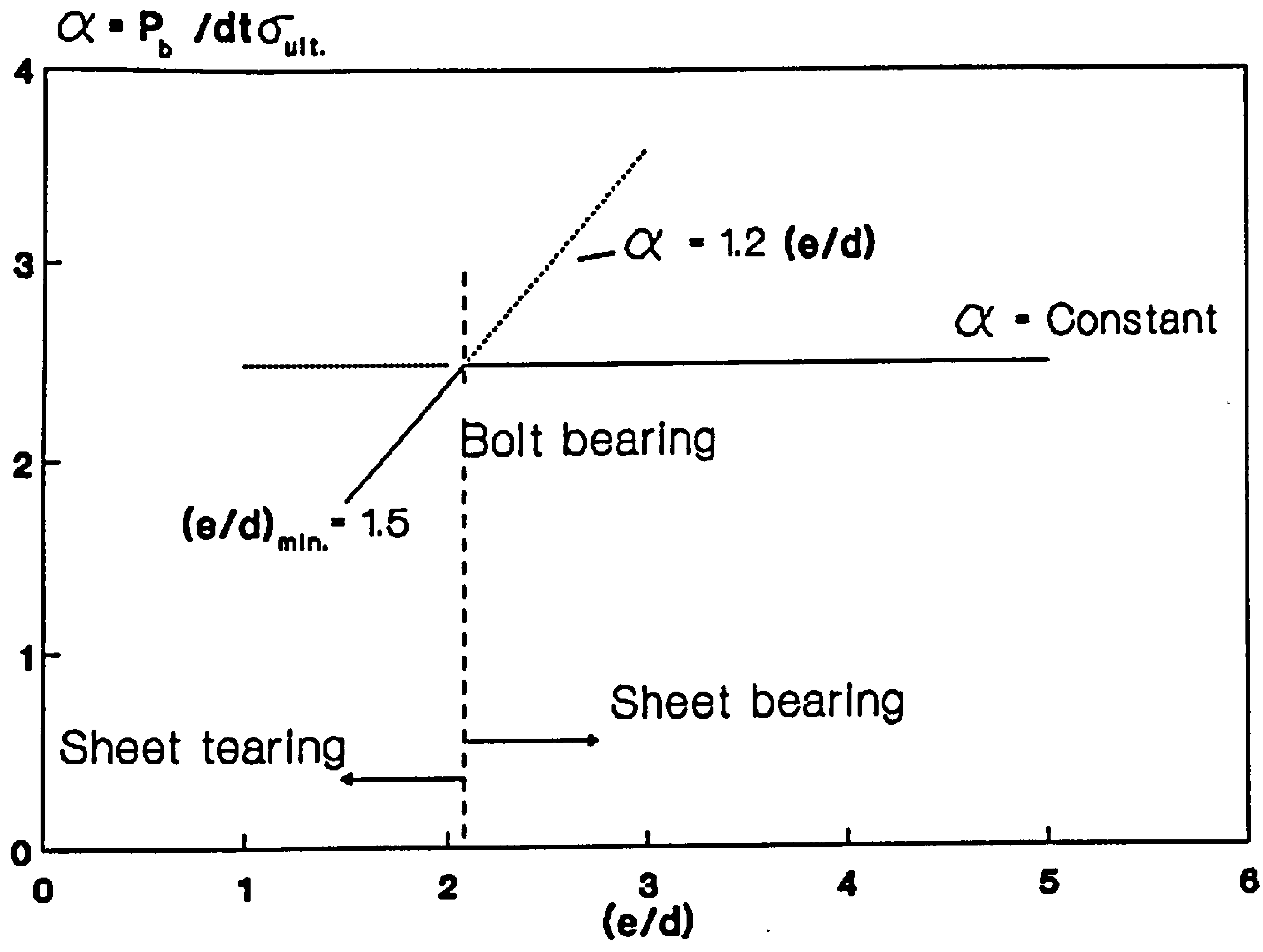


Fig. 1-8 : Bolt bearing mode of failure.

This approach will be adopted in the thesis and all the relevant factors shown above will be quantified in the following chapters.

<u>Tensile failure in net section</u>

This type of failure occurs when the strength of the fastening is more than the ultimate strength of the net section of steel, Fig. 1-9.

The adverse effect of stress concentration caused by:
- The presence of a hole(s),
- The concentrated localized force transmitted by the bolt to the sheets,

also encourages this mode of failure.

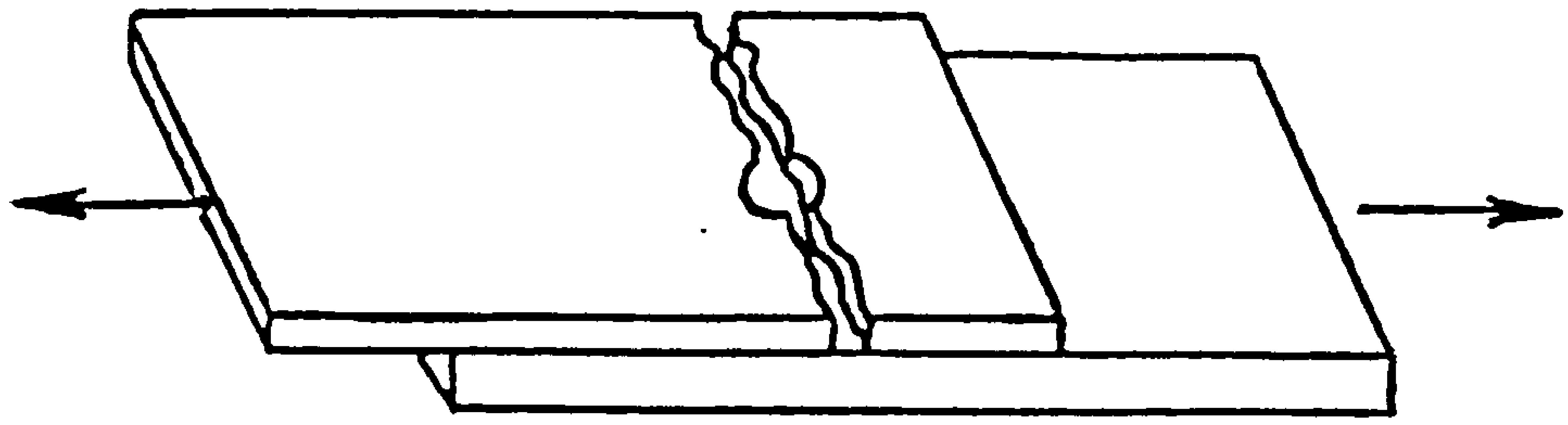


Fig. 1-9 : Net section mode of failure.

$$P_n = A_n \cdot \sigma_n$$

σ_n is limited to the yield stress (σ_y) in BS 5950 pt.5 and to the tensile strength (σ_{ult}) in Annex A, EC3.

Consideration of net section mode of failure is the same as in conventional hot rolled construction and does not require further attention here.

Shearing of bolt

This mode of failure occurs when the bearing strength of connected parts is greater than the shear strength of bolt, This is a sudden mode of failure and not common in cold formed steel. (Fig. 1-10)

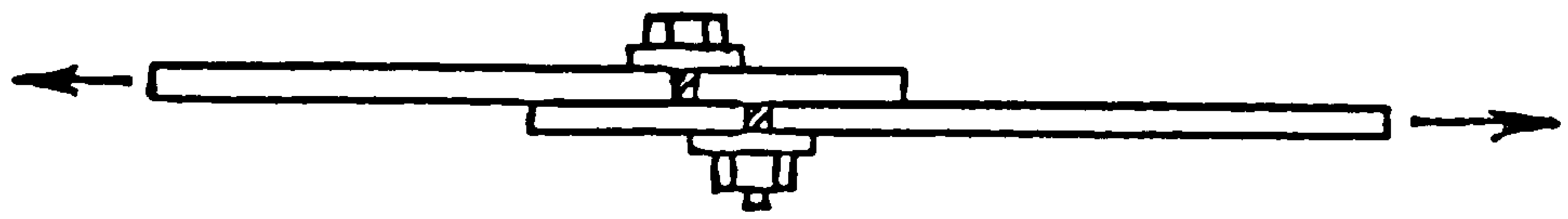


Fig. 1-10 : Shearing of bolt mode of failure.

$$P_s = A_s \cdot \sigma_s$$

σ_s depends on bolt grade, given in Table 11 BS 5950 pt.1 ; and
A_s depends on whether the shear plane is on threaded part or the plain shank of the bolt.

Shearing of bolt mode of failure although essentially the same as in hot rolled sections but hardly ever occurs, even in the thicker range of sheets (2.4 to 3.2 mm, say). This is due to the bolt tilting phenomenon. Bolt tilting will be described in the following section and the arguments put forward will be backed up by test results in later chapters.

1.2.3.1.1. The bolt tilting phenomenon

The most fascinating feature with regard to behaviour of bolted connections in cold formed steel, not seen in the hot rolled variety, is bolt tilting.

The material tends to fold into a ridge in front of the bolt and washer and form a tension field thus enhancing the ultimate bearing strength. The increase in the ultimate bearing strength however, is fairly marginal. The main effect of bolt tilting is to release the shear on the bolt and replace it by a state of shear and tension. This often eliminates the shearing of the bolt mode of failure and it is not accounted for in any of the design codes up to date.

Strictly speaking bolt tilting is not solely restricted to connections in cold formed steel. A connection in hot rolled steel will act in exactly the same manner provided that the bolt in the fastening is hard enough to resist failure in shear. The author has conducted tests with material as thick as 6 mm where the bolt tilted, just as it normally would with the thinner sheets (Fig. 1-11). For such tests however very hard cap screw bolts (obtained from the Department of Mechanical Engineering) hardly ever seen in structural engineering were used. Obviously with sheets thicker than 6 mm even harder bolts are required.

Whether a connection will fail by shearing of bolt or the bolt will tilt and hence sheet bearing mode of failure will dominate from there on, can be thought of as a point of bifurcation. If the load at which bolt tilting initiates is greater than the shear strength of the bolt, then the bolt will obviously fail in shear. Otherwise the bolt will tilt, and from there on the shear force on the bolt is gradually reduced and replaced by a tensile force. From this point on even though the applied load on the connection might exceed the shear strength of the bolt, the bolt will not fail and as said above bearing strength of connected parts will govern the failure load.

In cases where the bolt has tilted say 90° and the shear force on the bolt is totally replaced by a tensile force then by that time the connected parts are distorted so much that the bolt is simply pulled through the joint. This mode of failure is often seen with the smaller types of fasteners mentioned in §1.2.

The fact that bolt tilting is never seen to occur in bolted hot rolled steel connections is that the grades of structural bolts used in the construction industry are never strong enough to prevent failure of fastener in shear. In light gauge connections with the loss of bearing strength of connected parts, consequent upon the thinness of the material, common structural bolts are often strong enough to resist failure in shear.

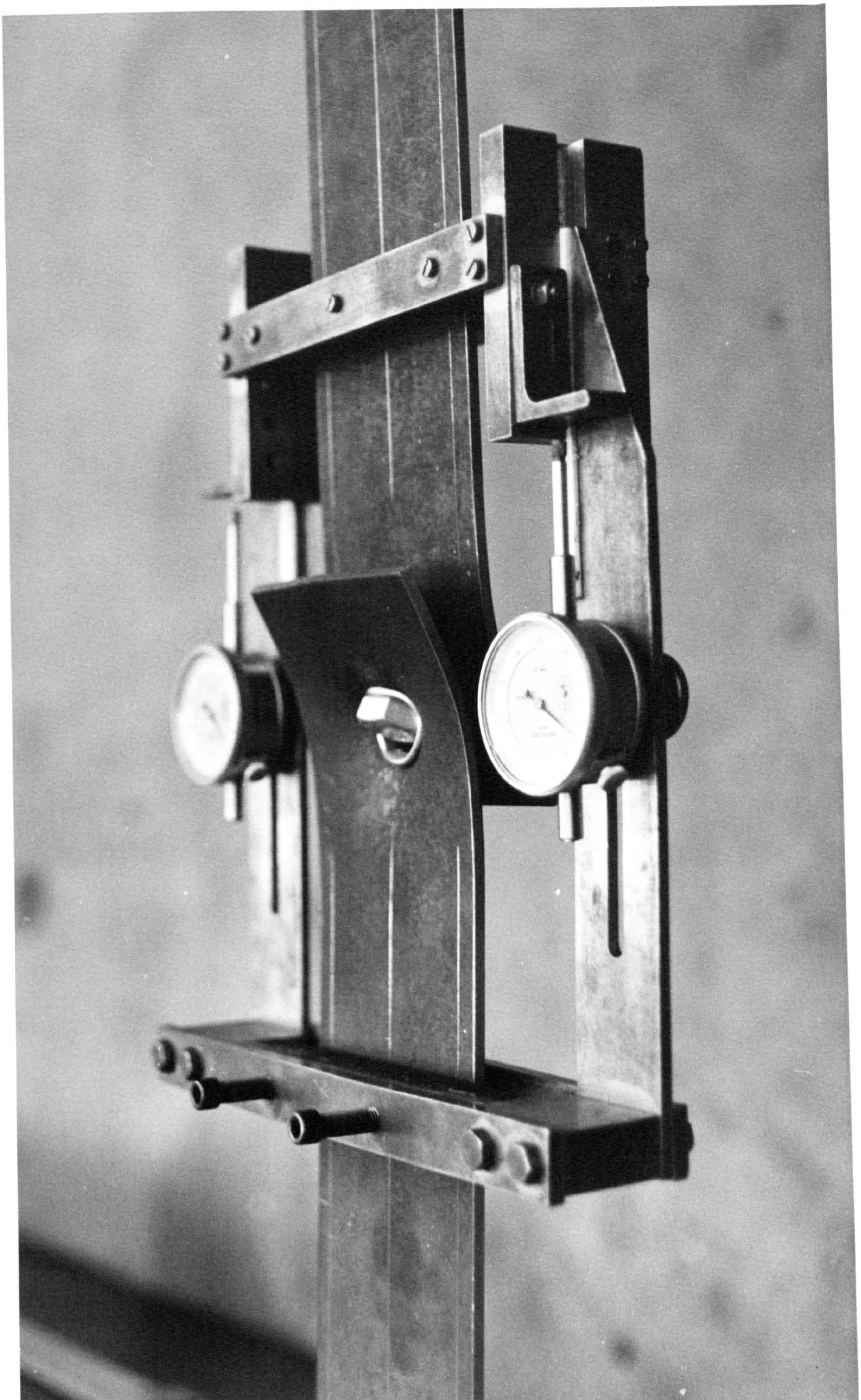


Fig. 1-11. Bolt tilting in a "thicker" specimen (t = 4.8mm)

1.3. Concluding remarks

It is shown that there is scope for innovation , at economical prices, in cold formed building components, and that cold formed steel is an ideal material for production line manufacturing techniques. Hence, it offers a competitive and versatile method of construction .

The provisions of new codes allow main frame sections, as well as the traditional secondary members, to be designed in cold formed steel; and have given a new impetus to the use of cold formed sections in structures.

Particular attention has been drawn to the benefits of components nesting and interlocking - a device not open to the hot rolled sections.

The importance of connections has been emphasized and their modes of failure defined in detail.

Chapter Two

Past tests carried Out at Salford

2. Past tests carried out at Salford

Summary

At Salford, a previous research student and project student had, to some extent, investigated the behaviour of bolted connections in cold formed steel. Their voluminous results were examined and analysed by curve fitting techniques. The general test arrangements and conclusions drawn on their results are described in this chapter. Research carried out by other investigators, considered in writing this chapter are also given in references [27] to [42], in chronological order.

2.1. Research work done by D. Corcoran

Tests carried out by D. Corcoran[24], a previous research student at Salford from 1978 to 1980 are described; the variables considered are discussed and the results obtained are summarized in this section.

2.1.1. Research objectives

The main purpose of the work was to establish load/extension characteristics of bolted connections in cold formed steel; and to determine the effects of various factors such as bolt tilting and sheet thickness upon the ultimate and slip load of such connections. To this aim lap joints where tested in shear.

2.1.2. Variables considered in testing lap joints

With due consideration given to relevant factors affecting the strength and rigidity of a bolted connection, it was decided to investigate the following parameters :

1. **Thickness of the connected sheets**; obviously sheet thickness is the foremost factor influencing the behaviour of a connection.

2. **Backing sheets of varying thickness**; the strength of a connection is primarily dictated by the thinnest of the connected sheets. Backing sheets of varying thickness were used to verify this and see whether they would influence various parameters, such as bolt tilting in any way.

3. **Type of bolt**; High Strength Friction Grip bolts (HSFG) were examined and compared to that of ordinary Black bolts.

   Black bolts (BS 4190[44]) are general mild steel fasteners which may be employed economically in lighter structures where loads are moderate.

   HSFG bolts (BS 4395[45]) differ in their use, in two respects from those of the ordinary black bolts.

(i) The material from which these bolts are made has about twice the tensile strength of ordinary bolts.

(ii) Nuts of high strength are torqued to prescribed amounts, using load indicating washers. The result is to induce a high prestress of approximately 70% of the ultimate tensile load of the bolt, in the shank of the friction grip bolts, to bring the adjoining sheets into intimate contact. This enables shear loads to be transferred by friction developed between the interfaces and makes for rigid connections highly resistant to movement.

With regard to these differences, it was the purpose of these tests to investigate :

(a) Whether the large connection friction, caused by (ii) above, has any beneficial effect on the ultimate load of the connection. Particularly that failure mainly occurs in sheet bearing of the parent material.

(b) To what degree the large connection friction affects the slip load, and if it could be utilized to reduce or eliminate this slip. Bolt slip, as it will be shown later, is a feature of connections with ordinary black bolts.

(c) With identical test parameters, whether the same modes of failure as those of black bolts would be obtained

4. **Bolt diameter**; effect of bolt diameter upon the ultimate and slip load was investigated.

2.1.3. Test parameters

It was decided that the required constants and variables yielding the correct mode of failure (i.e. sheet bearing) were as follows:

2.1.3.1. Test constants

(1) Number of bolts ; One bolt only with a single shear plane.

(2) Nominal 280 N/mm^2 yield steel.
(Actual values reported were : $\sigma_y = 339\ N/mm^2$,
$\sigma_{ult} = 404\ N/mm^2$)

(3) 2 mm clearance holes ; The investigator reported that the sheets were pulled into bearing prior to the tightening of the bolt.

If this was the case, then any connection slip would be eliminated. Load/extension characteristics produced (Fig. 2-3) suggest otherwise.

(4) End distance in the line of stress 60 mm.

i.e.	e/d	=	3.75	for	16 mm dia. bolts.
	e/d	=	3.0	for	20 mm dia. bolts.

2.1.3.2. Test variables

(1) Sheet thickness ; 1.55, 2.45, 3.05 mm thick sheets, tested in all possible (six) combinations.

Tests carried out by G. Winter[29] and later by Baehre and Berggren[31] show that galvanising gives the lowest coefficient of friction, i.e. maximum amount of slip, in bolted cold formed steel connections. Therefore galvanised sheets were used throughout the tests to be more representative of the in-situ conditions.

(2) Type of bolt ; as mentioned previously
Black bolts (grade 4.6) ; and
HSFG bolts (general grade).

(3) Bolt diameter ; 16 and 20 mm dia. bolts.

2.1.4. Specimen dimensions and test procedures

A series of preliminary tests were carried out to determine suitable specimen dimensions for the desired mode of failure (i.e. sheet bearing). Fig. 2-1 shows the final dimensions decided upon. These dimensions were kept constant throughout the tests.

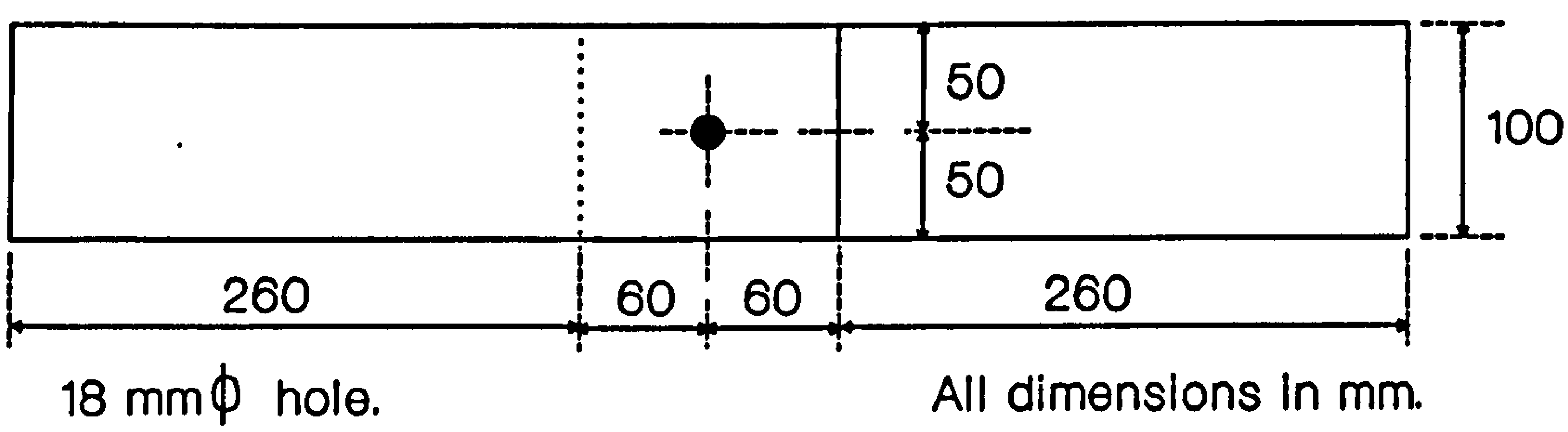


Fig. 2-1 : Specimen dimensions.

Many of the specimens showed a severe out of plane curling of the sheets under load. Since this condition was not desirable a restraint rig was designed to prevent such curling. (Fig. 2-2)

For each variable a minimum of three tests were carried out. For groups that contained inconsistent results further test were carried out until three reasonable sets of results were obtained. Fig. 2-3 shows a typical set of results.

In trying all possible combinations of sheet thickness, type of bolt and bolt diameters, mentioned in § 2.1.3, twenty four groups were obtained (twelve with

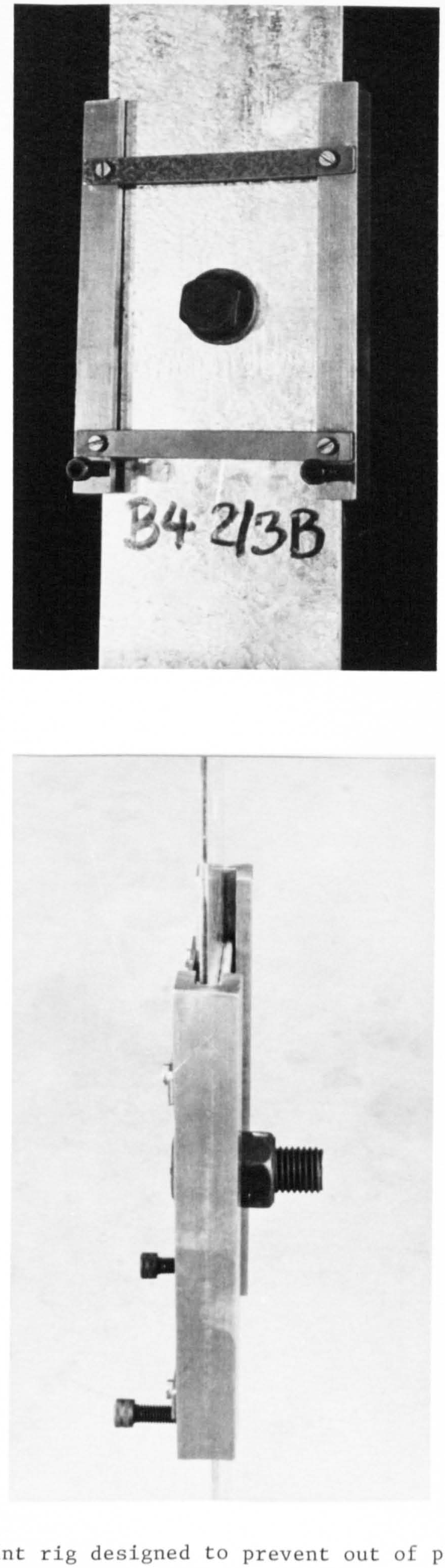


Fig. 2.2 Restraint rig designed to prevent out of plane curling of sheet

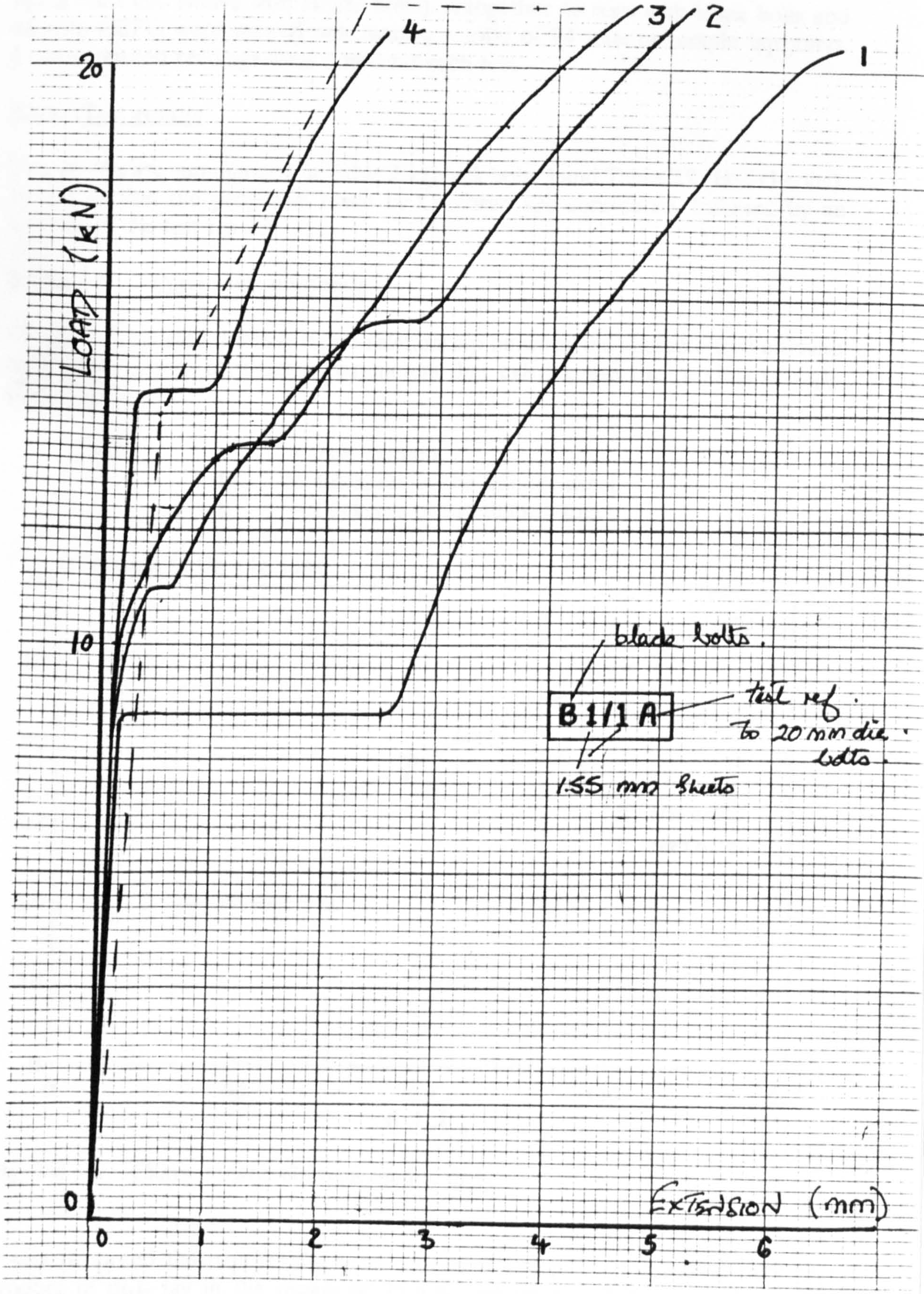


Fig. 2-3 : Typical test result.

black bolts and twelve with HSFG bolts). Altogether 48 tests with black bolts and 46 with HSFG were carried out, making a total of 94 tests on simple lap joints. Load/extension readings were noted for each test.

2.1.5. Test results

Test results are summarized in Table 2.1 (black bolts) and Table 2.2 (HSFG bolts). In this section the proposals made by Corcoran are considered, followed by an analysis of the test results.

2.1.5.1. Corcoran's proposals

Corcoran suggested the only direct relationship between the characteristic ultimate load and different connection parameters that existed, was the product of dt_1t_2. (Fig. 2-4)

Where d = bolt diameter.
t_1 = thickness of the thinner sheet.
t_2 = thickness of the thicker sheet.

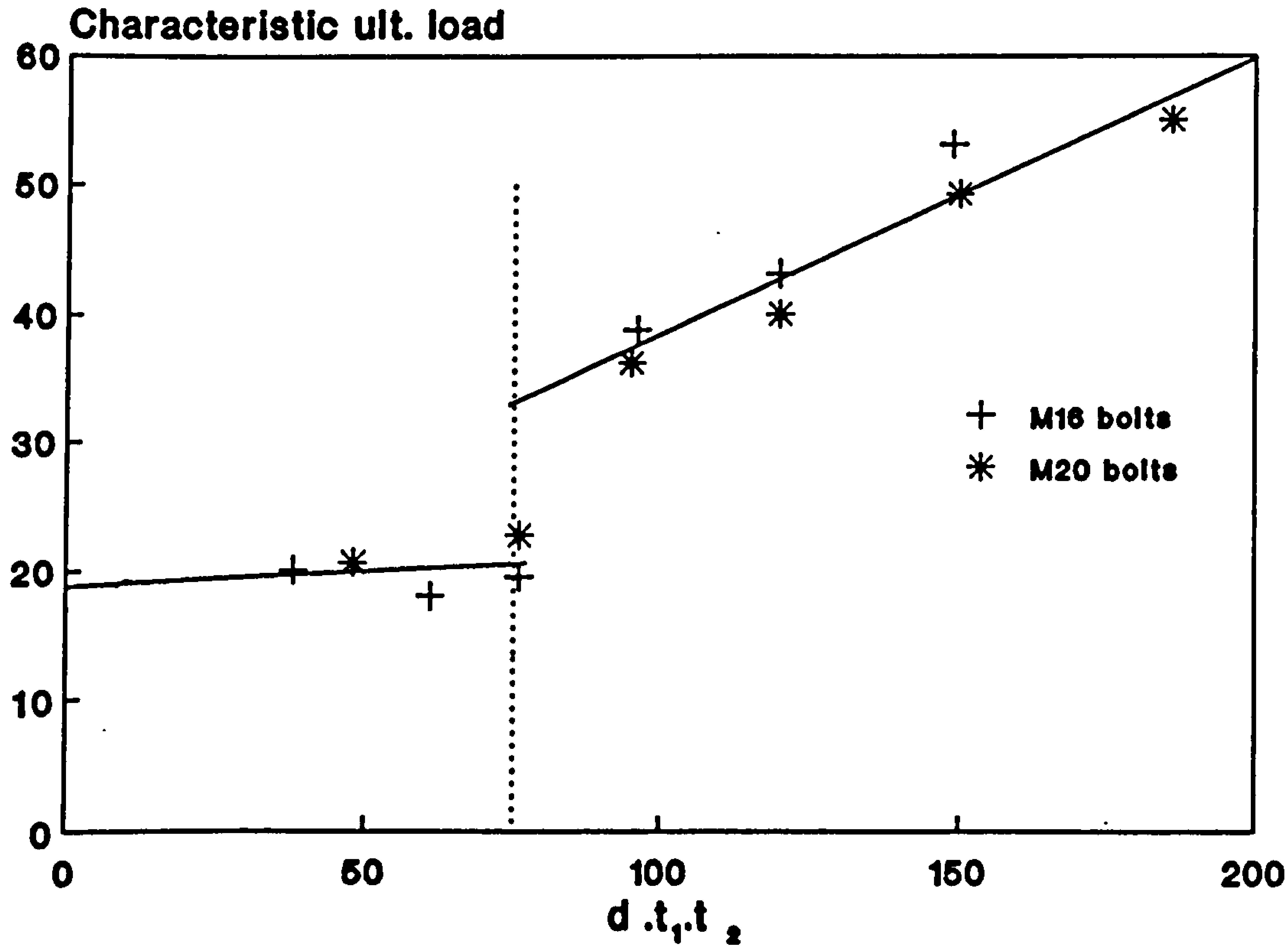


Fig. 2-4 : Characteristic ultimate load v. dt_1t_2

It was proposed that the results were consistent except for the region were the product of dt_1t_2 lay in the region of 75 mm^3, where a well defined split in the relationship could be seen. Bolt tilting was thought to be directly responsible for this discontinuity. For majority of samples, where $dt_1t_2 < 75$ mm^3, bolt tilting was not apparent. However, when $dt_1t_2 > 75$ mm^3 bolt tilting became increasingly significant.

It was further proposed that when a bolt tilts, washer diameter rather than the bolt diameter, is representative of the bearing width and hence the governing factor in determining the ultimate load of the connection. Therefore substituting washer diameter for bolt diameter, for $dt_1t_2 > 76$ mm^3, led to the relationship shown in Fig. 2-5, for **black bolts.**

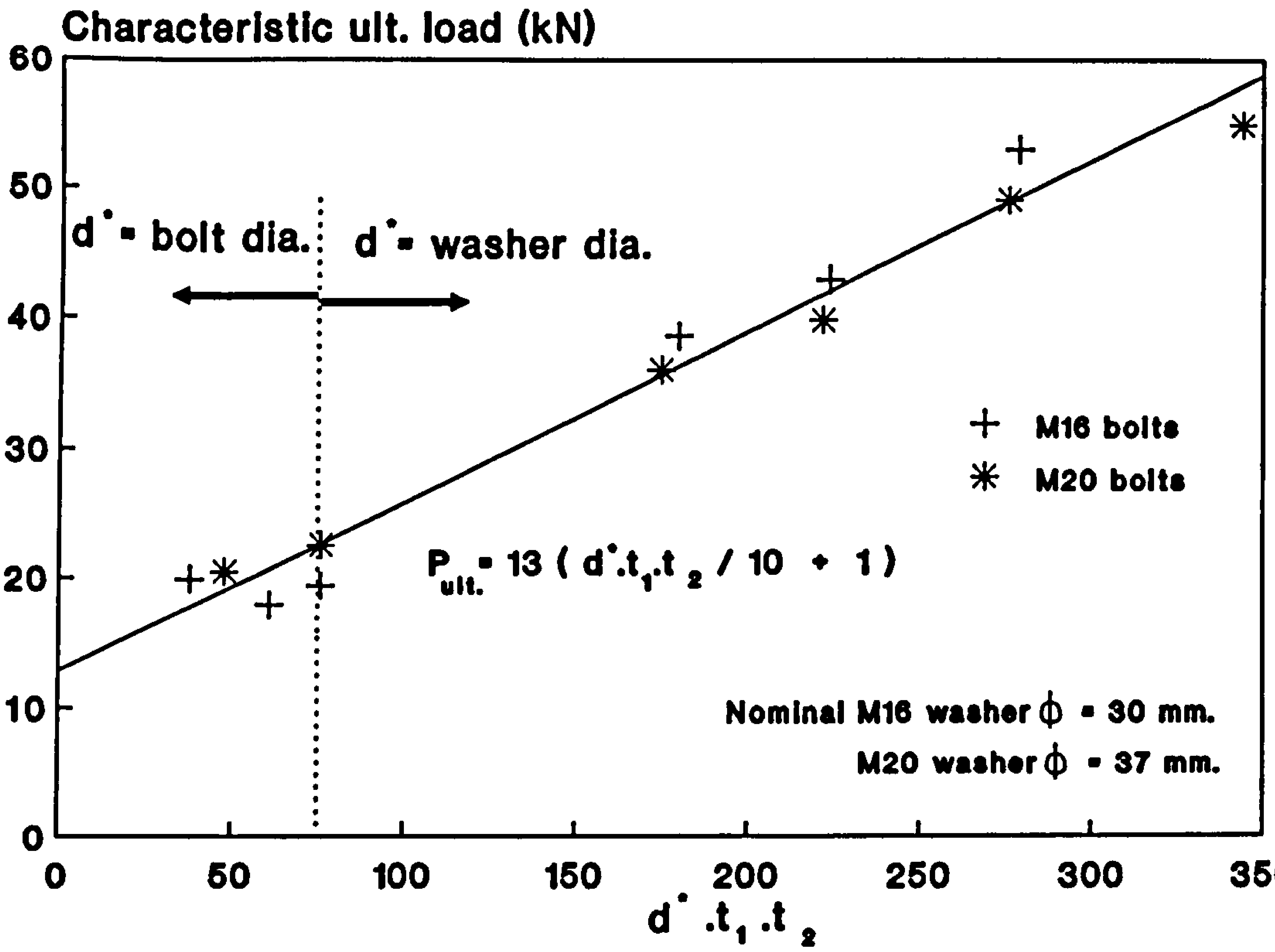


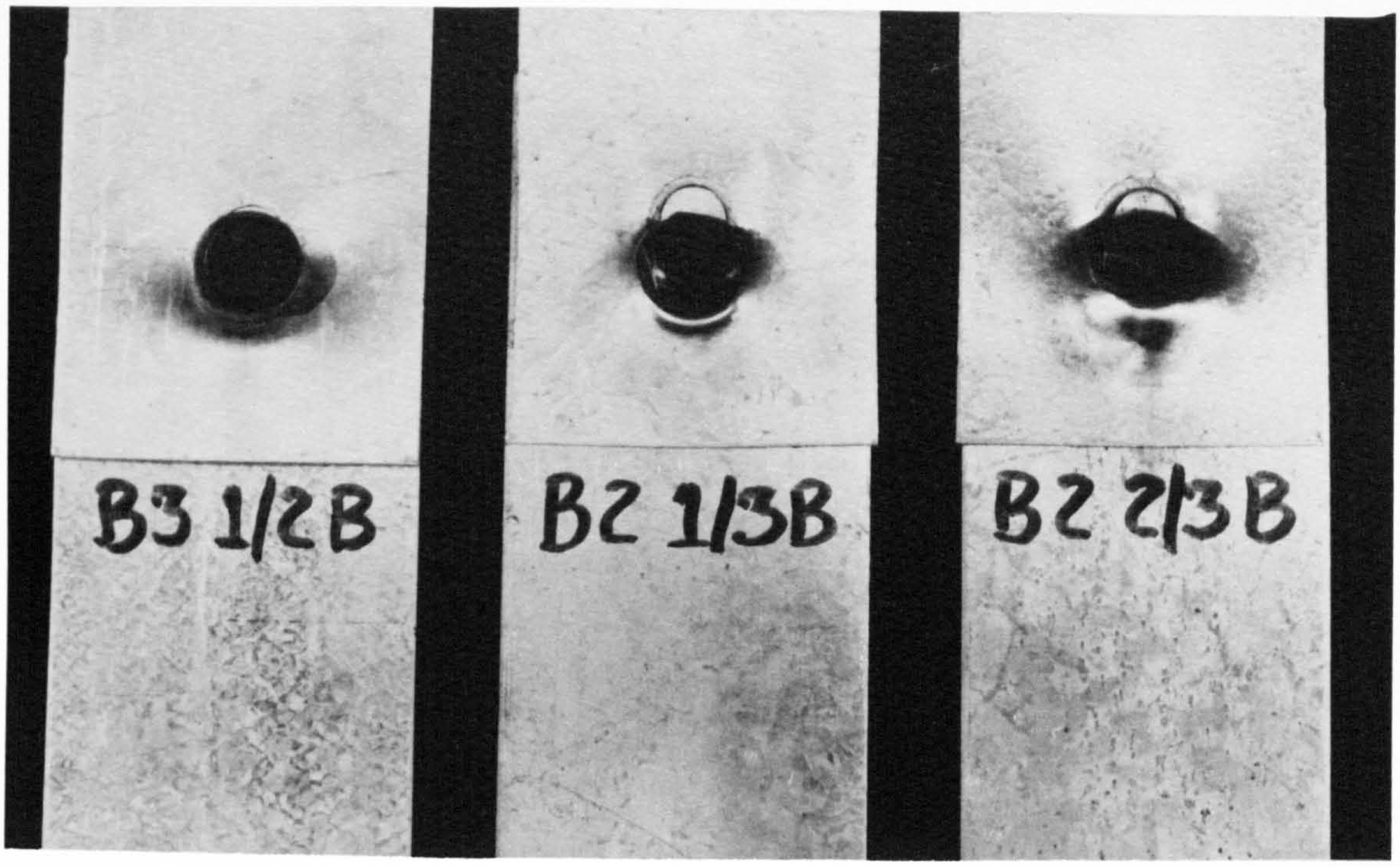
Fig. 2-5 : Using washer dia. in place of bolt dia., for $dt_1t_2 > 76$ mm^3.

After careful consideration of the Corcoran proposals, and with hindsight, it is believed that the observations made by Corcoran may have been particular to his test arrangements. A digression from the first principles outlined in § 1.2.3.1, is not recommended. It should also be noted that the results obtained by Corcoran, [Table 2.1 (page 31, 33) and Table 2.2 (page 37)], at a glance, are well in line with the stated principles, i.e. the strength of a joint is mainly determined by that of the thinnest connected sheet.

2.1.5.2. Black bolts

All specimens failed in sheet bearing with considerable bolt tilting in most cases. (Fig. 2-6)

The results obtained are summarized in Table 2.1



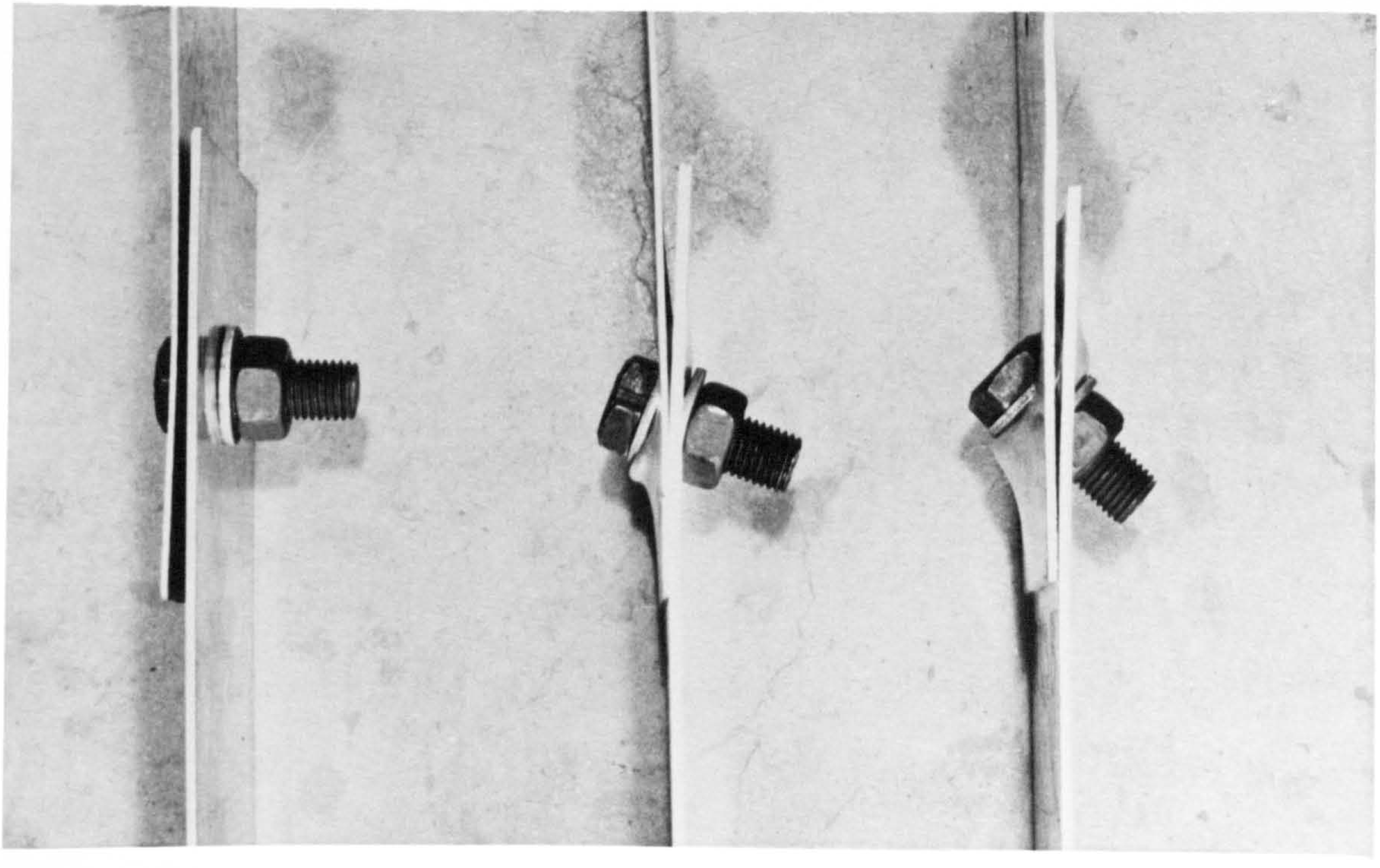


Fig. 2-6. Failed specimens with black bolts

Group No.	Bolt diameter, d (mm)	Top sheet thickness, t_1 (mm)	Backing sheet thickness, t_2 (mm)	No. of tests	Mean ultimate load (kN)	Sample standard deviation from mean	Character-istic# ultimate load (kN)	Mean slip load (kN)	Sample standard deviation from mean	$d.t_1.t_2$ (mm^3)	$d^{*}.t_1.t_2$ (mm^3)
1	16	1.55	1.55	4	22.9	1.9	20.0	8.5	1.7	38	38
2		1.55	2.45	4	27.1	6.2	18.0	9.0	0.9	61	61
3		1.55	3.05	4	29.0	6.4	19.5	8.5	2.1	76	76
4		2.45	2.45	4	41.3	1.7	38.8	9.7	0.5	96	180
5		2.45	3.05	4	45.7	1.8	43.1	8.8	1.9	120	224
6		3.05	3.05	3	53.9	0.5	53.1	10.0	2.0	149	279
7	20	1.55	1.55	4	23.6	2.1	20.6	11.9	2.5	48	48
8		1.55	2.45	4	25.5	1.9	22.7	12.0	2.0	76	76
9		1.55	3.05	4	38.1	1.3	36.2	17.3	1.7	95	175
10		2.45	2.45	5	46.1	4.0	40.0	14.2	3.1	120	222
11		2.45	3.05	4	50.7	1.0	49.2	10.4	3.9	150	276
12		3.05	3.05	4	59.1	2.8	55.0	15.5	1.0	186	344

Table 2.1 : Summary of results for black bolts.

From BS 4320, Nominal : M16 washer diameter = 30 mm.

M20 washer diameter = 37 mm.

Characteristics load has been calculated according to BS 5950 Part 5, Appendix E.

2.1.5.2.1. Ultimate load characteristics

A straight comparison between equivalent groups of equal sheet thickness, but different bolt diameter, shows that the mean ultimate load increased by 8.5% on average, as a result of increasing the bolt diameter from 16 to 20 mm.

2.1.5.2.2. Slip load characteristics

In considering the load at which the connection slipped, that is when the clearance between the bolt and hole diameter was taken out, no correlation could be found between the slip load and the test parameters.

In some cases a great scatter in the slip load and amount of slip, even within the samples in one group, could be found.

It is clearly not possible to produce a graph that is representative of every test, however based on the results produced, typical load/extension characteristics of a lap joint connected with a black bolt might be as illustrated in Fig. 2-7.

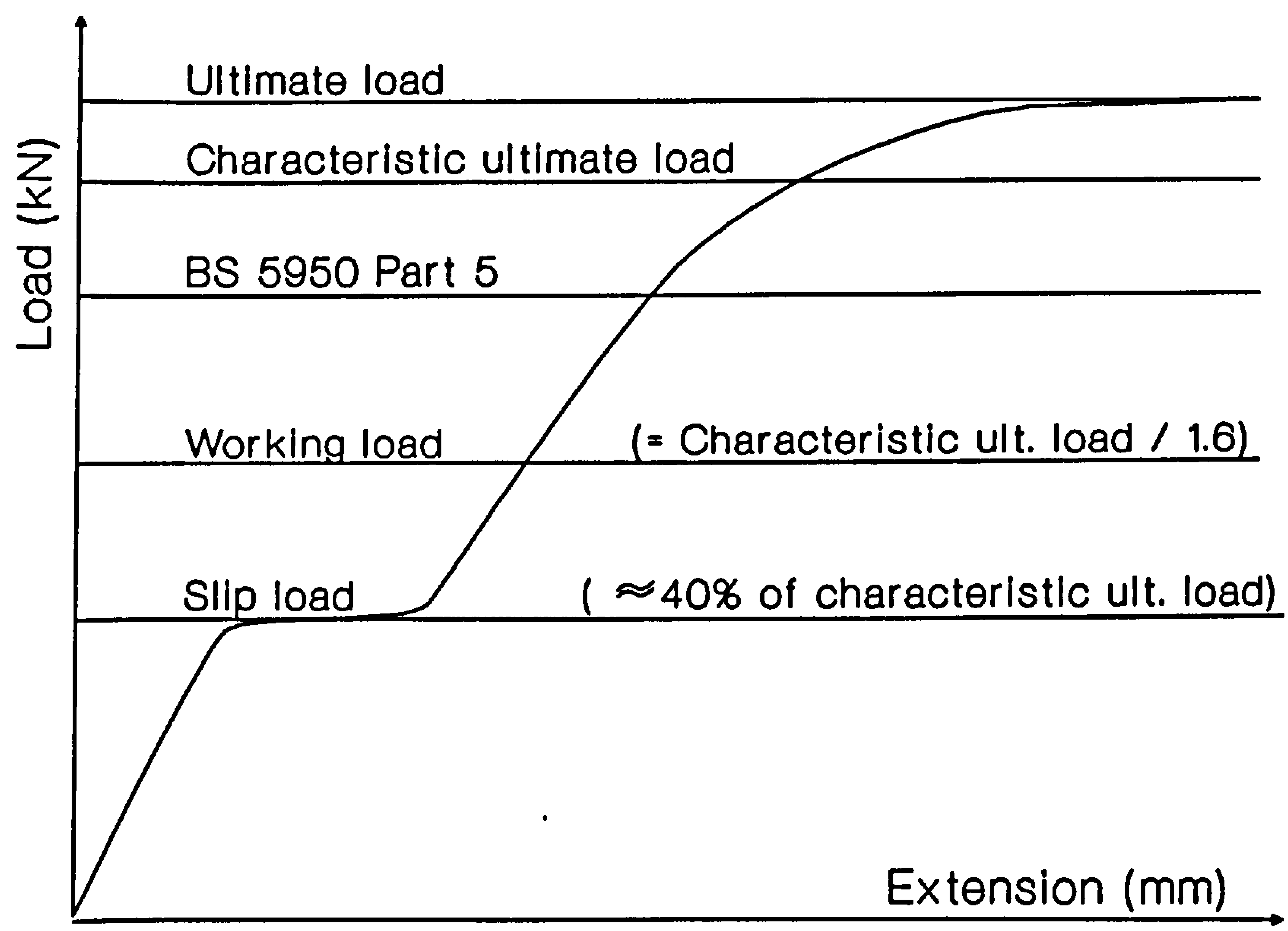


Fig. 2-7 : Typical load-extension characteristics

From Fig. 2-7, the following point should be noted :

- Connection slip cannot be prevented at working load level, when using black bolts in cold formed steel connections.

Furthermore, the results given in Table 2.1, indicate that slip load is independent of sheet thickness. From the same table, can be seen that slip load increased with increase in bolt diameter.

2.1.5.3. High Strength Friction Grip (HSFG) Bolts

The results obtained on HSFG bolts are considered to determine what advantages, if any, could be gained by using this type of fastener in preference to ordinary black bolts.

With regards to tightening of the bolts, it was found that HSFG bolts could be tightened in the thinnest of the sheets tested, up to the manufacturers recommended torques without any local deformation of the sheets.

All specimens failed in a similar way to that of black bolts, i.e. sheet bearing with considerable bolt tilting in most cases. (Fig. 2-8)

Results obtained are summarised in Table 2.2.

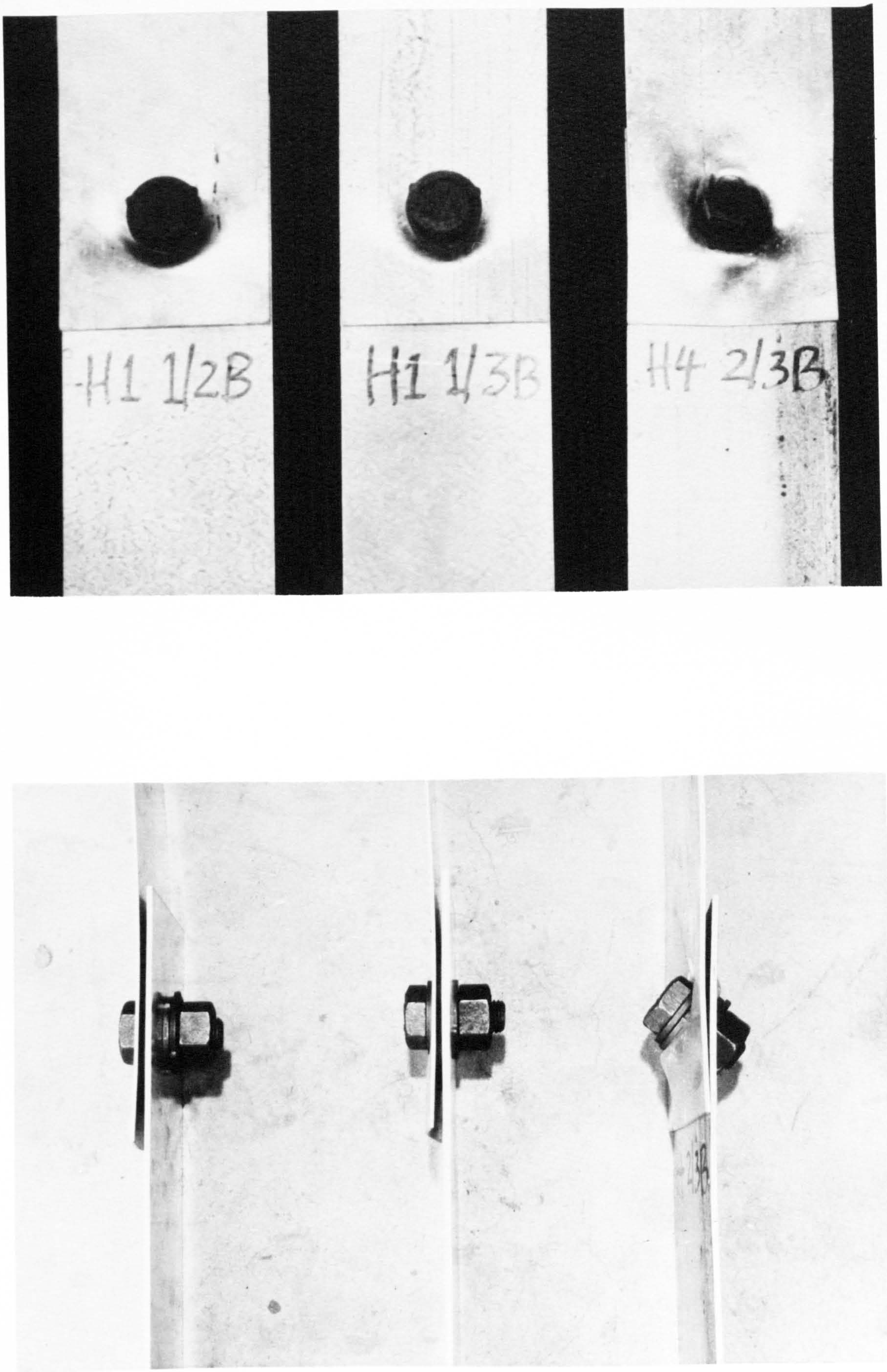


Fig. 2-8. Failed specimens with HSFG bolts

Group No.	Bolt diameter, d (mm)	Top sheet thickness, t_1 (mm)	Backing sheet thickness, t_2 (mm)	No. of tests	Mean ultimate load (kN)	Sample standard deviation from mean	Character-istic ultimate load (kN)	Mean slip load (kN)	Sample standard deviation from mean	$d.t_1.t_2$ (mm^3)	$d^*.t_1.t_2$ (mm^3)
13	16	1.55	1.55	4	37.1	3.1	32.5	-	-	38	38
14		1.55	2.45	4	39.0	1.1	37.4	31.7	23.3	61	61
15		1.55	3.05	4	37.6	2.3	34.3	-	-	76	76
16		2.45	2.45	4	55.9	1.5	53.7	31.9	10.8	96	180
17		2.45	3.05	4	59.0	0.6	58.2	15.7	18.6	120	224
18		3.05	3.05	4	66.9	1.3	65.0	32.4	6.4	149	279
19	20	1.55	1.55	4	48.5	1.7	46.1	-	-	48	48
20		1.55	2.45	4	48.4	2.9	44.2	-	-	76	76
21		1.55	3.05	3	49.3	1.0	47.9	-	-	95	175
22		2.45	2.45	3	61.2	0.3	60.7	27.0	23.4	120	222
23		2.45	3.05	4	67.0	3.0	62.6	45.5	8.3	150	276
24		3.05	3.05	4	75.6	4.8	68.5	54.9	7.9	186	344

Table 2.2 : Summary of results for HSFG bolts.

2.1.5.3.1. Ultimate load characteristics

The percentage increase gained in strength, as a consequence of using HSFG rather than black bolts is tabulated in Table 2.3.

Bolt diameter d, (mm)	Thickness (mm)		Mean ultimate load (kN)		% increase in strength
	top sheet	backing sheet	Black bolt	HSFG bolt	
16	1.55	1.55	22.9	37.1	38%
	1.55	2.45	27.1	39.0	31
	1.55	3.05	29.0	37.6	23
	2.45	2.45	41.3	55.9	26
	2.45	3.05	45.7	59.0	23
	3.05	3.05	53.9	66.9	19
20	1.55	1.55	23.6	48.5	51%
	1.55	2.45	25.5	48.4	47
	1.55	3.05	38.1	49.3	23
	2.45	2.45	46.1	61.2	25
	2.45	3.05	50.7	67.0	24
	3.05	3.05	59.1	75.6	22

Table 2.3 : HSFG bolts compared, in terms of strength, to black bolts.

It is interesting to note that in Table 2.3 the maximum percentage increase in ultimate load occurs for the combination of greater bolt diameter and thinnest sheets. For this group the percentage gain in strength is in the range of 40 to 50% ; for smaller bolt diameter and increasing sheet thickness it gradually reduces to 19%.

A detailed study of the test results showed that the increase in strength obtained for smaller sheet thickness and larger bolt diameter is due to the ultimate load of the connected elements being less than the slip load of the friction bolt, giving the equivalent of a rigid joint. Therefore, failure occurs when slip load occurs.

The friction force which resists connection slip depends on two factors:

(i) the bolt tension

(ii) the coefficient of friction between the connected surfaces.

The low coefficient of friction resulting from galvanised sheets reduces the friction force that can be developed between the bolt and the sheets, thus reducing the effectiveness of the bolts. Conversely, as the sheet thickness increases, the ultimate load of the sheets approaches and exceeds the slip load of the friction bolt, thus reducing the rigidity mentioned for the thinner sheets, and with it the effectiveness of the friction bolts. Hence the reduction in strength with increase in sheet thickness and the smaller bolt diameter observed above.

From Table 2.2, can be seen that the mean ultimate load increased by 14.6%, on average, as a result using a larger diameter bolt.

2.1.5.4. Slip load characteristics

As mentioned above, for the thinner range of sheets, the ultimate load of connected elements were often less than the slip load of the bolt; in such cases a slip load has not been noted in Table 2.2. Only in cases where at least half of the samples in a group slipped, is there any indication of the slip load. Also for such cases a standard deviation greater than 15 indicates that at least one or two of the samples in that group did not slip at all.

Slip load in general increased with increase in bolt diameter.

In conclusion, it may be said that the main advantage of using HSFG bolts over black bolts is that the connection slip is prevented at working load level.

2.1.6. Conclusions drawn on Corcoran's results

- Connection slip cannot be maintained at working load levels, when using black bolts.

- Slip load of a connection is independent of the sheet thickness (with black bolts).

- Results indicate that the slip load is proportional to bolt diameter. However, on the basis of the Corcoran test results alone it is not possible to justify this, since bolt torques were not monitored at the time of testing.

- In bolted connections in light gauge steel, where failure often occurs in the connected elements rather than the fastener, the use of bolts larger than 16 mm diameter does not justify the extra cost. In any case, percentage strength gained as a result of increasing the bolt diameter from 16 mm to 20 mm was, on average, below 10% .

- Given the same parameters, the modes of failure for both black and HSFG bolted connections, are identical.

- HSFG bolts are most effective when used with the thinnest of sheets, where the ultimate load of the sheets is less than the slip load of the bolt, hence resulting in a perfectly rigid joint until failure.

- On average, over the whole range of sheet thicknesses tested, the ultimate load increased by about 30% as a result of using HSFG bolts, compared to that of black bolts. The relatively small increase in the strength, as a result of using HSFG bolts, does not justify the extra costs involved in using these bolts. This is partly due to the low coefficient of friction, because of "surface treatment" that cold formed steel receives.

- Although the use of HSFG bolts is not economical in terms of increase in the ultimate strength, in view of large connection friction at the design loads HSFG bolts may prove advantageous where it is necessary to ensure a perfectly rigid joint at the design loads.

2.2. Research work done by G. Geha[25]

In the second part of this chapter, tests carried out by G. Geha, a previous research project student at Salford in 1984, are described and the results obtained are discussed.

2.2.1. Scope of the Research

From Corcoran's work, two important points with regard to bolted connections in cold formed steel were established. Namely ;

1. Use of HSFG bolts is not economical in cold formed steel connections.

2. 16 mm diameter bolts are adequate for all structural connections in light gauge steel. Hence a larger 20 mm diameter bolt would be of no practical importance in the industry.

It was decided to build on Corcoran's work and investigate further parameters not considered by him.

2.2.2. Test parameters

The following variables were considered in testing.

2.2.2.1. Test variables

(1) **Hole diameter;** 16.1 mm (perfect fit)
17.5 mm

The latter is classed as medium series clearance hole. (European Recommendation[16], § A.3.7.3)

(2) **Number of washers;**
- Two
- One under bolt head
- One under the nut
- None

(3) **Bolt torque;** 110, 120 and 135 N.m

(4) **End distance in line of stress;** expressed in terms of e/d.

Minimum	e/d	= 1.5
Maximum	e/d	= 6.0
Increments		= 0.75

2.2.2.2. Test constants

The following parameters were applied to every test.

(1)* Sheet thickness; 1.5 mm thick sheets of equal thickness for each connection (i.e. top and backing sheets of equal thickness).

A few tests were carried out with 2.4 mm thick sheets.

(2) Type and diameter of bolt - 16 mm diameter black bolts were used, as this is often the case in practice.

(3) Nominal 280 N/mm^2 yield steel.
(Actual values reported : σ_y = 314.8 N/mm^2 and;
σ_{ult} = 394.6 N/mm^2)

(4)* End distance in line of stress 60 mm, e/d = 3.75 .

(5)* Hole diameter, ϕ = 17.5 mm.

As with the previous investigator the specimens were reported to be pulled into bearing prior to the tightening of the bolt, but the load/extension characteristics plotted, contradict this.

(6)* Two washers, one under the bolt head and one under the nut .

(7)* Bolt torque 120 N.m.

After a series of preliminary tests, it was decided to keep to exactly the same specimen dimensions and specifications as that of Corcoran. (Fig. 2-1, page 27).

The test procedures adopted were also exactly the same.

2.2.3. Test results

All specimens failed in sheet bearing unless stated otherwise. Bolt tilting was reported to be present in the majority of cases.

The results obtained were as follows;

2.2.3.1. Hole tolerance

Perfect fit and clearance hole lap joints were tested in combination with varying number of washers. The purpose of these tests were to establish to what extent the use of washers affect the ultimate and slip loads of a connection. With perfect fit holes the effect of washers on slip load would obviously be eliminated, and its effect on the ultimate load was isolated.

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* Parameter should be regarded as "standard", unless a variable itself.

2.2.3.1.1. Use of washers

In the North America in particular, there is a tendency to accomplish light gauge steel bolted connections without any washers. Earlier work by Winter[27] had shown that the bearing capacity of bolted connections with washers was equal to :

$$\sigma_b = 4.9\ \sigma_y \qquad (e/d>3.5)$$

The above equation was based on a wide range of variables, i.e. sheet thicknesses of 0.9 to 4.1 mm, bolts of 6 to 25 mm diameter , lap joints with one and two bolts in single and double shear.

It was later shown by Chong and Matlock[34] that the bearing capacity of connections without washers, is reduced to ;

$$\sigma_b = 2.7\ \sigma_y \qquad (e/d>2.5)$$

based on tests carried out on a thickness range of 0.9 mm to 2.6 mm (with different surface coatings), and 5/16, 1/2 and 3/4" dia. bolts. That is a reduction of 45% compared to Winter's equation, given above.

However, it was later shown by Rhodes and Loughlan[40] that an ultimate bearing stress of 4.9 σ_y (for connections with two washers), originally proposed by Winter, was unsafe and :

$$\sigma_b = 4.2\ \sigma_y$$

was a more representative value. This latter equation is actually used for the ultimate bearing stress in the AISI, but expressed in terms of σ_u (i.e. $\sigma_b = 3.0\ \sigma_u$).

As a result of the above mentioned findings, the permissible bearing stress of bolted connections has been reduced by 26% (from 3.0 σ_u to 2.22 σ_u) in the AISI specifications.

A similar factor (25%) has been implemented in BS 5950 pt.5, for cases where either one or both washers are discarded.

Tests were carried out by Geha to verify the above.

Washers with perfect fit holes

Test results are shown in Table 2.4.

Results suggest the following :

- One washer under the nut is adequate to develop the full bearing capacity.

- The washer under the bolt head, has no significance on the ultimate load.

Type of assembly $\phi = 16.1$ mm	Mean Ultimate load (kN)	% of the maximum ultimate load
Two washers (one under bolt head and one under nut)	28.7	100%
One washer under nut	28.5	99%
One washer under bolt head	22.2	77%
No washers	22.9	80%

Table 2.4 : Effect of washers; perfect fit holes.

<u>Washers with clearance holes</u>

Results are shown in Table 2.5.

Type of assembly $\phi = 17.5$ mm	Mean ultimate load (kN)	% of the maximum ultimate load	Mean slip load (kN)
Two washers (one under bolt head and one under nut)	28.0	100%	14.0
One washer under nut	22.5	80%	13.6
One washer under bolt head	20.3	73%	13.4
No washers	19.2	69%	11.9

Table 2.5 : Effect of washers; clearance holes.

It is seen that the use of washers increased the slip load by about 15%. Connection slip, on average, occurred at a load of 13 kN.

With respect to the ultimate load, the results are intuitively more agreeable compared to perfect fit holes. There is however further evidence that as with the perfect fit holes, the washer under the nut is more effective than the washer under the bolt head.

Comparing the ultimate loads for clearance holes and perfect fit holes (with two washers), it is evident that hole tolerance has no significance on the ultimate load.

The reduction in bearing capacity in terms of the non-dimensionalized factor α, as a result of not using washers is as follows. (Table 2.5)

Two washers α $= P_{ult}/dt\sigma_y$
$= 28.0 / (16 \times 1.5 \times 314.8)$
$= 3.7$

No washers α $= 19.2 / (16 \times 1.5 \times 314.8) = 2.5$

i.e. $((3.7 - 2.5) / 3.7) \times 100 = 32\%$

It therefore appears that the 25% reduction in strength as a result of omitting washers, adopted by BS 5950 pt.5, is unsafe. In the case of AISI, this is absorbed by an excessive safety factor of 2.22.

2.2.3.2. Effect of bolt torque, T

Three identical connections, except for their bolt torques, were tested to failure. All three specimens slipped and the bolt clearance was taken up before failure.

Results are tabulated below.

Applied torque N.m	Ultimate load kN	Slip load kN
110	27.5	12
120	27.2	13
135	27.7	14.5

Table 2.6 : Effect of bolt torque.

Results clearly indicate that :

- Connection strength is independent of bolt torque.
- Slip load is proportional to bolt torque, where a "friction grip" type effect is indicated.

Conclusions drawn above are in line with results produced by Chong and Matlock[34].

2.2.3.3. Effect of end distance, e

Tests were carried out on 1.5 and 2.4 mm thick sheets.

Test approach adopted was to start with a shortest end distance of 24 mm, e/d = 1.5, and to increase this in increments of 12 mm, e/d increments of 0.75, up to a longest end distance of 96 mm, e/d = 6.0, in the case of 1.5 mm thick sheets; and 60 mm for 2.4 mm sheets.

2.2.3.3.1. Thickness of sheet, t = 1.5 mm

Two different modes of failure were observed.

(i) failure by sheet tearing, for $e/d < 2.25$.
(ii) failure by sheet bearing for $e/d \geq 2.25$.

The test results, together with the predicted ultimate loads given by BS 5950 pt.5 are tabulated in Table 2.7.

Overall the values given by BS 5950 pt.5 are on the conservative side.

Test results also indicate that ultimate load increases up to a (e/d) ratio equal to 5. This is contrary to the results obtained by the author and elsewhere[27][34][40].

2.2.3.3.2. Thickness of sheet, t = 2.4 mm

Results are only available for $e/d \leq 3.75$.

Three modes of failure were noted:

(i) failure by sheet tearing, for $e/d < 1.5$
(ii) failure by sheet bearing, for $e/d = 2.25$
(iii) failure by shear of bolt, for $e/d > 2.25$.

In the previous chapter, it was shown from first principles that the shearing of bolt mode of failure is independent of end distance of the fastener in the line of stress. On this basis the results produced are irrational, and definitely not in line with what will follow in later chapters.

One possible explanation for the uncharacteristic results obtained with respect to variation in end distance (for either sheet thicknesses), is the very high initial bolt torque of 120 N.m used in the tests. To give the reader a feel in this matter; in order to obtain a bolt torque this high, the nut has to be turned very hard indeed. Tests carried out by the author indicate that a bolt torque of 120 N.m induces an axial stress in the order of 190 N/mm^2 in a black bolt. This, according to BS 5950 pt.5 Table 11, is equal to the ultimate tensile stress of such bolts.

End distance (mm)	e/d	P_{ult} (from tests) (kN)	P_{bs} BS 5950 pt.5 (kN)	Slip load (kN)	α (= $P_{ult}/dt\sigma_y$) Tests	α (= $P_{bs}/dt\sigma_y$) BS 5950 pt.5	Failure mode
24	1.50	17.7	15.9	9.7	2.34	2.10	Sheet tearing
36	2.25	22.8	16.7	5.9	3.02	2.21	Sheet bearing
48	3.00	24.5	17.6	6.5	3.24	2.33	Sheet bearing
60	3.75	28.0	17.6		3.71	2.33	Sheet bearing
72	4.50	33.6	17.6	9.9	4.45	2.33	Sheet bearing
84	5.25	35.6	17.6	12.1	4.71	2.33	Sheet bearing
96	6.00	35.7	17.6	19.0	4.73	2.33	Sheet bearing

Table 2.7 : Variation of ultimate load with change in end distance, e.

d = bolt dia., 16 mm
t = sheet thickness, 1.5 mm
σ_y = yield stress of material, 314.8 N/mm^2

2.3. Conclusions drawn on Geha's results

- Hole tolerance has no significance on the ultimate load of a connection.
- The ultimate bearing capacity of a bolted connection is reduced by about 30%, as a result of omitting washers. On this basis a reduction of 25% proposed in BS 5950 pt.5 is unsafe.
- Slip load is only marginally dependent on the use of washers.
- Test results indicate that a washer under the nut is more effective than a washer under the bolt head.
- The ultimate load of a bolted connection is independent of the bolt torque.
- The slip load of a bolted connection is proportional to the bolt torque.

Chapter Three

Preliminary tests and experimental procedures in testing of lap joints

3. Preliminary tests and experimental procedures in testing of lap joints

Summary

In this chapter, current directives on testing of bolted connections in cold formed steel are outlined. A series of preliminary tests, carried out to standardize the succeeding test programme on bolted lap joints is described. Based on findings further recommendations on testing of fastenings to represent structural bolted connections in cold formed steel sections are made.

3.1. Introduction

The objectives of the thesis were clearly laid out in chapter one. That is, to consider practical factors in bolted connections, in cold formed steel sections, not previously or adequately accounted for. The research projects described in chapter two proved very useful in the early stages of this work. However, they only sufficed so far as giving some scope to the work that needed to be done - eliminating factors such as the type and diameter of fasteners and indeed raising others, such as whether the use of washers was adequately accounted for in the current codes of practice, or whether the end distance in line of stress did really have a bearing on the failure of the fastener as well as the fastening.

In order to realise the above objectives a series of preliminary tests was carried out to ensure that the subsequent testing and test procedures would represent the insitu conditions and at the same time yield the maximum amount of information with a minimum number of tests.

The main point of focus was the specifications drawn by the current codes of practice in testing of bolted lap joints in cold formed steel. These are briefly outlined below.

3.2. Testing of fastenings in cold formed steel

BS 5950 Part 5 does not give specific rules for testing of connections in cold formed steel. Instead, in § 10.5, it refers the reader to European Recommendations where definite guidelines have been drawn up by the ECCS (working group TC7).

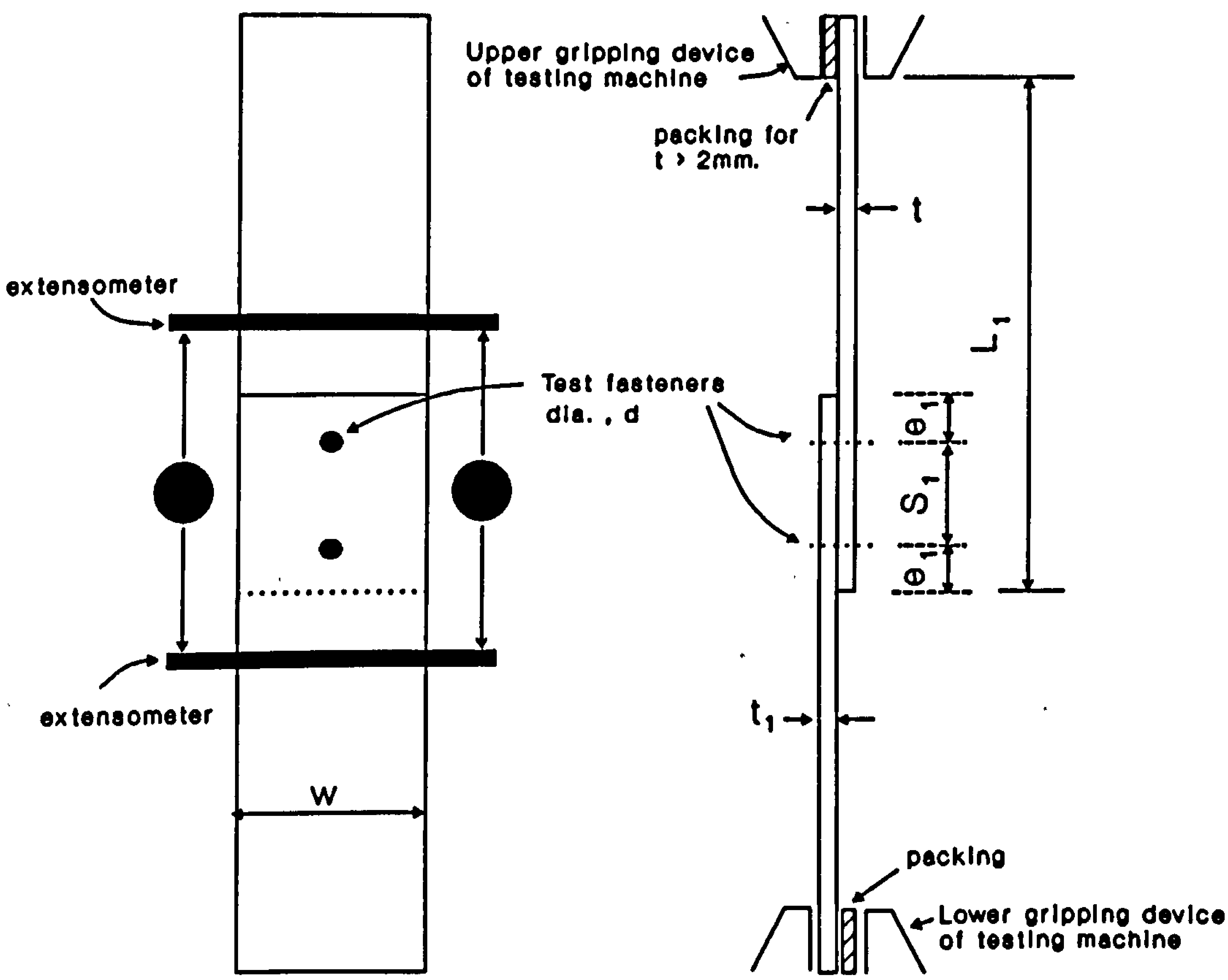
The procedures drawn by the ECCS in testing of connections in cold formed steel are essentially in two categories. First, the European Recommendation (E.R.) Publication No. 35, reference [16], which covers tests to determine the strength and rigidity of all common types of fasteners in light gauge steel. Second, the E.R. Publication No. 21, reference [46], comprising of test procedures to determine the properties of the fastenings.

The latter recommendation was obviously of main interest to this project. The important points, regarding testing of lap joints in shear, are described below.

3.2.1. Standard shear test of fastenings in light gauge steel

3.2.1.1. Specimen dimensions

The standard shear test commonly known as the "single lap joint shear test", in which a two fastener lap joint of standard dimensions is placed in testing machine and loaded to failure. The standard dimensions (for d > 6.5 mm) are shown in Fig. 3-1.



Fastener dia., d (mm)	Specimen dimensions, (mm)			
	w	L_1	e_1	S_1
> 6.5	10d	200 + 10d	5d	10d
Tolerance	± 2	± 5	± 1	± 1

Fig. 3-1 : Standard shear test specimen specified by the E.R.[46].

As can be seen the test incorporates two fasteners and the results obtained are an average of the two fasteners tested.

The E.R. allows a single fastener test to be used where it is deemed to be more representative of the conditions being tested than a double fastener test. In such cases the specimens length may be adjusted accordingly to provide the end distance required for the tests. The value of which should be in accordance with e_1 above, unless the end distance is being investigated itself.

3.2.1.2. Testing procedure

It is specified that during testing the rate of loading shall not exceed 1 kN/min. and rate of straining shall not exceed 1 mm/ min. Faster rates of loading may artificially lead to higher test results in terms of strength and stiffness of the connection.

3.2.1.3. Number of tests

The minimum number of tests required from which the fastening properties may be estimated, is influenced by the variability of the results. It has been specified that for test series that include one nominal cross section but several nominal thicknesses, where the actual difference between the thicknesses is at least 0.1 mm, a minimum of three tests should be carried out for each sheet thickness. The same principle applies when the mechanical properties of the sheets differ by more than 30 N/mm^2.

3.2.1.4. Deformation limit placed on the ultimate load

The ultimate load is defined as the lesser of the load recorded within 3 mm deformation or the load at which the first drop in the load-extension characteristics occurs. (Fig. 3-2)

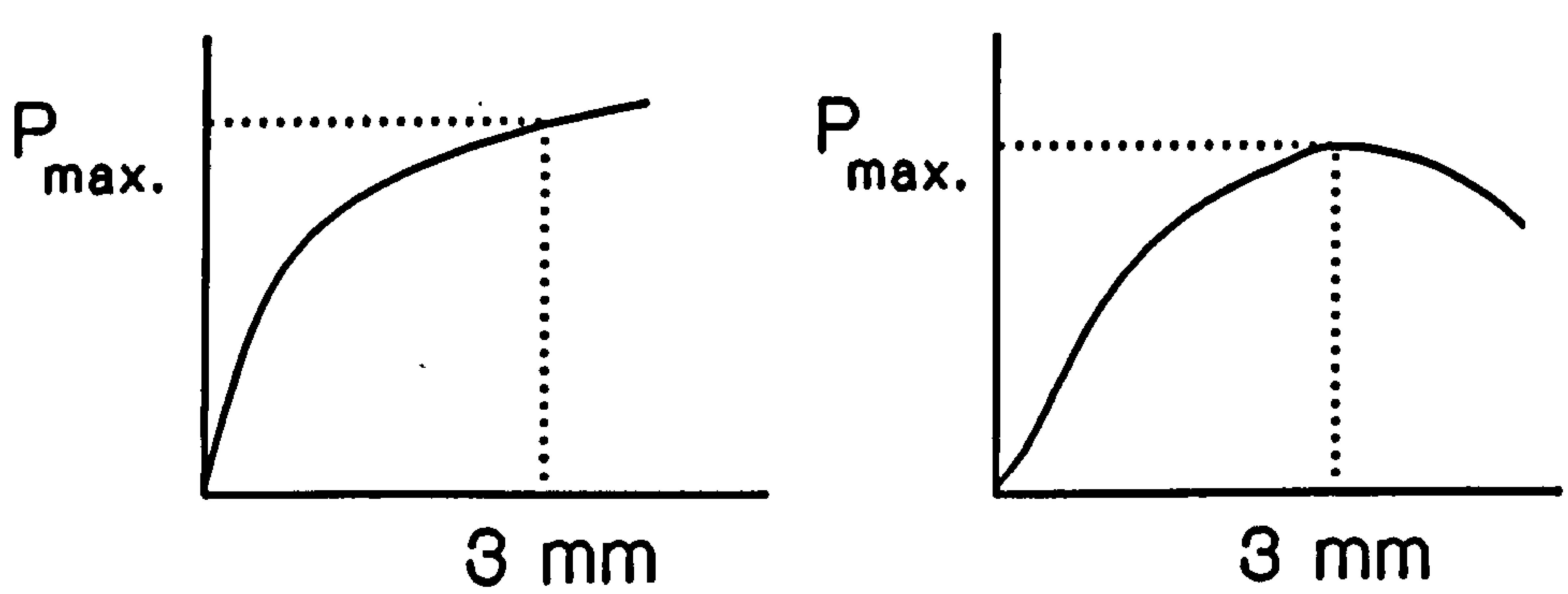


Fig. 3-2 : Deformation limit set on the ultimate load.

3.3. Scope of the tests

The dimensions specified by the European Recommendation summarized above, seemed to be too excessive. A detailed literature survey to the background of above specifications revealed that they were originally intended for testing of connections in roof and wall sheets to the under-structure. The strength and stiffness of such fastenings would often be relied upon in diaphragm action. This work has already been referred to in § 1.2.2. Reference [19] gives the background study to Fig. 3-1.

The diameters of fasteners considered in the above study were in the range of 4.8 to 6.3 mm.

It was obviously desirable both in terms of saving material and reducing test time, to test samples with one bolt only, made up of less onerous dimensions. Considering the range of tests intended at Salford, with 12 and 16 mm dia. bolts a specimen width of 10d (=160 mm) would not even fit in the test rig. The maximum feasible width was 150 mm.

As well as the main objective, i.e. to find the most economical and insitu-representative dimensions, it was also desirable to carry out calibration tests in order to standardize other factors such as restraints to out of plane movement of the sheets, the type of bolts and washers used, the bolt torque and the end distance in the line of stress, etc. in the subsequent testing.

To this purpose twenty lap joints were tested in shear with one and two bolts in the line of stress. The test parameters and results are described below.

3.4. Test parameters

3.4.1. Sheet thickness

Sheets at both the "thinner" and "thicker" range of cold formed steel sections were tested. The sheets properties were as follows:

Specimen thickness (mm)	Yield stress (N/mm^2)	Ultimate stress (N/mm^2)
1.73	331.9	379.7
3.00	280.3	383.2

All the sheets were galvanised

Specimens consisted of sheets of the same thickness.

Table 3.1 : Mechanical properties

3.4.2. Type of fastener

M16 x 50mm plated Grade 4.6 bolts and;
M16 x 70mm Grade 4.6 set screws were tested.

It has already been mentioned that most structural bolted connections in cold formed steel nowadays, are accomplished with galvanised 16 mm ϕ grade 4.6 bolts. From a structural point of view galvanising affects the coefficient of friction between the bolt and the parent material.

Set screws are differentiated from "bolts" here to refer to fasteners without a plain shank. That is, with set screws the threads run right up to the fastener head.

With bolts, the length of the plain shank between the bolt head and the last thread depends entirely on the bolt length. BS 4190[44] specifies a thread length of 2d + 6 mm for all bolts. To provide for structural applications where threads in the shear plane are not desired, bolts may be ordered with an alternative shorter length of 1½ d.

The most common lengths of 16 mm ϕ bolts used in structural connections are 30 or 35 mm. Therefore strictly speaking they are set screws. It follows that the occurrence of bolt threads in the shear plane of a connection is the norm, in cold formed sections.

One purpose of the tests was to examine to what extent the occurrence of the bolt threads in the shear plane affects the strength and rigidity of the connections.

3.4.3. Type and number of washers

Two washers were used with every test, one under the bolt head and one under the nut.

Soft commercial washers and precision engineering (hard steel) machined washers were used to examine their effect on the bearing width.

All washers were large dia., classified as Form F in BS 4320, with an outside dia. of 34 mm.

3.4.4. Number of fasteners in line of stress

Lap joints with a single fastener and;
Lap joints two bolts in the line of stress were tested.

Different bolt spacings of e, 1½e and 2e were examined, with specimens with two bolts in the line of stress. (e = end distance in the line of stress)

3.4.5. Specimen dimensions

The specimen dimensions specified by the E.R.[46], and smaller measurements were examined to obtain the most economical test dimensions.

3.4.6. Hole tolerance

Perfect fit (16mm ϕ) holes were used to eliminate the effect of clearance slip at this stage.

All bolt shanks were "turned and fitted" to obtain a perfect fit fastening.

3.4.7. Bolt torque, T

Obviously with perfect fit holes the effect of bolt torque on the slip load is eliminated. Nevertheless different torque settings ranging from 60 to 100 N.m were applied to further ensure that bolt torque did not affect the ultimate load in any way.

Further experiments were carried out, as will be described later, to standardize the bolt torque for 16 mm ϕ bolts in future tests.

3.5. Test method

The specimens were tested in a 100 tonne Losenhausenwerk universal testing machine. An extensometer with two dial gauges, each with an accuracy of 0.01 mm, was used to record the extension readings. Loads were applied and noted at a rate of 1 kN/minute. In addition load/extension curves were recorded by the machine to ensure an accurate register of data up to the failure load. The general test arrangement is shown in Fig. 3-3.

3.5.1. Practical test considerations

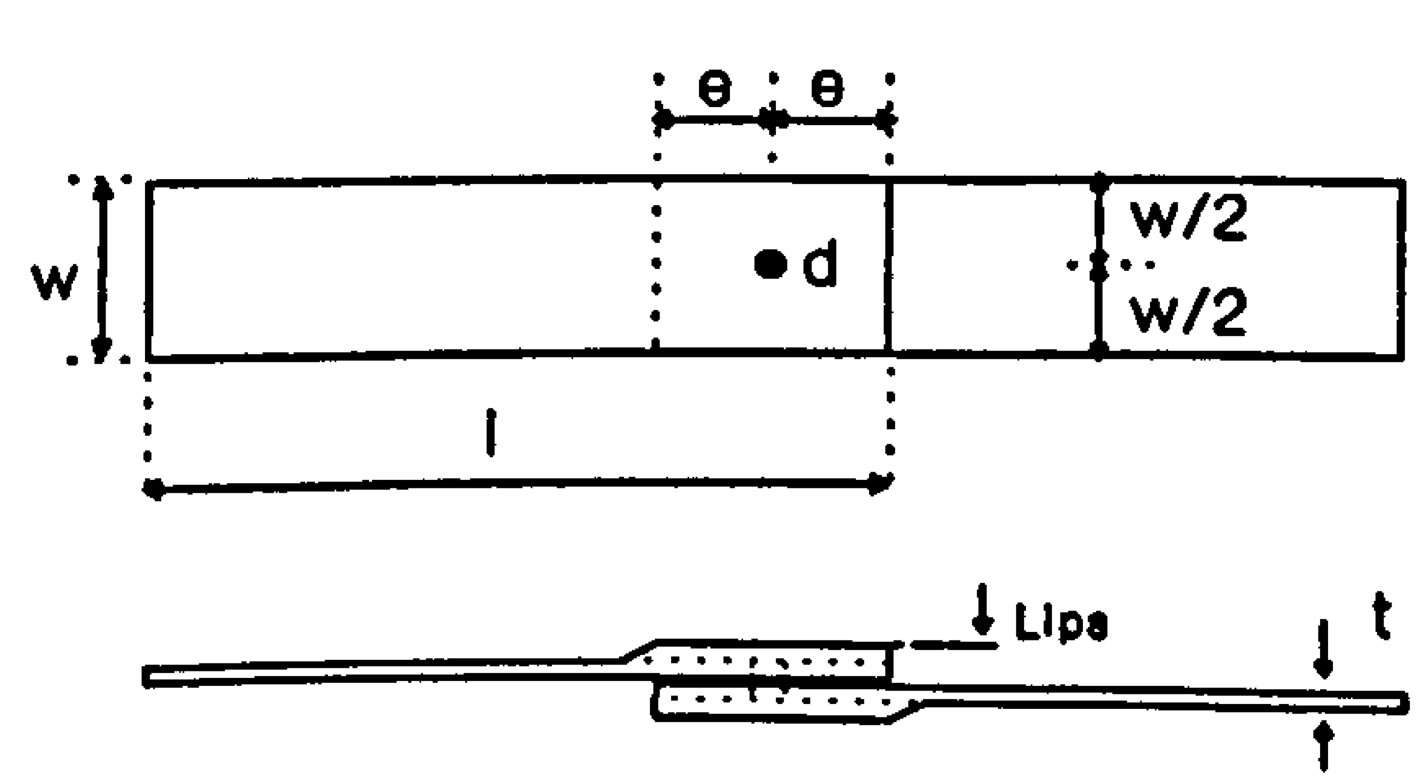
From the above it is evident that test procedures were kept as close as possible to that specified in E.R. publication No. 21. However, the deformation limit of 3 mm set by the above recommendation, § 3.2.1.4, is not realistic for the types of connections being tested here.

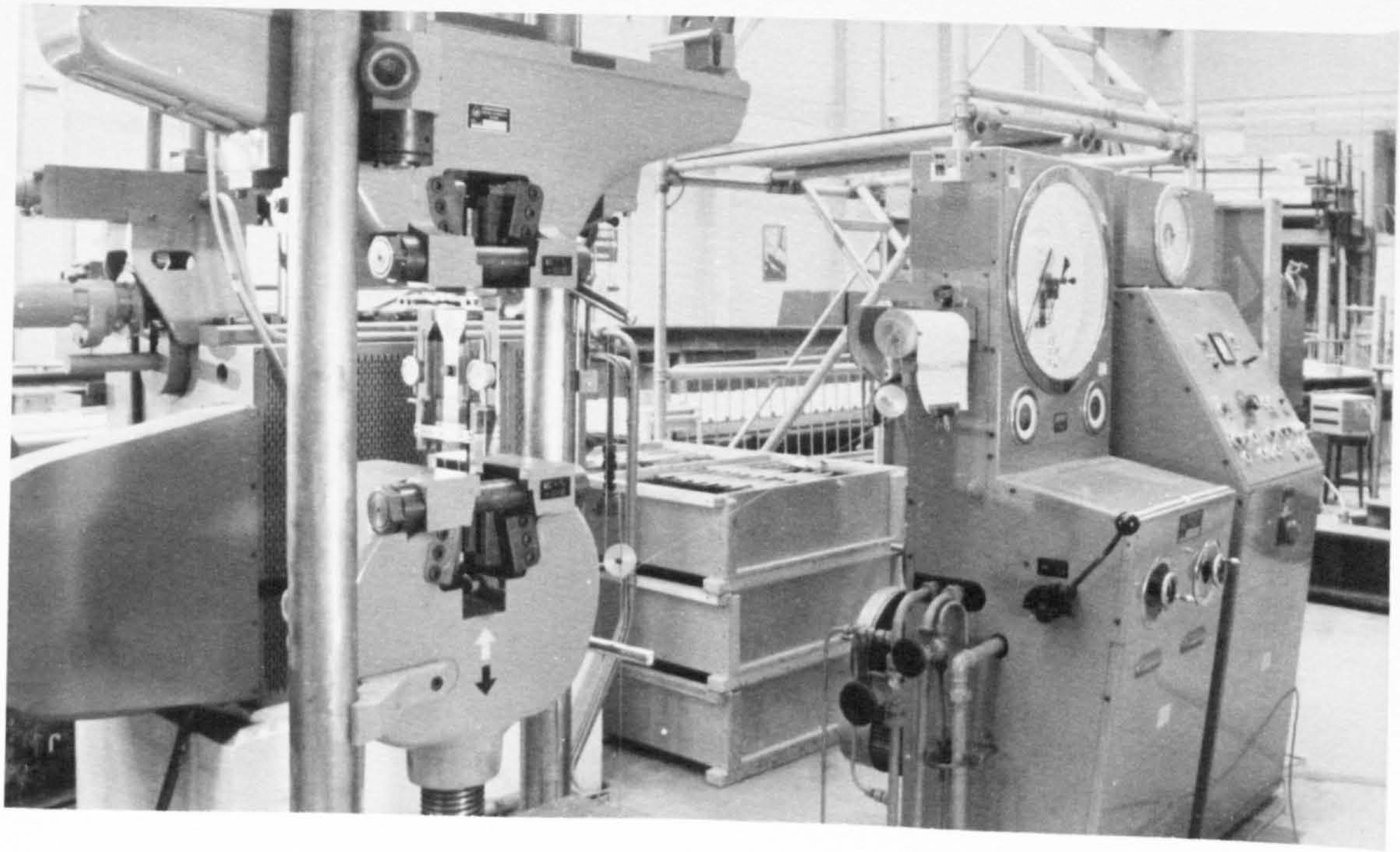
It will be seen in future tests that the deformation capacity of a bolted lap joint is of the order of 20 to 30 mm, before failure. Therefore a deformation limit of 3 mm will unnecessarily hamper the results. Furthermore in connections with 2 mm clearance holes, a 3 mm deformation will often correspond to the slip load of such joints. Again with reference to the background study leading to the E.R.[19] specification revealed that a 3mm deformation would amply correspond to the ultimate load of the type of fastenings considered there.

It is therefore recommended to discard any deformation limits placed on a bolted structural connection in light gauge steel and the ultimate load is simply taken as the first drop in the load/extension characteristics.

3.6. Test results

3.6.1. Lap joints with one bolt in the line of stress





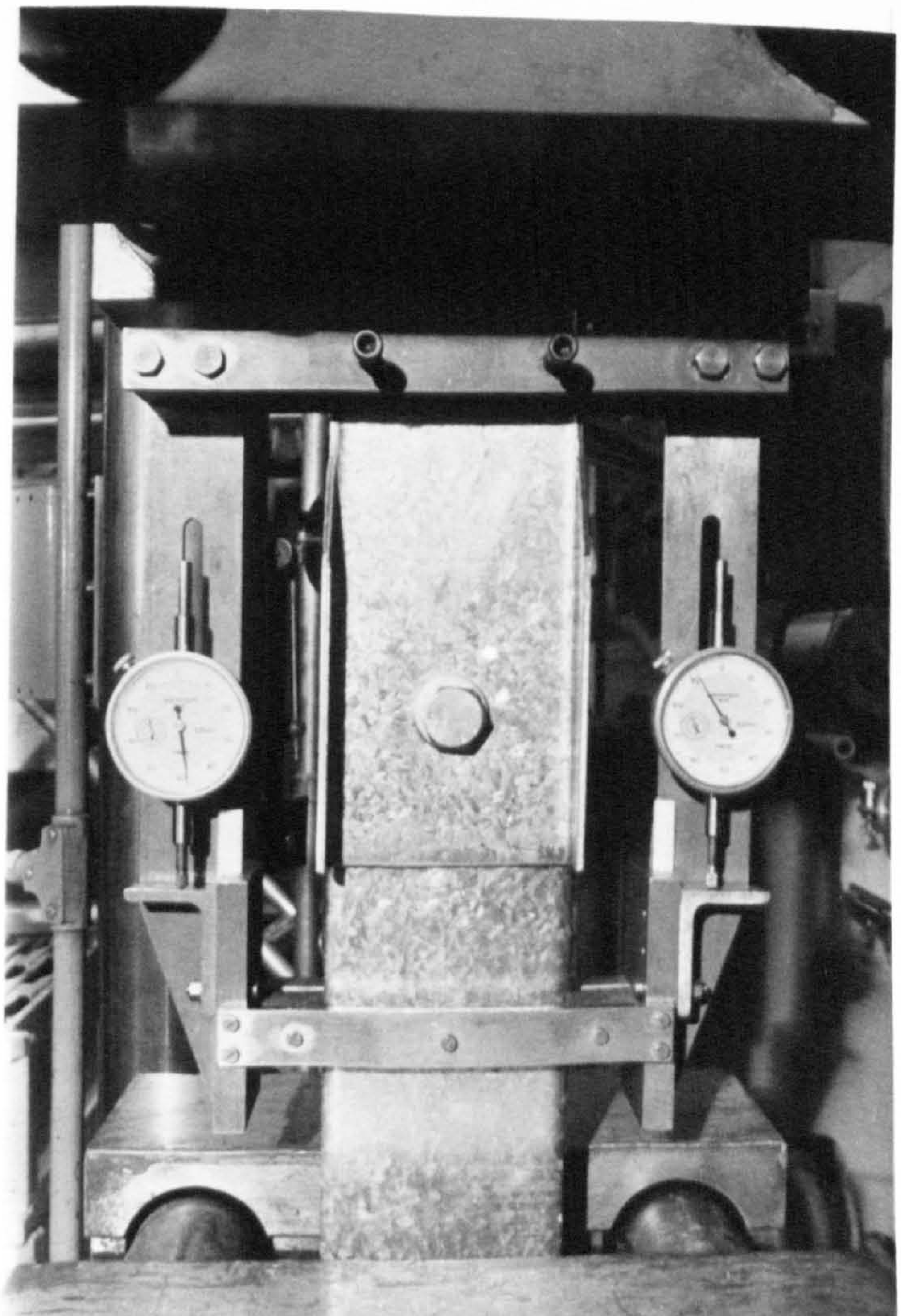


Fig. 3-3.

A specimen fully assembled and undergoing test

3.6.1.1. Sheet thickness, t = 1.7 mm

The results are tabulated in Table 3.2 .

Type of fastener	l mm	w mm	e/d	Lips	Washers	T N.m	Ult. load kN
Bolt	380	100	3.75	-	soft	65	30.0
Bolt	380	100	3.75	-	soft	100	29.0
Bolt	380	100	3.75	20	soft	70	31.0
Bolt	380	100	3.75	20	machined	65	34.0
Set screw	380	100	3.75	20	soft	70	30.0
Bolt	460	150	5.0	-	soft	100	30.0
Bolt	460	150	5.0	20	soft	65	31.0

Table 3.2 : Details of lap joints with a single fastener, t= 1.7 mm.

All specimens failed in sheet bearing. With specimens consisting of flat elements sheets curled out of plane. Increasing the specimen width had an adverse affect on sheet curling. In specimens where sheet curling out of plane was restricted by forming lips around the connection area, the fastener tilted instead.(Fig. 3-4)

Load/extension characteristics of the tests are plotted in Fig. 3-5.

In comparison of the above results the following points are noted :

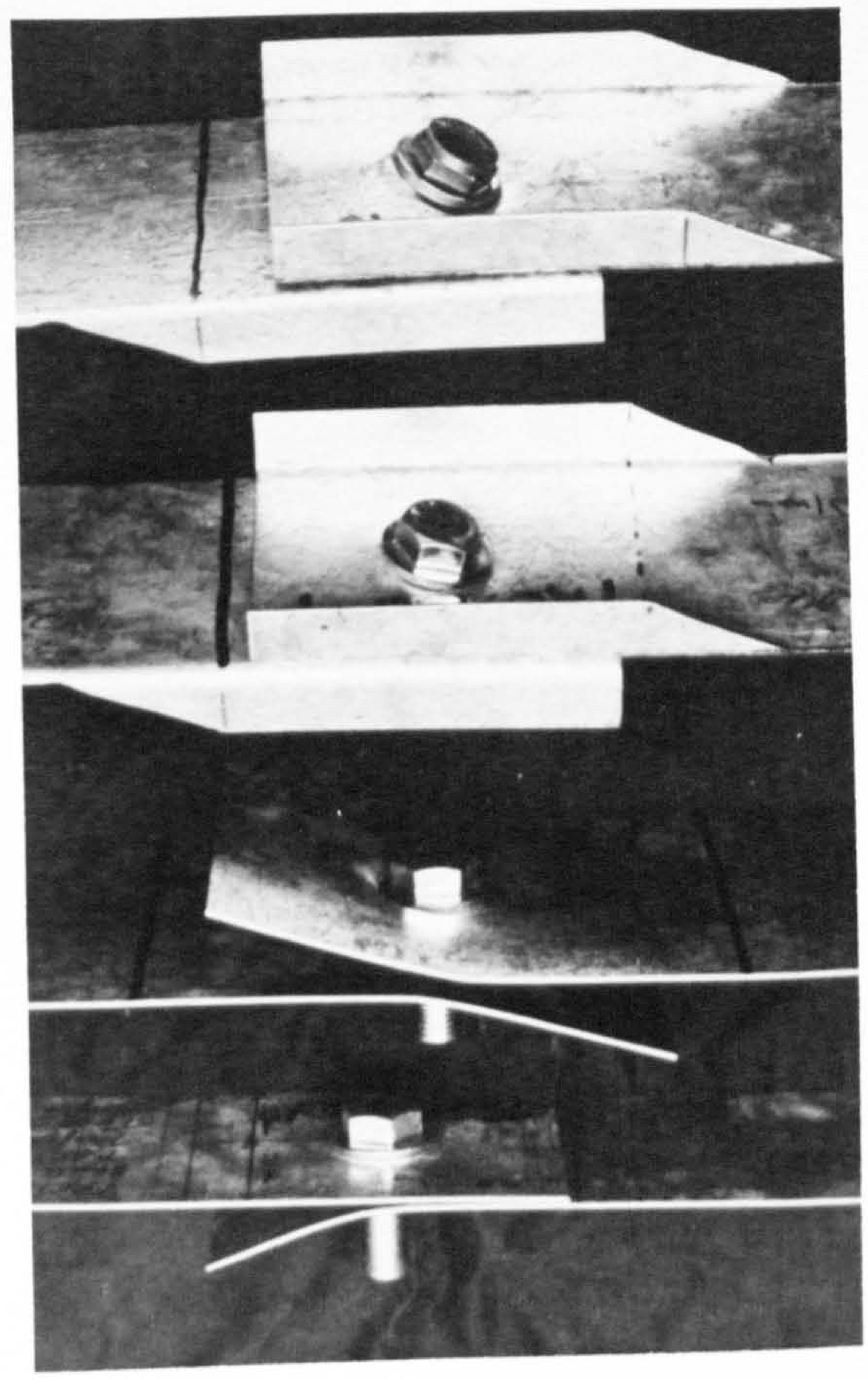
- A width of 100 mm is adequate to develop the desired mode of failure, i.e. sheet bearing.

- Restraining of sheet curling, by forming lips round the connection area, increased the deformation capacity of the lap test specimens, making them more representative of the in-situ conditions.

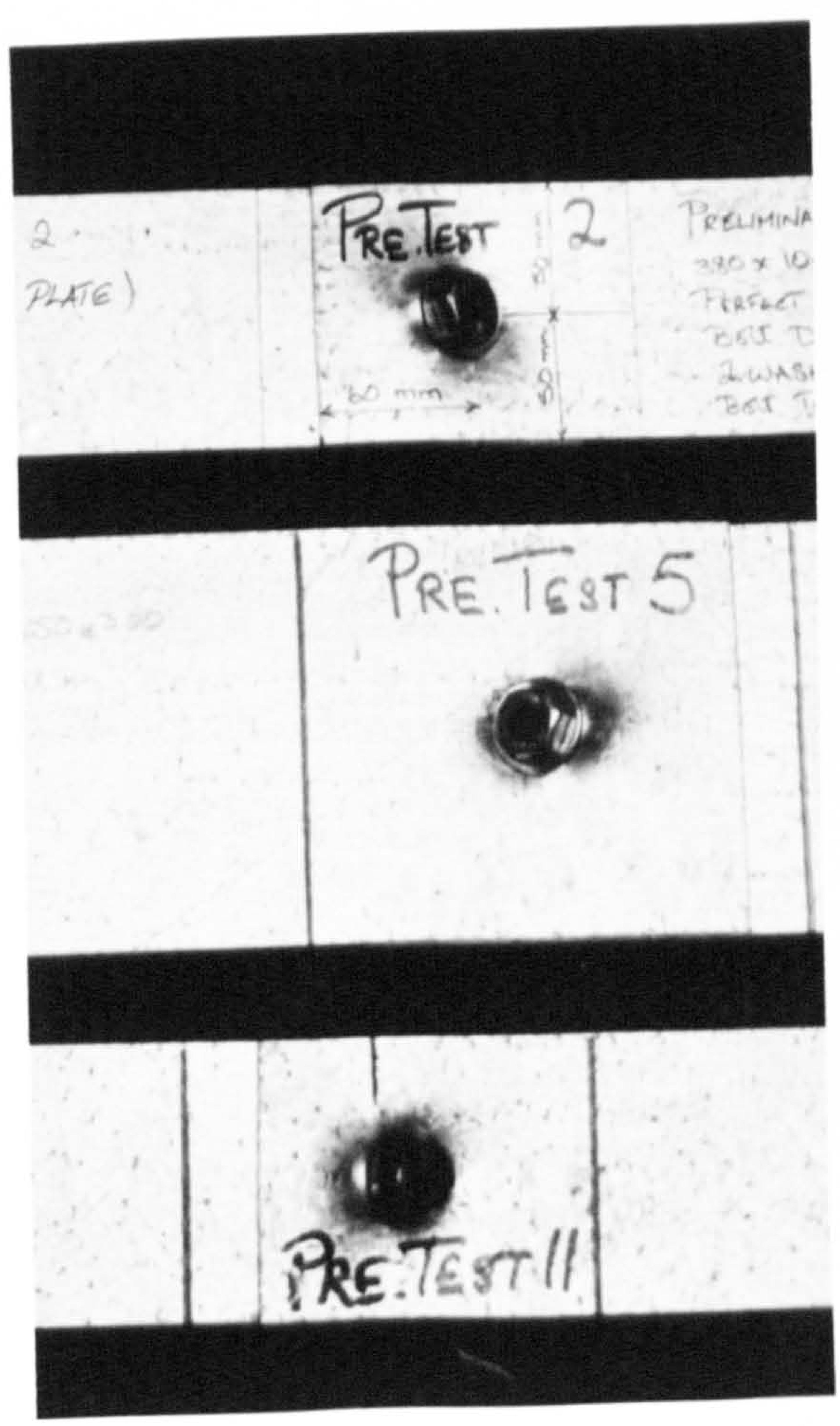
- Early indications are that occurrence of bolt thread in the shear plane increases the flexibility of the connection. A mechanical interlocking of the fastener and the fastening , as a result of threads digging into the parent material, was observed when bolt threads occurred in the shear plane.

- It is evident that bolt torque had no effect on the ultimate load, as expected.

- It appears that the type of washers used does bear a significance on the ultimate load obtained.



t = 1.7mm



t = 3.0mm

Fig. 3-4. Lap joints with one bolt in line of stress

Load-extension characteristics
Specimens with one bolt and t = 1.7 mm.

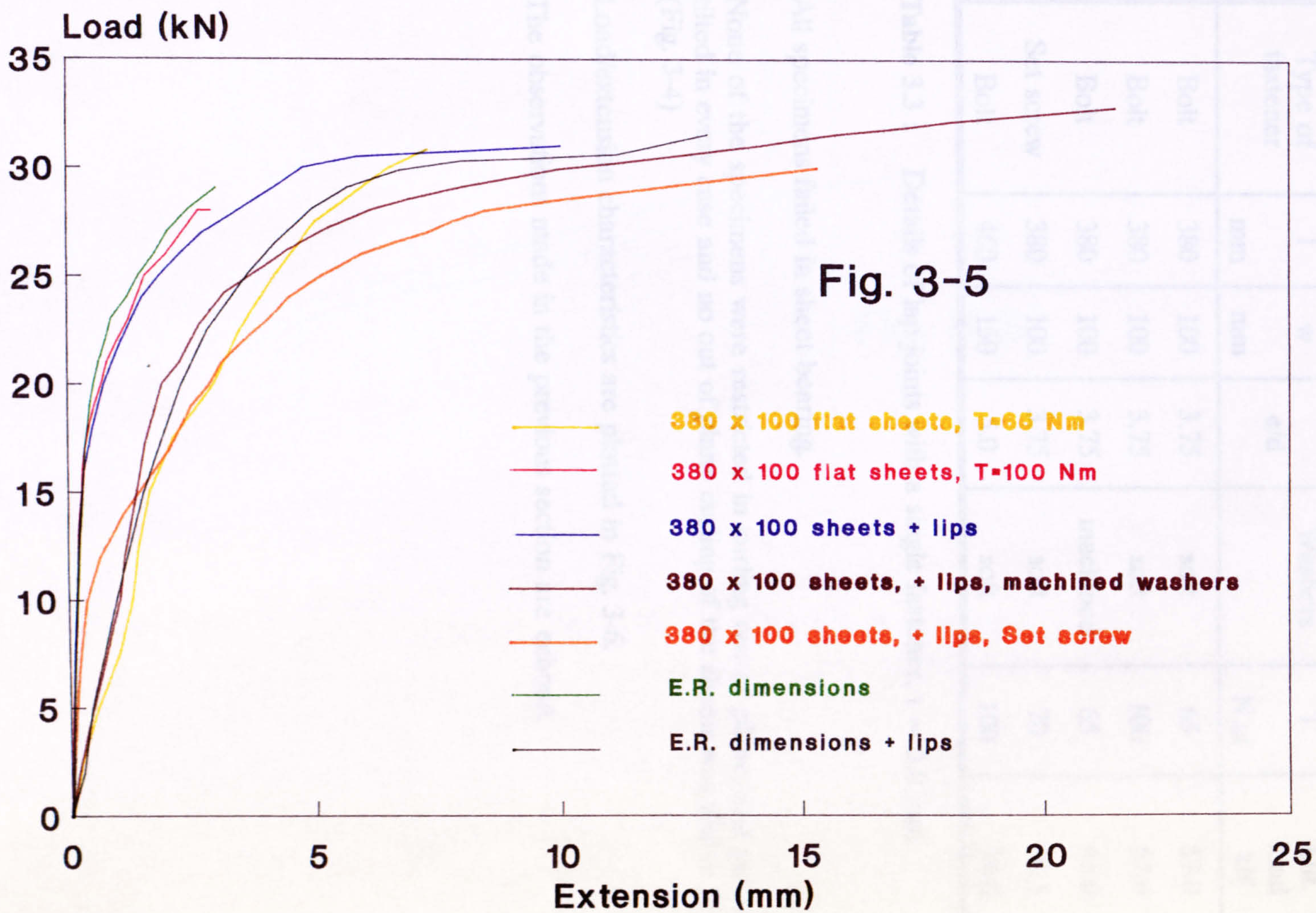


Fig. 3-5

3.6.1.2. Sheet thickness, t = 3.0 mm

The results are tabulated below.

Type of fastener	l mm	w mm	e/d	Washers	T N.m	Ult. load kN
Bolt	380	100	3.75	soft	65	59.0
Bolt	380	100	3.75	soft	100	57.0
Bolt	380	100	3.75	machined	65	65.0
Set screw	380	100	3.75	soft	70	60.5
Bolt	460	150	5.0	soft	100	58.0

Table 3.3 : Details of lap joints with a single fastener, t = 3.0 mm.

All specimens failed in sheet bearing.

None of the specimens were restricted in curling out of plane, and yet the bolts tilted in every case and no out of plane curling of the sheets was visible at failure. (Fig. 3-4)

Load/extension characteristics are plotted in Fig. 3-6.

The observations made in the previous section are echoed.

Load-extension characteristics
Specimens with one bolt and t = 3.0 mm.

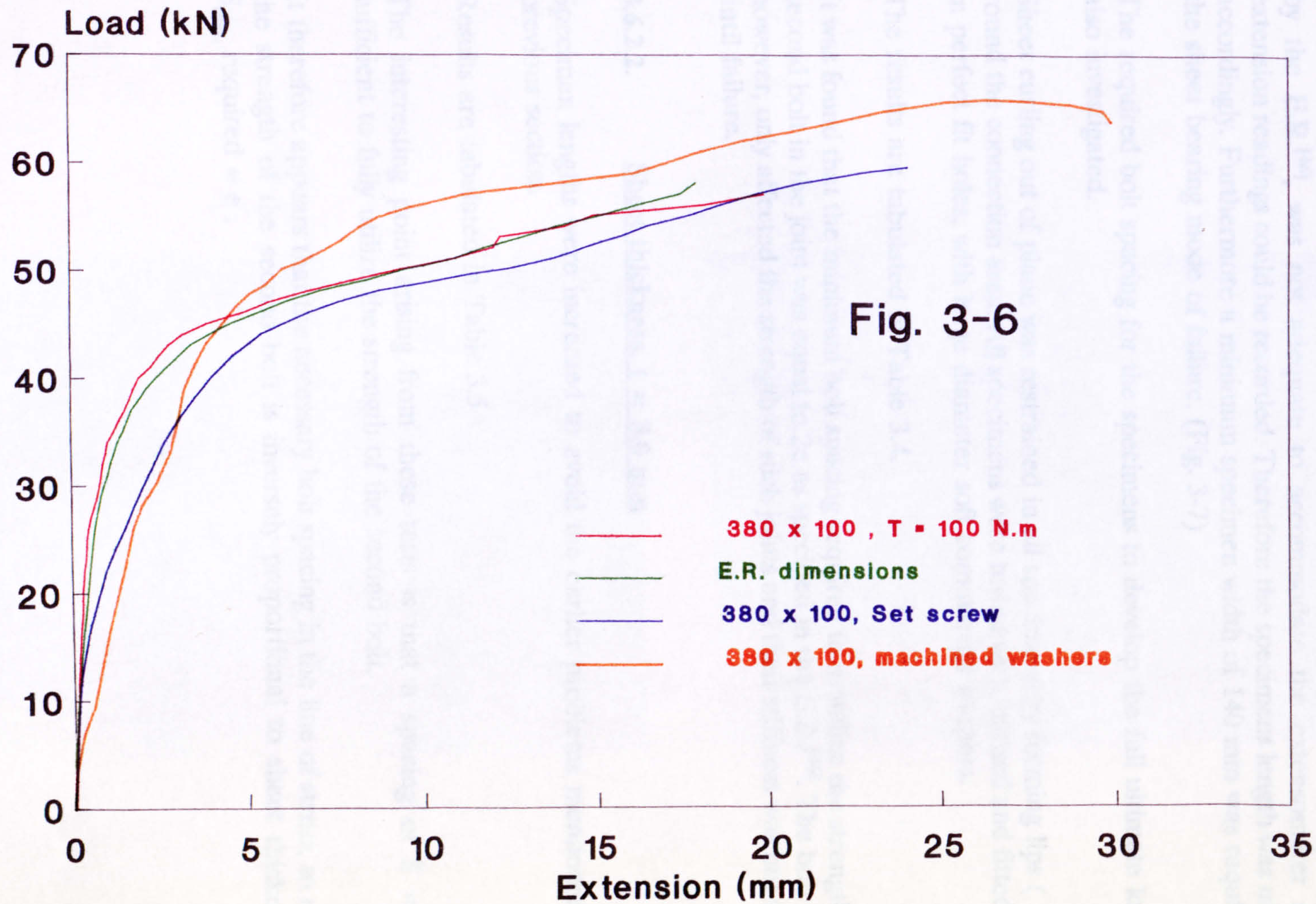


Fig. 3-6

3.6.2. Lap joints with two bolts in the line of stress

3.6.2.1. Sheet thickness, t = 1.7 mm

It was found that the due to the machine limitations the specimen length specified by the E.R.[46], was not adequate to accommodate the extensometer so that extension readings could be recorded. Therefore the specimens length was modified accordingly. Furthermore a minimum specimen width of 140 mm was required for the sheet bearing mode of failure. (Fig. 3-7)

The required bolt spacing for the specimens to develop the full ultimate load was also investigated.

Sheet curling out of plane was restrained in all specimens by forming lips ($\approx w/5$) round the connection area. All specimens were tested with "turned and fitted" bolts, in perfect fit holes, with large diameter soft commercial washers.

The results are tabulated in Table 3.4.

It was found that the minimum bolt spacing required to mobilize the strength of the second bolt in the joint was equal to 2e as specified in the E.R.[46]. The bolt spacing however, only affected the strength of such joints and their stiffness was not affected until failure.

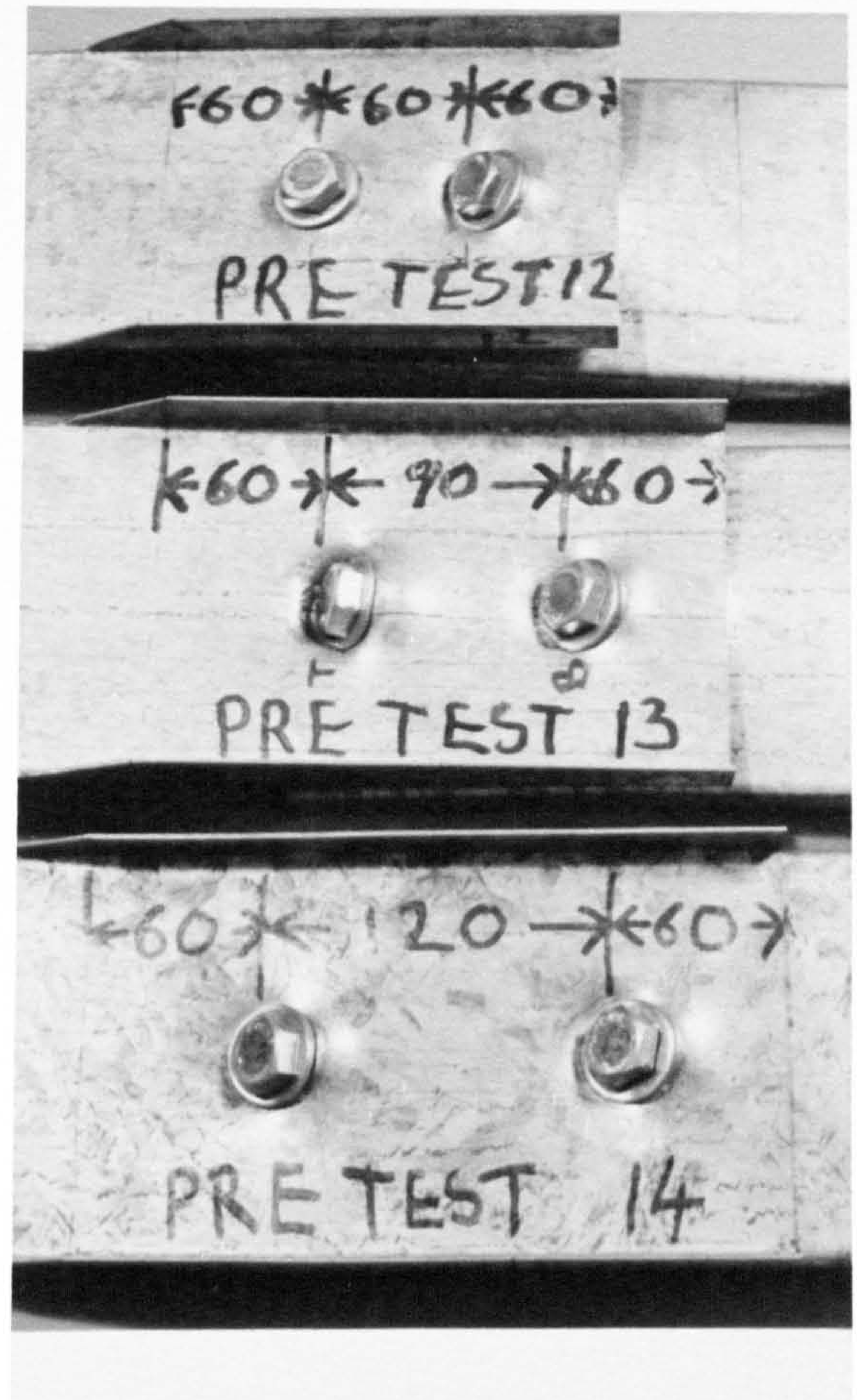
3.6.2.2. Sheet thickness, t = 3.0 mm

Specimen lengths were increased to avoid the earlier problems mentioned in the previous section.

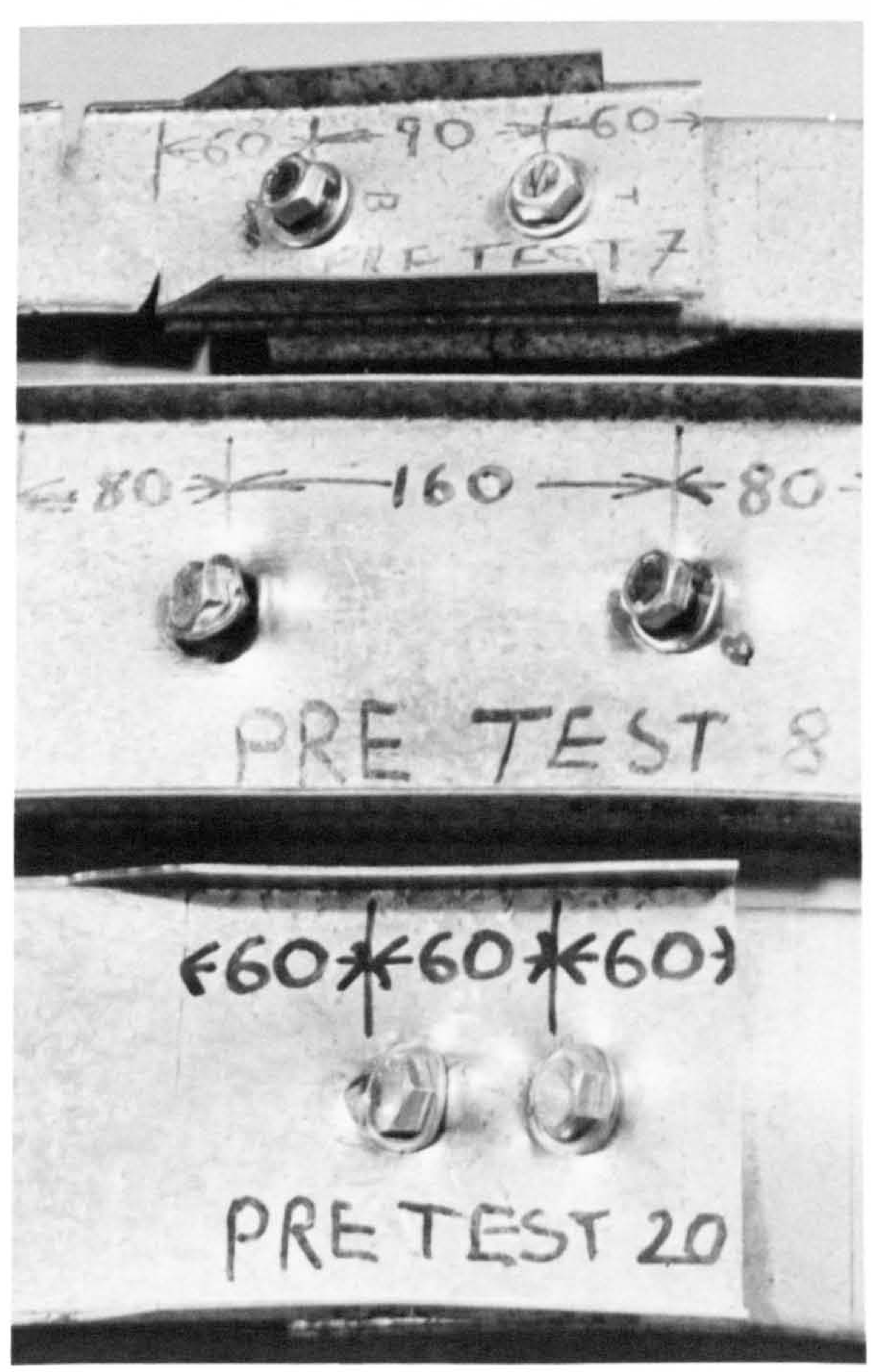
Results are tabulated in Table 3.5

The interesting point arising from these tests is that a spacing of $S = e$ was sufficient to fully utilize the strength of the second bolt.

It therefore appears that the necessary bolt spacing in the line of stress, to mobilize the strength of the second bolt is inversely proportional to sheet thickness. i.e. $S_{min.}$ required $\propto e$.



t = 1.7mm

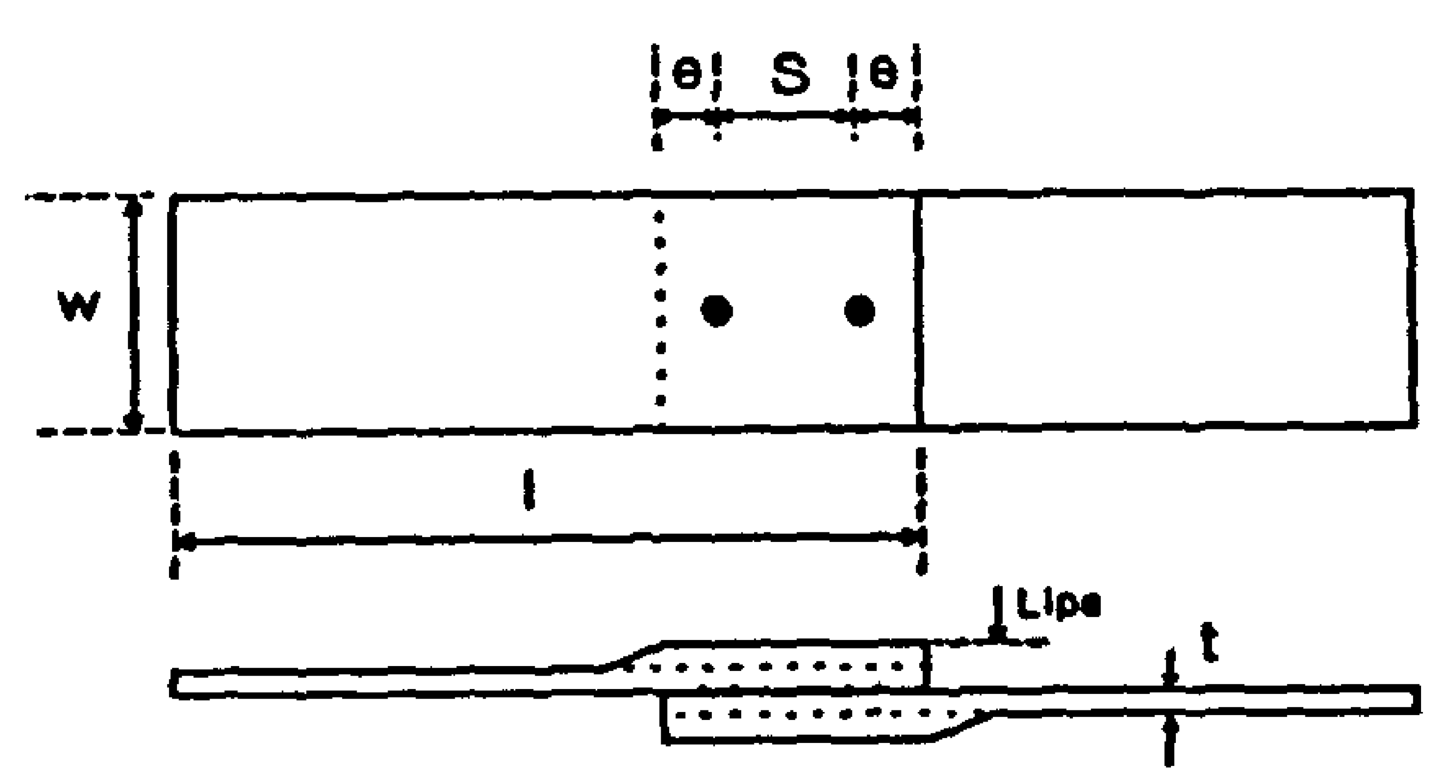


t = 3.0mm

Fig. 3-7. Lap joints with two bolts in line of stress

l mm	w mm	e/d	Bolt spacing, S mm	T N.m	Ult. load kN	Mode of failure	Remarks
380	100	3.75	1½ e	100	58	Net section	Specimen not wide enough
460	150	5.0	e	100	70	Sheet bearing	Length specified by the E.R.[46], not adequate to accommodate the extensometer.
480	140	3.75	e	65	59	Sheet bearing	Bolt spacing not adequate to mobilize the strength of the second bolt. The second bolt had hardly tilted at the time of failure.(Fig. 3-7)
510	140	3.75	1½ e	65	68	Sheet bearing	Behaviour improved compared to the above, but the strength of the second bolt still not fully utilized.
510	140	3.75	2 e	65	73	Sheet bearing	Adequate bolt spacing, bolts tilted equally.

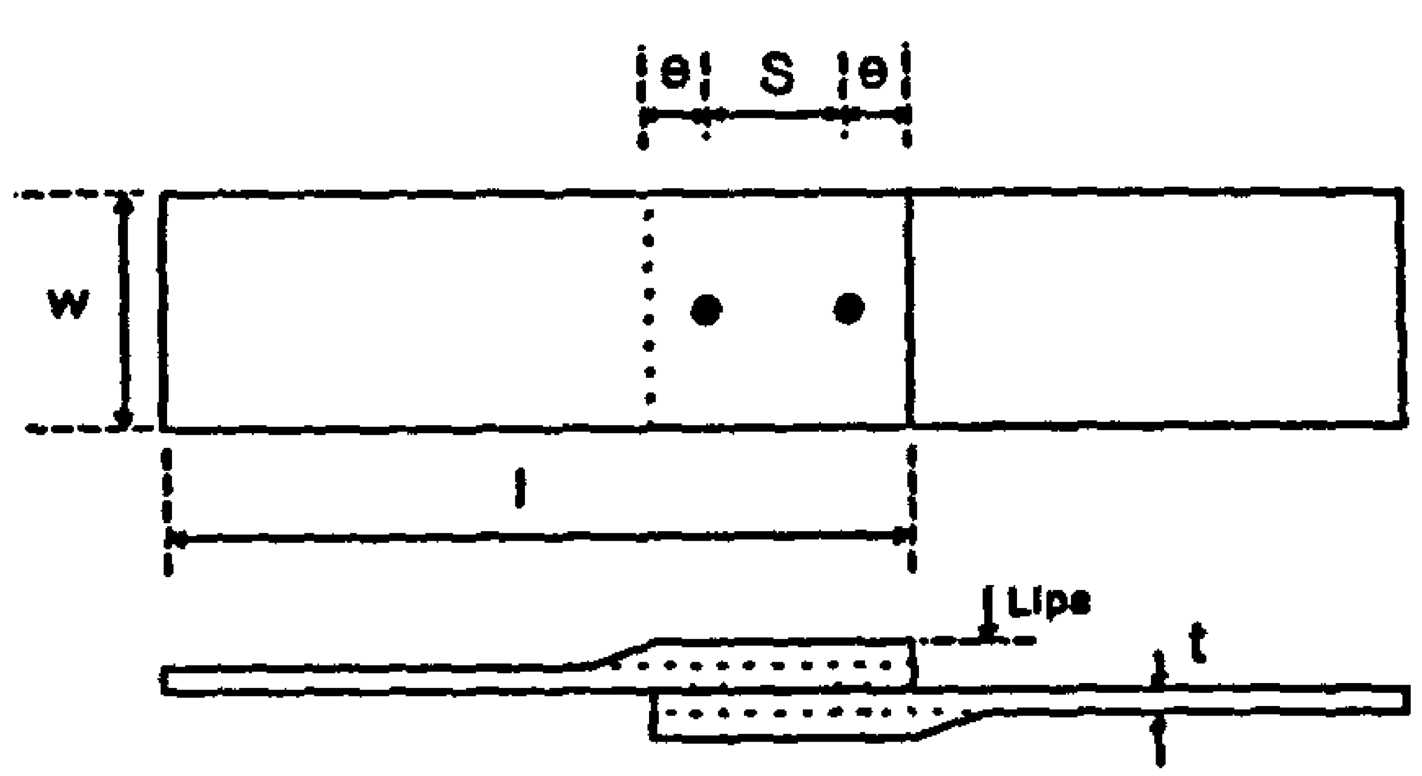
Table 3.4 : Details of lap joints with two bolts in the line of stress, t = 1.7 mm.



All specimens with perfect fit holes and 16 mm ϕ bolts.
Sheets mechanical properties as per Table 3.1
All specimens with 25 mm lips.

l mm	w mm	e/d	Bolt spacing, S mm	T N.m	Ult. load kN	Mode of failure	Remarks
460	100	3.75	1½ e	100	97	Net section	Specimen width not adequate
480	140	3.75	e	65	114	Sheet bearing	Bolt spacing of, S = e, was adequate to mobilize the strength of the second bolt. Equal bolt tilts.
560	150	5.0	2 e	100	117	Sheet bearing	Dimensions specified by the E.R.[46].

Table 3.5 : Details of lap joints with two bolts in the line of stress, t = 3.0 mm.



All specimens with perfect fit holes and 16 mm ϕ bolts.
Sheets mechanical properties as per Table 3.1
All specimens with 25 mm lips.

3.6.3. Comparison of test results for two and one bolt lap joints

The load/extension characteristics of a specimen with two bolts is compared with that of specimens with a single bolt in Fig. 3-8.

As can be seen the results obtained are very satisfactory and almost the same characteristic per bolt is obtained, therefore indicating that lap joints with a single bolt in the line of stress may be used to adequately predict the load-extension characteristics of structural bolted connections in cold formed steel.

3.7. Conclusions

1. Lap joints with a single bolt in the line of stress may be used to adequately predict the load/extension characteristics of structural bolted connections in cold formed steel.

2. The dimensions specified in the European Recommendations, publ. No. 21, for the standard shear test are excessive for load bearing structural bolted connections. The alternative dimension shown in Fig. 3-9 are recommended.

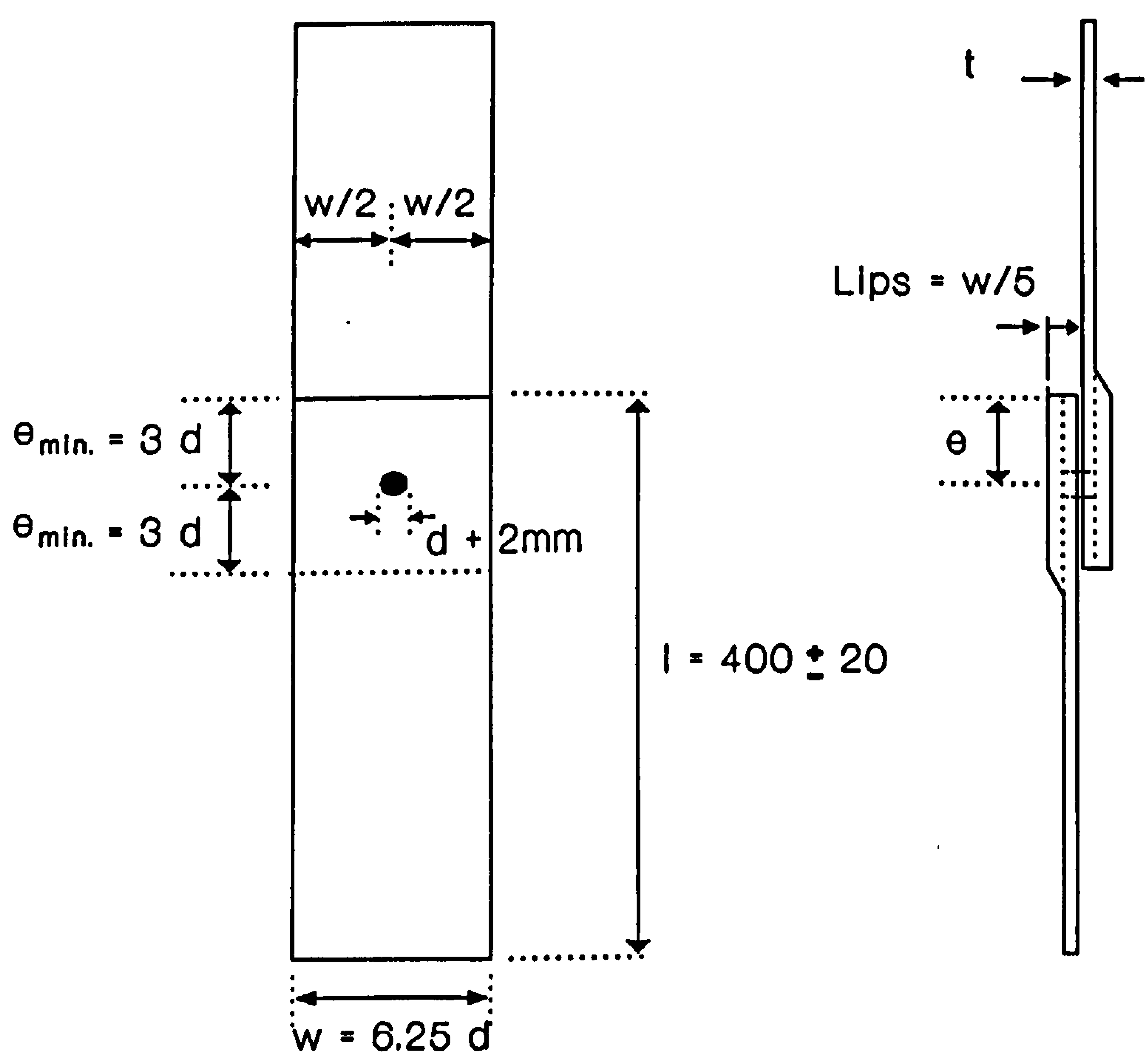


Fig. 3-9 : Recommended test dimensions for structural bolts in cold formed steel sections. (d≥ 12 mm)

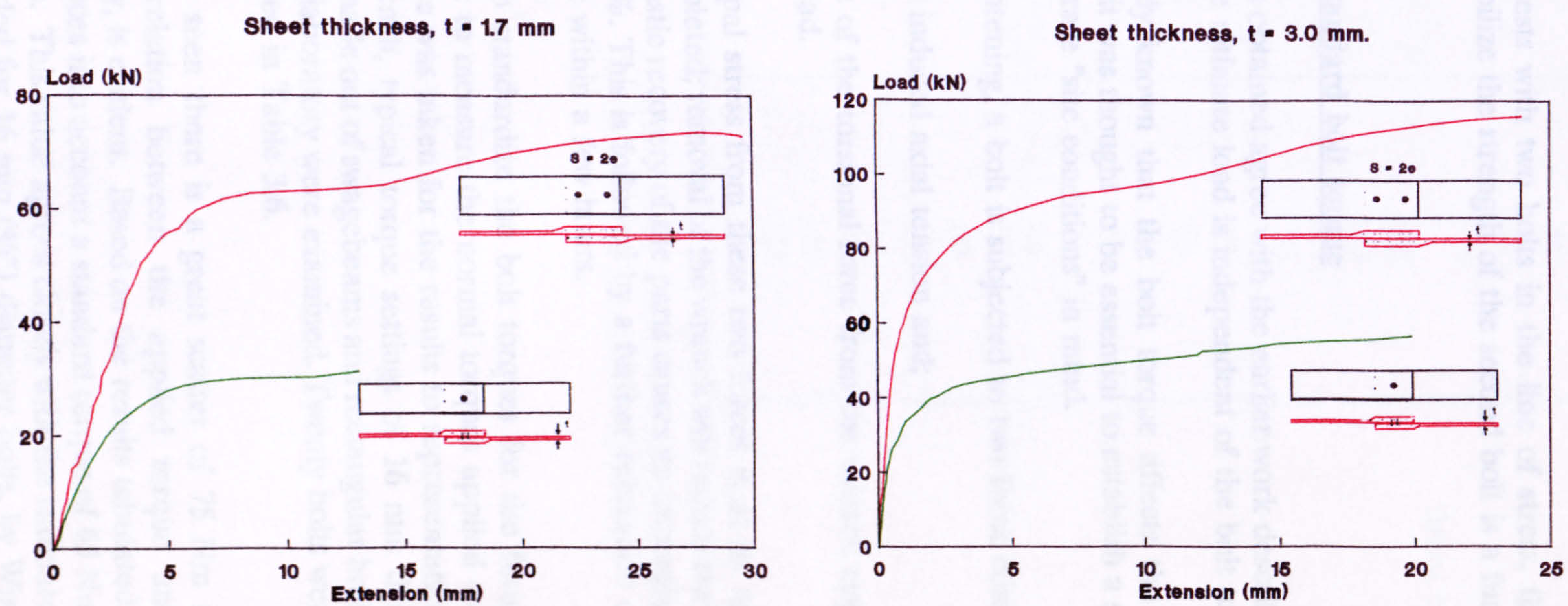


Fig. 3-8 : Comparison of load-extension characteristics of one and two bolt(s) specimens

With samples with two bolts in the line of stress, the specimen length should obviously be increased by the bolt spacing i.e. $l = (400 \pm 20) + S$. A specimen width of $w = 8.75\ d$ was found to be adequate to obtain the correct mode of failure.

3. In tests with two bolts in the line of stress, the required bolt spacing to mobilize the strength of the second bolt is a function of sheet thickness.

3.8. A standard bolt torque

The results obtained agree with the earlier work described in the previous chapter. That is, the ultimate load is independent of the bolt torque.

It is already known that the bolt torque affects the slip load of a connection. Therefore it was thought to be essential to establish a standard torque setting with the key phrase "site conditions" in mind.

During tightening, a bolt is subjected to two force components:-

(i) The induced axial tension and;

(ii) Part of the torsional force from the wrench applied to the bolt via the nut thread.

The principal stress from these two forces is at its maximum when tightening is being completed; removal of the wrench will reduce the torque component of stress, and the elastic recovery of the parts causes an immediate reduction in axial tension of some 5%. This is followed by a further relaxation of 4% of 5% most of which takes place within a few hours.

In order to standardize the bolt torques for the future tests , experiments were carried out to measure the normal torques applied to 16 mm ϕ bolts in practice. Special care was taken for the results to representative of site conditions. Using a torque wrench, typical torque settings of 16 mm diameter bolts of the storage platform, made out of swagebeams and rectangular hollow section stanchions, in the Structures laboratory were examined. Twenty bolts were tested at random. Results are tabulated in Table 3.6.

As can be seen there is a great scatter of 75 Nm in the results obtained. No apparent relation between the applied torque and bolt location, hence its accessibility, is evident. Based on the results tabulated and taking the relaxation of the bolt forces into account a standard torque of 65 Nm is recommended for all the future tests. This value agrees closely with the standard torque of 68 Nm (50 ft. lb.) recommended for 16 mm (⅝") diameter bolts, by Winter[27].

Bolt ref. number	Applied torque (N.m)	Bolt location
1	60	Beam to beam cleat bolts
2	75	
3	12	
4	115	
5	105	
6	115	
7	125	
8	95	
9	60	Stanchion bolts
10	60	
11	60	
12	60	
13	60	
14	60	
15	60	
16	60	
17	85	
18	110	
19	115	
20	135+	

Table 3.6 : Typical torque wrench settings of M16 bolts on the storage platform in the Structures laboratory.

Chapter Four

Bearing strength of bolted connections in cold formed steel sections

4. Bearing strength of bolted connections in cold formed steel sections

Summary

A comprehensive study of all factors influencing the strength of bolted connections in cold formed steel sections is carried out and a design expressions has been proposed. Reference is made to the present codes of practice where appropriate.

4.1. Introduction

A total of 228 tests on bolted lapped joints has been carried out including 20 preliminary tests, the results of which were described in the previous chapter. In this chapter the main body of tests on lap joints are described.

In addition the appropriate results of some 700 tests from other British and European research institutions have been considered. At this stage however, reference made to these data is restricted to use of their test results, where applicable, and the institution(s) associated with them. The background to these tests will be described in more detail in later chapters when a comparison of the proposed design expressions is made with other codes of practice.

The purpose of the lap tests carried out at Salford was to investigate all factors influencing the strength and flexibility of bolted joints in cold rolled steel frames and secondary members; to make design recommendations and draw up design expressions. The strength of such joints will be considered in this chapter. The results obtained will provide essential information on the real behaviour of bolted joints and enable cold rolled steel structures to be designed with full economy.

4.2. Test parameters

The test arrangements and experimental procedures were in accordance with the conclusions drawn on the basis of the preliminary tests described in the previous chapter.

Test constants and variables are described in the following sections. A sample of some of the specimens tested to failure is shown in Fig. 4-1.

4.2.1. Test constants

In order to reduce the large number of variables in the tests on lap joints, the following factors were fixed as constants unless they were being investigated as variables themselves :

(1) All specimens were tested with one bolt in the line of stress.

(2) Loading rate of specimens was kept to 1 kN/minute, until failure.

(3) All sheet steel and fasteners (i.e. bolts, nuts and washers) were galvanised.

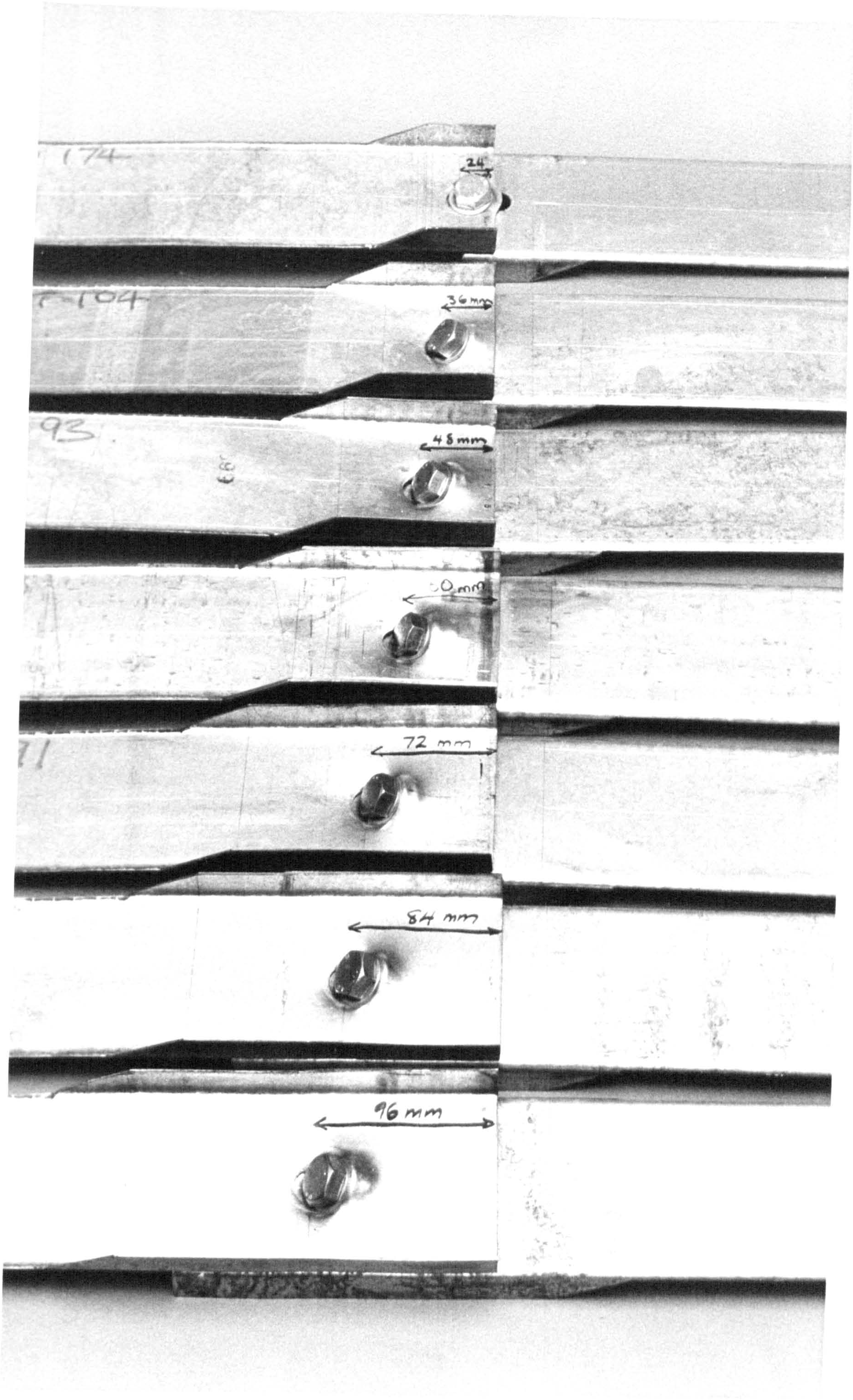


Fig. 4-1. Some failed specimens

(4) Nominal yield stress of sheets, 280 N/mm^2.

(5) Bolt diameter, d = 16mm (M16) , 30mm long.

Grade 4.6 bolts, with grade 8.8 nuts were used. Note that the term "bolt", is used here in its generic sense. Strictly speaking a 30mm long M16 is a set screw

(6) 2 mm clearance holes were punched and reamered in every sheet. In each test sheets were pushed in together prior to the tightening of the bolt, to leave the maximum possible clearance, i.e. 4mm, in the joint

(7) 2 mild steel washers (M16), one under bolt head and one under nut.

The washers were classified as form E in BS 4320, with an average outside diameter of 30mm and a nominal thickness of 3mm.

(8) End distance in line of stress (to centre of bolt), e = 3.75 d.

(9) Bolt torque, T = 65 Nm.

(10) Specimen length, l = 380-420 mm; width, w = 100mm.

(11) All specimens had 20mm lips (w/5), around the lapped joint.
i.e. sheet edges were restrained from out of plane curling.

These measures ensured that failure occurred in the primary mode being investigated, i.e. sheet bearing.

The principal range of sheet thickness considered was 1.5 to 3.2 mm. This range covers the vast majority of cold formed sections used in the industry at the present.

4.2.2. Test variables

The test variables are shown succinctly in the form of a tree diagram in Fig. 4-2.

4.3. Test results - Lap joints

For each variable considered the relevant tree branch in Fig. 4-2 is enlarged, giving more details on the tests carried out, in form of further tree diagrams in Appendix 1. In these latter diagrams all the test variables, number of tests, their mechanical properties, the test constants different from § 4.2.1 above and the ultimate load of each test are depicted thoroughly and concisely. Load-extension characteristics of all tests are classified in groups, according to their test variables, and plotted in Appendix 2. The important results are tabulated within the text in the following sections.

In this way it is hoped to give a comprehensive account of the tests and the results

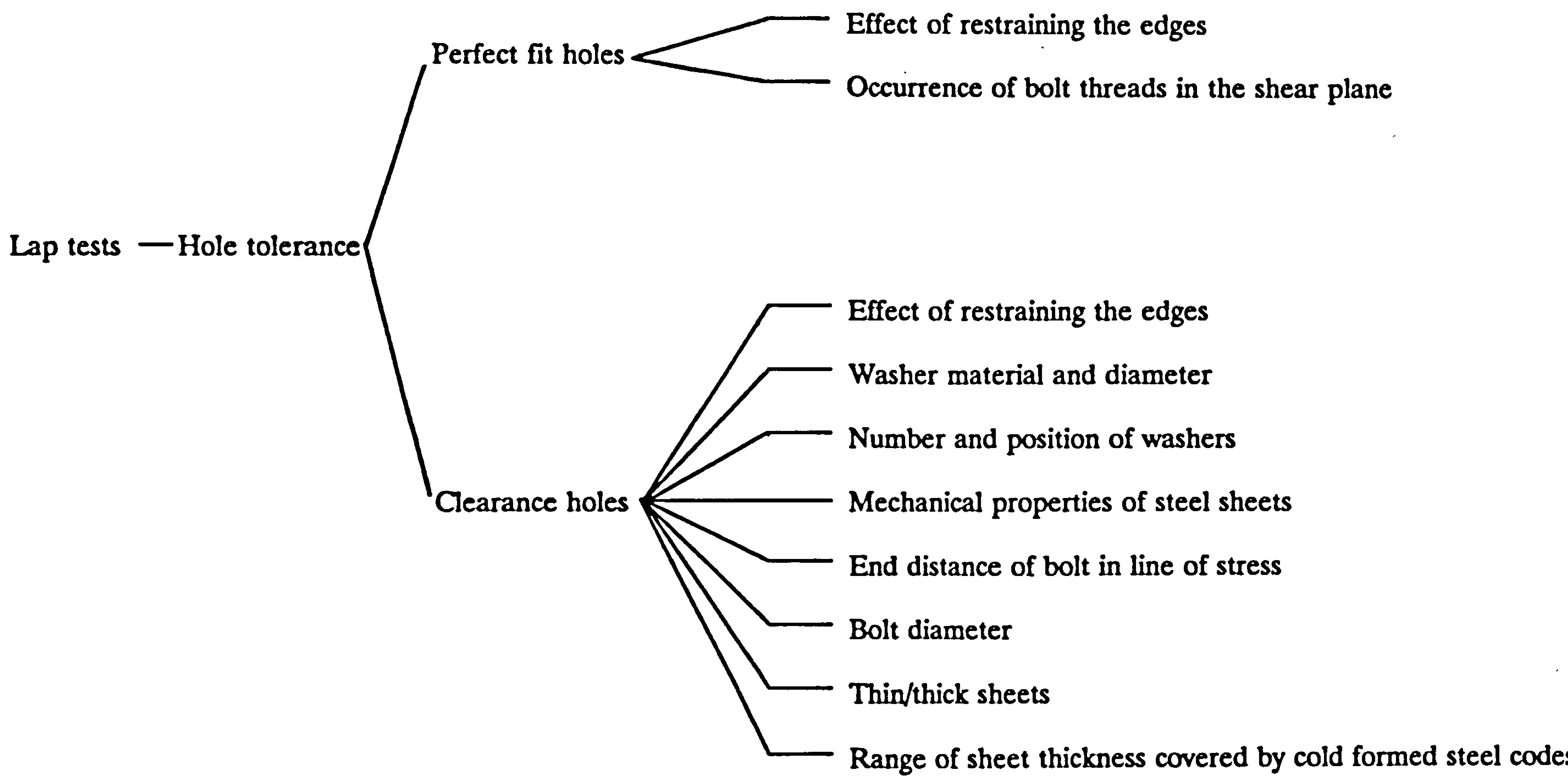


Fig. 4-2 : General test programme on lap joints obtained, with all the necessary details, without interrupting the flow of the discussion on the test results. It may be worth mentioning that the succeeding headings in this section follow the pattern of the tree diagram in Fig. 4-2, and the subsequent sub-headings are illustrated in the relevant diagrams in Appendix 1.

The influence of each factor is considered separately. Design recommendations are later brought together to form a final design expression.

One point in producing the test results should be noted that in the majority of cases there are at least three identical tests for any one set of variables. The results produced are the average of these tests. In some cases the test specimens within a group may have been made up from different steel batches and tested at different times. It was therefore impossible to ensure that all the specimens within one group had precisely the same thickness at all times. So in some cases there may be a small difference, in the order of 0.1 to 0.15mm between the specimen thicknesses in one group. In such cases the ultimate loads are scaled up (or down) to a nominal thickness that is the most common value in that group. Clearly a linear interpolation is adequate to render accurate results for such small discrepancies in the thicknesses.

Hole tolerance

The majority of tests were carried out on specimens with 2mm clearance holes. However some tests were made on specimens with exact diameter holes, i.e a perfect fit was obtained between the fastener and the fastening.

Load-extension characteristics of exact diameter holes are compared with 2mm clearance holes in Figs. A2.1 to A2.3 (Appendix 2). Test parameters are shown in Figs. A1.2 (perfect fit) and A1.4 (clearance holes) in Appendix 1. The shear plane occurred on the fastener threads in every case. Two large diameter, machined bright steel washers were used in each test.

On the basis of the test results it is evident that, once the joints with clearance holes are pulled into bearing their behaviour is very similar to that of exact diameter holes thereafter. It is therefore concluded that normal hole tolerance is not detrimental to the joint strength.

4.3.1. Perfect fit lap joints

A series of tests were carried out using "turned and fitted" bolts with a plain shank in the shear plane of the lap joints. That is, the bolt hole was drilled and reamered to the exact bolt diameter. The bolt shank and threads were also turned down to perfectly fit in the reamered hole. Therefore a perfect fit was obtained representing an "ideal connection". This was to serve two purposes:

(1) The perfect fit eliminates the "connection slip". The results obtained were then compared to 2mm clearance connections; thus adequate slip load provisions could be provided for design conditions. This will be dealt with in

later chapters

(2) A plain shank was contrived to verify the effect of occurrence of bolt threads in the shear plane of a joint.

Note in § 4.3 above, the results do perfect fit holes with bolt threads in the shear plane were referred to as "exact diameter" holes. This was to distinguish their subtle difference with perfect fit holes, i.e. with perfect fit holes the threads did not occur in the shear plane.

A plain shank in the shear plane was ensured by using longer bolts, M16 x 50 mm, than the usual 30 to 35 mm long 16mm ϕ bolts used in the industry.

4.3.1.1. Effect of restraining the sheet edges - perfect fit joints

The effect of restraining the sheet edges was investigated by forming lips around the lapped part of the perfect fit joints. The test parameters are depicted in Fig. A1.1. The load extension graphs for all the specimens tested are shown in Figs. A2.4 to A2.7 (Appendix 2) for sheet thicknesses of 1.5 to 3.2mm respectively.

In Chapter Two it was mentioned that Corcoran suggested bolt tilting occurs when $dt_1t_2 > 75\ mm^3$.

It was decided to investigate this further. The results obtained were very consistent as evident in their load-extension characteristics. (Figs A2.4 to A2.7, Appendix 2)

After examining the failed specimens it is concluded that bolt tilting initiates, in specimens with unrestrained edges, in the region of dt_1t_2 equal to 60, where at this point the failure in the case of the unrestrained specimens was by a combination of sheet curling and bolt tilting as well as sheet bearing. However, in this lower region, sheet curling was the dominant factor. As the product of dt^2 increased, then bolt tilting became more and more significant, to the point that for higher products of dt^2 sheet curling was completely eliminated in specimens with or without edge restraints. This point is very well illustrated in Table 4.1.

Nominal sheet thickness (mm)	Average ultimate load for specimens :		Ratio $\frac{(1)}{(2)}$	Product dt^2 $(t_1 = t_2)$ (mm^3)
	(1) with lips (kN)	(2) without lips (kN)		
1.50	35.8	23.6	1.52	36
1.96	35.8	29.1	1.23	61
2.48	62.0	53.9	1.15	98
3.02	71.4	72.0	1.00	146

Table 4.1 : Effect of edge restraint, perfect fit holes

It is seen that the ultimate load ratios for the specimens with and without lips reduces with increase in dt^2, to the point that the ratio is unity for dt^2 of 146 (nominal sheet thickness of 3.02mm). The edge restraints, i.e lips. were of no significance with these "thicker" sheets and the failure was by sheet bearing and bolt tilting, in every case. On the other hand a ratio of 1.52 exists for specimen thickness of 1.5mm (dt^2 equal to 36). In this case the specimens without lips essentially failed as the sheets yielded round the bolt hole and the sheets curled up very rapidly with very little deformation capacity (Fig. A2.4). In the case of 1.5mm thick specimens with lips however, there was significant bolt tilting with sufficient deformation capacity, resulting in a "ductile" failure. This illustrates the necessity of edge restraints in testing of lap joints, for the test results to be representative of the in-situ conditions. Although the presence of edge restraints is emphasized here, it should be noted that lips are of no design significance, but purely an experimental technique that effectively represents flat elements in practice.

4.3.1.2. Effect of occurrence of bolt threads in the shear plane - perfect fit joints

A distinction between set screws and bolts was drawn in Chapter Three. It was also established that the occurrence of bolt threads in the shear plane is the norm in bolted connections in cold formed steel.

In calculating the bearing strength of bolted connections however, the nominal bolt diameter is always used. For sake of convenience and practicality, it is obviously desirable to ensure that this is so. This will be taken into account in design expressions produced later on, i.e. they will be based on the assumption that the shear plane occurs on the threads.

Tests were carried out to investigate and quantify the effect of occurrence of bolt threads in the shear plane. The result could then be implemented as an "add on" factor in cases where it can be shown that bolt threads do not occur in the shear plane. This concept has already been employed for the shear capacity of fasteners in BS 5950 pt.5. That is the full shank area of the bolt can be used, in place of the tensile stress area, provided that it can be shown that bolt threads do not occur in the shear plane.

In order to investigate the effect of occurrence of bolt threads in the shear plane, tests were carried out on perfect fit lap joints with bolts and set screws. In the case of bolts; a plain shank in the shear plane was ensured by using a longer length bolt, M16 x 50mm, as described in § 4.3.1.

Test parameters are depicted in Fig. A1.2 (Appendix 1). Load-extension characteristics of all the tests carried out are plotted in Figs. A2.8 to A2.10 (Appendix 2).

A study of the failed specimens revealed that under load, in the case of bolts, the sheet steel is squashed in bearing, in the area of contact with the bolt. Whereas with set screws the threads dig into the bearing area and precipitate tearing. This affected both strength and flexibility of the tested joints. The effect on flexibility will be considered in later chapters. The effect on connection strength is considered here. The results obtained are tabulated in Table 4.2.

Nominal sheet thickness (mm)	Average ultimate loads (kN)			Ratio
	(1) Bolts, bright steel washers	(2)		(1)/(2)
		Set screws	Type of washers	
1.50	35.8	31.1 25.3	bright (harder) steel Mild steel	0.87 -
2.57	55.8	54.6	bright (harder) steel	0.98
3.17	75.9	54.25* 63.8	bright (harder) steel Mild steel	N.A. 0.84

* failed by shearing of bolt.

Table 4.2 : Effect of the occurrence of bolt threads in the shear plane - perfect fit joints.

Before discussing the effect of occurrence of bolt threads in the shear plane however, two points in the results tabulated above should be noted.

i) For lap joints with 2.57mm thick sheets, a rather wider band of test results was obtained. The ultimate loads of the specimens ranged from 47.5 to 63.4 kN, i.e. a 15% variation. This was unusual since, for the vast majority of test on the lap joints, the ultimate loads and load-extension characteristics varied within 5%. The inconsistency in the results for this particular thickness is also reflected in the last column of Table 4.2 above, where a reduction in the ultimate load of only 2% compared to 13 and 16% for 1.5 and 3.2mm thick sheets respectively, is obtained.

This discrepancy is believed to be due to lack of edge restraints on the out of plane curling of the sheets in the case of 2.57mm thick specimens (Fig. A1.2). For $t = 2.5$mm ($dt^2 = 98$) the bolt tilting/sheet curling mode of failure is still in a "transitional" stage. Although bolt tilting, combined with sheet bearing, is the dominant mode of failure, even with unrestrained edges, sheet curling could still effect the ultimate load. In fact, the lowest ultimate load of 47.5 kN, mentioned above, in this case occurred precisely because of this where failure was by sheet bearing and sheet curling.

In case of 1.5mm thick specimens where sheet curling was effectively restrained by the presence of lips, and with 3.2mm thick sheets where sheet curling was insignificant, the results obtained were very consistent.

ii) The use of large diameter bright steel washers with perfect fit lap joints resulted in a stiffer joint than usual, and therefore less prone to bolt tilting. In the case of 3.2mm thick sheets with bolts, where the shear plane occurred on the plain shank of the fastener, the shear strength of the full core diameter of the bolts was adequate to pass the threshold of bolt tilting and all the specimens failed in sheet bearing and bolt tilting. With the same sheet thickness but set screws however, where the shear plane was on the threads, the extra stiffness of the harder washers prevented the occurrence of bolt tilting long enough for the specimens to fail in shearing of bolt at an average load of 54.25 kN. It is very interesting to note that use of normal diameter mild steel washers with the same sheet thickness and set screws resulted in a joint less resistant to bolt tilting and in the actual test the bolt tilted and the specimen failed at a higher load 63.8 kN, in sheet bearing.

From BS 5950 pt.5 (Table 11) the shear capacity of a 16mm ϕ fastener, with the shear plane on the bolt threads, is equal to :

$P_s = A_t \cdot p_s = 157 \times 0.160 = 25.1$ kN.

Where A_t = tensile stress area (=157 mm^2 for M16 bolts)

On the other hand with the thinner specimens it can be seen from the same table that using normal diameter, mild steel washers, has resulted in 19% reduction in ultimate strength, compared to large diameter hard steel washers.

The interaction between bolt tilting and sheet bearing mode of failure will be dealt with in later chapters.

Returning to the original discussion regarding the occurrence of bolt threads in the shear plane and the results produced in Table 4.2, it is concluded that there is a reduction in the order of 13% in the ultimate strength of a bolted connection when the shear plane is on the threads. This corresponds to an equivalent reduction in effective diameter of the fastener.

It is therefore recommended to allow an increase of $(d/d_{eff.})$ in the ultimate bearing capacity when threads do not occur in the shear plane.

$(d/d_{eff.})$ can be expressed in terms of fastener bearing area as :

$$\frac{d}{d_{eff.}} = \sqrt{\frac{\frac{\pi d^2}{4}}{A_t}} = \frac{d}{2}\sqrt{\frac{\pi}{A_t}}$$

The above ratio has been considered for a range of fastener diameters, relevant to bolted structural connections, in Table 4.3.

Fastener dia., d (mm)	Tensile stress area, A_t (mm^2)	$\frac{d}{2}\sqrt{\frac{\pi}{A_t}}$
10	58.0	1.16
12	84.3	1.16
16	157	1.13
20	245	1.13

Table 4.3 : Ratios of full core to effective diameter areas.

Based on the above table it is concluded that the bearing strength may be increased by 15% provided it can be shown that bolt threads do not occur in the shear plane.

4.3.2. Lap joints with clearance holes

The results obtained on specimens with 2mm clearance holes, which form the main bulk of the tests on lap joints, are examined and discussed in this section. For the range of fasteners in cold formed steel connections, 1 to 2 mm clearance holes are the most common form of practice in the industry. The following are the main findings :-

4.3.2.1. Effect of restraining the edges - clearance holes

Effect of restraining the sheets edges on bolt tilting was also studied with clearance holes. Nominal sheets thicknesses of 2.5 and 3mm were considered.

Test parameters are depicted in Fig. A1.3. Load-extension characteristics are given in Figs. A2.11 and A2.12 for 2.48 and 3.02mm thick sheets respectively.

The ultimate loads obtained are tabulated below.

Nominal sheet thickness (mm)	Average ultimate load for specimens :		Ratio $\frac{(1)}{(2)}$	Product dt^2 ($t_1 = t_2$) (mm^3)
	(1) with lips (kN)	(2) without lips (kN)		
2.48	47.2	37.9	1.24	98
3.02	56.3	54.4	1.04	146

Table 4.4 : Effect of edge restraint, perfect fit holes

The results obtained are very much along the same lines as that of Table 4.1 - that is, the ratio of the ultimate bearing strength of specimens with and without lips reduce to near unity with increase in the dt^2 product.

Figs. A2.11 and A2.12, in Appendix 2, also show that the slip load of the joint is independent of the sheet thickness and (in this case) ranged between 3 to 10 kN.

Having considered restraining of sheets edges with perfect fit and clearance hole lap joints; it is concluded that restraining lips are essential, over the lapped portions, to ensure that the test set-up is representative of practical joints. This device eliminates sheet curling - particularly in the thinner sheets - which would otherwise badly distort the test results.

4.3.2.2. Washer material and diameter

Tests were made on three different types of washers apart from those used as standard for all other lap tests. The purpose of these tests was two-fold :

(i) To investigate whether there was any advantage to be gained from using large diameter washers.

(ii) whether the quality of washers has any significant effect on the characteristics of a connection.

Washers are often punched out of the scrap end of steel strips and usually manufactured by the cheapest available means. Four types of washers, listed below, were tested.

(a) Normal diameter, mild steel galvanised washers used as standard for all tests carried out on lap joints. (30mm diameter washers). BS 4320, Form E, (Table 4.5).

(b) Normal diameter, mild steel galvanised washers, obtained from a different manufacturer compared to (a) above.

(c) Large diameter, mild steel galvanised washers, 34mm outside diameter. BS 4320, Form F, (Table 4.5).

(d) Large diameter, bright steel washers, - high quality engineering washers with machined edges and chamfered round the outside edge, known as Turned and Chamfered washers in practice. These washers ruptured in a more brittle mode indicating a harder steel compared to those above. These washers were also used as "standard" for the perfect fit lap joints.

Table 4.5, extracted from BS 4320, shows the required specifications for normal and large diameter washers for nominal M16 bolts.

Nominal bolt dia. (mm)	A Inside dia. (mm)			B Outside dia. (mm)			C Thickness (mm)			D Mass in kg per 1000 ±30%
	Nom.	Max	Min	Nom.	Max	Min	Nom.	Max	Min	
M16	Normal diameter (Form E)									11
	18	18.5	18	30	30	29.2	3	3.6	2.4	
	Large diameter (Form F)									16
	18	18.5	18	34	34	32.8	3	3.6	2.4	

Table 4.5 : Black washers to BS 4320.
Due to thickness tolerance, the mass can vary by as much as 30%.

A random batch of 10 washers of each of the four types listed above was tested against the required specifications tabulated above. The results are shown in Table 4.6.

BS4320 Form :	Washer Group	A Inside dia. (mm)		B Outside dia. (mm)		C Thickness (mm)		D Mass (gr.)	
		Mean	Std. deviation	Mean	Std. deviation	Mean	Std. deviation	Mean	Std. deviation
E	(a)	18.1	0.16	30.0	0.35	2.3	0.06	7.8	0.06
	(b)	17.1*	0.00	29.4	0.23	1.9*	0.02	6.6*	0.06
F	(c)	17.3*	0.07	35.0*	0.04	3.1	0.11	16.6	0.60
	(d)	16.5*	0.00	34.8*	0.02	3.0	0.02	16.6	0.14

* failed BS4320 requirements

Table 4.6 : Tested washers compared with BS 4320 requirements

It should be noted that except for case (d) the emphasis was on testing standard washers used in practice. Therefore Table 4.6 should be regarded as typical values.

Fig. 4-3 shows a sample of each washer intact and after testing. Note the distortion caused by bolt tilting.

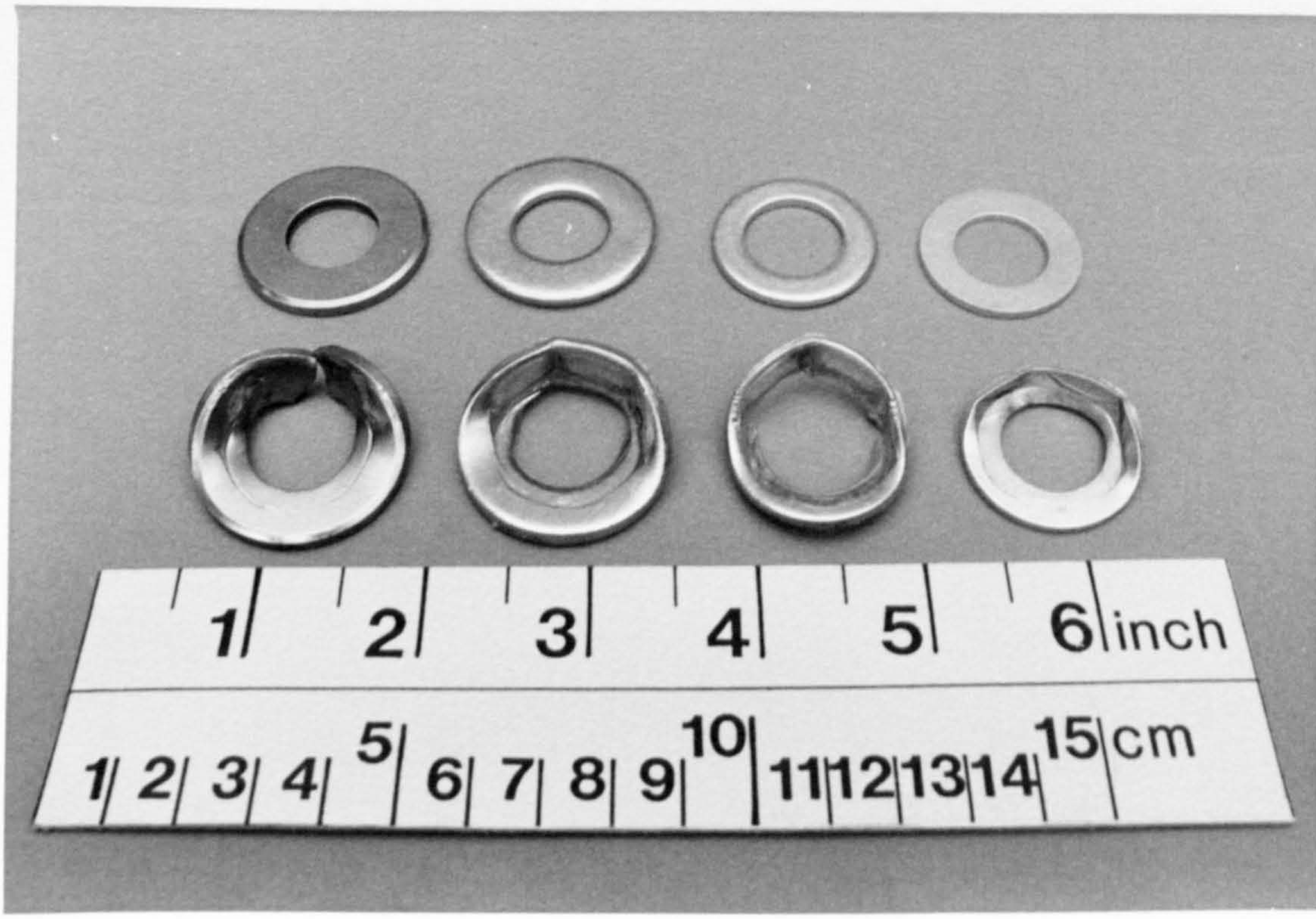


Fig. 4-3 : Washer distortion due to bolt tilting.

From left to right case (d), (c), (a), (b)

Test parameters are shown in Fig. A1.4. Load-extension characteristics are plotted in Figs. A2.13 to A2.15, where a comparison of the characteristics of each case with that of case (a) and also a comparison between cases (c) and (d), for each sheet thickness are made.

Table 4.6 shows that washers (a) were of slightly better quality than that of (b); and washers (d) although fractionally thinner than that of (c), due to their higher quality produced better results.

The ultimate results obtained are shown in Table 4.7. Based on the results tabulated the following are concluded: (Table 4.7)

(1) The improvement in the strength of a lap joint by using a large washer instead of a normal washer, varied from 25% for 1.6mm thick sheets to 5% for 3mm sheets.

This is because with thinner sheets the extra washer diameter provides an added restraint against bolt tilting, hence an improved load carrying capacity. As the sheet thickness increases to the same order as that of the washers the restraining effect of washers compared to that of the connected sheets themselves against bolt tilting reduces, hence a reduction in the extra load carrying capacity.

Nominal sheet thickness (mm)	Nominal washer diameter (mm)	Washer group	Average ultimate load (kN)	Av. ult. load as a ratio of case (a)	Av. ult. load as a ratio of case (c)
1.63	30	(a)	26.5	1.00	-
	30	(b)	26.5	1.00	-
	34	(c)	33.7	1.27	1.00
	34	(d)	34.1	1.29	1.01
2.48	30	(a)	47.2	1.00	-
	30	(b)	42.9	0.91	-
	34	(c)	51.9	1.10	1.00
	34	(d)	53.7	1.14	1.03
3.02	30	(a)	56.3	1.00	-
	30	(b)	49.7	0.88	-
	34	(c)	59.2	1.05	1.00
	34	(d)	62.5	1.11	1.05

Table 4.7 : Effect of type of washers

In terms of design specifications this increase may be incorporated as follows:

15% for $t \leq 2$mm.
5% for $t \leq 3$mm.
No increase should be allowed if $t > 3$mm.

(2) The effect washer material quality on the ultimate strength is proportional to sheet thickness. Compare (a) with (b) and (d) with (c) in Table 4.7 above. The reason being that superior quality washers are less susceptible to bolt tilting, and hence convey an increase in the ultimate load carrying capacity. With thinner steel, the sheets fail at an earlier stage and hence do not utilize the full strength of washers.

The effect of washer quality on the ultimate strength may be ignored. Since a classification of washers in terms of their material quality is not practical and in any case its influence is found to be of secondary importance. (Table 4.7)

Furthermore, examining the load-extension characteristics of the tests (Figs. A2.13 to A2.15 Appendix 2) reveals that washer material or diameter does not bear any significant outcome on the joint characteristics or the slip load.

4.3.2.3. Number and position of washers

Tests were carried out on specimens with normal diameter mild steel washers, i.e. case (a) in the previous section, for all possible assemblies, that is:

(1) 2 washers, one under bolt head and one under nut
(2) 1 washer under bolt head
(3) 1 washer under nut
(4) No washers.

Test parameters are depicted in Fig. A1.5. Load-extension characteristics are plotted and compared with the standard practice of 2 washers, for each case in Figs. A2.16 to A2.18.

The results obtained were extremely consistent and are tabulated below.

Nominal sheet thickness (mm)	Number and position of washers	Average ultimate load (kN)	Average ultimate load as a percentage of specimens with 2 washers
1.63	2 1 under bolt head 1 under nut None	26.5 22.2 21.3 18.2	100% 84% 80% 69%
2.48	2 1 under bolt head 1 under nut None	47.2 39.8 37.9 34.0	100% 84% 80% 72%
3.02	2 1 under bolt head 1 under nut None	56.3 49.1 45.1 39.7	100% 87% 80% 71%

Table 4.8 : Number and position of washers

In the above table if the strength for case (1) is taken as 100%, then the strengths for cases (2), (3) and (4) are 85%, 80% and 70% respectively.

After studying the failed specimens it was realised that the 5% difference or so in strength between cases (2) and (3) was due to the bolt head having sharper corners than the nut. So if there was no washer under the head, its corners would bite into the sheets and precipitate tearing at an earlier stage.

From the load-extension characteristics (Figs. A2.16 to A2.18) it can be seen that the slip load was independent of sheet thickness and the use of washers. The slip load varied between 3 to just over 10 kN. In terms of joint characteristics the only tangible difference was in the deformation capacities. That is, the deformation capacity was greater in case (1) compared to (2) and (3), and those in turn greater than case (4).

Based on the results obtained, it is proposed that a 20% reduction in design bearing strength should be made if only one washer is used, and a 30% reduction should be made if no washers are used. No such allowance is made in Annex A of EC3 at present, and an incorrect allowance is made in BS 5950 Part 5.

4.3.2.4. Mechanical properties of steel sheets

The object of the tests was to see whether an increase in the mechanical properties of the connected sheets would translate into an equal increase in the bearing strength of a bolted joint, as it is presently assumed in all codes of practice. Moreover it was wished to see whether yield or ultimate strengths of sheet steel would best represent the bearing strength of the joint.

To this purpose a series of tests was carried out with a nominal yield stress (σ_y) of 350 N/mm^2 (nominal ultimate stress, σ_{ult} = 490 N/mm^2) and compared with that of the standard tests, i.e. σ_y = 280 N/mm^2.

Test parameters are depicted in Fig. A1.6 (Appendix 1). Load-extension characteristics are plotted and compared with that of standard tests (σ_y = 280 N/mm^2) in Figs. A2.19 to A2.21, Appendix 2.

The average ultimate loads are tabulated in Table 4.9 for nominal 350 N/mm^2 and 280 N/mm^2 yield steels respectively.

The results obtained from this table are then used to compare the increases in the ultimate loads, as a result of using a higher grade of steel, to equivalent increases in the mechanical properties in Table 4.10. The columns of Table 4.9 have been numbered to help illustrate the mathematical operation carried out in Table 4.10.

Type of steel	Sheet thickness (mm)	Actual average : (N/mm^2)		Average ultimate load, P_{ult} (kN)
		σ_y	σ_{ult}	
Nominal		(1)	(3)	(5)
350 N/mm^2	1.57	436.40	482.03	32.5
	2.37	395.96	479.66	47.8
yield steel	3.11	397.20	473.46	65.4
Nominal		(2)	(4)	(6)
280 N/mm^2	1.50	309.11	395.03	24.4
	2.48	337.31	411.98	47.2
yield steel	3.02	317.23	392.83	55.8

Table 4.9 : Test results and actual mechanical properties

Nominal* sheet thickness (mm)	σ_{ult} / σ_y		$\dfrac{P_{ult}\ (\sigma_{y\,350})}{P_{ult}\ (\sigma_{y\,280})}$	$\dfrac{\sigma_{y\,350}}{\sigma_{y\,280}}$	$\left(\dfrac{\sigma_{y\,350}}{\sigma_{y\,280}}\right)^{1/2}$	$\dfrac{\sigma_{ult.\,350}}{\sigma_{ult\,280}}$	$\left(\dfrac{\sigma_{ult\,350}}{\sigma_{ult\,280}}\right)^{1/2}$
	Nominal 350 N/mm^2 yield (4) / (2)	Nominal 280 N/mm^2 yield (3) / (1)	(5) / (6)	(1) / (2)	[(1) / (2)]$^{1/2}$	(3) / (4)	[(3) / (4)]$^{1/2}$
1.5	1.28	1.10	1.27	1.42	1.19	1.22	1.10
2.5	1.22	1.21	1.06	1.17	1.08	1.16	1.08
3.0	1.24	1.19	1.15	1.25	1.12	1.21	1.10

- The actual ultimate loads have been worked out according to these thicknesses , linear interpolation has been used. Therefore in calculating the ratios of ultimate loads, i.e. column (5) / (6), allowance has been made for scaling the actual results to these values.

Table 4.10 : Various statistics relating the test results to their mechanical properties. All figures tabulated are based on actual values.

Emboldened numbers in brackets, refer to the relevant columns numbers in Table 4.9. Compare the values listed in (5) / (6) column to the succeeding ratios.

From Table 4.10 it is clearly evident that the bearing strength does not increase linearly with the yield or ultimate strengths of the connected sheets - but it is proportional to $\sigma_y\ (280/\sigma_y)^{1/2}$ or $\sigma_{ult}\ (390/\sigma_{ult})^{1/2}$. A linear assumption is therefore unsafe.

A numerator of 280 N/mm^2 yield steel has been used as a common bench mark here to correlate the variation in bearing strength, to the yield strength of the connected sheets. This is logical since 280 yield steel is the most common grade of steel used in practice at the present.

Furthermore in order to decide between the yield or ultimate stresses of the connected sheets, which is best used to define the bearing strength - a survey of the actual and assumed design values of the mechanical properties of all the lap tests was carried out. It was found that whenever a nominal 280 N/mm^2 steel was specified, the actual yield stress was in every case at least this, but more often well in excess of 280 N/mm^2. However, an assumed design σ_{ult}/σ_y ratio of 1.4 was not necessarily so. That is a nominal 280 N/mm^2 yield steel did not necessarily imply a minimum 390 N/mm^2 ultimate stress steel. In fact, 43% of the specimens failed to meet the σ_{ult}/σ_y = 1.4 requirement, and their ultimate stresses fell short of the assumed 1.4 ratio. This is also reflected in Table 4.10, where σ_{ult}/σ_y ratio for either steel and all sheet thicknesses was always less than 1.4. Apparently this problem is even worse with imported steels.

It follows that it is more reliable and logical to base the ultimate bearing strength of a joint on the yield stress rather than the ultimate stress of the connected sheets.

Therefore in terms of design specifications, a design strength factor of $\left(\frac{280}{\sigma_y}\right)^{\frac{1}{2}}$ is recommended.

4.3.2.5. <u>End distance of bolt in line of stress</u>

A series of tests were carried out to study the influence of end distance on the mode of failure and the ultimate bearing strength of a bolted connection.

The test programme adopted was to start with shortest end distance of 24mm (e/d = 1.5) and increase the end distance in steps of 12mm (e/d = 0.75) up to a maximum end distance of 96mm (e/d = 6.0). A sample of failed specimens with varying end distances was shown in Fig. 4-1.

Test parameters are depicted in Fig. A1.7. Load-extension characteristics are plotted in Figs A2.22 to A2.24, for sheet thicknesses of 1.63, 2.48 and 3.02mm.

The percentage of variation of ultimate load for any identical sets of test parameters, varied from a maximum of 10.5% to a minimum of 1.6%, with a mean value of 5.3% and a standard deviation 2.3%. It is therefore evident that the results obtained were very consistent.

With some specimens in this group after the connection had slipped the loading was dropped gradually to near zero and then re-loaded back to the original value. It was found that this re-cycling of load did not in any way impair the load extension characteristics of a joint.

The modes of failure observed were as follows:-

(1) sheet tearing - for $e/d \leq 1.5$
(2) sheet bearing - for $e/d \geq 2.25$

Due to the great volume of the data, the results are best represented graphically in Fig. 4-4.

In this figure the non-dimensionalised parameters α ($=P_{ult}/dt\sigma_y$) and e/d are plotted for all tested sheet thicknesses. Each * plotted is the average of at least three identical tests.

It is seen that the ultimate strength P_{ult} increases with end distance up to $e/d = 2.5$, whereafter it remains constant.

It is interesting to note that this in agreement with the results obtained by Chong and Matlock[34], already referred to in § 2.2.3.1.1, page 43 Chapter Two.

4.3.2.5.1. <u>Correlation of the end distance and sheet thickness with the ultimate bearing strength</u>

From Fig. 4-4, it is evident that

for $1.5 \leq e/d < 2.5$

$\alpha = f(e/d, t)$ Where $\alpha = \dfrac{P_{ult}}{dt\sigma_y}$

and for $2.5 \leq e/d$

$\alpha = f(t)$ i.e. α is independent of e/d

Derivation of the correlation between α and sheet thickness for $e/d \geq 2.5$ was a straight forward matter.

Plotting α against t, for $e/d \geq 2.5$, and fitting a lower bound least squares estimate to the data gives : (Fig. 4-5)

$$\alpha = (2.6 + 0.35\,t) \qquad \text{For } e/d \geq 2.5 \qquad (1)$$

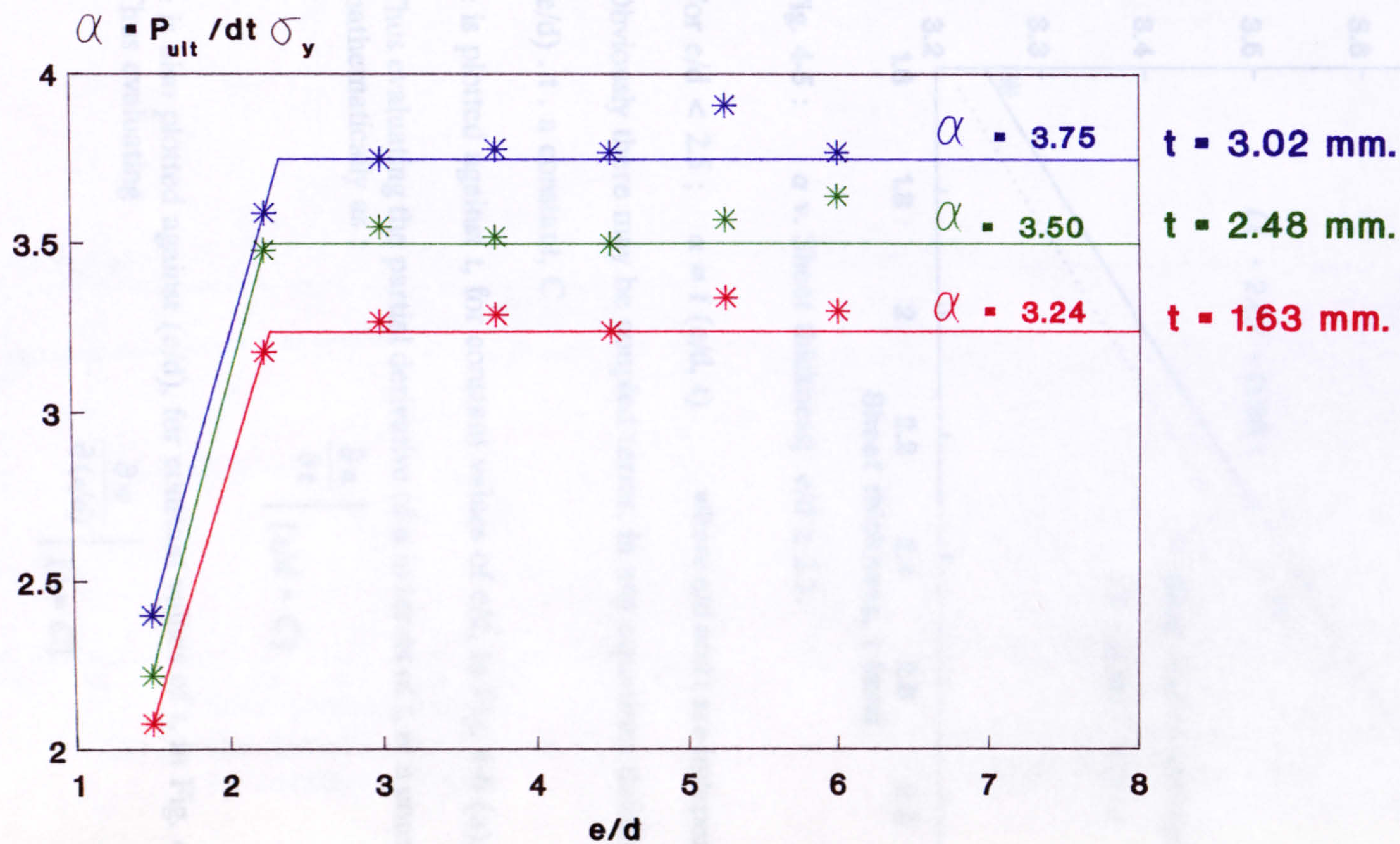


Fig. 4-4 : α v. (e/d)

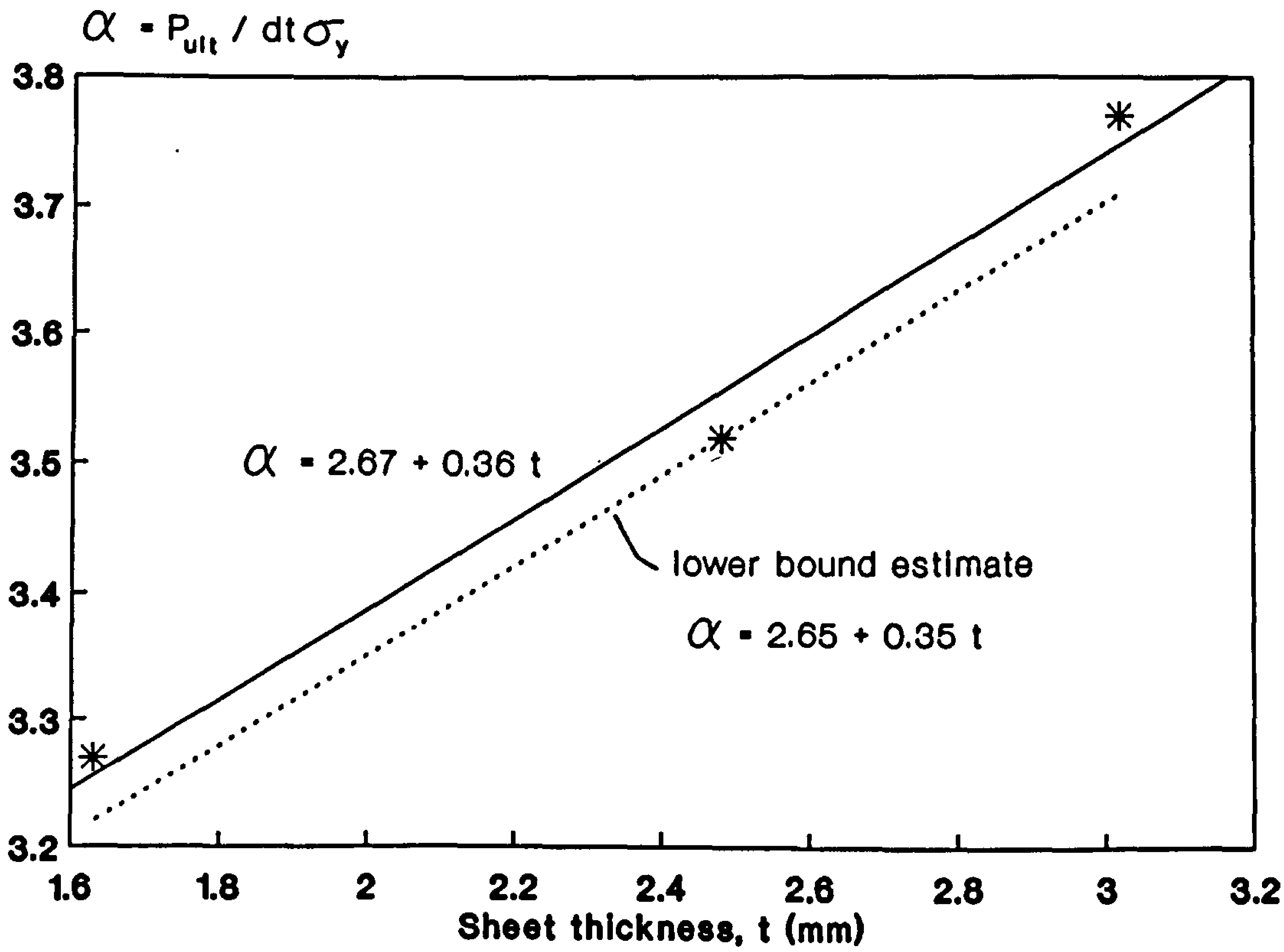


Fig. 4-5 : α v. Sheet thickness; $e/d \geq 2.5$.

For $e/d < 2.5$; $\alpha \equiv f(e/d, t)$ where e/d and t are independent of each other.

Obviously there may be coupled terms, in any equations defining α, in form of ;

(e/d) . t . a constant, C

α is plotted against t, for constant values of e/d, in Fig. 4-6 (a).

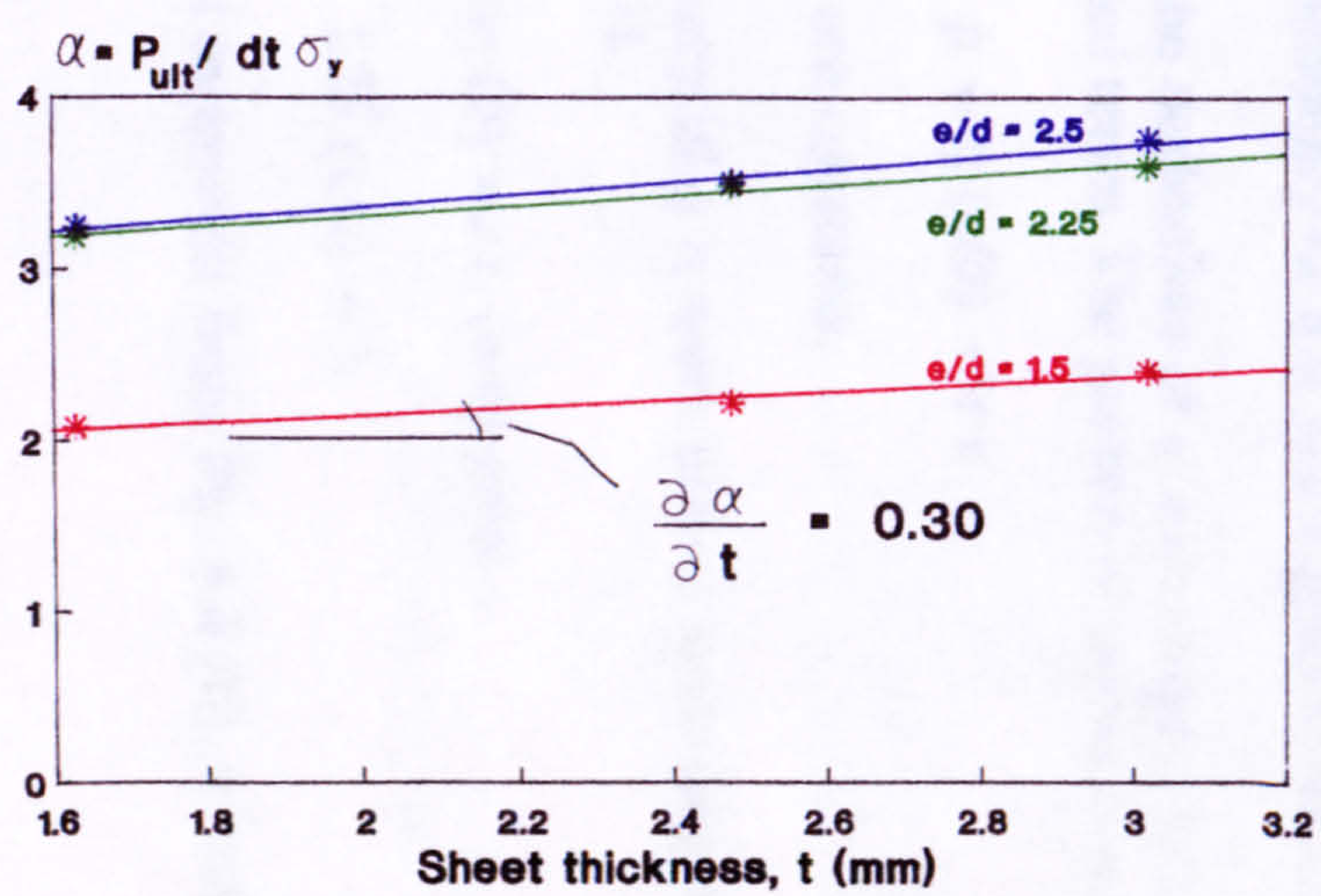
Thus evaluating the partial derivative of α in terms of t, at a constant e/d. Expressed mathematically as :

$$\left.\frac{\partial \alpha}{\partial t}\right|_{[e/d = C]}$$

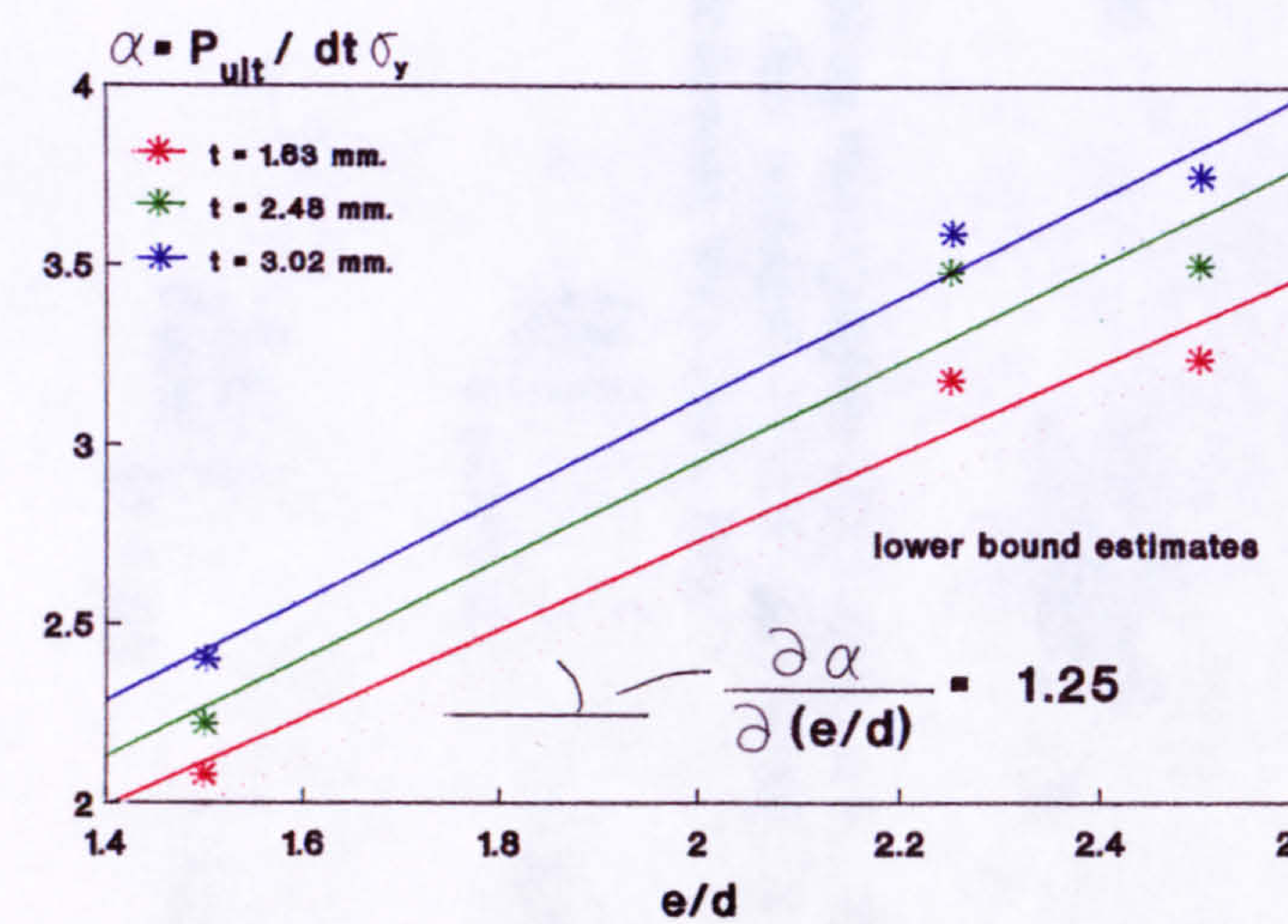
α is also plotted against (e/d), for constant values of t, in Fig. 4-6 (b).
Thus evaluating

$$\left.\frac{\partial \alpha}{\partial (e/d)}\right|_{[t = C]}$$

For any set of data a best fit is obtained by the least squares method.



(a) Partial derivative of α w.r.t. t, for constant (e/d)'s.



(b) Partial derivative of α w.r.t. (e/d) for constant sheet thickness, t.

Fig. 4.6 : (e/d) < 2.5

From the above mentioned figures it may be seen that in each case the rate of change of gradient with respect to the other variable is negligible.

i.e. in Fig. 4-6 (a)

$$\frac{\partial \left(\dfrac{\partial \alpha}{\partial t} \right)}{\partial (e/d)} = 0$$

and in Fig. 4-6 (b)

$$\frac{\partial \left(\dfrac{\partial \alpha}{\partial (e/d)} \right)}{\partial t} = 0$$

Therefore uncoupling the equation defining α. A lower bound estimate is fit to each case (i.e. taking the smallest of the gradients in each figure) to obtain the derivatives. The following was deducted;

Fig. 4-6 (a)
$$\left. \frac{\partial \alpha}{\partial t} \right|_{[e/d = C]} = 0.30 \tag{2}$$

Fig. 4-6 (b)
$$\left. \frac{\partial \alpha}{\partial (e/d)} \right|_{[t = C]} = 1.25 \tag{3}$$

Therefore the dependency of α is much greater on e/d rather than t.

It is evident that the derivatives of α with respect to t and (e/d) are constant; and there are no coupled terms. The general equation of α may therefore be defined as:

$$\alpha = p + q\,(e/d) + r\,t \tag{4}$$

Where p, q and r are constants.

Since the dependency of α is more biased towards (e/d), more weight is therefore put on equation (3).

Integrating equation (3) w.r.t. (e/d) gives :

$$\alpha = 1.25\,(e/d) + k$$

where $k \equiv f(t)$ and as evident from Fig. 4-6 (b), it will be a linear function.

From Fig. 4-6 (b), the following relationships are obtained:

$$\text{for} \quad t = 1.63 \quad \alpha = 1.25\,(e/d) + 0.11$$
$$t = 2.48 \quad \alpha = 1.25\,(e/d) + 0.35$$
$$t = 3.02 \quad \alpha = 1.25\,(e/d) + 0.53$$

plotting the constant terms above, against t gives k as a linear function of t.

i.e. $k = 0.35\,t - 0.5$

The factor 0.35 above agrees closely to that in equation (2). It also ties in with that in equation (1).

Hence equation (4) becomes

$$\alpha = -\,0.5 + 1.25\,(e/d) + 0.35\,t$$

Therefore from a lower bound analysis of least squares fit to data, it is concluded that the results are best defined by the following equations :

For $1.5 \leq e/d < 2.5$

$$\alpha = \{1.25\,(e/d) - 0.5\} + 0.35\,t$$

For $e/d \geq 2.5$

$$\alpha = 2.6 + 0.35\,t$$

It is evident that the reduction in the ultimate bearing strength as a result of having end distances less than 2.5 in the line of stress is given by a factor , say λ, as :

$$\lambda = \frac{1.25\,(e/d) - 0.5}{2.6}$$

It was realised that the approach adopted in EC3 Part 1 in defining λ (e/3d, for e/d > 3) for hot rolled steel connections could be modified to adequately represent the above equation. i.e.;

$$\lambda = \frac{e}{2.5\,d}$$

A comparison of the above two equations is carried out in Table 4.11.

e/d	λ: $\frac{1.25\,(e/d)\ -\ 0.5}{2.6}$	λ: $\frac{e}{2.5\,d}$
1.5	0.53	0.60
2.25	0.89	0.90
2.5	1.00	1.00

Table 4.11 : Reduction factors for bearing strength, for (e/d) ≤ 2.5.

The strength may therefore be generally described by making α the lesser of e/2.5d or 1.

This is a splendidly simple solution to what used to be an extremely set of complicated equations in previous drafts of EC3 Annex A, involving natural logarithmic design expressions with two different gradients for different ratios of e/d.

The same comments equally apply to BS 5950 Part 5 design expressions which are essentially a conservative simplification of the previously existing equations in Annex A.

The proposed expression also defines the ultimate strength more accurately than either of the design expressions referred to above.

4.3.2.6. Bolt diameter

The great majority of tests were made with 16mm diameter bolts. Nevertheless, the test programme was extended to cover connections with 12mm diameter bolts.

It has already been mentioned that 16mm diameter bolts are the most common form of fasteners used in structural connections in cold formed steel sections. One leading manufacturer in the U.K. however, due to the form and size of their sections, which does not allow the necessary space for handling and fitting of M16 bolts, uses grade 8.8 M12 bolts as standard in their main structural connections. This is more of an exception rather than the general rule.

The test parameters are depicted in Fig. A1.8. Load-extension characteristics are plotted and compared with that of 16mm diameter bolts in Figs. A2.25 to A2.27.

From load-extension plots it is seen that the characteristics of lap joints with different size bolts followed a similar pattern. The slip load however was, on average, higher than that of 16mm diameter bolts for the same bolt torque of 65 Nm. This is reasonable since the same torque applied to a smaller diameter bolt should have a more "gripping effect". This should also be the case in site conditions. Therefore any prediction of the slip load for 16mm diameter bolts should safely apply to 12mm diameter bolts.

The ratios of the ultimate loads achieved were not proportional to the bolt diameters, as it has been assumed in all current codes of practice, but were approximately proportional to the square root of the diameters as shown in Table 4.12.

Nominal sheet thickness (mm)	Average ult. load for 12mm ϕ bolts (kN)	Ratio of av. ult. loads for 16mm ϕ and 12mm ϕ bolts	$\frac{d_{16}}{d_{12}}$	$\left(\frac{d_{16}}{d_{12}}\right)^{1/2}$
1.63	23.1	1.15	1.33	1.15
2.45	39.8	1.17		
3.12	50.7	1.14		

Table 4.12 : Ultimate strength of lap joints with 12mm and 16mm ϕ bolts.

The scope of the test results have been widened by considering bolt diameters further a field. That is, including results for 10 and 16mm ϕ from Strathclyde University and 20mm ϕ bolts from Chapter Two.

The non-dimensionalised parameter α is plotted against bolt diameter for d = 10, 12, 16 and 20mm , with t = 3.0 mm in Fig. 4-7. This covers the entire range of bolts, and over, in the structural bolted connections in cold formed steel.

From Fig. 4-7 it is clearly evident that $\alpha \approx (16/d)^{1/2}$ and a linear assumption between the ultimate load and bolt diameter taken by all present codes of practice is not correct.

16mm diameter bolt is used here as a common bench mark since, as mentioned previously, it is the most common form of bolt used in structural connections in cold formed sections.

It may thus be taken that the bearing strength of a lap joint with bolt diameter d

is proportional to : $$d\left(\frac{16}{d}\right)^{1/2} .$$

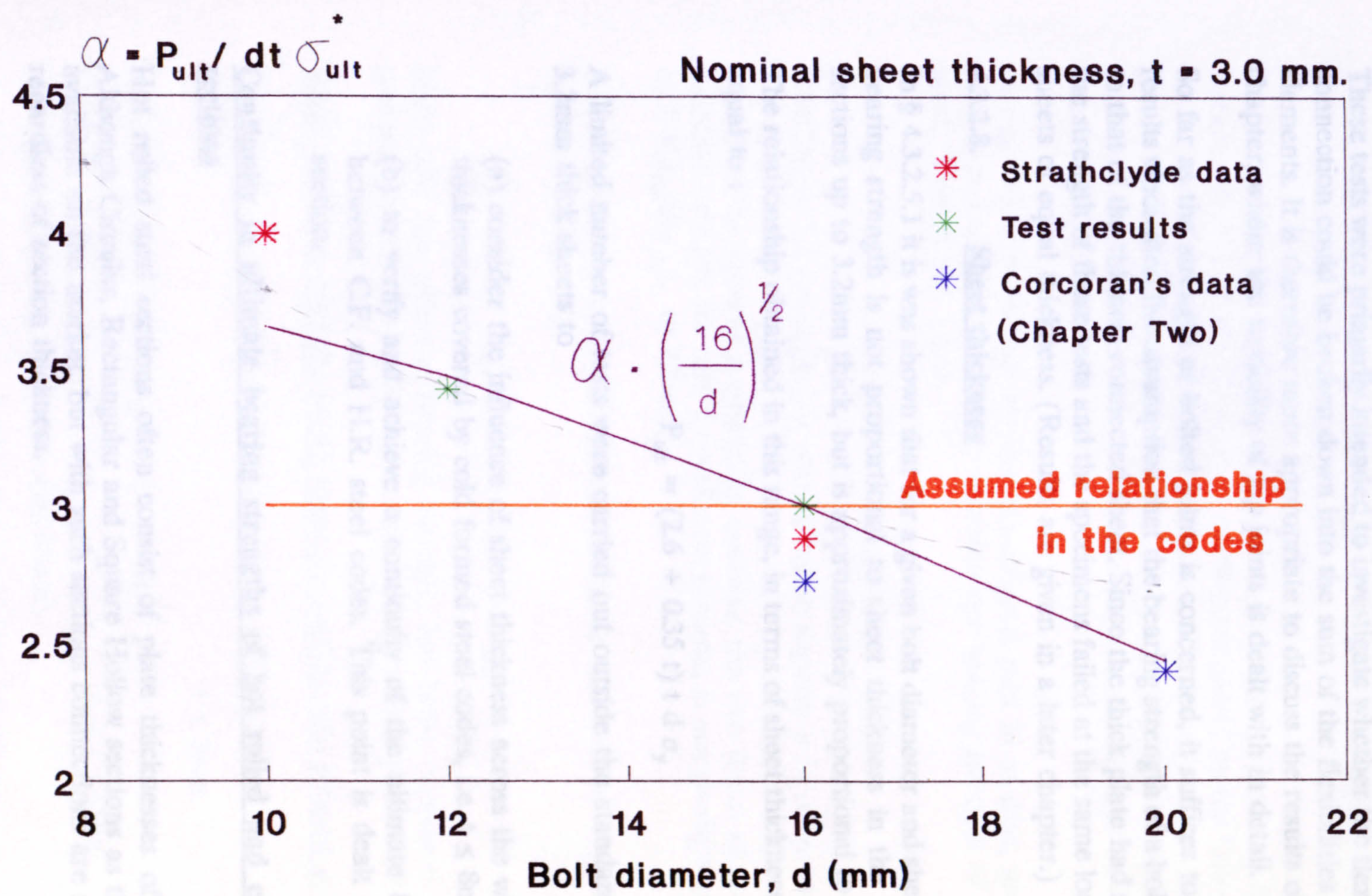


Fig. 4-7 : α v. bolt diameter, d

*Note that α is expressed in terms of σ_{ult}

4.3.2.7. Thin/thick sheets

Tests were carried out on single cold formed sheets (for the standard thickness range of 1.5 to 3.2) bolted to a reinforced 9mm thick plate.

These tests were primarily intended to investigate whether the flexibility of a bolted connection could be broken down into the sum of the flexibilities of its constituent elements. It is therefore more appropriate to discuss the results obtained in a later chapter, where the flexibility of lap joints is dealt with in detail.

So far as the strength of bolted joints is concerned, it suffices to say that the test results vindicated the assumption that the bearing strength of a bolted joint, is equal to that of the thinnest connected sheet. Since the thick plate had no influence over the strength of these tests and the specimens failed at the same loads as that of two sheets of equal thickness. (Results are given in a later chapter.)

4.3.2.8. Sheet thickness

In § 4.3.2.5.1 it is was shown that for a given bolt diameter and sheet properties, the bearing strength is not proportional to sheet thickness in the normal range of sections up to 3.2mm thick, but is approximately proportional to $(\text{thickness})^2$.

The relationship obtained in this range, in terms of sheet thickness, was found to be equal to :

$$P_{ult} = (2.6 + 0.35\ t)\ t\ d\ \sigma_y$$

A limited number of tests were carried out outside the standard range of 1.5 to 3.2mm thick sheets to

(a) consider the influence of sheet thickness across the whole spectrum of thicknesses covered by cold formed steel codes, i.e. $t \leq 8$mm; and

(b) to verify and achieve a continuity of the ultimate bearing strengths between C.F. and H.R. steel codes. This point is dealt with first, in this section.

Continuity in ultimate bearing strengths of hot rolled and cold formed steel sections

Hot rolled steel sections often consist of plate thicknesses of 6mm and over. Although Circular, Rectangular and Square Hollow sections as thin as 2.0mm are available on the market, but with such sections connections are made by welding, regardless of section thickness.

It follows that there is an area of overlap between the two codes of practice, approximately in the thickness range of 6 to 8 mm. It is obviously desirable to obtain a continuity between the two forms of steel.

The overall perspective of the codes for the two forms of steel is shown in Fig. 4-8.

In this figure α ($=P_{ult}/dt\sigma_{ult}$) has been plotted against sheet thickness. Note that here, α has been based on the ultimate strength and not the yield strength of the connected sheets. This is a requirement of EC3 that all equations defining ultimate load carrying capacities <u>must</u> be based on σ_{ult} and not σ_y. A ratio of $\sigma_{ult}/\sigma_y = 1.4$ has been assumed in plotting Fig. 4-8.

From Fig. 4-8, can be seen that, as a result of efforts made by Salford, continuity between EC3 Part 1 and Annex A has been achieved by the "improved" expression for Annex A, but not in the BS 5950 values. Fig. 4-8 also shows that BS 5950 Part 1 gives very conservative values for bearing strength of hot rolled steel sections. It is recommended that steps should be taken by the appropriate bodies to bring this in line with EC3 Part 1.

In Fig. 4-8, it is also shown that *the original best fit to data, given in § 4.3.2.5.1*, results in a small discontinuity between the bearing strength of the two forms of steel. That is, for $t \geq 3$

$$\alpha = (2.6 + 0.35 \times 3) / 1.4 = 2.6$$

Division by 1.4 ($= \sigma_{ult}/\sigma_y$) is carried out to express α in terms of σ_{ult}.

Therefore to achieve a continuity between the two forms of steel the maximum ultimate strength, for cold formed steel sections, is marginally suppressed from 2.6, by $0.1(P_{ult}/dt\sigma_{ult})$, to 2.5 $dt\sigma_{ult}$.

The modified equation therefore defining the influence of sheets up to 3mm thick becomes : (Fig. 4-8)

$$P_{ult} = (1.9 + 0.2\ t)\ t\ d\ \sigma_{ult}$$

Note that the ultimate bearing strength has been expressed in terms of σ_{ult}, assuming a ultimate / yield stress ratio of 1.4, to satisfy the EC3 requirements.

<u>Sheet thickness over the whole range covered by cold formed steel codes</u>

From Fig. 4-8 it is deducted that for

$t \leq 3$mm

the ultimate bearing strength $\propto$ (thickness)^2

and for $3 < t \leq 8$mm

the ultimate bearing strength $\propto$ (thickness)

Tests were carried out to cover a range of 0.9 to 6.2mm thick sheets. The results are plotted in Fig. 4-9. Each point plotted in this figure is the result of a lower bound analysis on a great many number of tests. For the thinnest sheets (t=0.9mm) data

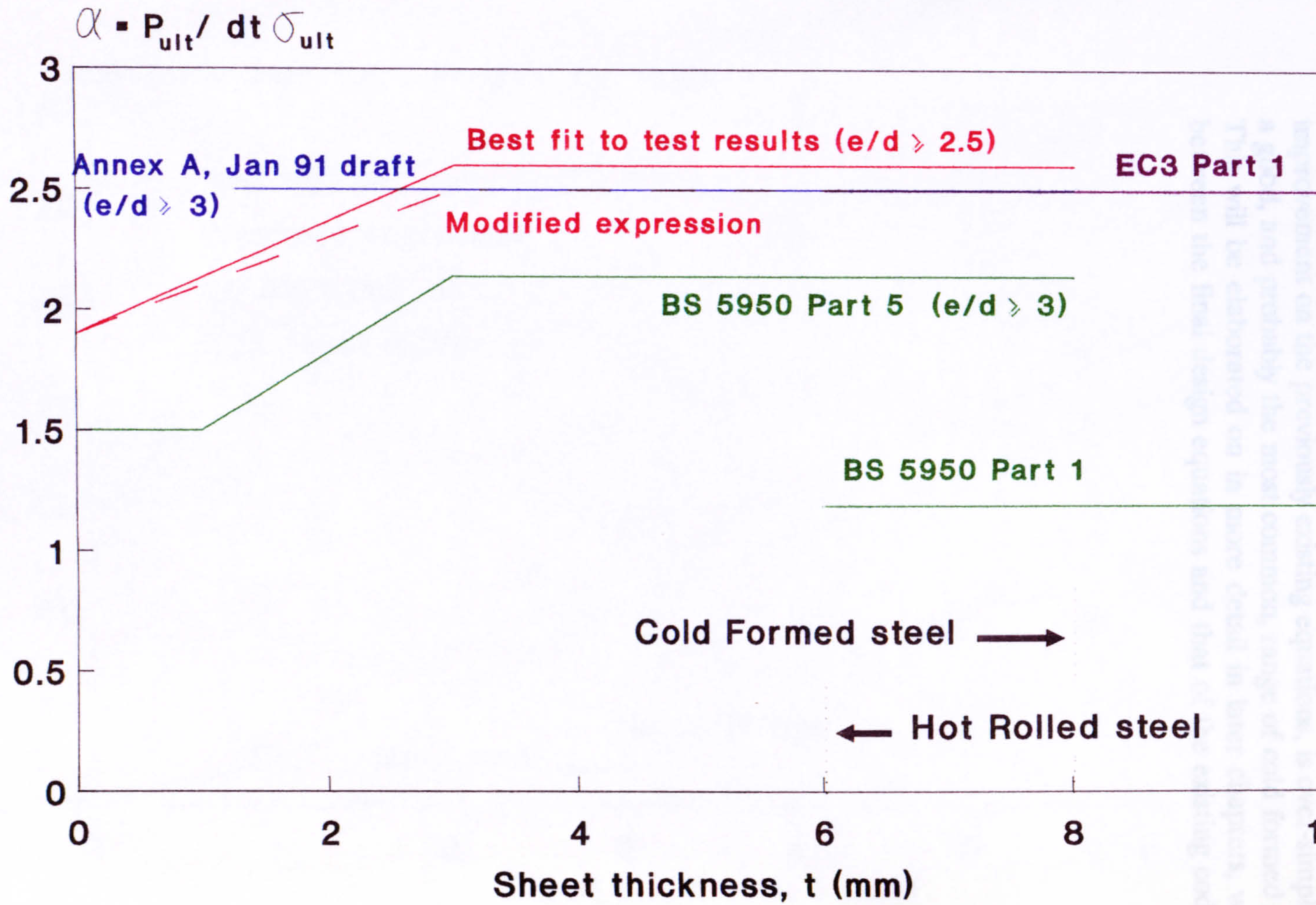


Fig. 4-8
Ultimate bearing strength of bolted connections in CF. & HR. codes.

obtained from Strathclyde University has been used.

It is concluded that the presumption made above (i.e. $P_{ult} \propto t^2$ for $t \leqslant 3mm$) yields the most logical fit to the data.

It is also believed that the "modified" EC3 Annex A equation, although a major improvement on the previously existing equations, is over-simplified and unsafe for a good, and probably the most common, range of cold formed section thicknesses. This will be elaborated on in more detail in later chapters, where a comparison between the final design equations and that of the existing codes is made.

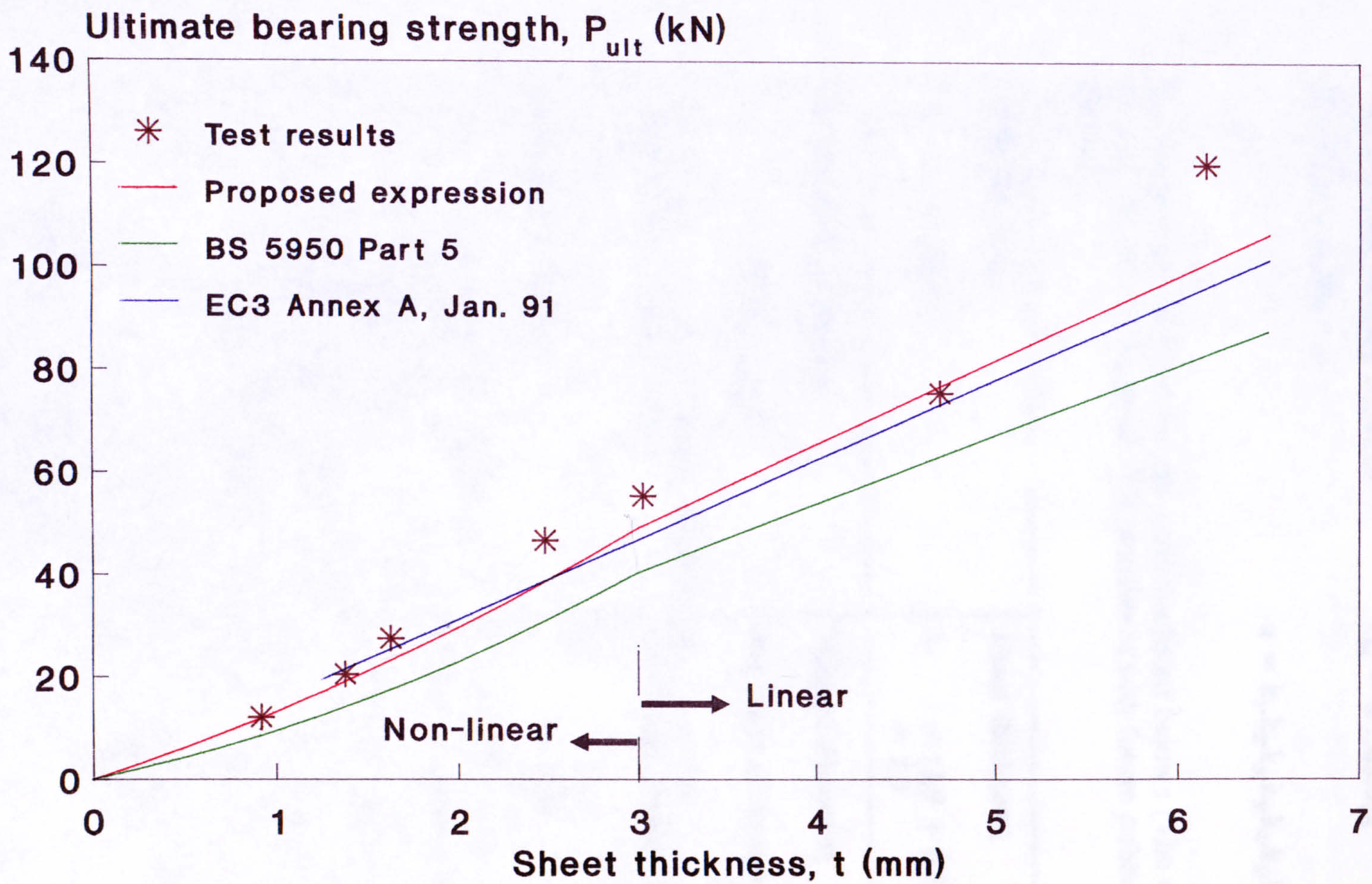


Fig. 4-9 : P_{ult} v. t

Assumed d = 16mm ϕ , σ_y = 280 N/mm^2

4.4. Design expression for strength - Lap joints

The design expression for bearing strength, P_{bs}, taking into account all factors described in § 4.3 is defined as :

The Ultimate bearing strength $\quad P_{bs} = \alpha \, . \, d.t.\sigma_{ult}$

Where α is defined as :

$$\alpha = k_1.k_2.k_3.k_3.k_4.k_5.k_6.k_7$$

k_1 to k_7 are factors given for the variables listed below : (The order in which these factors are listed is designed to be consistent with future publications based on this thesis.)

Bolt diameter; $k_1 \quad = (16/d)^{½}$	**Sheet thickness;** $k_2 \quad = (1.9 + 0.2\,t)$ for $t \leq 3mm$ $\quad\;\; = 2.5 \quad\quad 3 < t \leq 8mm$
Mechanical properties; $k_3 \quad = (390/\sigma_{ult.\,design})^{½}$ where $\sigma_{ult.\,design}$ is the design ultimate stress of the sheet material.	**Washer diameter;** For Normal diameter washers, (Form E, BS 4320) $k_4 \quad = 1.0.$ For Large diameter washers, (Form F, BS 4320) $k_4 \quad = 1.15$ for $t \leq 2mm$ $\quad\;\; = 1.05$ for $2 < t \leq 3mm$ $\quad\;\; = 1.0$ for $t > 3mm$
Number of washers; $k_5 \quad = 1.0$ when two washers are used. $\quad\;\; = 0.8$ when only one washer is used. $\quad\;\; = 0.7$ when no washers are used.	**End distance in the line of stress;** $k_6 =$ the lesser of $(e/2.5d)$ and 1. where $(e/d) \geq 1.5$.
Shear plane on the plain shank or threads of bolts; $k_7 \quad = 1.15 \quad$ where it can be shown be shown that the shear plane occurs over the full shank diameter. $\quad\;\; = 1.0 \quad$ otherwise	

It should be mentioned that had it not been for EC3 requirements, it would have been preferred to define the bearing strength in terms of σ_y, assuming a σ_{ult}/σ_y ratio of 1.4, rather than σ_{ult}.

4.5. Conclusions

The strength of bolted connections in cold formed sections is influenced by a number of factors which have been investigated and individually quantified. A design expression has been propounded. In the following chapters it will be shown that the proposed expression is a significant advance on those given in the existing codes of practice. The proposed expression defines all significant factors in the present day industry, most of which have not even been considered in the existing codes, with considerable accuracy.

A continuity in the ultimate bearing strength of bolted connections in cold formed and hot rolled steel sections has been achieved.

Chapter Five

Interaction of bearing strength with other modes of failure

5. Interaction of bearing strength with other modes of failure

Summary

Interaction of sheet bearing with other modes of failure of bolted connections is briefly considered in this chapter.

5.1. Sheet tearing

Sheet tearing is defined implicitly in the design equation propounded in the previous chapter. Bolt bearing was described as a more general mode of failure, comprising of both sheet tearing and bearing modes of failure in Chapter One, (Fig. 1-8) page 20. This mode of failure (bolt bearing) is effectively what the proposed design expression represents.

In Chapter One it was shown that the end distance in line of stress was the governing factor on whether connection failure occurs in sheet bearing or sheet tearing.

The governing equation was shown to be equal to :

$$\alpha = 1.2\ (e/d)$$

It has been shown in the previous chapter that the factor α, defining the level of the bearing strength, in itself depends on other parameters.

Considering the range of structural cold formed steel sections i.e. $t = 1.2$ to 3.2 mm, then α is given as :

$\alpha = 1.9 + 0.2\,t$ (assuming other factors are equal to unity)

$t = 1.2$ mm : $\alpha = 2.14$

$t = 2.0$ mm : $\alpha = 2.3$

$t \geq 3$ mm : $\alpha = 2.5$

It follows that the end distance at which the mode of failure changes from sheet tearing to sheet bearing, varies from ;

$t = 1.2$ mm : $(e/d) = 2.14 / 1.2 = 1.8$

$t = 2.0$ mm : $(e/d) = 2.3 / 1.2 = 1.9$

$t \geq 3$ mm : $(e/d) = 2.5 / 1.2 = 2.1$

Considering the mid range of sheet thickness in cold formed steel sections, t = 2 mm (say), then (e/d) = 1.9 may be taken as a typical intersection of the two modes of failure.

This is strongly supported by the test data available at Salford and various other institutions.

5.2. Net section failure

The intersection of sheet bearing and net section mode of is considered here for completeness.

$$P_{bs} = P_n$$
$$\alpha\, d.t.\sigma_{ult} = A_n \,.\, \sigma_n$$

Since the bearing strength is considered in terms of σ_{ult}, in accordance with EC3 requirements, for consistency, σ_n will also be taken as σ_{ult} as specified by EC3.

$$\alpha\,.\,d.t.\sigma_{ult} = b_n.t.\,\sigma_{ult}$$
$$\alpha = b_n/d$$

b_n is the effective breadth of the net section. Its value is governed by the bolt diameter and the spacing of bolts perpendicular to the line of stress, which is multiplied as a factor to σ_y in BS 5950 Part 5; and σ_{ult} in Annex A. b_n is obviously limited to a maximum of specimen width minus the bolt(s) hole diameter(s).

To take a simple example, consider a lap joint consisting of flat elements in the mid range of sheet thicknesses in cold form steel bolted connections, say t = 2 mm. Then for simplicity, assuming $\alpha = (1.9 + 0.2\ t)$, with other factors assumed to be equal to unity, and b_n being the specimen width minus the bolt hole diameter. Then with a 16mm diameter bolt:

$$b_n = (1.9 + 0.2 \times 2)\ 16 = 37 \text{ mm.}$$

That is for flat 2 mm specimens with only one 16 mm ϕ bolt. In case of specimens with lips then an effective area concept, taking account of the effect of lips may be considered. In such cases, from BS 5950 Part 5 then ;

$$A_n = a_1\,(3a_1 + 4a_2)\ /\ (3a_1 + a_2)$$

where a_1 = the net sectional area of the connected leg; (i.e. b_n x t , as above)

and a_2 = the gross sectional area of connected leg or legs. (say 2 x b/5 x t, assuming lips to be one fifth of specimen width)

Since this is of no significant interest in this thesis, it will not be considered any further.

5.3. Shearing of bolt

What has been described so far in this chapter, has mainly been of academic interest. The interaction of sheet bearing and shearing of bolt modes of failure however, has a considerable design significance in bolted connections in cold formed steel sections. This is primarily because of the bolt tilting phenomenon.

Shearing of bolt mode of failure was defined in Chapter One as :

$$P_s = A_s \cdot \sigma_s$$

Assuming that the shear plane is on the threaded part of the bolt, A_s is equal to the tensile stress area of the bolt. Values of σ_s are given for different grades of bolts in Table 11 of BS 5950 Part 5.

The interaction of sheet bearing and shearing of bolt modes of failure is best illustrated in Fig. 5-1.

The shear capacities of grades 4.6, 8.8 and 10.9 bolts, have been expressed in terms of α ($=P_s/dt\sigma_{ult}$) and compared against the ultimate bearing strength of the connected sheets as given by the proposed design equation in Chapter Four, BS 5950 Part 5 and Annex A of EC3.

Assuming a bolt diameter of 16 mm (M16) and an ultimate tensile stress of 390 N/mm^2, for the connected sheets, the equations defining the shear capacities, of the above mentioned grades of bolts in terms of α is obtained as :

(Shear strength of bolts are calculated in accordance with Table 11, BS 5950 pt.5.)

Grade 4.6 bolts :

$$P_s = A_s \cdot \sigma_s = 157 \times 160 = 25.120 \text{ kN}$$

$$\alpha = P_s/dt\sigma_{ult} = 25120 / (16 \times t \times 390) = 4.03/t$$

Grade 8.8 bolts :

$$P_s = 157 \times 375 = 58.875 \text{ kN}$$

$$\alpha = 58875 / (16 \times t \times 390) = 9.44/t$$

Grade 10.9 bolts:

$$P_s = 157 \times 480 = 75.360 \text{ kN}$$

$$\alpha = 75360 / (16 \times t \times 390) = 12.08/t$$

Considering grade 4.6 bolts (which are the most common grade used in structural bolted connections, in cold formed sections, in the U.K.) and comparing their shear

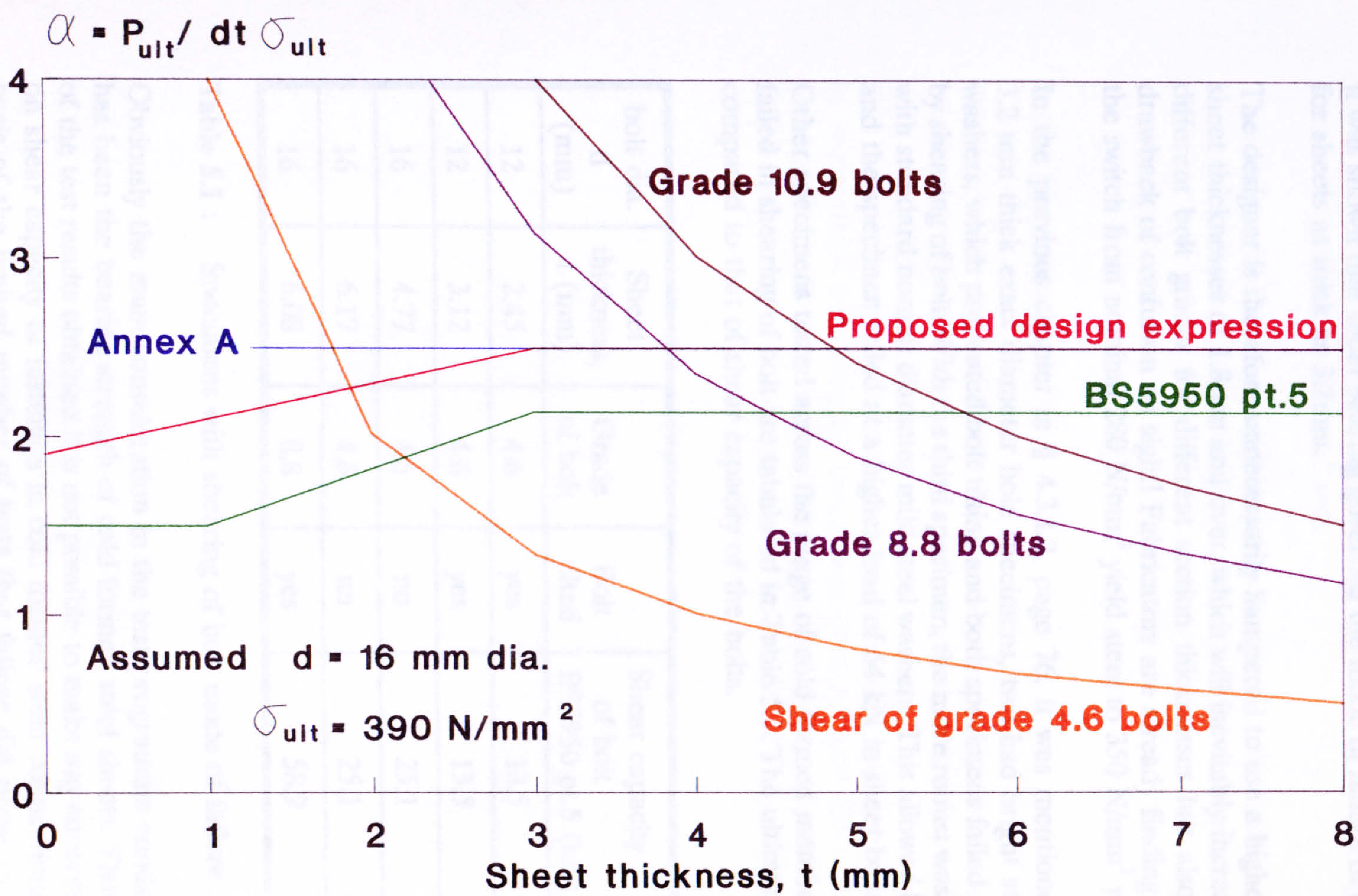


Fig. 5-1
Interaction of sheet bearing & shearing
of bolt modes of failure

strength capacity with that of the bearing strength of the connected sheets, as given by the proposed design expression - it is seen from Fig. 5-1 that the shearing of bolt mode of failure dominates the design considerations for sheet thicknesses of 1.8mm and over. With grade 8.8 and 10.9 bolts this thickness is equal to 3.8mm and 4.8mm respectively. Yet in the previous chapter in tests on lap joints with grade 4.6 bolts, it was shown that sheet bearing governed the mode of failure of bolted connections for sheets as thick as 3.2mm.

The designer is therefore unnecessarily hampered to use a higher grade of bolt for sheet thicknesses of 1.8mm and over, which will inevitably increase the cost. Using different bolt grades for different section thicknesses has also a more practical drawback of confusion on sight! Fabricators are already finding this difficulty with the switch from nominal 280 N/mm^2 yield steel to 350 N/mm^2 yield.

In the previous chapter in § 4.3.1.2, page 76, it was mentioned that with three 3.2 mm thick exact diameter hole specimens, two had bright steel large diameter washers, which prevented bolt tilting and both specimens failed at a load of 54 kN, by shearing of bolt. With the third specimen, the more robust washers were replaced with standard normal diameter mild steel washers. This allowed bolt tilting to occur and the specimen failed at a higher load of 64 kN in sheet bearing.

Other specimens tested across the range of cold formed members thickness which failed in shearing of bolt are tabulated in Table 5.1. The ultimate loads obtained is compared to that of shear capacity of the bolts.

bolt dia. d (mm)	Sheet thickness, t (mm)	Grade of bolt	Bolt tilted	Shear capacity of bolt BS5950 pt.5 (kN)	Ultimate load (kN)
12	2.45	4.6	yes	13.5	36.0
12	3.12	4.6	yes	13.5	35.0
16	4.77	4.6	no	25.1	56.8
16	6.17	4.6	no	25.1	57.3
16	6.08	8.8	yes	58.9	72.0

Table 5.1 : Specimens with shearing of bolt mode of failure.

Obviously the main consideration in the test programme conducted for this thesis has been the bearing strength of cold formed steel sheets. Therefore on the basis of the test results obtained it is not possible to make any concrete recommendation on shear capacity of fasteners in cold formed steel connections. However, on the basis of the limited number of tests that failure did occur in the fastener - it is suggested that a more favourable shear capacity for grade 4.6 bolts in connections with sheets less than 3.2 mm thick, where bolt tilting is known to occur and the actual mode of failure for the majority of design cases is known to be in sheet bearing, is prudent.

Based on the results above and other test results in Appendix 2, which clearly show that the strength of connected sheets dominates the mode of failure of bolted lap joints, even with the thicker range of sheet thicknesses - a bolt tilting factor of 2 may be used in conjunction with bolted connections made of grade 4.6 bolts and sheet thicknesses less than 3.2 mm.

The alternative is to use grade 8.8 bolts for the thicker range of cold formed sections, whilst its known that grade 4.6 bolts will be adequate.

5.4. Conclusions

The mode of failure of bolted connections changes from sheet tearing into sheet bearing at a typical end distance - bolt diameter ratio of; (e/d) = 1.9.

The shear capacity of grade 4.6 bolts may be increased by a bolt tilting factor of 2, in connections consisting of sheets less than 3.2 mm thick. However more test information with bolts from different manufactures and different diameters are required before the conclusion drawn can be fully justified.

Chapter Six

Comparison of the proposed design expressions for the bearing strength and other codes of practice

6. Comparison of the proposed design expressions for the bearing strength with other codes of practice

Summary

The ultimate bearing strengths of bolted lap joints are listed and compared with that predicted by the proposed design expression in Chapter Four, EC3 Annex A and BS 5950 Part 5. Results of other research institutions are also considered. First however, the background to the design equations in the two above mentioned codes is given.

6.1. EC3, Annex A

First point of contact of Salford with Annex A was in December 1988. When it was first realised that the bearing strength equations given in what was then the latest draft of Annex A did not match the overall format of the test results emerging at Salford. The design equations for bearing strength were as given below :

$$F_b^* = \alpha \, d.t.f_y$$

where for $t \leq 1mm$:

$$\alpha = 2.1$$

for $1mm < t < 3mm$:

for $(e/d) \leq 6$ $\quad \alpha = \{ 2.6 - 0.5t + 0.9(t-1) \ln(e/d) \}$

for $(e/d) > 6$ $\quad = (1.0 + 1.1\,t)$

for $t \geq 3mm$:

for $(e/d) \leq 6$ $\quad \alpha = \{1.1 + 1.8 \ln(e/d) \}$

for $(e/d) > 6$ $\quad = 4.3$

With $(e/d)_{min.} = 1.5$.

Three months later, in March 1989 draft of Annex A, the above equations were expressed in terms of f_u, in the following form ;

$$F_b^* = \alpha \, d.t.f_u$$

Where for $t \leq 1mm$:

$$\alpha = 2.1$$

for $1mm < t < 3mm$:

for $(e/d) \leq 3$ $\quad \alpha = \{ 2.1 + (0.3\,(e/d) - 0.45)(t-1) \}$

for $3 < (e/d) \leq 6$ $\quad = \{ 2.1 + (0.22\,(e/d) - 0.21)(t-1) \}$

for $(e/d) > 6$ $\alpha = (1.0 + 1.1\,t)$

for $t \geq 3$mm

for $(e/d) \leq 3$ $\alpha = \{1.2 + 0.6\,(e/d)\}$

for $3 < (e/d) < 6$ $= \{1.68 + 0.44\,(e/d)\}$

for $(e/d) \geq 6$ $= 4.3$

Therefore the original natural logarithmic curve, expressing the variation of bearing strength with the end distance in line of stress for $(e/d) \leq 6$, was replaced with two separate linear gradients, for any given sheet thickness. More importantly, the bearing strength was effectively increased by a factor of 1.4, in replacing f_y by f_u, in the new expressions.

Following this Salford expressed its concern followed by publication of a report in July 1989. As a result the March 89 proposals were replaced by $f_u/1.4$.

This was then followed by a meeting at Delft, attended by TNO, Aachen and Salford, in September 1989 to particularly discuss the bearing strength of structural connections in detail. At this meeting Salford aired two main points :

1. Bearing strength equations for bolted connections, given in Annex A were not representative of the insitu conditions.

2. For the first time attention was drawn to the incompatibility of design equations for the two forms of steel, i.e hot rolled and cold formed.

This then sparked off a period of exchange of ideas and discussions over the bearing strength of structural bolted connections, discussed at the following meetings of working group TC7, in the course of the year that followed.

It was also generally agreed that yield strength f_y, was more representative in defining the bearing strength, than f_u. But it was decided to adhere to f_u to satisfy EC3's requirements. That is, equations defining rupture should be expressed in terms of σ_{ult}.

In the meetings that pursued however, some crucial goals were achieved ;

1. The hideously complex and at the same time unrepresentative equations for bearing strength in Annex A were withdrawn.

2. The end distance at which the Ultimate load levelled off with increase in end distance in the line of stress was reduced from 6 to 2.5.

3. The continuity of the two forms of steel was ensured and hence the illogical step functions existing previously between the two forms of steel were eradicated.

The design equation, defining the ultimate bearing strength, as a result was changed from the forementioned equations to :

$$F_b^* = 2.5\ \alpha\ dtf_u \qquad \text{(for } t \geq 1.25 \text{ mm)}$$

Where α is the lesser of e/2.5d and 1.

Obviously the above equation is a significant improvement to what otherwise would have been included in Annex A. It is also the result of an extensive collaboration within the EC3 sub-committee, which has been at a most valuable level that can be envisaged and the benefits derived will serve the industry for some years to come.

The final form of the above equation is still the subject of debate. It is believed that there is still significant room for improvement and it is hoped that the conclusions drawn in this thesis will be further incorporated in Annex A.

Following the September 89 meeting in Delft, Aachen also expressed interest in this matter and prepared a first draft of what was called, the background document A.01[50] to Annex A, later on that year.

In document A.01, based on a statistical analysis carried out at Aachen, it has been suggested that the end distance at which the ultimate load levels off should be increased from 2.5 to 3.0. This, makes the bearing strength equations in hot rolled and cold formed codes identical. This is the present position in Annex A. That is :

$$F_b^* = 2.5\ \alpha\ dtf_u \qquad \text{(for } t \geq 1.25 \text{ mm)}$$

Where α is the lesser of e/3d and 1.

The shortcomings of the present equation in Annex A and the analysis carried out in document A.01 are discussed in the following section.

6.1.1. Background to document A.01

Chapter 2 of EC3 part 1 lays down the general principles for design of civil engineering structures. It broadly defines the characteristic values, partial factors of safety and design values needed to verify the performance of such structures.

Background document 2.01[51], to chapter 2 of EC3 Part 1, describes the detailed procedures required to determine such values. This a common framework which should be followed, and satisfied, by all the design rules given in EC3.

Document 2.01 essentially proposes a reliability-oriented concept used to establish safety factors for design rules, supported by test data, on a limit state basis. This procedure is outlined in Appendix 3.

In document A.01 a statistical analysis is carried out, based on 2.01. A series of design equations given by various institutions, and previous drafts of Annex A have been considered. Each design model is supported by a series of test results.

6.1.2. Design models considered in document A.01

In A.01, design equations defining the bearing strength, given in the following documents have been considered ;

i) Annex A , March 1989 draft.

ii) EC3 part 1 proposals. That is to retain the equation defining the bearing strength of bolted connections in hot Rolled steel, for that of the cold formed variety, as mentioned in page 111.

iii) TNO proposals to replace f_u in EC3 part 1 equation, by 1.3 f_y in Annex A.

iv) Salford proposals in report No. 89/233 July 1989.

The design equations given in (i) is no longer applicable and therefore irrelevant. The design expression proposed by Salford in (iv) above, was also based on an early analysis of only a part of the test results, and has also been superseded by a more substantial study. The results of the full test programme at Salford had been made available to all members of the EC3 sub-committee TC7 in November 89, according to the minutes of the previous sub-committee meeting in Frankfurt in October 89. This was then followed by a substantial report in February 90[49], giving the results of the full test programme. These results and the final proposed design expression however, have not been considered in either May 90 or August 90 drafts of document A.01.

Therefore in document A.01, in effect, the design models upon which a valid statistical analysis has been carried out are the EC3 Part 1 bearing strength equation, and the same with f_u replaced by 1.3 f_y. Based on this, it has been proposed to retain the same design expression as EC3 part 1 for Annex A, this is obviously misleading.

To retain the same equation in both hot rolled and cold formed steel codes has undoubtably a cosmetic attraction, but at the expense of a few serious shortcomings.

For one, in the characteristic bearing strength equation in EC3 part 1, the effect of sheet thickness, in the factor 2.5 α, has been ignored.

So far as it relates to bolted connections in hot rolled steel this is satisfactory. Since as it was shown in Chapter Four it safely estimates the bearing strength for the range of sheet thicknesses that it covers, i.e 6 mm and over. Therefore it complies with EC3 part 1 requirements.

However, for the range of sheet thicknesses that Annex A should cover, i.e

$t \leq 8$mm, with reference to Fig. 4-4 in Chapter Four[‡], it is seen that the EC3 Part 1 equation renders unsafe values for $t < 2.5$ mm. Clearly this is not acceptable. Especially since a good majority of structural bolted connections in cold formed sections fall within this range with hardly anything over 3.0 mm over thick ! Therefore the proposed equation fails to adequately cater for its most important range of application.

Furthermore there are a number of other inherent differences between bolted connections in hot rolled and cold formed steel connections that affect their load bearing characteristics in some respects.

In Chapter Four it was shown that bolt diameter has a significance on the ultimate load bearing capacity of a bolted connection in cold formed sections. This may be due to fact that bolt diameter/sheet thickness ratios (d/t), in hot rolled steel is a fraction of those in cold formed steel connections.

It was also shown that the mechanical properties of the connected sheets affect the factor α. This may not be of equal significance to the generally one (or two) popular grades of steel used in hot rolled steel. In the cold formed construction industry however, a larger variety of steels are in use, ranging from a yield stress of 200 to 350 and even 550 N/mm^2.

Washers are sometimes discarded in bolted connections in cold formed steel. This is already an established practice in the North America, which may later be over in this continent, but it is not the case in hot rolled steel.

Similarly for thicknesses greater than 3mm, washer diameter has no significance on the ultimate bearing strength of a bolted connection. So its not surprising that its has been ignored in EC3 Part 1. However, it has been equally shown that for $t < 3$mm, a large washer diameter does improve the strength of bolted connections. So why ignore it, only because it doesn't affect the range of hot rolled steel bolted connections ?

The full shank diameter of a bolt can easily occur in the shear plane of a connection in cold formed but not in hot rolled steel.

These are all important aspects common to the present day bolted connections in cold formed steel. They need to be properly addressed in a progressive code if it is to provide adequate design guidance for the everyday practice in the industry and to keep in pace with it.

Obviously, the characteristic bearing strength given by EC3 part 1 does not consider the above factors, since they do not relate to bolted connections in hot rolled steel.

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‡ Note that in Fig. 4-4, α has been plotted in terms of σ_y, hence values shown should be divided by 1.4.

Nevertheless in document A.01, it has been suggested that the EC3 part 1 design equation should be in adopted in Annex A. Since its deficiency mentioned previously, in overestimating the strength connections for sheet thicknesses less than 2.5 mm, is covered by the factor of safety for the bolt material γ_{Mb} (=1.25), required by EC3 Part 1.

This argument is not valid, firstly because the database considered in A.01 will be examined in the following section and it will be shown that none of above factors would be reflected in the statistical analysis carried out in A.01, simply because the these variables were not included.

Secondly a factor of safety for <u>bolt</u> material should be regarded as the factor of safety for that material, and not knowingly used to cover other shortcomings of a design equation.

For instance what if two purlins each 1.8 mm thick are connected with bolts without washers ?

As it has been shown in Chapter Four, a loss of 30% in the ultimate strength would result for discarding the washers. So what factor of safety is then used to cover the shortcoming of the EC3 equation in estimating a safe value for this very common range of thickness ?

This is just one example out of the very many that can be thought of, which exposes the deficiencies of the existing equation in Annex A.

It is therefore author's belief that it is not good enough to simply dismiss the effects of sheet thickness, washer diameter and usage, bolt diameter, mechanical properties of steel sheets etc. hoping that they will all be covered by a factor of safety which in turn is primarily intended for something else.

This in turn poses the question that whether a common level of safety for bolted connections in hot-rolled and cold formed steel has been achieved, or the equations given in the Annex A and EC3 Part 1 merely look the same.

The database used by Aachen in document A.01, which has been relied upon to achieve the same factors of safety as in EC3 Part 1 is considered in the following section.

6.1.3. <u>Database on bolted connections used in document A.01</u>

The statistical evaluation carried out in A.01 is based upon the following database ;

-Braunschweig[52]	: 22 tests.	(1985)
-Linz[53]	: 110 tests.	(1976)
-Strathclyde[54]	: 141 tests.	(1975)
-Salford[55]	: 71 tests.	(1989)
-Braunschweig[56]	: 540 tests.	(1984)

The legitimacy of any sort of evaluation only extends as far as the reliability of the data which form its basis. It is therefore intended to scan through all the actual data given in the above database, and examine their relevance to the bearing strength of structural connections, pertinent to the current cold formed steel construction industry.

i) Braunschweig[52]

There are 18 tests reported with sheet tearing / bearing modes of failure. These tests comprise of the following lapped joints ;

Test	constants :	Thin/thick (3mm / 20mm) sheets. Grade 10.9 bolts. d = 20 mm diameter.
	variables :	Specimen width ranging from 48 to 200 mm. e/d ratios ranging from 1.0 to 10.0.

Test results indicate that the ultimate strength increases with specimen width. This contradicts the first principles outlined in Chapter One. To some extent, the increase in the ultimate strength with increase in the specimen width, for the narrower specimens, is that their width is at a value close to the intersection of sheet bearing and net section modes of failure. This would therefore affect the results. However, the effect of specimen width is to the extent that α ($=F_u/dtf_u$) ranges from 5.9 to 8.2 (up to 194 kN for 3mm thick steel!) for 200 mm wide specimens.

There is a general tendency for the results to be on the high side compared to those found elsewhere.

The combination of a very thick backing plate and a strong bolt might have affected the ultimate strength of the specimens. In tests carried out at Salford with thin/thick sheets, in one case where a very strong bolt was used, it caused the thicker plate to bend which artificially enhanced the strength of the lap joint, which was considered to be for a 3mm thick sheet.

If only regarding some of the results mentioned above as uncharacteristic and considering those with; $w \leq 140$ mm and $e/d \geq 1.5$, 12 tests remain.

ii) Linz[53]

56 tests, have been reported with sheet tearing / bearing modes of failure.

The following parameters have been considered ;

Test variables :	Sheet thickness 3 & 5 mm thick (sheets of equal thickness). Bolt diameter 6, 10 & 16 mm. (Grade 10.9) e/d ratio ranging from 0.9 to 6.4.

There is no information given on specimen width, but it seems as if it may have been held constant for any one given bolt diameter.

The original expressions in Annex A were a best fit to Linz data. Since structural bolted connections are of concern in Annex A, then 6 mm diameter bolts are neither relevant nor is there any application for such bolts in the context of 3 and 5 mm thick sheets in any sort of connection! Therefore these should not be considered. The same can also be said about 10 mm diameter bolts with 3 & 5 mm, for that matter.

However, considering the 10 & 16 mm diameter bolts, with $e/d \geq 1.5$, then 27 tests remain.

iii) Strathclyde[54]

121 test results have been tabulated in the Aachen database. Based on the information received directly from Strathclyde however, it has been established that a total of 171 tests were carried out in the early 70's at Strathclyde.

These tests comprised of 20 tests with two bolt lapped joints which have appropriately been left out in the Aachen database. A further 151 tests were carried out on single bolt lap joints.

Out of the 151 tests on single bolt lap joints, 109 relate to sheet tearing / bearing modes of failure.

The test parameters were as follows ;

Test constants : Grade 8.8 bolts.
Mechanical properties of the steel sheets: σ_y ranging from 161 to 297 N/mm^2 with a corresponding σ_{ult} range of 262 to 404 N/mm^2.
Thin/thin flat sheets, channel/channel and channel/thick plate lapped joints.

variables : Bolt diameter, 5, 10 and 16 mm.
Specimen width, ranging from 40 to 120 mm.
Sheet thickness 1, 3 and 5 mm.

The purpose of these tests were to define and establish various modes of failure in bolted connections in cold formed steel. 89 of the 171 tests carried out were considered to be relevant to the bearing strength of bolted connections in cold formed steel sections.

The relevant test results to bearing strength of structural bolted connections (i.e. 10 and 16 mm ϕ bolts) will be considered later in this chapter.

iv) Salford[55] ;

It has already been mentioned that a total of 228 tests on bolted lap joints have been carried out at Salford. However, only 71 of these tests have been included in the Aachen database. That is, only the tests shown in Figs. A1.6 and A1.7, in Appendix 1, have been considered and the rest of the results have been ignored even in the later updates of document A.01. Discarding the 20 preliminary tests and 10 tests which failed in shearing of bolt mode of failure, 197 tests remain.

These tests were based on the experience gained from two past research projects at Salford consisting of some 119 tests. The test parameters considered were therefore carefully designed to have a direct relevance to BS 5950 Part 5 and Annex A.

Indeed, it can be said with confidence that over 90% of structural bolted connections that Annex A should be concerned with, fall within the range of variables considered here. With the overlap between Salford and Strathclyde results, then it can be said that all relevant variables to bolted connections in structural bolted connections in cold formed steel sections have been considered.

v) Braunschweig[56]

530 tests have been reported with sheet bearing mode of failure.

The tests consist of lap joints with two bolts perpendicular to the line of stress. The test parameters consist of ;

Test	constants :	2 (Grade 10.9) bolts perpendicular to the line of stress. End distance in the line of stress = 30mm. Specimens ultimate stress ≈ 370 N/mm^2.
	variables :	Bolt diameter 8, 10 and 12 mm. Sheet thickness ranging from 0.75 to 2.5 mm. Surface coatings : Aluminium, Zinc, Plastics and Enamel.

The test specimens consist of a rather odd geometry. That is, two bolts perpendicular to the line of stress. (Fig. 6-1)

There is no information available to establish whether this represents the same case as single bolt lap joints considered up to here. Bearing in mind that the two bolt lap tests of Strathclyde and elsewhere have been omitted for the same reason in the Aachen database.

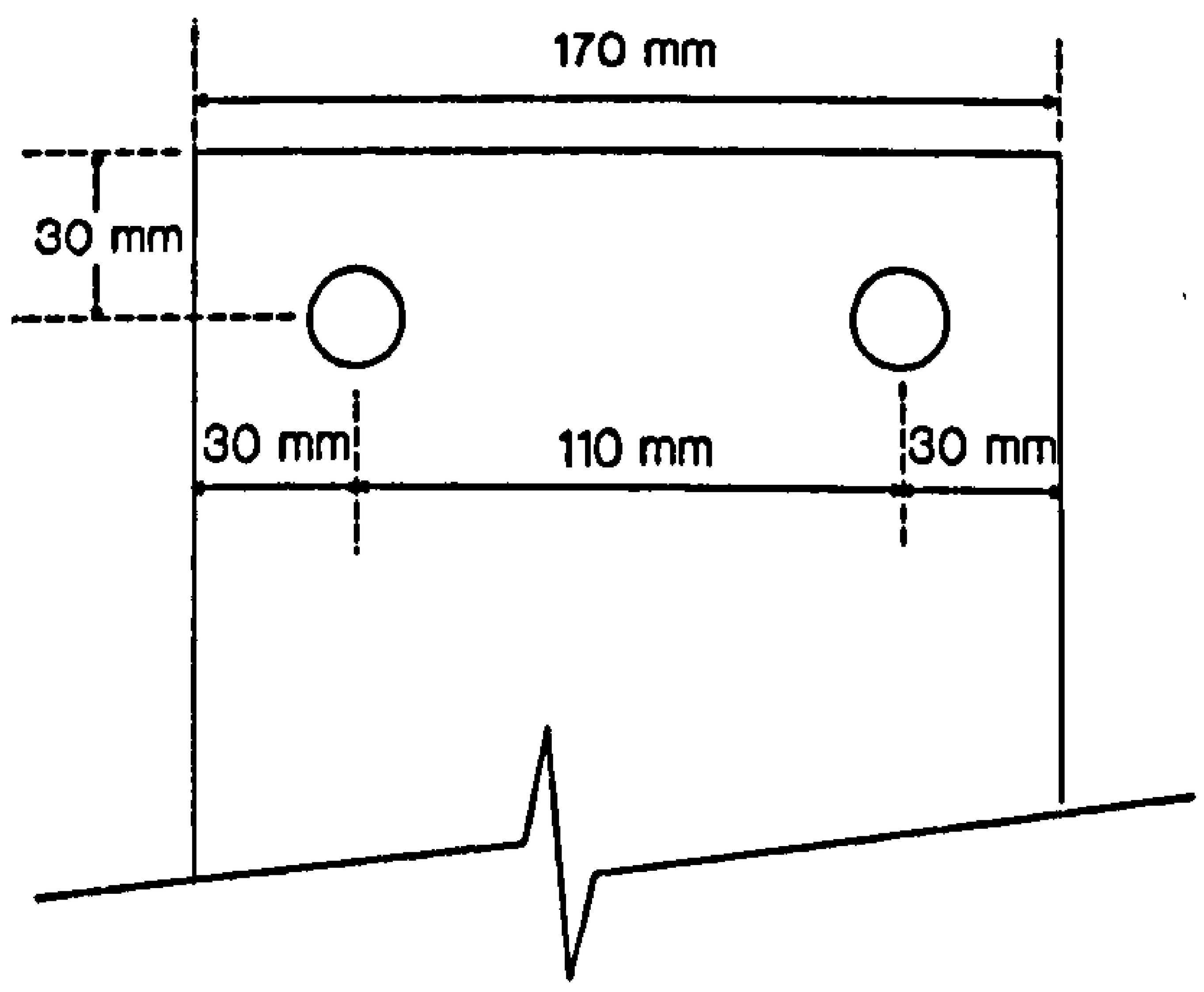


Fig. 6-1

Furthermore 8 mm diameter bolts are not of particular relevance to structural connections.

The tests carried out at Braunschweig, would have been an ideal exercise to establish the effects of different surface coating on the coefficient of friction and hence slip load of bolted connections. Unfortunately there is no information available on this matter, in the database.

Considering 10 and 12 mm bolts however, about 350 tests remain.

6.1.4. Conclusions drawn on document A.01

Having extensively considered the information and results published in document A.01, it is concluded that in A.01 a very legitimate reliability analysis based on background document 2.01, has been carried out. It is believed that this is a very worthwhile exercise to maintain a common ground for safety margins in all the equations given by the EC3. In A.01, based on this analysis, it has been concluded that the design equation for bearing strength of bolted connections in hot rolled steel given in EC3 part 1 gives the lowest factor of safety required compared to that of other design equations. But there are two main points to bear in mind.

i) Two of the design models considered in A.01 were outdated. In A.01, the design models considered are therefore effectively that of EC3 Part 1 compared to EC3 Part 1 with a small modification.

ii) Not all the data used in the database used to carry out the analysis are relevant. In A.01, none of the forementioned factors i.e., bolt diameter, use of washers, mechanical properties of connected sheets, etc have been considered. These factors are therefore not reflected in the final safety margins tabulated in A.01. The information presented in A.01 is therefore not wholly representative of the insitu conditions.

The above points however, have been studied to their full extent in recent tests at Salford and their effects have been understood and quantified. It is therefore important that they are incorporated in Annex A.

It is believed that the statistical analysis carried out in A.01 should be mainly based on the results obtained at Salford and Strathclyde, since they represent the more relevant factors in current practice in the industry. Braunschweig[56] results would be a very valuable addition to this database, provided that it can be shown that the connection shown in Fig. 6-1 is characteristic of that of single bolt lap joints, as this is the case with the rest of the database.

It is believed that the main objects of the exercise should be :

- to ensure that the design equations in both hot rolled and cold formed codes have been derived from a common set of first principles.

- to maintain a continuity between the bearing strengths of the two codes of practice for hot-rolled and cold-formed steel.

- to account for all the common factors relevant to the present day cold formed steel industry.

It is therefore suggested that a statistical analysis carried out on the proposed design expression in this thesis, and EC3 Part 1 design equation, based on the following database, would be a much more valuable exercise;

Braunschweig	(1985) : 12 tests
Linz	(1976) : 27 tests
Strathclyde	(1975) : 89 tests
Salford	(1990) : 197 tests
Braunschweig‡	(1984) : 350 tests

Therefore it is not important if the equations for the bearing strength of hot-rolled and cold-formed steel don't look the same. So long as they have each covered all the factors which are relevant to the contemporary practice in their industries. It is believed that the present equation in Annex A, although vastly improved compared to what it had been before, still falls short of this goal.

6.1.5. An editorial note regarding Annex A

Before deliberating the test results further in this chapter, an editorial flaw in chapter A8 of Annex A should be aired.

Chapter 6 of EC3 part 1, § 6.5.5, defines the Design resistance, R_d of the bolted connections. In Annex A (§ A8), however, the Characteristic strength, R_k, of such connections are defined. The difference between the two being the required factor of safety for the bolt material, namely γ_{Mb}. That is $R_d = R_k / \gamma_{Mb}$. Therefore the strength values obtained from § A8 of Annex A should be divided by a factor of γ_{Mb} (=1.25). However, this has not been manifested anywhere in Annex A. So a designer can easily be misled to accept the values obtained from the equations in § A8 as the Design strengths of bolted connections. This matter should therefore be rectified in the final version of Annex A.

It is also arguable that a lesser factor of safety for bolts, in connections with cold formed steel sections may be acceptable, since failure often occurs in the connected material, rather than the fasteners.

Note however that, in what follows in § 6.3 the characteristic values given in Annex A, have been tabulated.

‡ Provided that the tests are characteristic of single bolt lap joints.

6.2. BS 5950 Part 5

The following equations govern the ultimate bearing strength of a bolted connection in BS 5950 Part 5:

$$P_s = \alpha \, dt\sigma_y$$

where for $t \leq 1mm$:

$$\alpha = 2.1$$

for $1mm < t \leq 3mm$:

for $(e/d) \leq 3$ $\quad \alpha = \{ 2.1 + [0.3 (e/d) - 0.45](t - 1) \}$

for $(e/d) > 3$ $\quad = (1.65 + 0.45 t)$

for $3mm < t \leq 8mm$:

for $(e/d) \leq 3$ $\quad \alpha = [1.2 + 0.6 (e/d)]$

for $(e/d) > 3$ $\quad = 3.0$

With $(e/d)_{min.} = 1.5$.

Note that the ultimate strength is expressed in terms of σ_y.

The above equations are a conservative simplification of the December 1988 draft of Annex A, (originally Published in European Recommendations, Publication No. 21[46]).

The simplifications in BS 5950 Part 5, effectively avoid the more complicated design expressions involving natural logarithms, described at the beginning of this chapter. The ultimate bearing strength has also been truncated for end distance ratios (e/d) greater than 3, compared to an increasing ultimate bearing strength up to (e/d) equal to 6 in December 88 draft. This served to reduce the discontinuity between Parts 1 and 5 of BS 5950 and to bring the maximum design value of the bearing strength more in line with other codes. (cf. 4.3 $dt\sigma_y$ in Dec. 88 draft and 3.0 $dt\sigma_y$ above.)

6.3. Salford test results compared to various design expressions

Average ultimate bearing strengths of test results produced in Chapter Four are tabulated and compared with that given by the proposed design expression in this thesis, EC3 Annex A and BS 5950 Part 5. The actual mechanical properties (σ_y and σ_{ult}) given in Appendix 1, have been used in calculating the predicted bearing strength of each connection.

The format of the following sections has been kept similar to that of Chapter Four, as much as possible.

6.3.1. Perfect fit lap joints

6.3.1.1. Plain shank on the shear plane

Test particulars ; Large diameter bright steel washers. See Fig. A1.1 (only specimens with lips have been considered, since it was shown in Chapter Four that they are more representative of the insitu conditions).

Note that 1.96 mm thick specimens have a nominal yield stress of 210 N/mm^2, see Fig. A1.1 (nominal ultimate stress, 270 N/mm^2). The average ultimate load obtained with these specimens is the same as that of 1.5 mm thick specimens, in spite of having a thicker cross section.

Nominal sheet thickness (mm)	Ultimate bearing strength (kN)			
	Average ult. load, tests	Proposed design expression	EC3 Annex A	BS 5950 Part 5
1.5	35.8	26.9	23.1	16.8
1.96	35.8	34.7	23.8	16.4
2.57	62.0	49.6	42.5	39.4
3.17	71.4	57.6	50.1	48.5

6.3.1.2. Shear plane on bolt threads

Test particulars : Two different types of washers, classified according to § 4.3.2.2 in Chapter Four, are listed :

(a) standard, normal diameter mild steel washers.
(d) large diameter bright steel washers.

See Fig. A1.2

Nominal sheet thickness (mm)	Ultimate bearing strength (kN)				
	Type of washers	Average ult. load, tests	Proposed design expression	EC3 Annex A	BS 5950 Part 5
1.50	(a)	25.3	20.9	23.7	17.2
	(d)	31.1	24.0	23.7	17.2
2.57	(d)	54.6	38.3	37.7	37.3
3.17	(a)	63.8	55.2	55.2	43.2

6.3.2. Clearance holes

6.3.2.1. Washer diameter

Test particulars ; (a) Normal diameter, standard washers
(b) Same as (a), but from a different manufacturer
(c) Large diameter, mild steel washers
(d) Large diameter bright steel washers

See Fig. A1.4.

Nominal sheet thickness (mm)	Ultimate bearing strength (kN)				
	washer dia.	Average ult. load, tests	Proposed design expression	EC3 Annex A	BS 5950 Part 5
1.50	(a)	24.4	20.7	23.5	17.1
1.63	(b)	26.5	21.8	24.4	20.5
	(c)	33.7	25.0	24.4	20.5
	(d)	34.1	25.0	24.5	20.6
2.48	(a)	47.2	38.8	40.5	37.1
	(b)	42.9	38.4	40.1	35.6
	(c)	51.9	40.3	40.1	35.6
	(d)	53.7	40.4	40.2	35.5
3.02	(a)	56.3	46.4	46.4	44.5
	(b)	49.7	45.3	45.3	43.3
	(c)	59.2	46.5	46.5	44.5
	(d)	62.5	46.5	46.5	44.5

6.3.2.2. Number of washers

Test particulars ;
(a) 2 washers
(b) 1 washer under bolt head
(c) 1 washer under nut
(d) no washers

With 3.02mm thick sheets, both specimens with and without lips have been considered, since it was shown in Chapter Four that with this range of sheet thickness, presence of lips did not affect the test results. In this case a weighted average of the mechanical properties listed in Fig. A1.5 has been used to calculate the predicted ultimate loads given by different expressions.

Nominal sheet thickness (mm)	Ultimate bearing strength (kN)				
	No. of washers	Average ult. load, tests	Proposed design expression	EC3 Annex A	BS 5950 Part 5
1.50	(a)	24.4	20.7	23.5	17.1
1.63	(b)	22.2	17.2	24.2	15.2
	(c)	21.3	17.2	24.2	15.2
	(d)	18.2	15.1	24.2	15.2
2.48	(a)	47.2	38.8	40.5	37.1
	(b)	39.8	31.6	41.2	27.7
	(c)	37.9	31.6	41.2	27.7
	(d)	34.0	27.2	40.5	27.8
3.02	(a)	56.3	46.4	46.4	44.5
	(b)	49.1	40.1	50.2	36.5
	(c)	45.1	39.4	49.3	35.8
	(d)	39.7	33.3	47.5	34.4

6.3.2.3. End distance in line of stress

See Fig. A1.7.

Nominal sheet thickness (mm)	Ultimate bearing strength (kN)				
	e/d	Average ult. load, tests	Proposed design expression	EC3 Annex A	BS 5950 Part 5
1.63	1.50	17.9	13.0	12.2	18.0
	2.25	27.3	19.5	18.3	19.2
	3.00	28.1	21.7	24.4	20.5
1.50	3.75	24.4	20.7	23.5	17.1
1.63	4.50	27.6	21.6	24.2	20.1
	5.25	28.6	21.6	24.2	20.1
	6.00	28.1	21.6	24.2	20.1
2.48	1.50	29.3	23.5	20.4	27.8
	2.25	45.5	35.2	30.6	32.2
	3.00	46.4	39.1	40.1	36.6
	3.75	47.2	38.8	40.5	37.1
	4.50	46.6	39.1	40.8	36.8
	5.25	46.5	39.0	40.7	36.7
	6.00	47.5	39.0	40.2	36.7
3.02	1.50	35.4	27.6	23.0	30.9
	2.25	51.3	41.4	34.5	37.5
	3.00	55.4	46.0	46.0	44.1
	3.75	56.3	46.4	46.4	44.5
	4.50	56.1	46.4	46.4	48.6
	5.25	57.7	46.0	46.0	44.0
	6.00	55.7	46.0	46.0	44.0

6.3.2.4. Mechanical properties of steel sheets

Test particulars : Nominal 350 N/mm^2 yield stress steel (450 N/mm^2 ultimate stress).

See Fig. A1.6, actual mechanical properties have been used in calculating the predicted ultimate bearing strengths.

Nominal sheet thickness (mm)	Ultimate bearing strength (kN)			
	Average ult. load, tests	Proposed design expression	EC3 Annex A	BS 5950 Part 5
1.57	32.5	25.0	30.3	25.8
2.37	47.8	40.2	45.5	40.1
3.11	65.4	54.8	58.9	59.3

6.3.2.5. Bolt diameter

Strathclyde data

Results of Strathclyde University obtained for 10 and 16mm diameter bolts(each with 0.91 mm and 3.10 mm thick sheets) are plotted in Fig. 6-2 and Fig. 6-3 respectively.

Strictly speaking, Annex A does not cover the thinner sheets (t = 0.91 mm), since it only caters for t $\geq$ 1.25 mm sheets.

The mechanical properties of specimens were as follows;

> for 0.91 mm thick sheets, σ_y = 251 and σ_{ult} = 325 N/mm^2, (i.e. nominal 250 N/mm^2 yield and 350 N/mm^2 ultimate stress steel, see BS 5950 Pt. 5 Table 4)
>
> for 3.10 mm thick sheets, σ_y = 297 and σ_{ult} = 404 N/mm2, (i.e. nominal 280 N/mm^2 yield and 390 N/mm^2 ultimate stress steel)

It is also interesting to note from these figures that for specimens width, w = 40 mm especially with 16 mm diameter bolts - although the reported mode of failure was in sheet bearing, the actual ultimate load was much less than that expected. The reason, as described in the previous chapter being that, for the given specimen width considering that the specimens had lips, the mode of failure is at the intersection of sheet bearing and failure of section modes of failure.

It is seen that the results obtained are totally in line with design expressions proposed in this thesis.

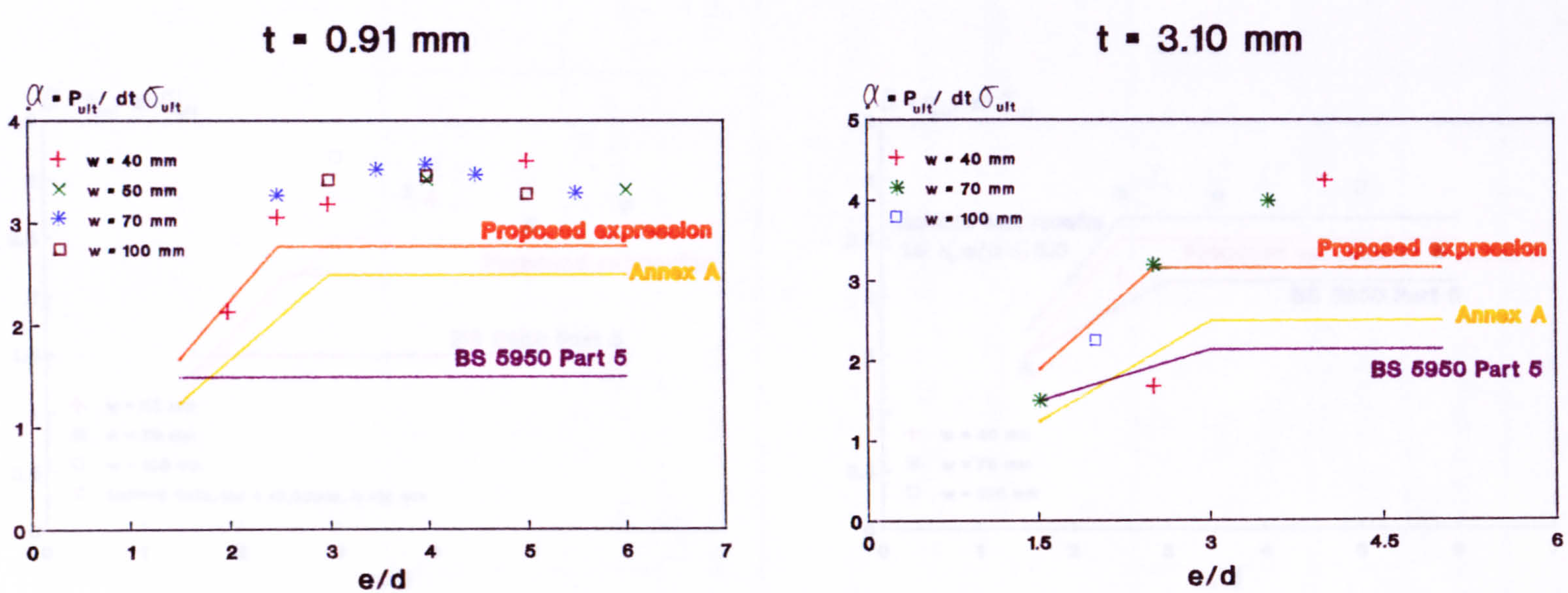


Fig. 6-2

Strathclyde data, 10 mm ϕ bolts

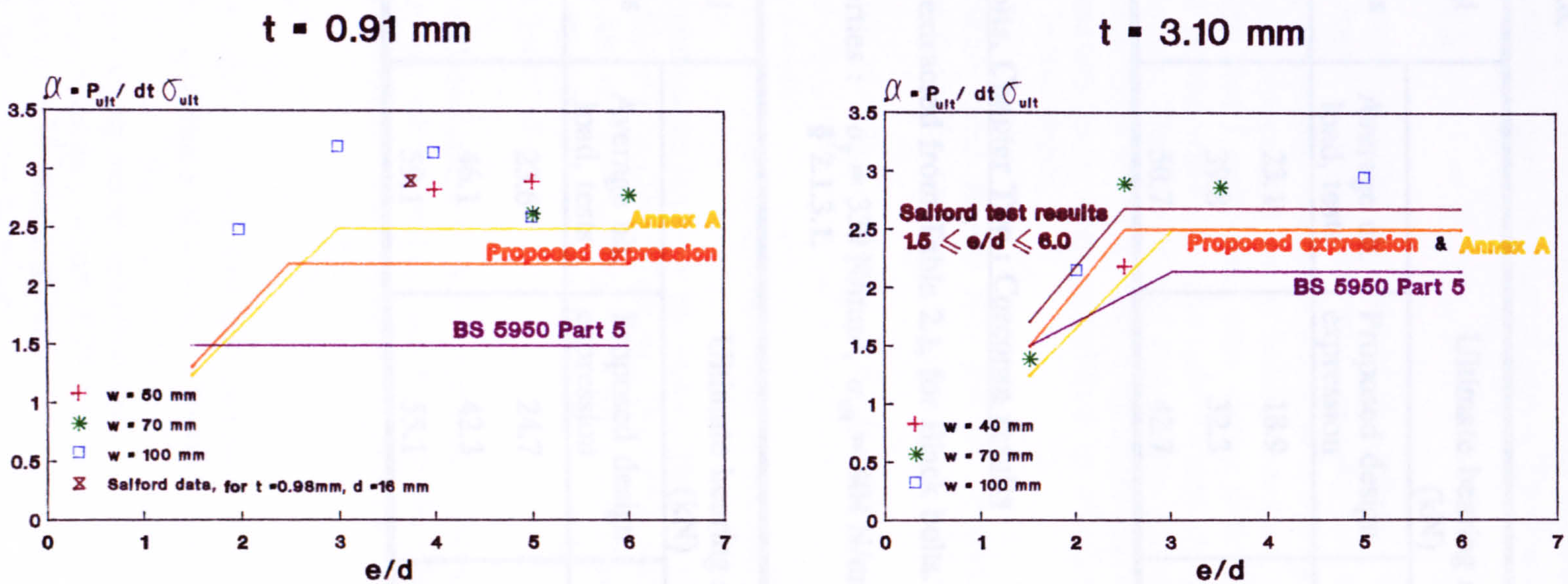


Fig. 6-3

Strathclyde data, 16 mm φ bolts

Salford 12mm ϕ bolts data

Test particulars : All grade 8.8 bolts, except for 1.63 mm thick sheets.

See Fig. A1.8.

Nominal sheet thickness (mm)	Ultimate bearing strength (kN)			
	Average ult. load, tests	Proposed design expression	EC3 Annex A	BS 5950 Part 5
1.63	23.1	18.9	18.4	15.5
2.45	39.8	32.3	29.3	25.4
3.12	50.7	42.7	36.9	34.8

20 mm ϕ bolts, Chapter Two : Corcoran results

Results are extracted from Table 2.1, for Black bolts.

Sheet properties : $\sigma_y = 339$ N/mm^2, $\sigma_{ult} = 404$ N/mm^2, as listed in Chapter Two § 2.1.3.1.

Nominal sheet thickness (mm)	Ultimate bearing strength (kN)			
	Average ult. load, tests	Proposed design expression	EC3 Annex A	BS 5950 Part 5
1.55	23.6	24.7	31.3	24.7
2.45	46.1	42.3	49.5	45.7
3.05	59.1	55.1	61.6	62.0

6.4. Conclusions

The backgrounds to the bearing strength equations of EC3 Annex A, and BS 5950 Part 5 have been described. Test results obtained at Salford and Strathclyde have been compared against that predicted by the proposed design expression and the above mentioned codes. The following are concluded :

BS 5950 Part 5

Overall it is concluded that the bearing strength of a bolted connection, as defined in BS 5950 Part 5 is over-conservative. The expressions in BS 5950 Part 5 are an intelligent application of earlier expressions in Annex A, with practicality in mind. As a result of tests carried out at Salford however, the parent expressions upon which BS 5950 Part 5 was based, were shown to be unrepresentative of the actual behaviour of bolted connections in cold formed steel connections and have since been abandoned. This leaves the expressions given in BS 5950 Part 5 defining obsolete conditions in terms of sheet thicknesses and end distances in line of stress. These parameters have been quantified by much simpler and more accurate expressions, in this thesis.

It has effectively been shown that the set of complicated equations given in BS 5950 Part 5, can be replaced by :

$$P_{bs} = (2.6 + 0.3\,t)\,\alpha\,dt\sigma_y \qquad \text{for} \quad t \leq 3\text{ mm}$$
$$= 3.5\,\alpha\,dt\sigma_y \qquad \text{for } 3 < t < 8\text{ mm}$$

where α is the lesser of (e/2.5d) and 1.

The above expressions improve the values given in BS 5950 Part 5 by a factor of about 1.4, achieve a continuity between the two forms of steel (assuming a σ_{ult} / σ_y ratio of 1.4) and are undoutably much more acceptable than those listed in § 6.2. The above expression incidentally, is the k_2 factor proposed in Chapter Four, expressed in terms of σ_y.

Apart from end distance and sheet thickness, it has been shown that an incorrect allowance for the use of washers has been made. Other factors considered in this thesis have not been accounted for in BS 5950 Part 5.

EC3 Annex A

Overall there has been a marked improvement in the design equation defining the ultimate bearing strength of a bolted connection in Annex A.

The design philosophy has nevertheless shifted towards over-estimating the strength of connection is in terms of sheet thickness, curtailing the lower limit of permissible sheet thickness at 1.25 mm, and relying on other margins to subsidise each other. For instance the relative increase in the ultimate bearing strength, in terms of α (= $P_{ult} / dt\sigma_{ult}$), as a result of using a 12 mm diameter bolt with 1.6 mm thick sheets has been ignored, but instead counterbalanced by over-estimating the strength of the connection for the mentioned sheet thickness in the first place (see page 128). A factor of safety for the bolt material is to be relied upon as a buffer for any other unforeseen circumstances. This is more reminiscent of the permissible stress design philosophy.

In this chapter, the deficiencies of the current equation in Annex A, have been brought to light particularly in terms of bolt diameter, use of washers and in cases where the plain shank occurs in the shear plane.

The lack of accuracy and safety in the Annex A equation in terms of sheet thickness and yield strength of material is borne out more clearly in the following chapters, in tests on full moment connections, which are essentially the main purpose of any design equation.

It is therefore believed that in a progressive code based on limit state design, it is essential to account for all common foreseeable factors as accurately and simply as possible. It should not give over-simplified expressions and adopt a "rent a factor" policy to subsidise for other short comings of a equation.

Furthermore no reliability analysis on any given design equation, no matter how advanced, could justify the accuracy of that equation if it is not subjected to the correct parameters representing the contemporary practice in the industry.

It is concluded that the propounded design equation is a significant advance on those given in existing codes of practice. Every common factor has been isolated and quantified separately and accurately. Any factor of safety can be applied explicitly, with confidence that it is being applied for the purpose that it has been intended for in the majority of cases. This is more in line with the limit state design philosophy. The proposed expression can also be safely used for the whole range of sheet thicknesses covered by cold formed steel codes and there is no need to impose any limits on its value, as it is presently the case in Annex A.

It is also interesting to note that with the proposed design expression, every thing fits in logically and in full agreement with the test results obtained at Salford and elsewhere not only in terms of the sheet bearing mode of failure, but also in an overall interaction of different modes of failure of bolted connections in general. A comparison of various modes of failure has already been carried out and reference has been made to test results that vindicate the earlier analysis. For example the interaction between the net section and sheet bearing modes of failure observed in Strathclyde results agreed with what was predicted earlier. In the previous chapter it was shown that the interface predicted between sheet tearing and sheet bearing modes of failure was also in perfect agreement with test results. The same however, can not be said about the design equations in existing codes of practice.

Chapter Seven

Flexibility of bolted connections in cold formed steel sections

7. Flexibility of bolted connections in cold formed steel sections

Summary

So far the emphasis in this thesis has been on the strength of bolted connections in cold formed steel. In this chapter the flexibility of such connections is considered. A design expression defining the load/extension characteristics of bolted lapped joints in cold formed steel is propounded. As such for the first time, designers will be able to predict the moment/rotation characteristics of bolted connections in cold formed steel, without resorting to testing.

7.1. Introduction

With the prominent publication of BS 5950 : Part 5 "Code of Practice for the design of cold formed sections" and EC3 Annex A, soon to be published in its final form, cold formed steel sections have assumed a greater importance in the construction industry. For the first time designers have been given sufficient information to design main frame of structures (as distinct from secondary members) in cold formed steel. Such systems are already on the market and their behaviour is verified by full scale tests.

It has long been realised that the successful use of steel in any building requires a sound and economical connection design. However traditionally, connections have been designed as either *pinned* (implying no moment transfer) or *rigid* (implying complete rotational continuity) in steel frames - alternatively referred to as *simple* and *continuous* construction.

At the present cold-formed steel sections are often designed using traditional simple construction, often accompanied by manufacturers recommended safe load design tables, based on their test results.

The notions of pinned and rigid joints are simply extreme cases of true joint behaviour. Most pinned connections possess some rotational stiffness, while rigid connections often display some flexibility. It would therefore seem more appropriate to regard all steel frames under the more general heading of Semi-Rigid Construction, treating the simple and continuous construction as the extremes. (Fig. 7-1)

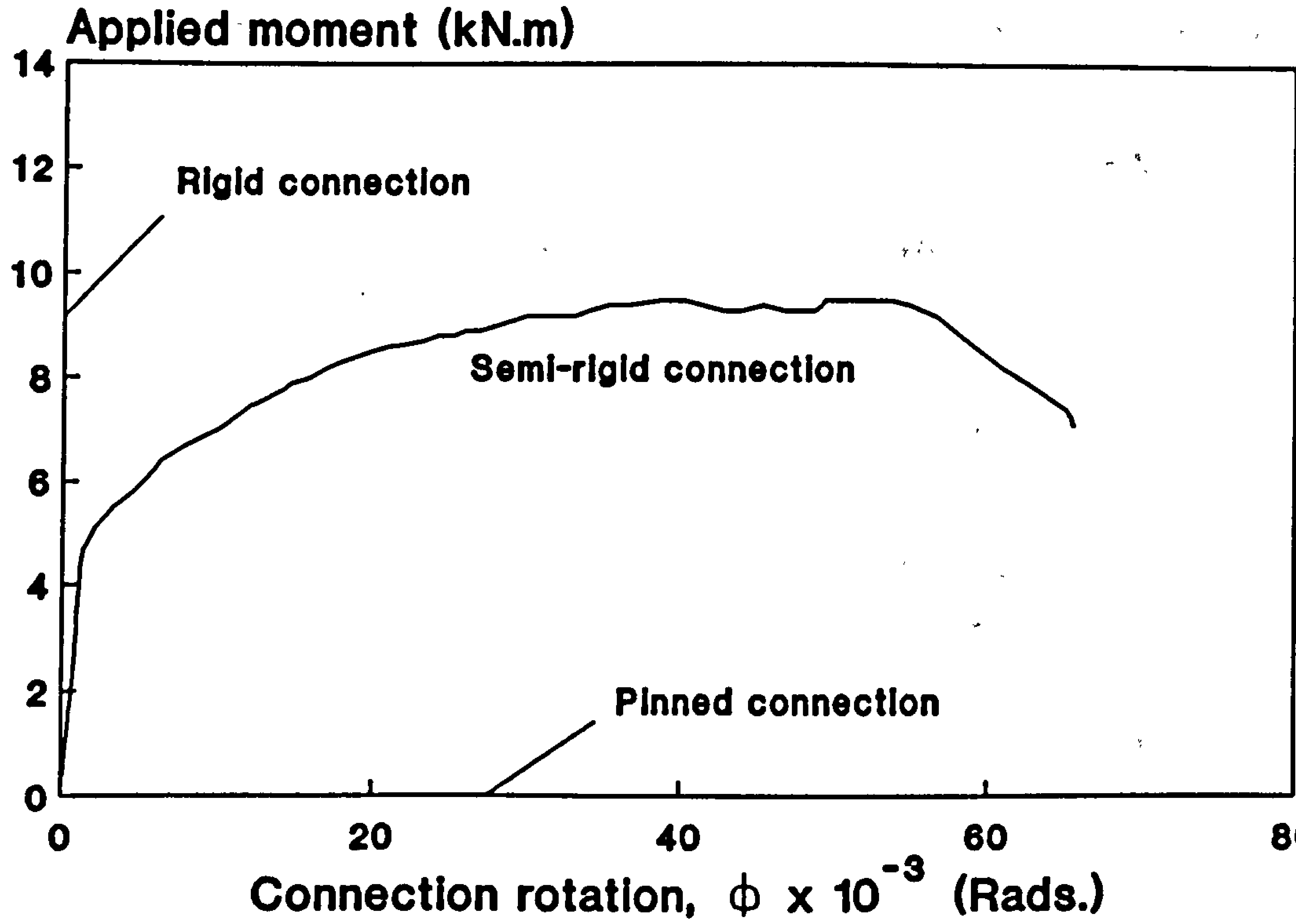


Fig. 7-1 : Typical moment-rotation characteristics of a bolted connection.

The most obvious advantage of semi-rigid connection design is that beam moments are reduced leading to lighter sections.

Fig. 7-2 shows the Bending Moment Distributions for a pinned, rigid and semi-rigid beam.

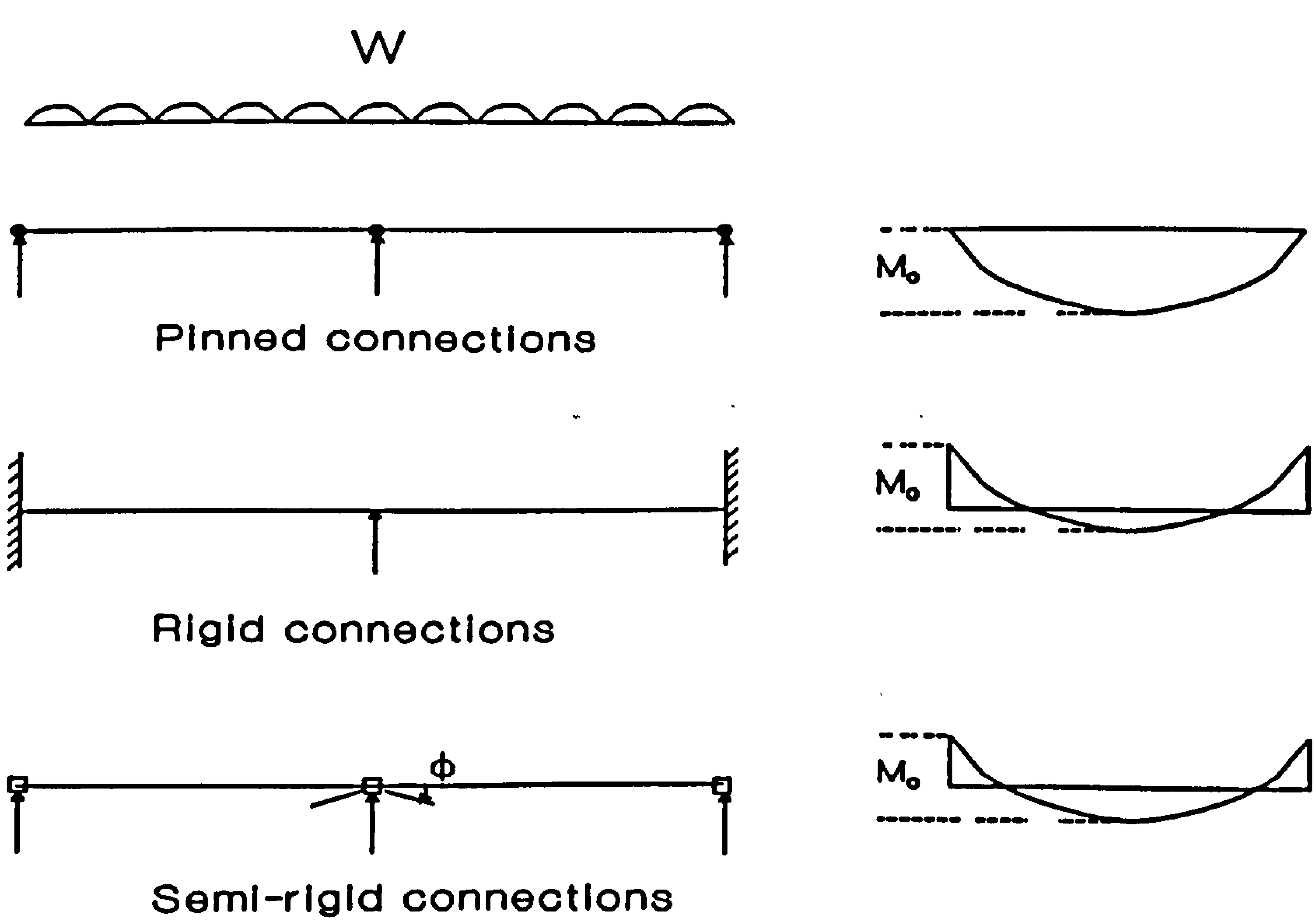


Fig. 7-2 : Bending moment distribution depending on end conditions.

It can be seen that with pinned connections the mid-span moment is critical. Whereas, with rigid joints end-moments are critical for beam design. If semi-rigid joints are assumed these two moments may be more nearly balanced. The optimum solution obviously being when the mid-span moment is equal to end-moments.

This makes apparent that consideration of the moment/rotation characteristics can be of vital importance to economic design.

Extensive test programmes have been carried out at the University of Sheffield[61] and BRE to study the semi-rigid joint action in bolted connections in hot-rolled steel. Various moment/rotation characteristics depending on the type of connection has resulted. (Fig. 7-3)[62]

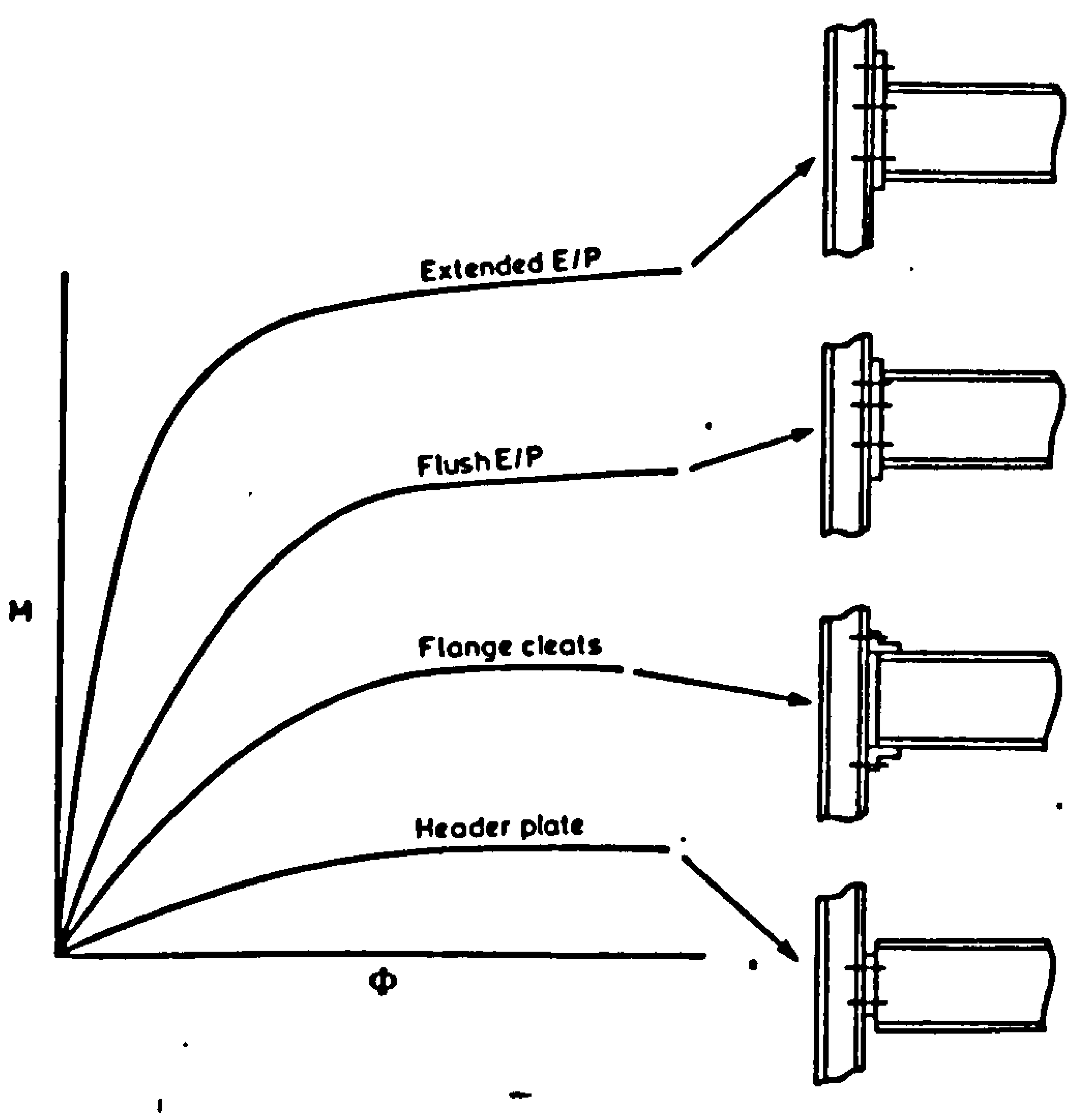


Fig. 7-3 : M/ϕ characteristics for various types of bolted connections in hot rolled steel.

For reasons mentioned in earlier chapters these characteristics are not directly applicable to bolted connections in cold formed steel.

Since connections in cold formed steel sections are almost always made with bolts, it was decided to embark on a fundamental and systematic study of their rigidity as well as strength.

It is now intended to address the rigidity of a bolted connection in cold formed steel, so that this information can be fed into design programmes to radically affect the elastic design of cold formed steel structures. This should then enable the optimum design of such structures by specifying joints with the appropriate characteristics.

Due to the simpler nature of cold formed steel compared to that of hot rolled steel construction, it is possible to obtain a more general expression applying to all bolted connections in cold formed steel. Therefore a classification of the type of the connection, as in Fig. 7-3, is not necessary.

7.2. Objectives

It has been clearly demonstrated that an understanding of the moment/rotation characteristics is essential to design of bolted connections in cold formed steel. This in turn requires a full knowledge of basic load/extension behaviour of simple lap joints. The information thus obtained can be readily translated to predict the moment/rotation characteristics of such connections.

The analyses carried out in order to determine the load/extension characteristics bolted lap joints, are described in the following sections.

7.3. Load / extension characteristics of lap joints

The total extension at a given load consists of two components:

i) Extension due to the clearance between a bolt and its oversize hole, resulting in a rigid body slip until bearing is established. The load at which the rigid body slip occurs is defined as the **Slip Load.**

Therefore slip load is defined as the load at which there is an increase in extension, without any increase in load. Prior to slip, the rigidity of a connection is governed by the frictional forces between the connected sheets, induced as a result of the axial stress in the connecting bolt(s). After slip, connection rigidity is dependent upon the stiffness of the connected sheets.

In the lap joints tested the theoretical maximum amount of slip is twice the hole clearance; since connected sheets were pushed in together prior to the tightening of the bolts. In practice however, with multiple bolted connections the slip will hardly be the clearance itself. Firstly due to imperfections in punching the bolt holes in precisely the right locations. However, most cold formed steel sections are nowadays punched at the fabrication stage, by computer operated machines, to client- specifications. So such imperfections are reduced to a minimum. Secondly, and mainly, because the bolts will be subjected to forces in different directions, in contrast to single bolt lap joints pulled in tension. The maximum possible slip, therefore, can not exceed the hole clearance, provided the initial set up is aligned.

ii) Extension due to deformation, mostly non-linear, of the joined elements.

Clearance extension is usually a fraction of the deformation extension.

7.3.1. Properties of the Slip load

It has been shown in earlier chapters that the slip load has the following three important properties.

i) Connection slip cannot be prevented at the working load level in common (black,high tensile...) bolted connections. Only HSFG. bolts may prevent this in some circumstances, as it was described in Chapter Two.

ii) Slip load is independent of the sheet thickness.

iii) Slip load is proportional to the Bolt Torque.

7.3.2. Characteristics of the clearance slip

A 100 tonne Losenhausenwerk Universal testing machine was used to test the lap joints. This machine was equipped with a load/head movement Recording Drum. At this stage it may be appropriate to take advantage of this and show typical slip patterns, taken from actual lap joint tests.

Clearance slip can be broadly divided into three patterns;

i) there can be a "sudden slip", with little loss of stiffness prior to its happening, followed by a sudden drop in load until the clearance has been taken out and full bearing is established. The load carrying capacity is then rapidly restored, as in case (a).

ii) alternatively, there may be a "gradual slip" with a gradual loss of stiffness, but no drop in load, until the clearance is taken out and bearing is established. The stiffness is then restored to its initial value, prior to slip, case (b).

iii) there can also be an "ideal slip", as defined in § 7.3, i.e. a sudden rigid body slip with no change in load until the clearance is taken out and bearing is established, case (c).

As such, with a varied pattern in slip loads, it is difficult to base any analysis on the actual test load/extension characteristics. For instance in the case of a gradual slip, what would be the slip load and the amount of slip?

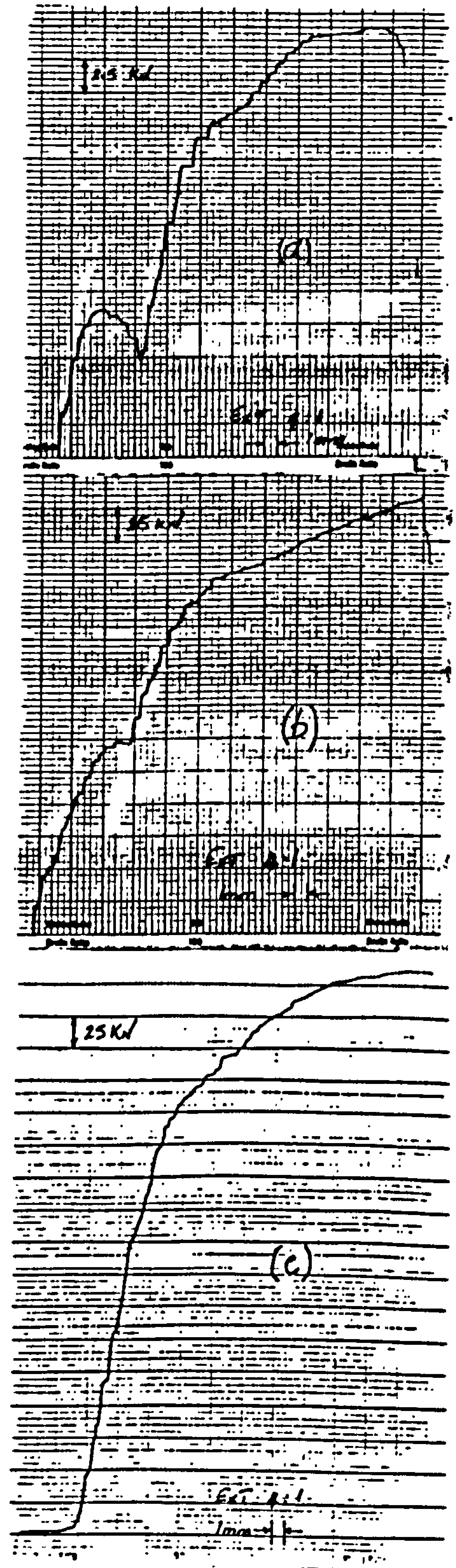


Fig. 7-4 : Typical clearance slip patterns.

Therefore in order to be able to predict the load/extension characteristics of a bolted connection, it was thought necessary to standardize the slip patterns. Hence, reasonable estimates of the slip load and amount of slip could be obtained.

The effect of bolt torque on the slip load was eliminated by applying a standard torque, taking account of the insitu conditions, to all lap joints, as described in Chapter Three.

The slip itself was idealized in load/extension plots, similar to case (c), taking the following steps :

> The bolt hole was made precisely to the required specification, by drilling in a hole one step lower than the desired size and then reamering to the exact diameter.
>
> The precise bolt diameter at the shear plane was measured by a micrometer.

Therefore at the analysis stage, knowing the exact hole clearance and using the test readings together with the load/head movement plot, recorded by the machine, an "ideal slip" was obtained in accordance with the slip definition in § 7.3. An example of actual slip load readings, compared to the idealised slip load is shown in Fig. 7-5.

7.4. Flexibility of lap joints

Flexibility of a connection is defined as the deformation sustained per unit load applied.

Therefore flexibility is the inverse of stiffness.

$$c = \frac{\delta}{P} \qquad (mm/kN)$$

The effect of various parameters on flexibility of a connection is examined in the following sections.

7.5. Design philosophy on connection flexibility

Various solutions in trying to estimate the flexibility of a bolted connection, and hence predict its moment/rotation characteristics, is possible. Fig. 7-6 shows two possible solutions.

It is the author's view that what is required for practical design is a simplified linear representation of the connection, yielding a reasonable approximation to its characteristics. More elaborate design models are believed to be unnecessary and will lead to unwieldy expressions.

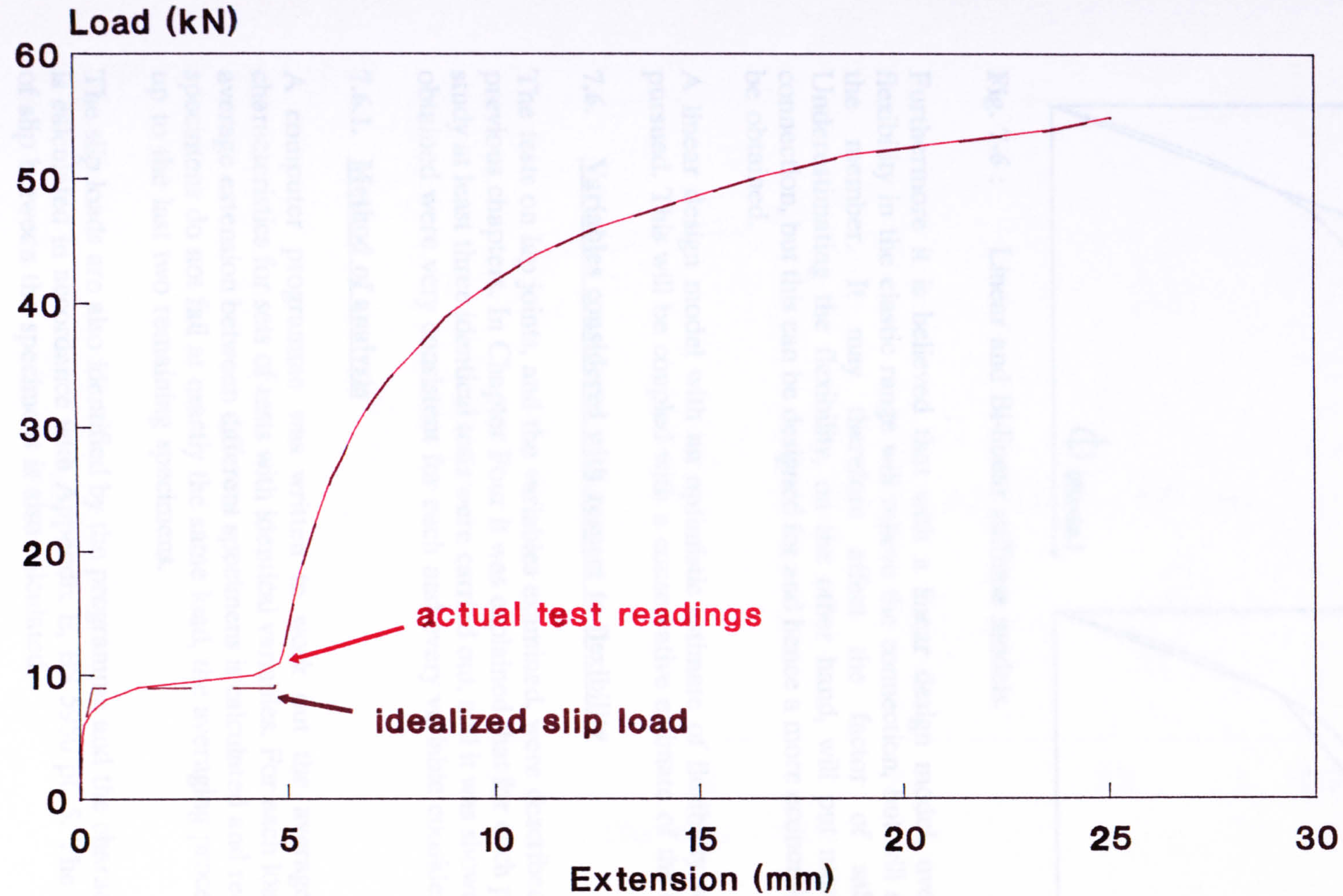


Fig. 7-5
Example of an idealized slip load

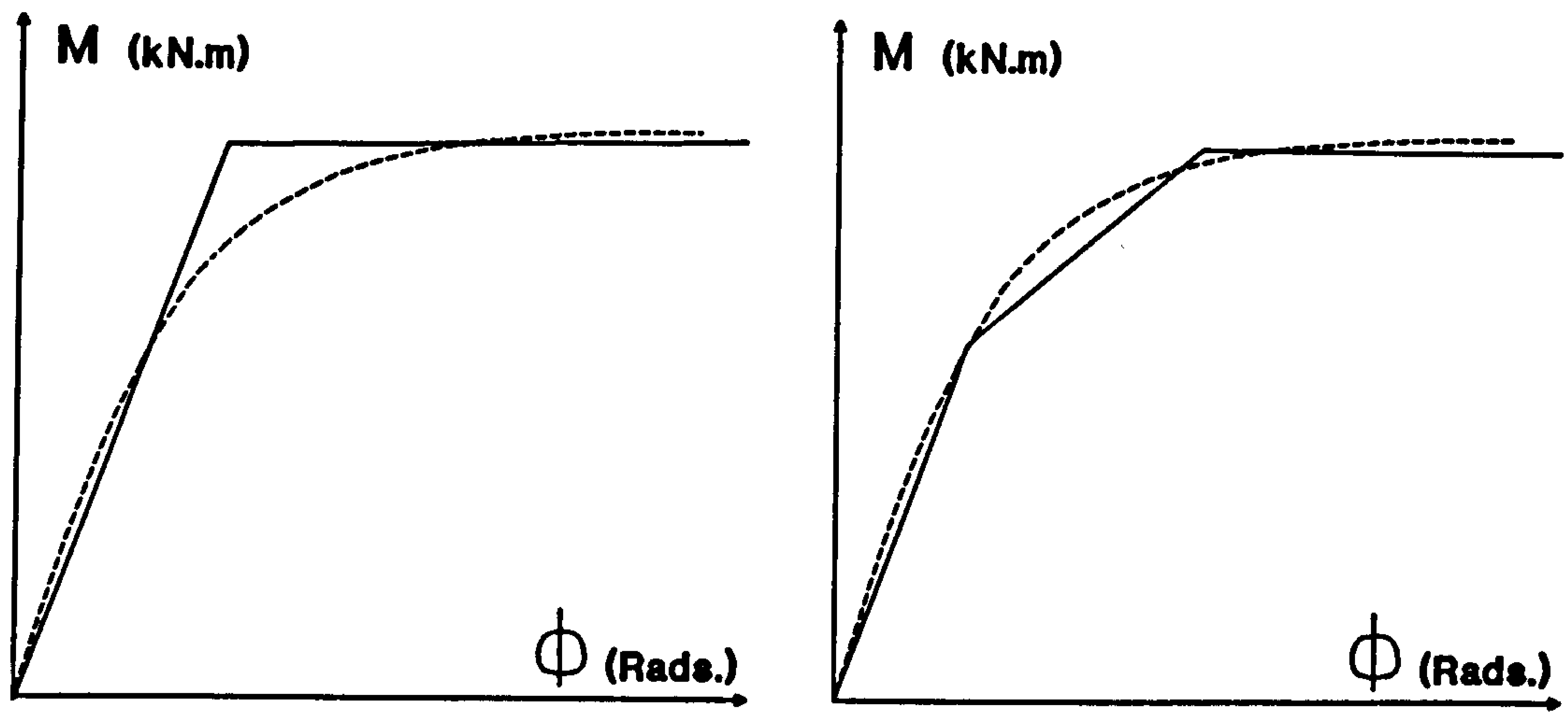


Fig. 7-6 : Linear and Bi-linear stiffness models.

Furthermore it is believed that with a linear design model, overestimating the flexibility in the elastic range will relieve the connection, but will shed moment in the member. It may therefore affect the factor of safety elsewhere. Underestimating the flexibility, on the other hand, will put more strain on the connection, but this can be designed for and hence a more economical solution may be obtained.

A linear design model with an optimistic estimate of flexibility will therefore be pursued. This will be coupled with a conservative estimate of the slip load.

7.6. Variables considered with respect to flexibility

The tests on lap joints, and the variables examined, were described in detail in the previous chapters. In Chapter Four it was explained that for each parameter under study at least three identical tests were carried out, and it was shown that the results obtained were very consistent for each and every variable considered.

7.6.1. Method of analysis

A computer programme was written to work out the average load/extension characteristics for sets of tests with identical variables. For each load increment the average extension between different specimens is calculated and recorded. Since all specimens do not fail at exactly the same load, the averaging process is carried out up to the last two remaining specimens.

The slip loads are also identified by the programme and the characteristic slip load is calculated in accordance with Appendix E, BS 5950 pt. 5. The average amount of slip between the specimens is also calculated.

A typical result is the average load/extension characteristics, plotted in red in Fig. 7-7 compared to the actual tests for a given set of variables.

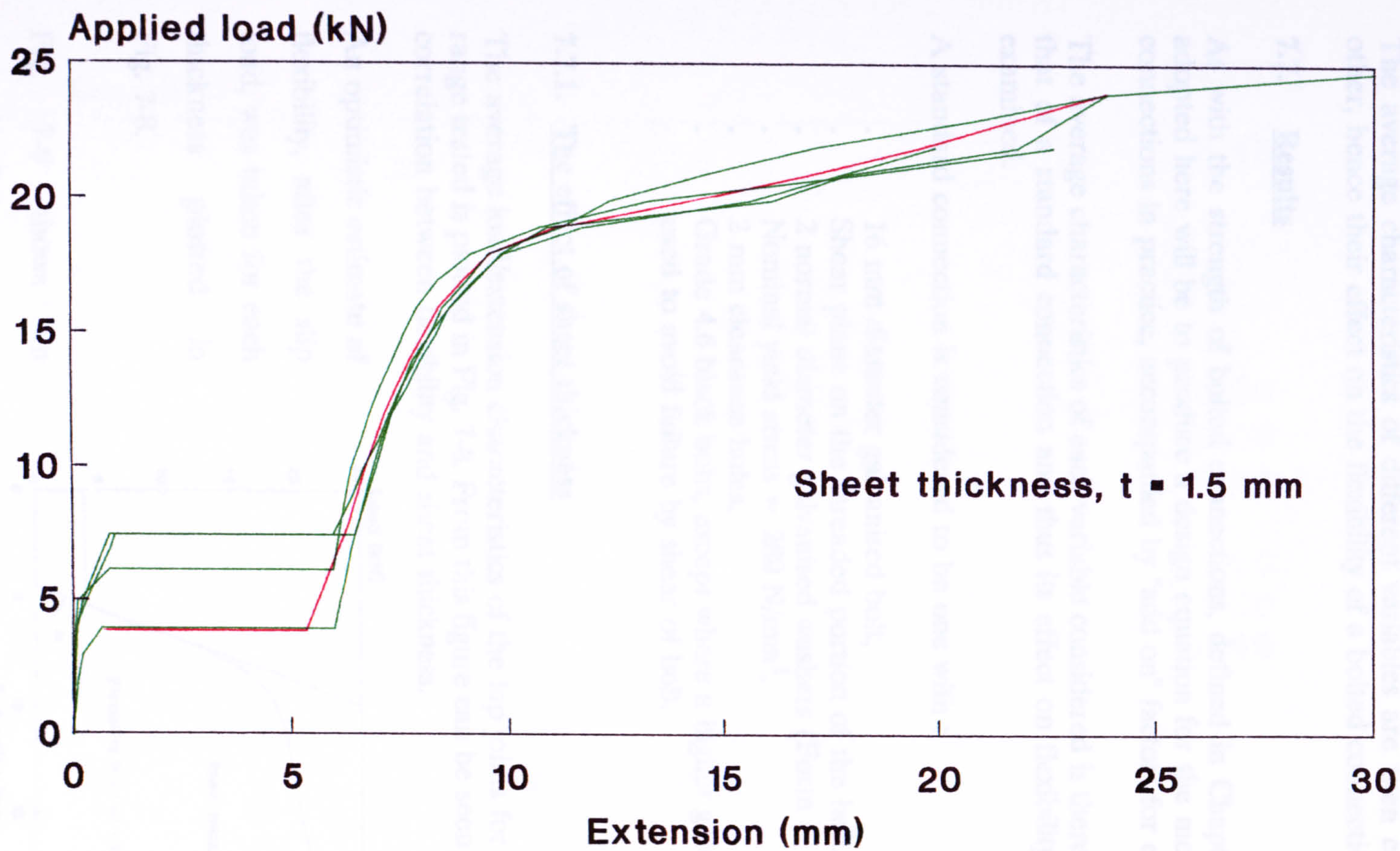


Fig. 7-7
Average characteristics plotted against actual tests.

Note that in Fig. 7-7, only the slip load has been based on a characteristic value, (i.e P_{slip} = $mean_{slip}$ - ks) and all extension values, considering that they are all closely packed together, have been based on average values.

The average characteristics of different variables are then compared with each other, hence their effect on the flexibility of a bolted connection is examined.

7.7. Results

As with the strength of bolted connections, defined in Chapter Four, the policy adopted here will be to produce a design equation for the most common form of connections in practice, accompanied by "add on" factors for extra effects.

The average characteristics of each variable considered is therefore plotted against that of a standard connection and thus its effect on flexibility of a connection is examined.

A standard connection is considered to be one with :

- 16 mm diameter galvanised bolt,
- Shear plane on the threaded portion of the bolt.
- 2 normal diameter galvanised washers (Form E, BS 4320),
- Nominal yield stress = 280 N/mm^2,
- 2 mm clearance holes,
- Grade 4.6 black bolts, except where a higher grade of bolt had to be used to avoid failure by shear of bolt.

7.7.1. The effect of sheet thickness

The average load/extension characteristics of the lap joints for the whole thickness range tested is plotted in Fig. 7-8. From this figure can be seen that there is a clear correlation between flexibility and sheet thickness.

An optimistic estimate of flexibility, after the slip load, was taken for each thickness plotted in Fig. 7-8.

Fig. 7-9 shows an example of this.

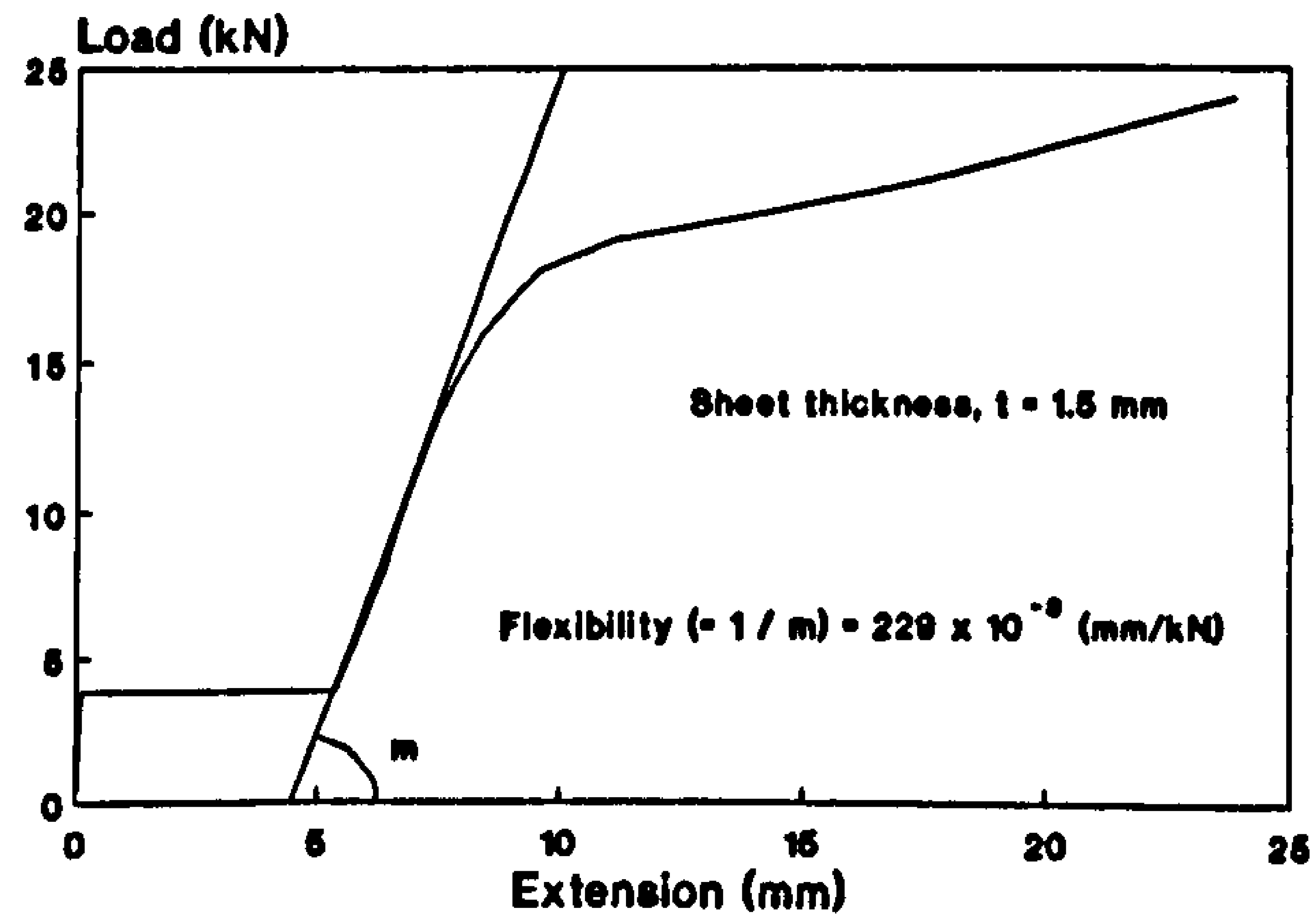


Fig. 7-9 : Estimated flexibility of 1.5 mm thick specimens.

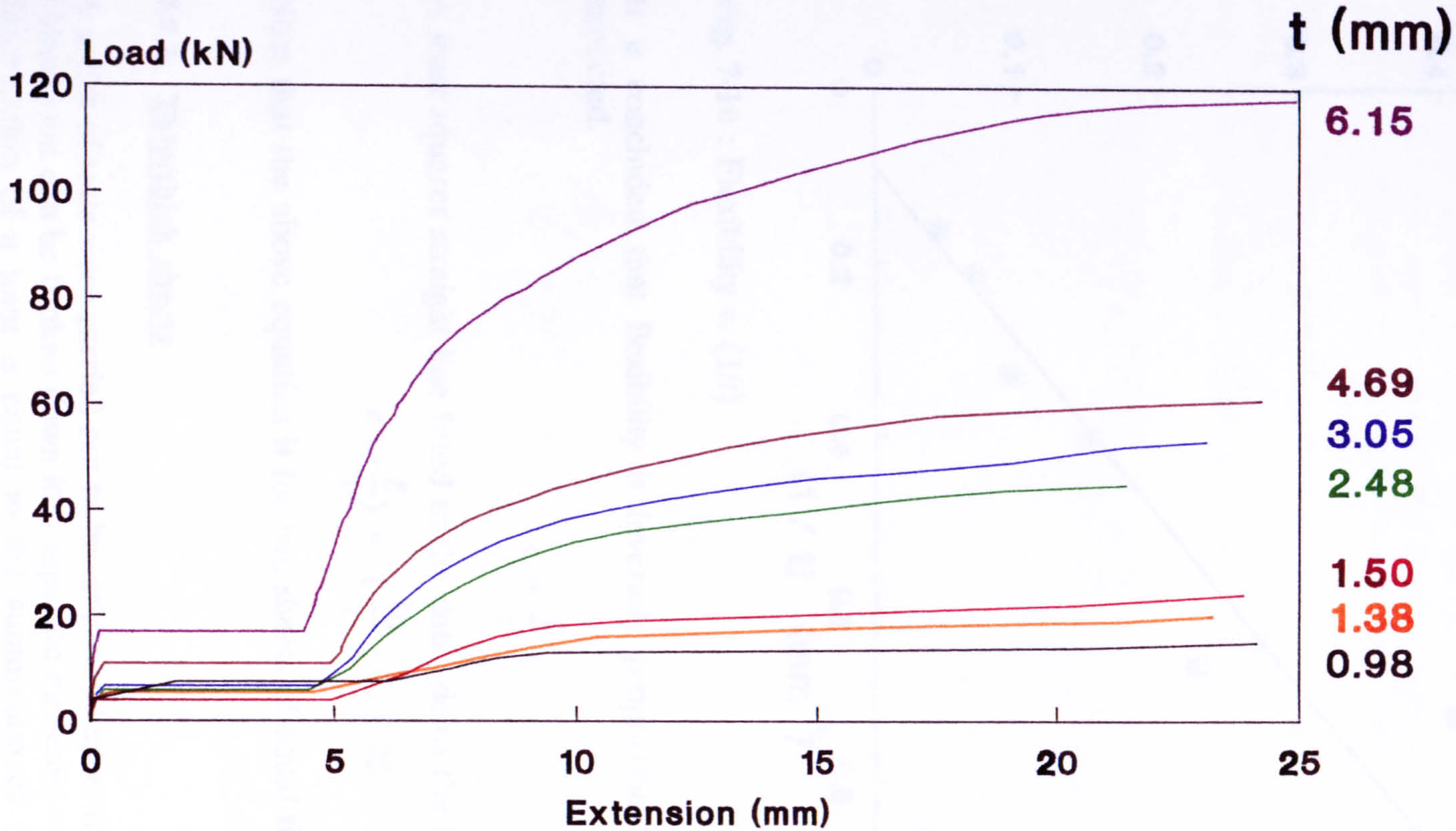


Fig. 7-8
Average load-extension characteristics
across the thickness spectrum

The flexibilities thus obtained, for the whole thickness range tested, are plotted against (1/t) in Fig. 7-10.

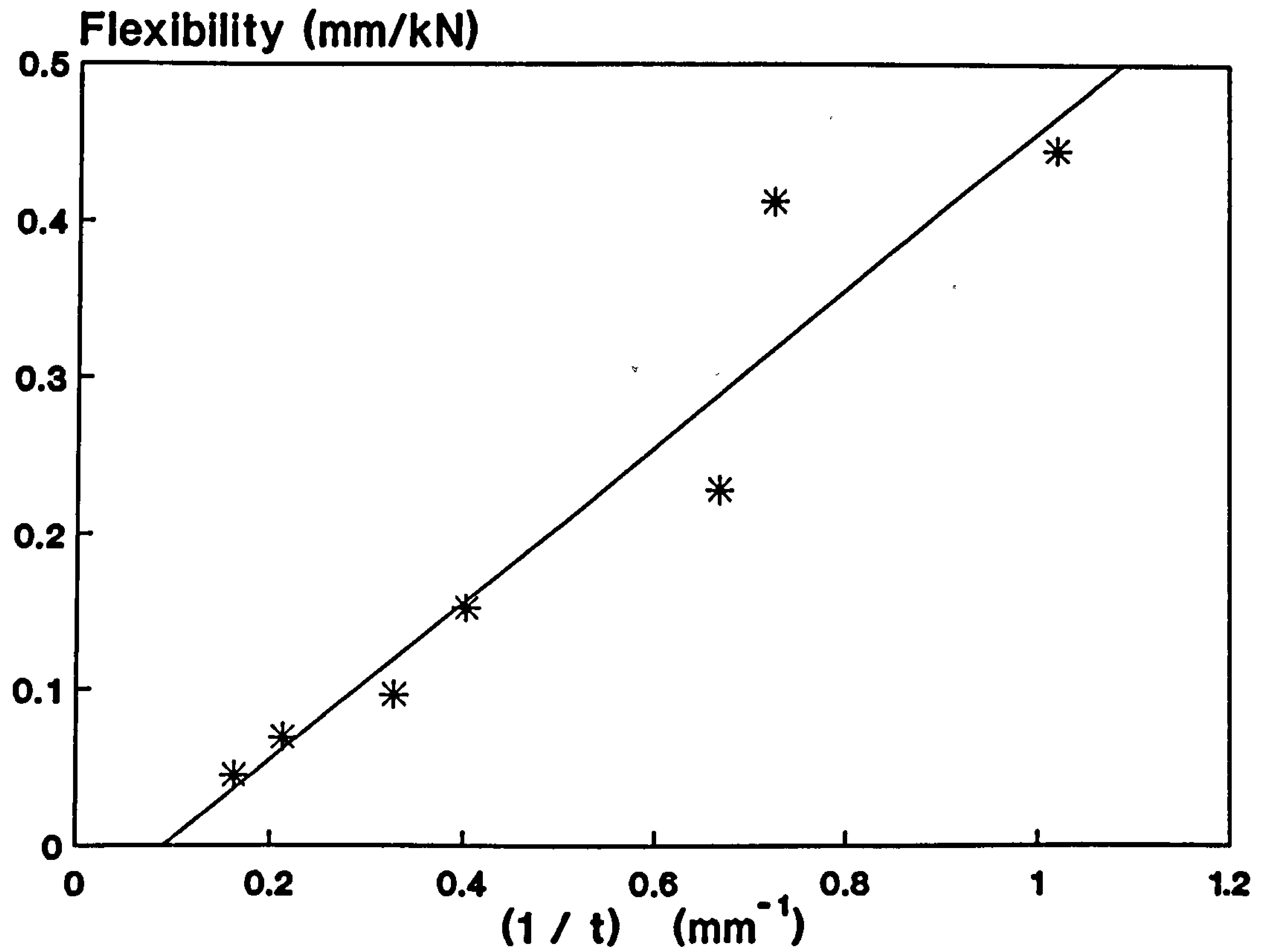


Fig. 7-10 : Flexibility v. (1/t)

It is concluded that flexibility is inversely proportional to sheet thickness, as expected.

$$c \propto \frac{1}{t}$$

A least squares straight line fitted to the data, yields the following expression :

$$c\ (=\frac{\delta}{P}) = (\frac{10}{t} - 1)\ 0.05$$

Note that the above equation is for two sheets of equal thickness.

7.7.2. Thin/thick sheets

A series of tests were carried out to investigate whether the total deformation of a bolted joint can be broken down into separate elements - that is, whether the total deformation of a joint is equal to the summation of the deformations of the connected sheets.

Therefore if the flexibility (deformation per unit load δ/P) of each of the individual connected sheets is known, then the total flexibility of the joint may be predicted.

To this purpose single sheets, with 2mm clearance holes were bolted to a 9mm thick mild steel plate. A thin galvanised sheet was packed in between the two connected elements to maintain the same coefficient of friction as that of lap tests with two sheets of equal thickness. Both the thick plate and packing had 16mm perfect fit holes. The bolt hole in the thick plate was reinforced with a hard steel bush, to make it infinitely stiff compared with the thin sheet. Therefore all the deformation in the joint was due to that of the sheet. Three identical tests, for each sheet thickness considered were carried out, making a total of nine tests.

Since bolt tilting was inhibited by the presence of the thick steel plat; for sheet thicknesses equal to and greater than 2.48 mm ($t \geq 2.48$), grade 8.8 bolts had to used to avoid failure by shearing of the bolts. Fig. 7-11 shows a specimen tested to failure.

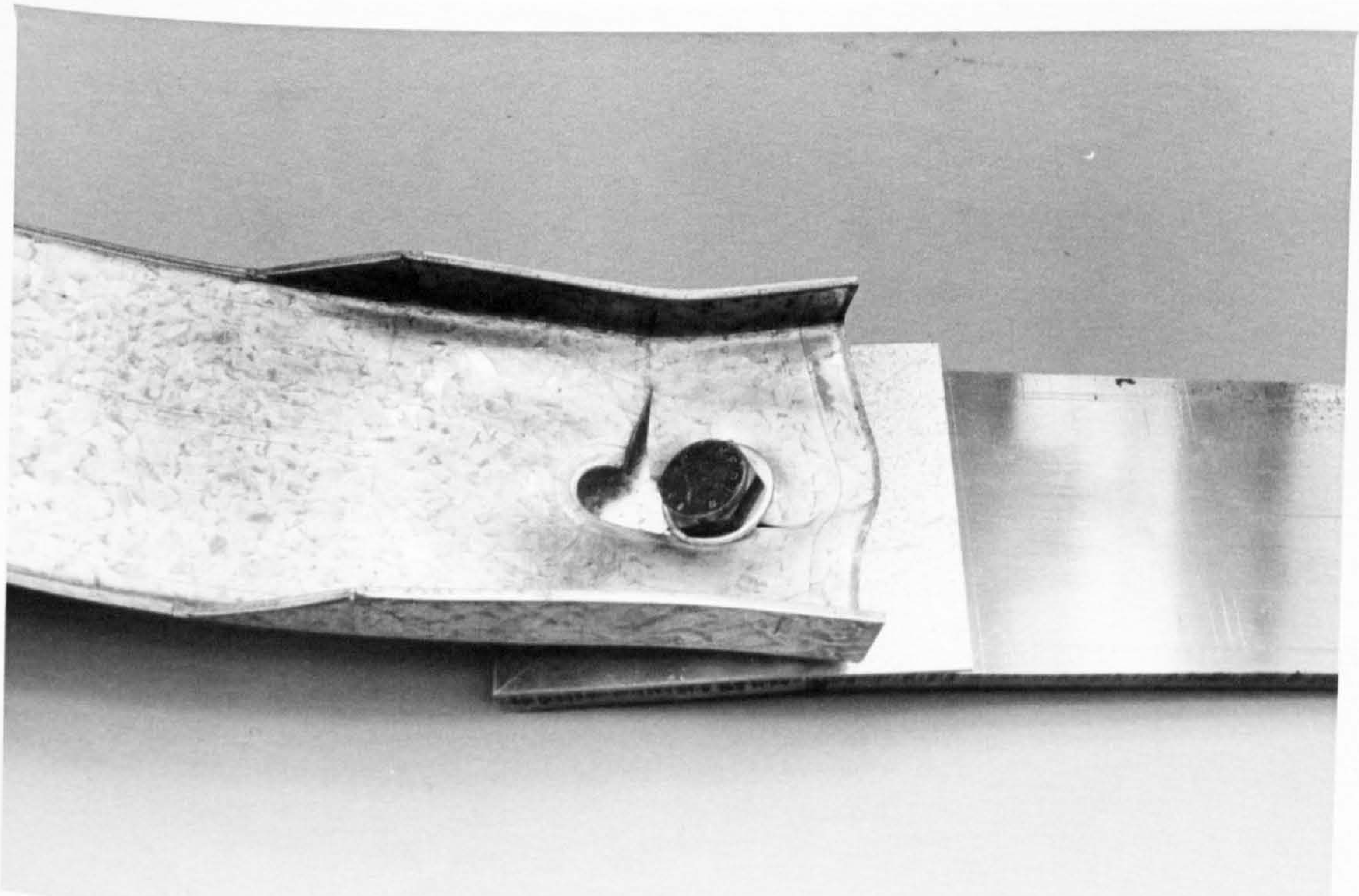
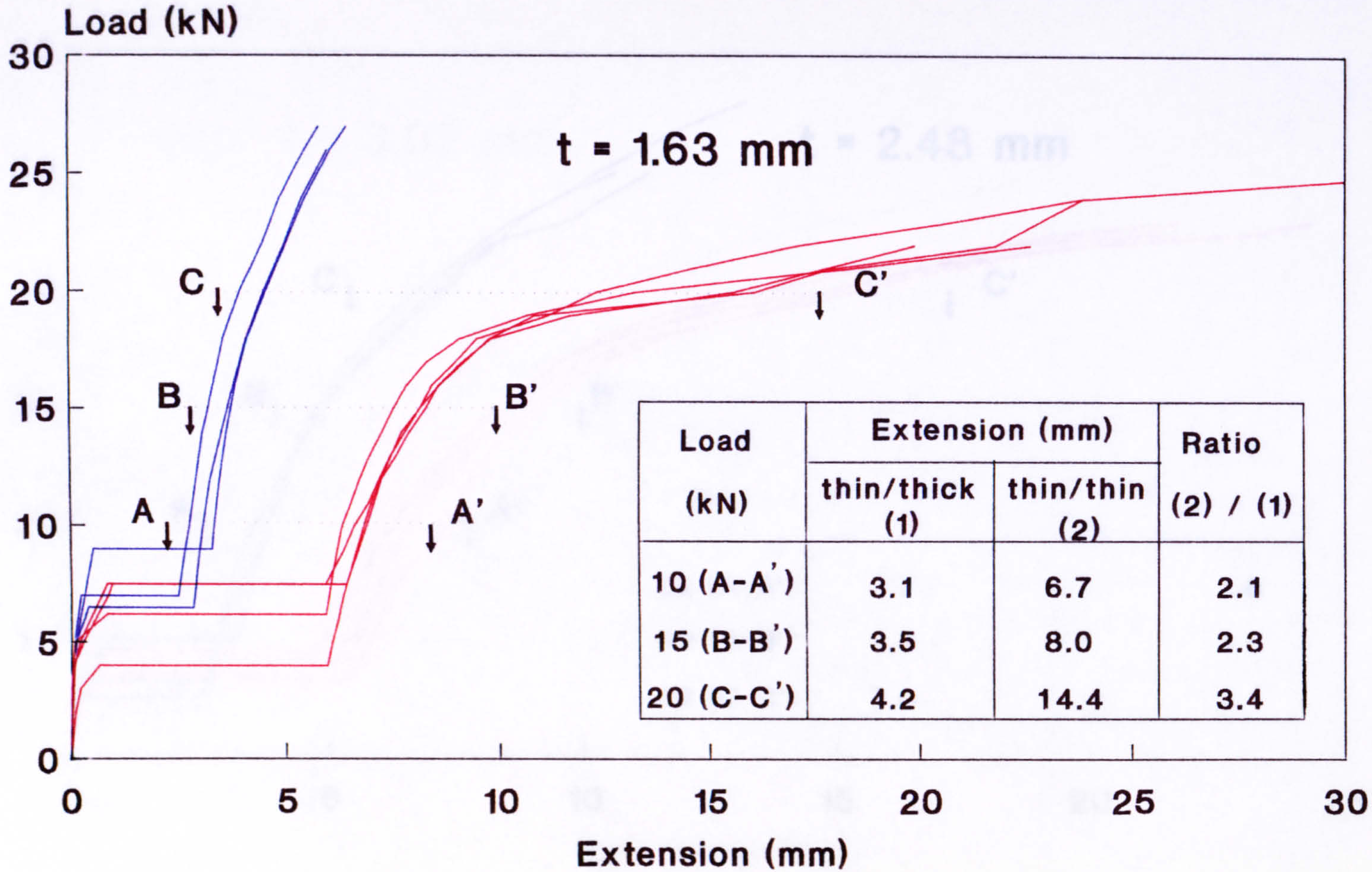


Fig. 7-11 : A thin/thick specimen tested to failure.
Galvanised sheet / Galvanised packing / Thick mild steel plate.

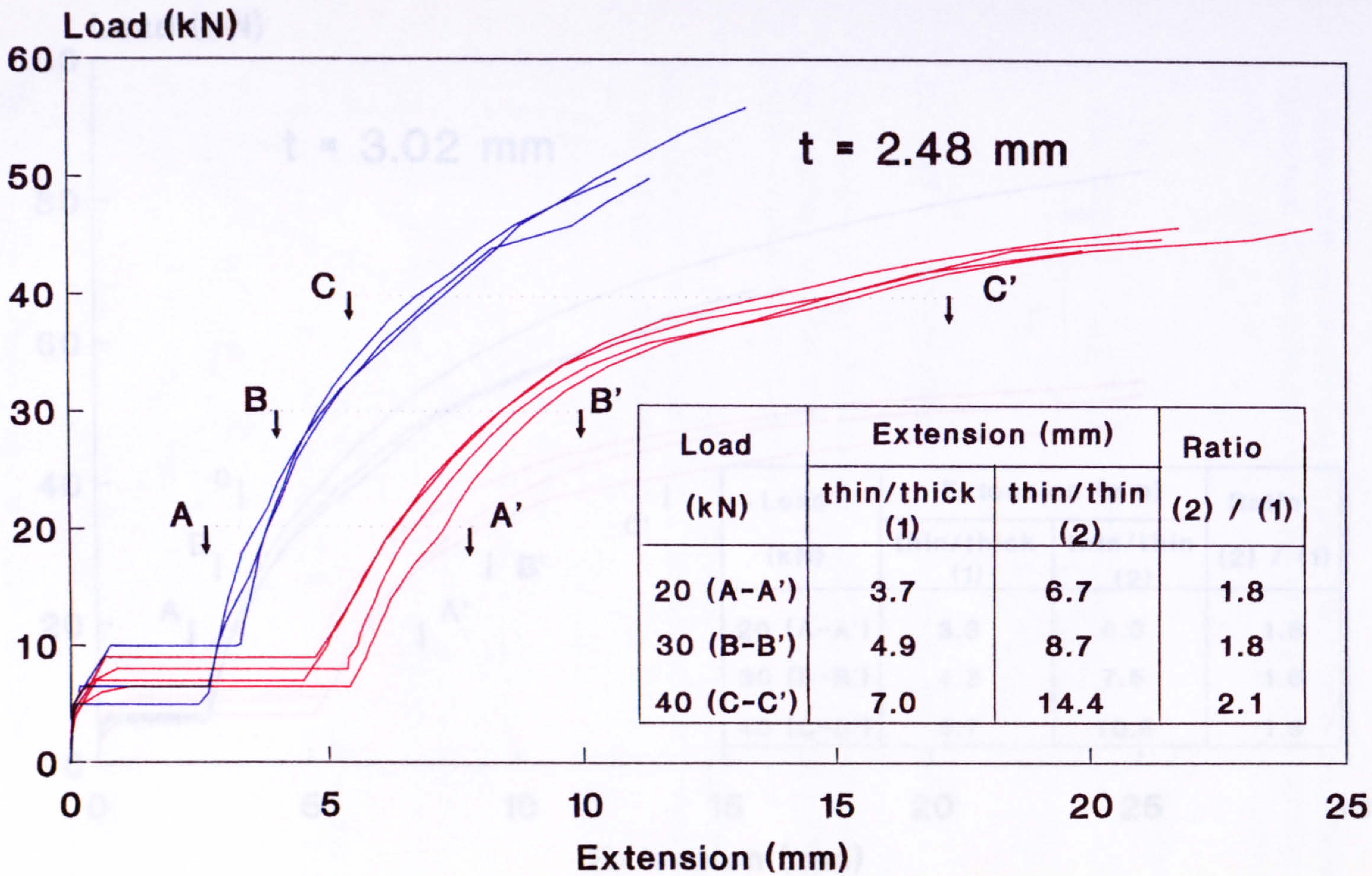
The results are plotted in Fig. 7-12, Fig. 7-13 and Fig. 7-14; for 1.63, 2.48 and 3.02 mm thick sheets respectively. In these figures load/extension characteristics of the tests described above are compared with that of corresponding lap joints, with two galvanised sheets of equal thickness, at different load levels.

From the above mentioned figures can be seen that, the extension ratios of two galvanised sheets, of equal thickness, is very nearly 2:1 compared with that of a



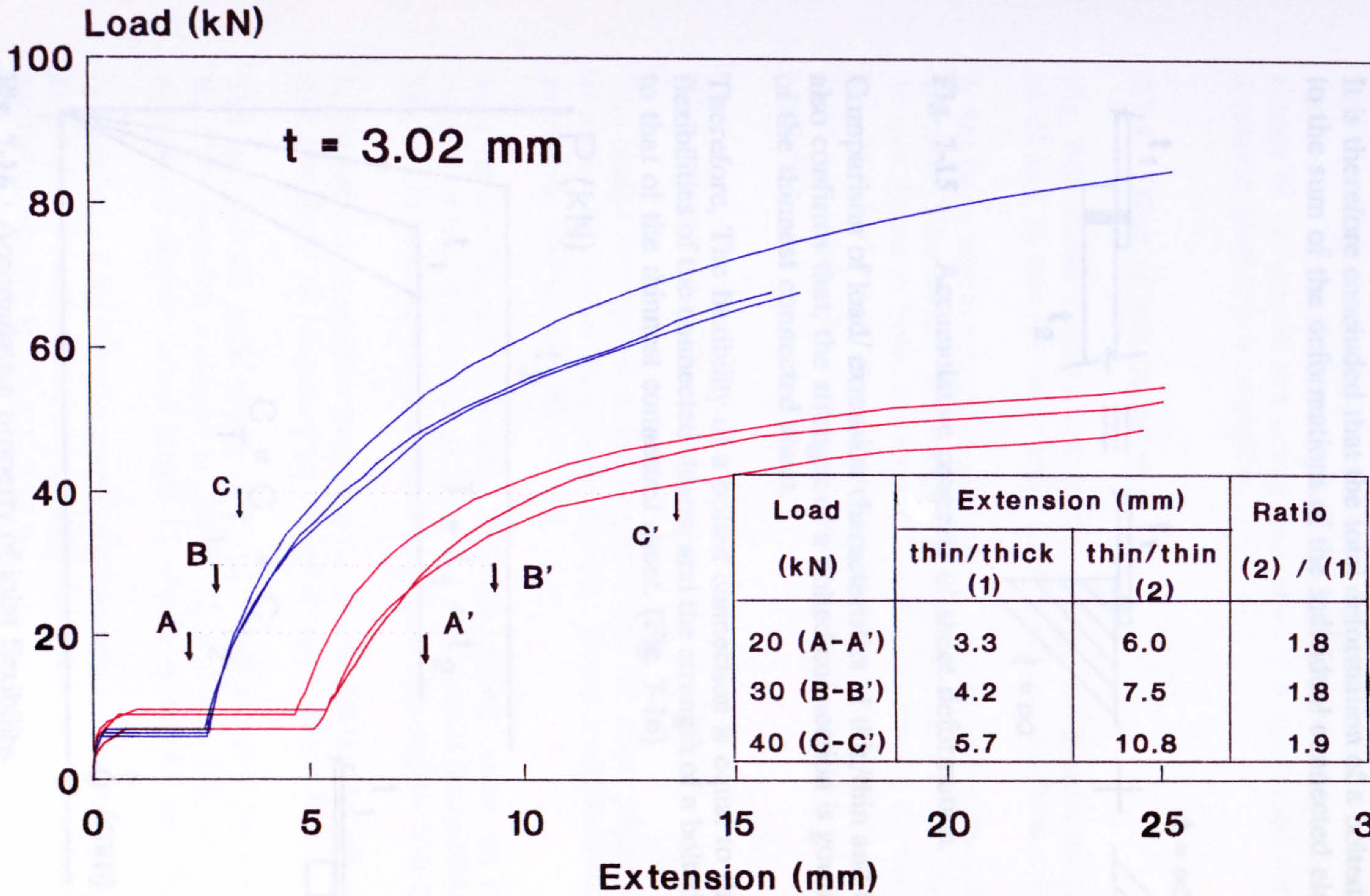
Load (kN)	Extension (mm) thin/thick (1)	Extension (mm) thin/thin (2)	Ratio (2) / (1)
10 (A-A')	3.1	6.7	2.1
15 (B-B')	3.5	8.0	2.3
20 (C-C')	4.2	14.4	3.4

Fig. 7-12
Comparison of thin/thick and thin/thin sheets



Load (kN)	Extension (mm) thin/thick (1)	Extension (mm) thin/thin (2)	Ratio (2) / (1)
20 (A-A')	3.7	6.7	1.8
30 (B-B')	4.9	8.7	1.8
40 (C-C')	7.0	14.4	2.1

Fig. 7-13
Comparison of thin/thick and thin/thin sheets



Load (kN)	Extension (mm) thin/thick (1)	Extension (mm) thin/thin (2)	Ratio (2) / (1)
20 (A-A')	3.3	6.0	1.8
30 (B-B')	4.2	7.5	1.8
40 (C-C')	5.7	10.8	1.9

Fig. 7-14
Comparison of thin/thick and thin/thin sheets

single galvanised sheet (of equal thickness) and an infinitely stiff plate. Only near to failure, the extension of two sheets of equal thickness, exceeds the 2:1 ratio compared to the thin/thick specimens. This was because of the bolt tilting phenomenon, since as it was mentioned previously, it was restricted by the presence of the thick backing plate, in thin/thick specimens.

It is therefore concluded that the total deformation of a bolted connection is equal to the sum of the deformations of the individual connected elements.(Fig. 7-15)

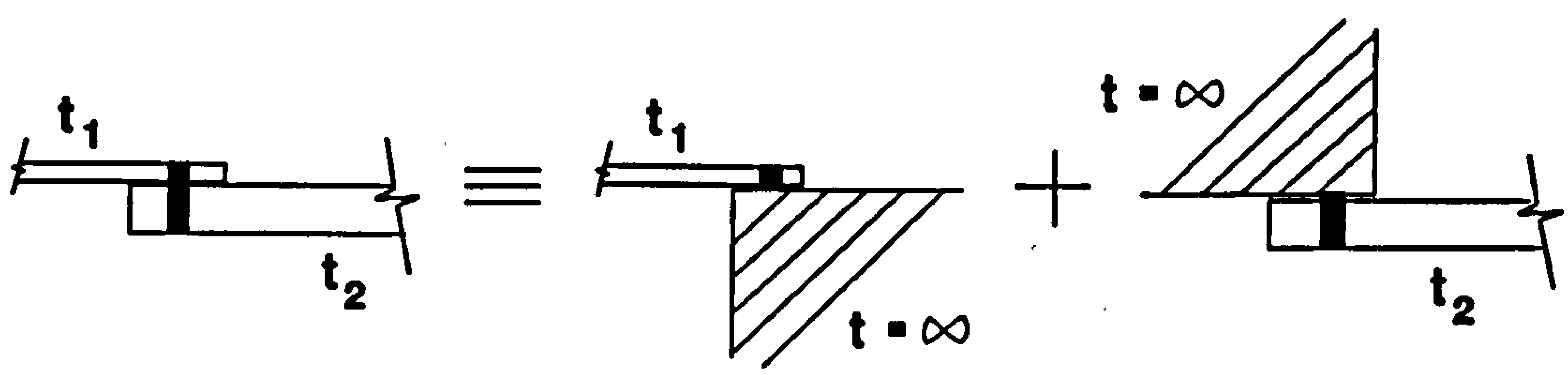


Fig. 7-15 : Accumulative property of sheet deformation.

Comparison of load/ extension characteristics of thin/thin and thin/thick specimens also confirms that, the strength of a bolted connection is governed by the strength of the thinnest connected sheet.

Therefore, The flexibility of a bolted connection is equal to the summation of the flexibilities of the connected sheets; and the strength of a bolted connection is equal to that of the thinnest connected sheet. (Fig. 7-16)

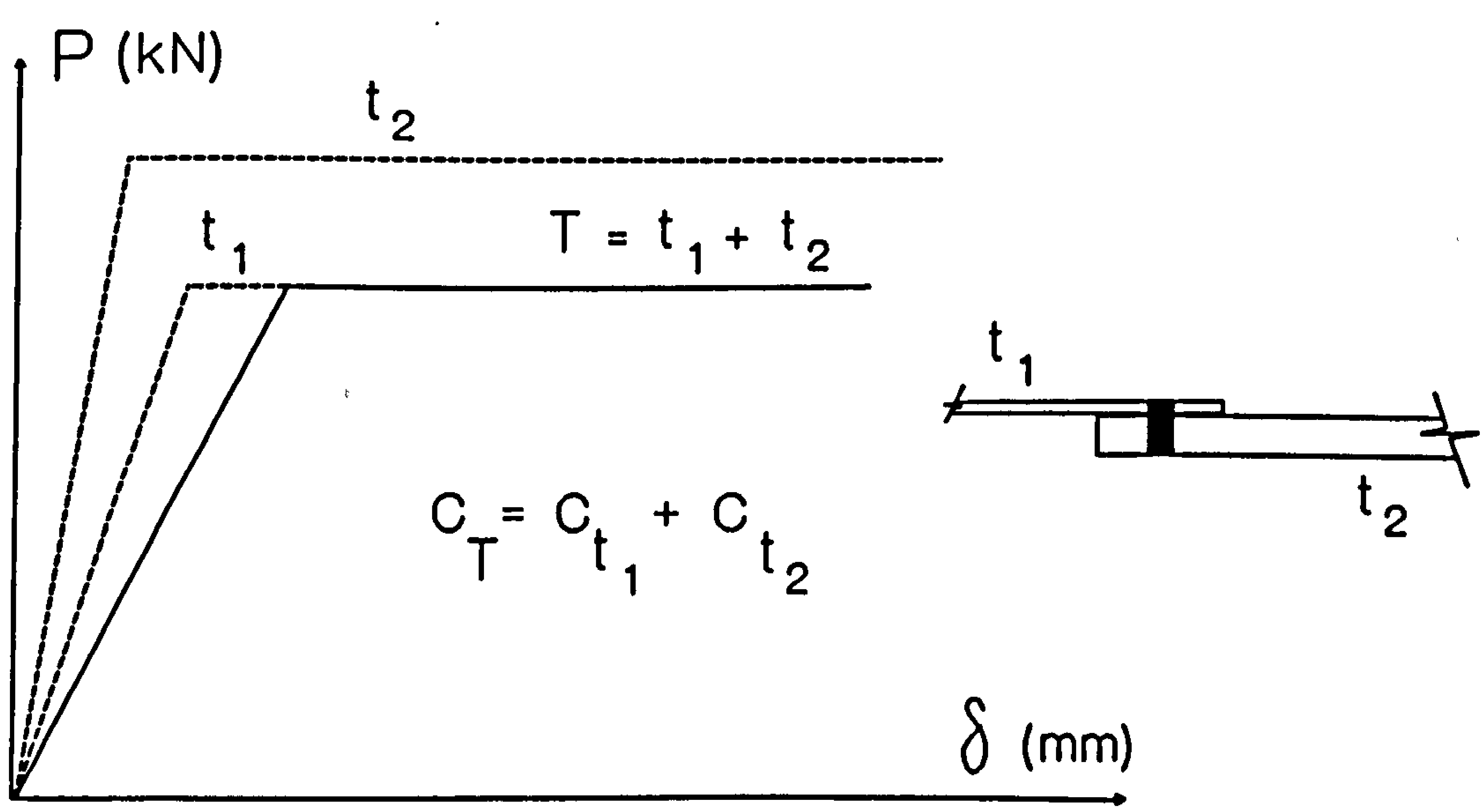


Fig. 7-16 : Accumulative property of joint flexibility.

7.7.3. Effect of Yield Stress

The average load/extension characteristics of specimens with a nominal yield stress of 350 N/mm^2 are plotted against that of 280 N/mm^2 yield steel, for a range of 1.5 to 3.2 mm thick specimens in Fig. 7-17.

The flexibility values c, are noted in each figure, for 280 and 350 N/mm^2 yield steels. As can be seen the flexibility is little affected by using a harder steel. Since the slopes of the graphs are nearly the same, it is concluded that the effect of yield stress on flexibility is negligible.

7.7.4. Effect of bolt diameter

Fig. 7-18 shows a comparison of the average load/extension characteristics of that of 12 and 16 mm diameter bolts, for a specimen thickness range of 1.5 to 3.0 mm.

It has been mentioned previously that a constant bolt torque (65 Nm) was used in all tests. This torque produced a greater grip on 12 mm diameter bolts than in 16 mm diameter bolts, resulting in a higher slip load in the former case. Otherwise, it is seen that the slope of load/extension characteristics of 16 and 12 mm diameter bolts are nearly the same. The effect of bolt diameter on flexibility is therefore negligible.

7.7.5. Shear plane on plain shank or thread of bolts

The design assumption so far, has been that the shear plane occurs on the threads of a fastener, rather than its plain shank. Under load the threads dig into the area in bearing and precipitate tearing.

Test results indicate that when the shear plane occurs on the threaded portion instead of the plain shank of bolts, the flexibility of connections is nearly doubled. The results will be discussed further in the following section. (Fig. 7-19)

It is therefore propitious to design for the correct bolt length whenever possible, so that the shear plane occurs on the plain shank of the bolt.

The amount of plain shank in a bolt is determined by the length of the bolt. BS 4190 specifies a thread length of 2d + 6 mm. For structural applications an alternative shorter thread length of 1½ d may also be specified.

In specifying a bolt length care should be taken to ensure that the thread length is long enough to allow ample travel for the nut to reach the connected sheets, or the washer under the connected sheets.

It therefore follows that the length of the plain shank in no circumstances should exceed :

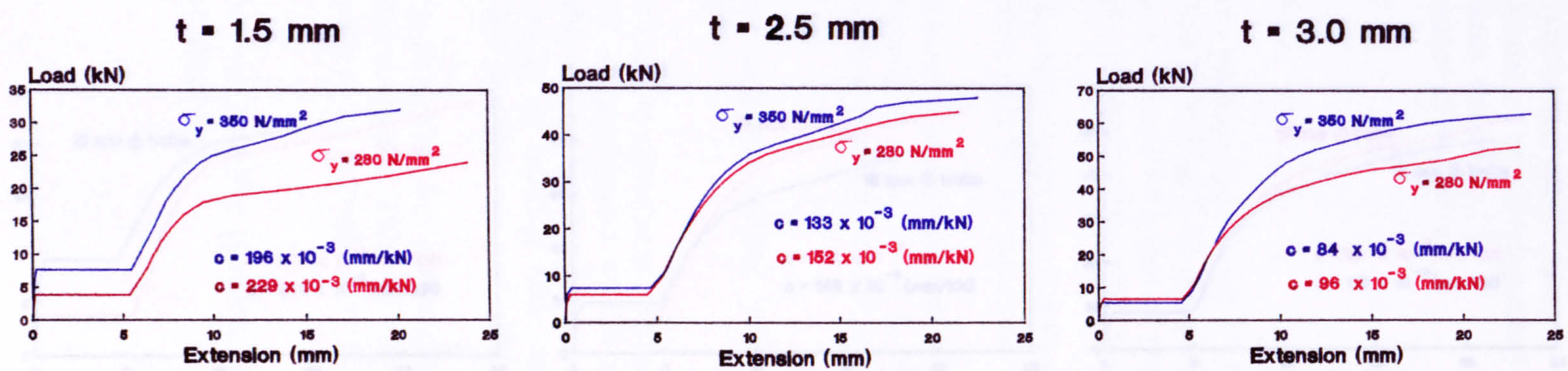


Fig. 7-17

Effect of yield stress on joint flexibility

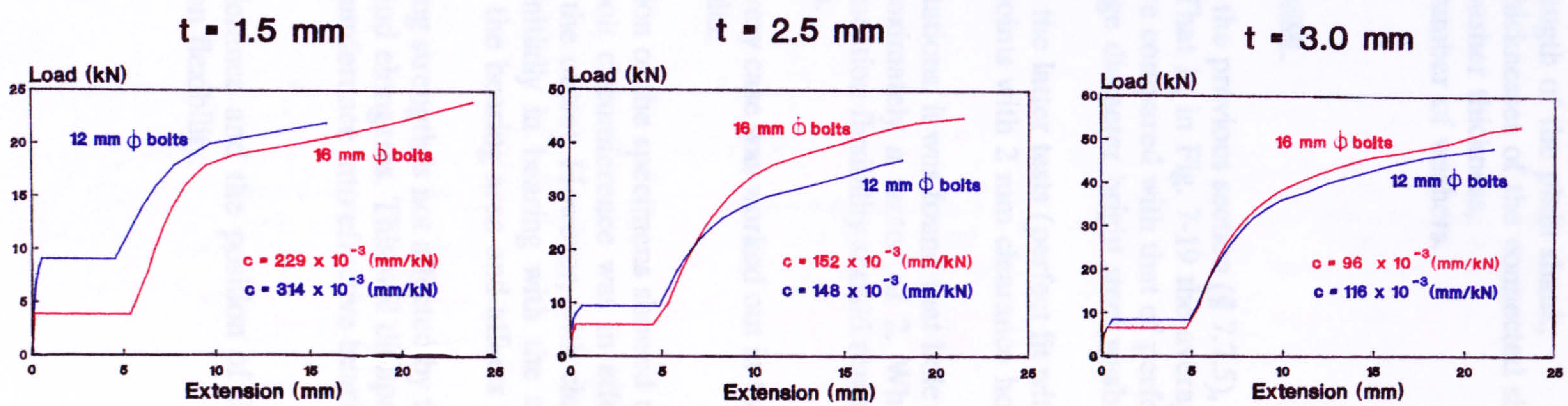


Fig. 7-18

Effect of bolt diameter on joint flexibility

$$l_{ps} \leq (t_1 + t_2) + n_w.t_w$$

Otherwise the bolt can not be tightened.

l_{ps} = length of the plain shank,
t_1,t_2 = thicknesses of the connected sheets,
t_w = washer thickness,
n_w = number of washers.

7.7.6. Hole tolerance

Tests described in the previous section (§ 7.7.5), were carried out on lap joints with perfect fit holes. That is, in Fig. 7-19 the average characteristics of perfect fit lap joints with bolts are compared with that of perfect fit lap joints with set screws. All specimens had large diameter bright steel washers.

In the same figure the latter tests (perfect fit with set screws) are compared to that of equivalent lap joints with 2 mm clearance holes.

Contrary to expectations, it was found that hole tolerance increases the connection flexibility by approximately a factor of 2. Where it might have been originally expected that, connection flexibility should remain unaffected once the initial slack has been taken up.

The flexibility in every case was worked out in the same load range, so that it would not affect the results.

A closer examination of the specimens showed the reason for this. With perfect fit holes the whole bolt circumference was in effective bearing with the connected sheets, right from the outset. However, with clearance holes only part of the bolt circumference is initially in bearing with the connected sheets. This leads to a localized stress on the bearing area and affects the flexibility.

The ultimate bearing strength is not affected by this, since as the load increases, the bolt hole distorts and elongates. This will dissipate the localized stress and brings the whole bolt circumference into effective bearing by the time the ultimate load is reached.

Therefore hole tolerance and the position of the shear plane, have a significant effect on connection flexibility.

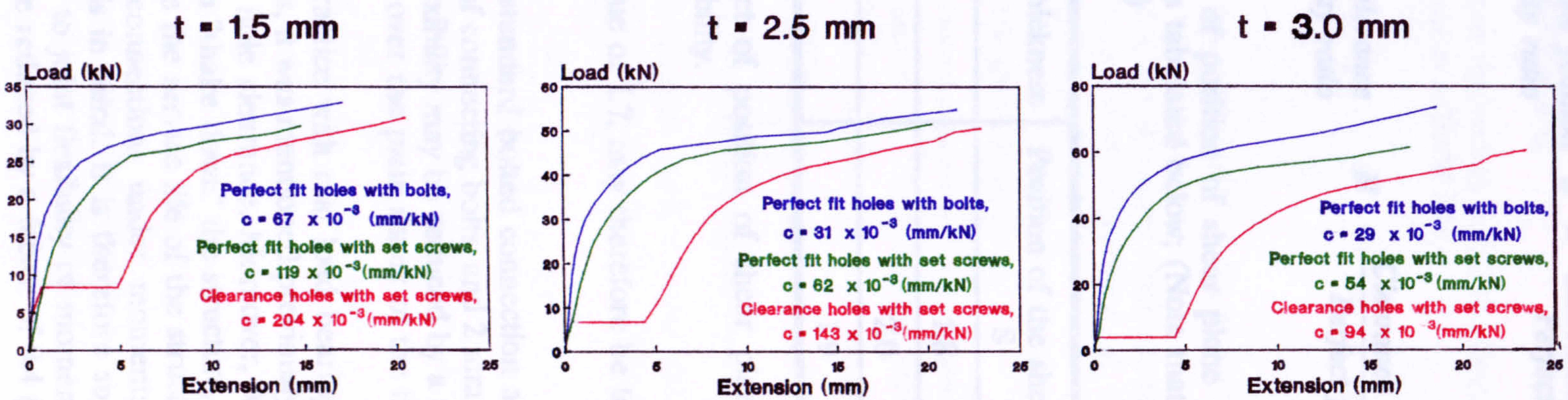


Fig. 7-19

Effect of hole tolerance and position of shear plane on joint flexibility

With reference to Fig. 7-19, if S and H are defined as :

$$\textit{Shear plane position flexibility ratio} \quad S = \frac{\textit{Perfect fit set screw flexibility}}{\textit{Perfect fit bolt flexibility}}$$

$$\textit{Hole tolerance flexibility ratio} \quad H = \frac{\textit{Clearance holes set screw flexibility}}{\textit{Perfect fit set screw flexibility}}$$

then the effects of position of shear plane and hole tolerance, i.e. S and H respectively, are as tabulated below; (Note that all the values required above, are listed in Fig. 7-19.)

Nominal sheet thickness (mm)	Position of the shear plane, **S**	Hole tolerance, **H**
1.5	1.8	1.7
2.5	2.0	2.3
3.0	1.9	1.7

Table 7.1 : Effect of position of shear plane and hole tolerance on joint flexibility.

A conservative value of 1.7, may therefore be taken for each of the above factors, i.e. S and H.

Having defined a standard bolted connection as that with the shear plane on the threaded portion of connecting bolts, and 2 mm clearance holes in § 7.7 - it follows that connection flexibility may be reduced by a factor of 0.4 (= 1 - 1/S) should the shear plane occur over the plain shank of the bolts.

Furthermore in practice with real load bearing moment connections, often with three or more bolts, it was mentioned previously that the connection clearance can hardly exceed the hole clearance. Moreover, it is believed that dead loads, wind loads etc. will soon "shake down" the structure, and any initial hole clearance will be taken up before the service life of the structure begins. It is therefore believed that with actual connections under moment; connection flexibility should be calculated with this in mind. It is therefore suggested that a factor of (1 - 1/H) should be applied to joint flexibility of moment connections. That is to say joint flexibility should be reduced by a factor of 0.4 (=1 - 1/1.7), for connections under load in practice.

Results of full moment connections given in the following chapters, will justify the above premise.

7.8. Design joint flexibility

Based on conclusions derived in § 7.7, the flexibility of bolted connections in cold formed steel sections is defined as :

$$c = 5n\left(\frac{10}{t_1} + \frac{10}{t_2} - 2\right) \times 10^{-3} \qquad \left(\frac{mm}{kN}\right)$$

where c = the joint flexibility (in mm/kN)

t_1 and t_2 are the sheet thicknesses (in mm)
note that from definition of cold formed sections $t_1 \leq$ 8mm and $t_2 \leq$ 8mm.

The factor n is given as follows:

Position of the shear plane on the bolts	For joints in tension	For joints under moment
Full shank diameter	3	1.8
Threaded portion	5	3

For instance if two purlins each 1.8 mm thick, are bolted together, then the flexibility of the connection is equal to : (assuming the shear plane to be on the threaded portion of the bolts)

$$c = 5 \times 3\left(\frac{10}{1.8} \times 2 - 2\right) \times 10^{-3} = 137 \times 10^{-3} \qquad (mm/kN)$$

Equally if one section is 1.8 and the other 2.4 mm thick, then the flexibility of the connection will be:

$$c = 5 \times 3\left(\frac{10}{1.8} + \frac{10}{2.4} - 2\right) \times 10^{-3} = 116 \times 10^{-3} \qquad (mm/kN)$$

7.9. Slip load

Properties of slip load were described in § 7.3.1. For a bolt torque representative of the insitu conditions a conservative slip load of 4 kN is recommended.

7.10. Amount of slip

In § 7.3 was mentioned that the maximum theoretical amount of slip, for lap joints, is equal to twice the hole clearance. For 2 mm clearance holes tested this will be 4 mm.

The average load/extension characteristic plots in Fig. 7-8, shows that the amount of slip is often in excess of 4 mm. Typical slip values are between 4 to 5 mm. This extra amount can be accounted for by two factors. First is the manufacturing tolerance of the bolt diameter. For a nominal 16 mm diameter bolt the actual bolt diameter is usually about 15.7 mm. Second for a 16 mm diameter bolt the thread pitch is 2 mm. So with the thinner specimens one of the connected sheets could enter this gap until the pitch narrows down to the sheet thickness.

With multiple bolted connections however, for the reasons described in § 7.3 the amount of rigid body slip could hardly reach the clearance itself, if indeed hole clearance is to be considered at all.

The recommended amount of slip in defining the load/extension and hence the moment/rotation characteristics of a connection is therefore equal to the hole clearance.

7.11. Conclusions

The flexibility of the bolted connections in cold formed steel has been analysed. All relevant factors have been identified and quantified. Based on this and results described in the previous chapters on the strength of such connections, the general load/extension characteristics of bolted connections may be defined as follows: (Fig. 7-20)

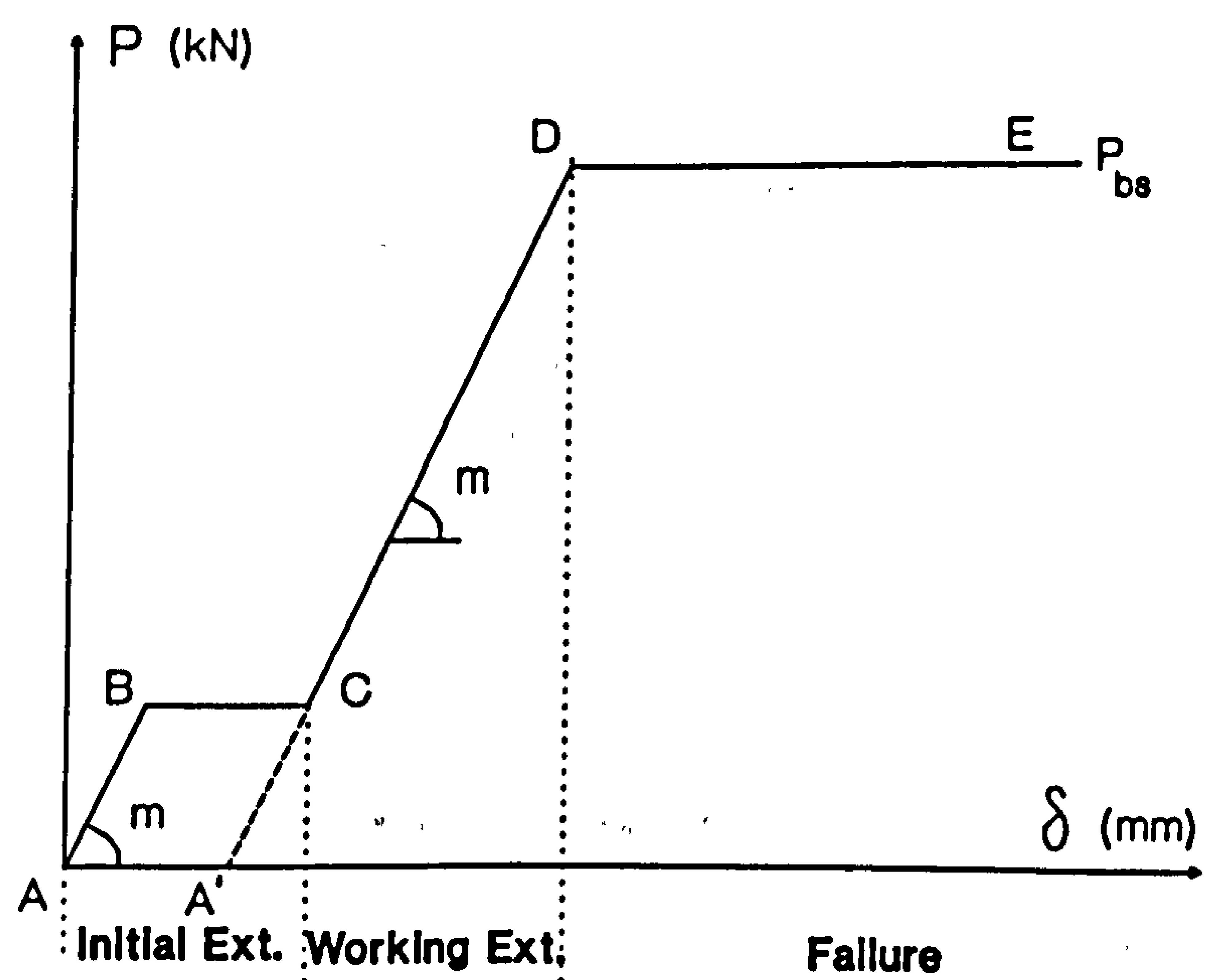


Fig. 7-20 : General Load/Extension characteristics of bolted connections in cold formed steel sections.

The variables listed in Fig. 7-20 above, are as follows:

The Ultimate bearing strength $P_{bs} = \alpha \cdot d.t.\sigma_{ult}$

Where α is defined as :

$$\alpha = k_1.k_2.k_3.k_3.k_4.k_5.k_6.k_7$$

k_1 to k_7 are factors given for the variables listed below :

Bolt diameter; $k_1 = (16/d)^{1/2}$	**Sheet thickness;** $k_2 = (1.9 + 0.2\,t)$ for $t \leq 3mm$ $= 2.5$ $\quad 3 < t \leq 8mm$
Mechanical properties; $k_3 = (390/\sigma_{ult.\ design})^{1/2}$ where $\sigma_{ult.\ design}$ is the design ultimate stress of the sheet material.	**Washer diameter;** For Normal diameter washers, (Form E, BS 4320) $k_4 = 1.0.$ For Large diameter washers, (Form F, BS 4320) $k_4 = 1.15$ for $t \leq 2mm$ $= 1.05$ for $2 < t \leq 3mm$ $= 1.0$ for $t > 3mm$
Number of washers; $k_5 = 1.0$ when two washers are used. $= 0.8$ when only one washer is used. $= 0.7$ when no washers are used.	**End distance in the line of stress;** $k_6 =$ the lesser of $(e/2.5d)$ and 1. where $(e/d) \geq 1.5$.
Shear plane on the plain shank or threads of bolts; $k_7 = 1.15$ where it can be shown be shown that the shear plane occurs over the full shank diameter. $= 1.0$ otherwise	

The **flexibility** of a connection (1/m, in Fig. 7-20) is defined as :

$$c = 5n\left(\frac{10}{t_1} + \frac{10}{t_2} - 2\right) \times 10^{-3} \qquad \left(\frac{mm}{kN}\right)$$

where c = the joint flexibility (in mm/kN)

t_1 and t_2 are the sheet thicknesses (in mm)
note that from definition of cold formed sections $t_1 \leq$ 8mm and $t_2 \leq$ 8mm.

The factor n is given as follows:

Position of the shear plane on the bolts	For joints in tension	For joints under moment
Full shank diameter	3	1.8
Threaded portion	5	3

The **slip load** of a bolted connection may be taken as **4 kN.**

The **amount of slip** may be taken as the **bolt clearance.**

If the design assumption is that the shear plane occurs over the plain portions of bolts, the length of the plain shank of the bolts in no circumstances should exceed :

$$l_{ps} \leq (t_1 + t_2) + n_w . t_w$$

Otherwise the bolt can not be tightened.

l_{ps} = length of the plain shank,
t_1, t_2 = thicknesses of the connected sheets,
t_w = washer thickness,
n_w = number of washers.

The general load-extension characteristics may be defined by the following coordinates (Fig. 7-20):

A (0, 0)
B (4c, 4)
C [(4c+ hole clearance), 4]
D [(P_{bs} c + hole clearance), P_{bs}]

When the initial extension of a connection, depicted in Fig. 7-20, is considered to occur under the self weight of a structure - in other words it is to be ignored in the design process, then the A'DE portion of the general load-extension characteristics apply and the origin is shifted to point A'. i.e. :

A'(0, 0)
D (P_{bs} c, P_{bs})

Chapter Eight

Introduction and experimental procedures in testing of full moment connections

8. Introduction and experimental procedures in testing of full moment connections

Summary

Most joints in cold formed steel structures are required to carry moment. Obviously truss and lattice structures are the exception. In this chapter the procedures adopted in testing of typical bolt groups of two, four and three bolts are described. Two different test arrangements, used to test full moment connections, are illustrated and methods of analysing the data are outlined.

8.1. Introduction

In the previous chapter the load-extension characteristics of bolted connections in cold formed steel sections were defined. Using these values, it is possible to calculate the ultimate moment capacity of a group of bolts, and the joint rotation without resorting to tests.

Of course any "nesting" or "interlocking" of sections is not taken into account in such calculations. These will be considered separately and suitable factors will be introduced.

8.2. Application of the proposed design expressions

The primary purpose of the design expressions conceived in this thesis is to define the moment-rotation characteristics of typical moment connections in cold formed steel sections.

In order to check that these expressions, see § 7.11 Chapter Seven, are valid for all types of bolt configuration a series of tests on full moment connections was carried out.

The moment-rotation characteristics of these tests would hence be compared with that predicted by the design expressions.

From the strength point of view of such connections, the proposed and actual test results are also compared to that given by Annex A.

In this chapter however, the testing procedures adopted are described. Test results will follow in the succeeding chapters.

In testing full moment connections two different test set-ups were used wherever possible. This was to ensure that the results obtained were independent of test conditions.

8.3. Sections Used for testing

To enable a straight comparison between the moment-rotation characteristics of a full moment test and that given by the design expressions in § 7.11, simple lipped channels were used. (Fig. 8-1)

Although the overall dimensions of the section were designed to be typical of the insitu conditions, the section itself is by no means what can be regarded as typical of that used in practice. There are now more advanced profiles available on the market that render superior section capacities for the same material content.

The main bulk of cold formed sections nowadays are about 170-240 mm deep. However, sections as deep as 300 mm such as Swage beams, UltraZeds, Multibeams (Sigma sections), etc. are being more and more frequently used for larger spans.

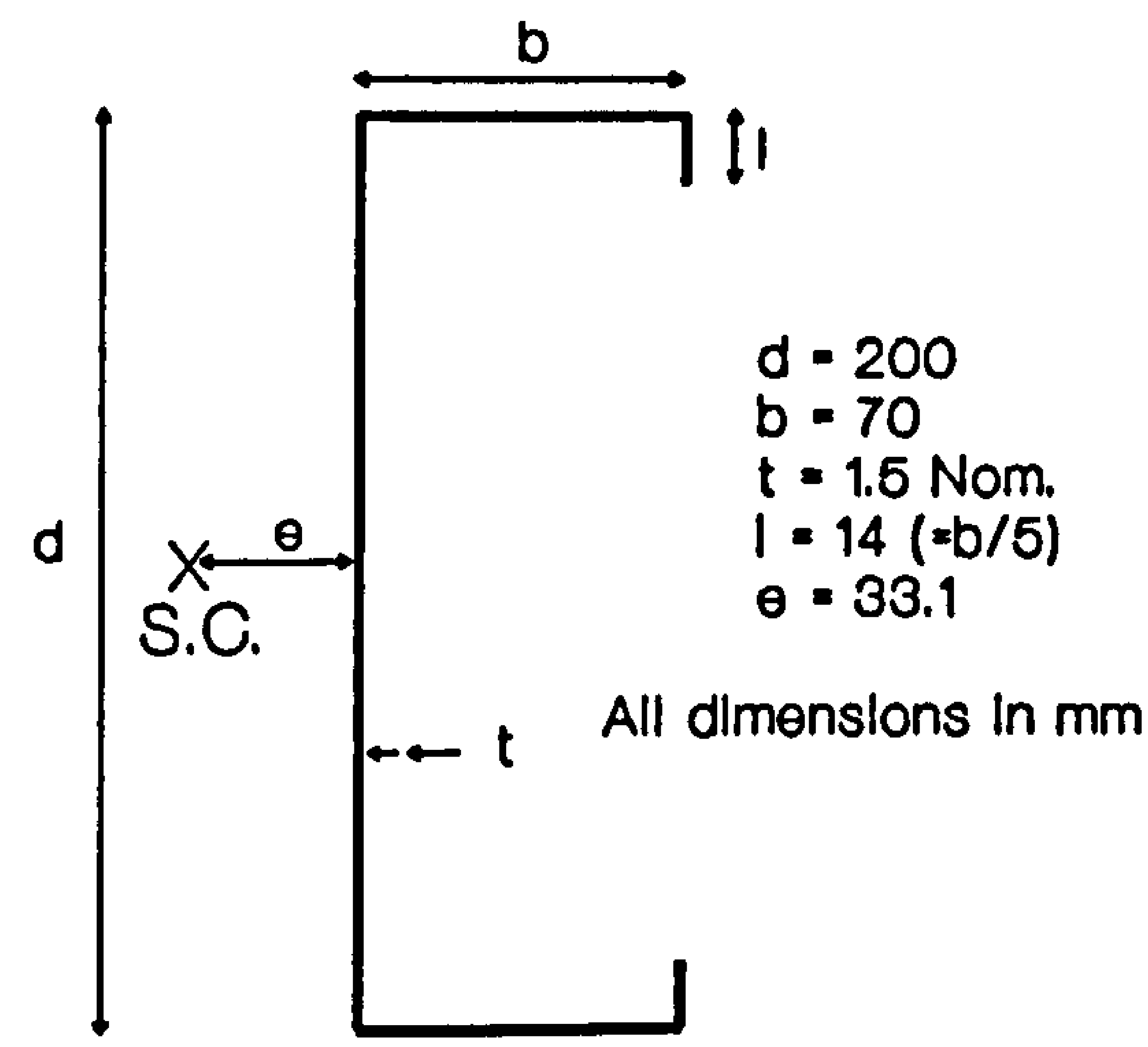


Fig. 8-1 : Profile of the section used for moment tests

The depth of the flange lips was designed in accordance with BS 5950 pt.5 (§ 4.6) to be equal to one fifth of the flange width. This was required to give the compression flange adequate bending rigidity to maintain straightness along its free edge under load.

The main advantage of the section used was that the moment-rotation characteristics could be directly compared with that of the predicted behaviour, since the applied forces could solely be resisted in sheet bearing by the connections. It was also a convenient section to use since it could be easily formed by press braking from flat sheets at the laboratory to the precise requirements.

However the main disadvantage of conventional channel profiles, in general, is that their shear centre lies outside the section. So that load applied through the web resolves itself into a load through the shear centre and an applied torsion. A few precautions were taken to keep the loading symmetrical and restrain the twisting of the section to as little as possible. These will be described later. The results would also be compared with similar tests on Zed sections with the shear centre in the plane of the web, so that the effect of torsion, if any, could be investigated.

The location of the shear centre of the section was calculated to be 33.1 mm outside the web, by considering the shear stress distribution within the section. Alternatively the position of the shear centre could be adequately estimated by the rule of thumb, e = b/2 = 35 mm.

8.4. Test Arrangements and procedures

The following parameters were adopted in every test, irrespective of the sections used or test arrangements.

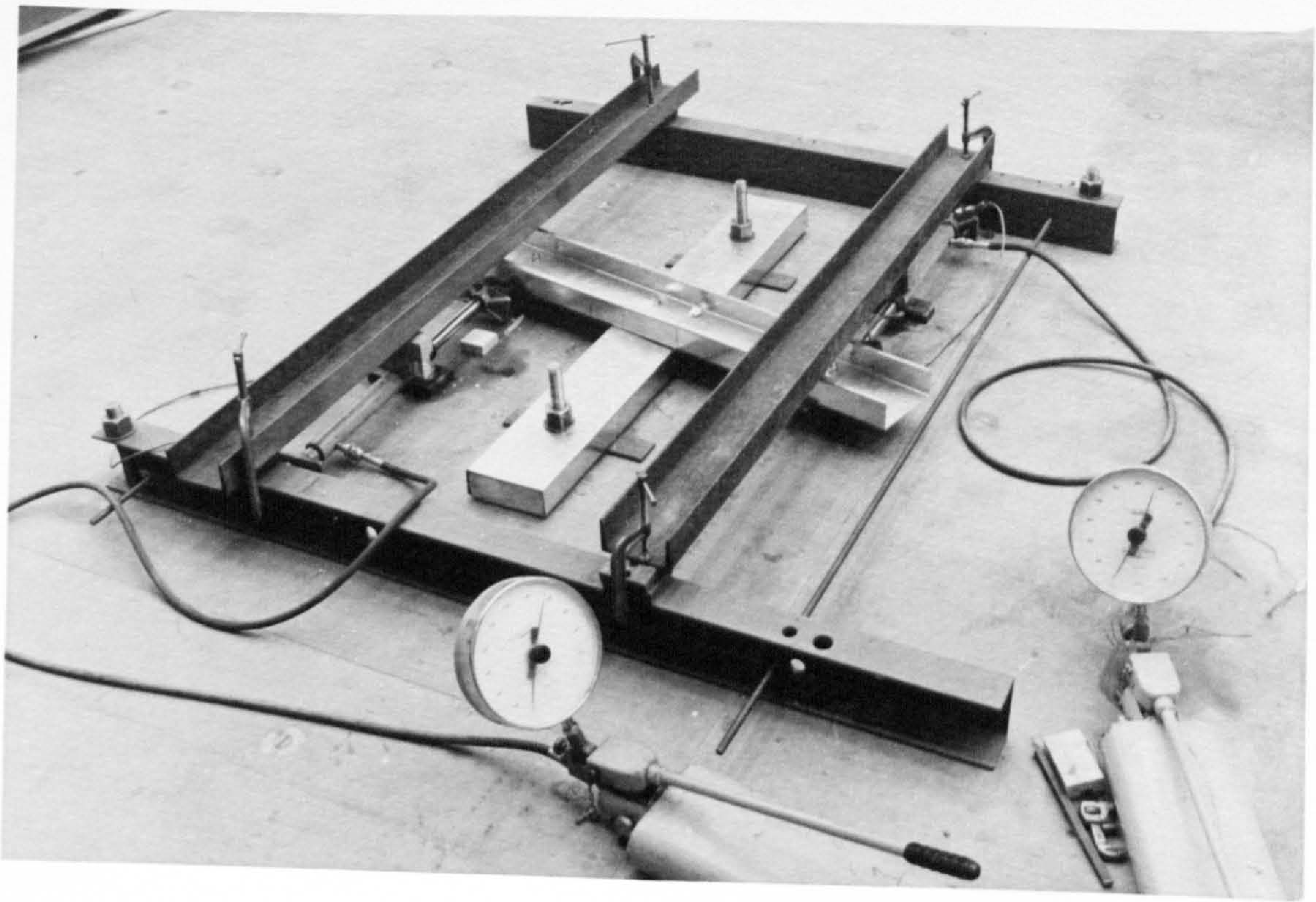
- 16mm diameter bolts were used throughout the tests.
- All bolts were of grade 4.6 galvanised mild steel, with grade 8.8 galvanised nuts.
- 18 mm diameter holes (2 mm clearance) were punched and reamered into the sections.
- All holes were aligned prior to the tightening of the bolts.
- All connection bolts were tightened to a bolt torque of 65 N.m . By connection bolts it is meant the bolts that were to resist the applied forces in sheet bearing. Other bolts, for instance those holding the loading cleats, etc. were fastened as tight as reasonably possible.
- Two galvanised normal diameter washers (Form E, BS 4320), one the under bolt head and one under the nut, were used with every bolt.
- All sections had a nominal yield stress of 280 N/mm^2, unless stated otherwise.

8.4.1. The floor, pure moment, test arrangement

Two channel sections were bolted back to back in form of a cross. Fig. 8-2 shows the test rig and a connection undergoing test.

The bottom channel was firmly bolted down to the laboratory strong floor by means of a large steel anchor bolt at each end. To prevent any local deformation of this section due to the tightening of the anchor bolts and ensure that it was firmly fixed in position - Two solid blocks of timber were made to the exact dimensions to slide into each end of the bottom channel before bolting it down to the floor.

Load was applied through two Epco hydraulic jacks (and measured with load cells) loaded via two hydraulic pumps. It would have been preferred to load the two jacks via one pump only, with the pressure distributed to the jacks through a junction box. However, a junction box was not available at the time of testing, so two separate pumps had to be used. The only disadvantage of this system was that loading one jack would to some extent relieve the load on the other. So the loading at each increment had to be carried out iteratively until both loads converged to the required value. Whereas using one pump and a junction box, the applied loads would have equalized automatically. However, apart from this mere inconvenience the system performed equally well.



A connection undergoing test in the floor arrangement

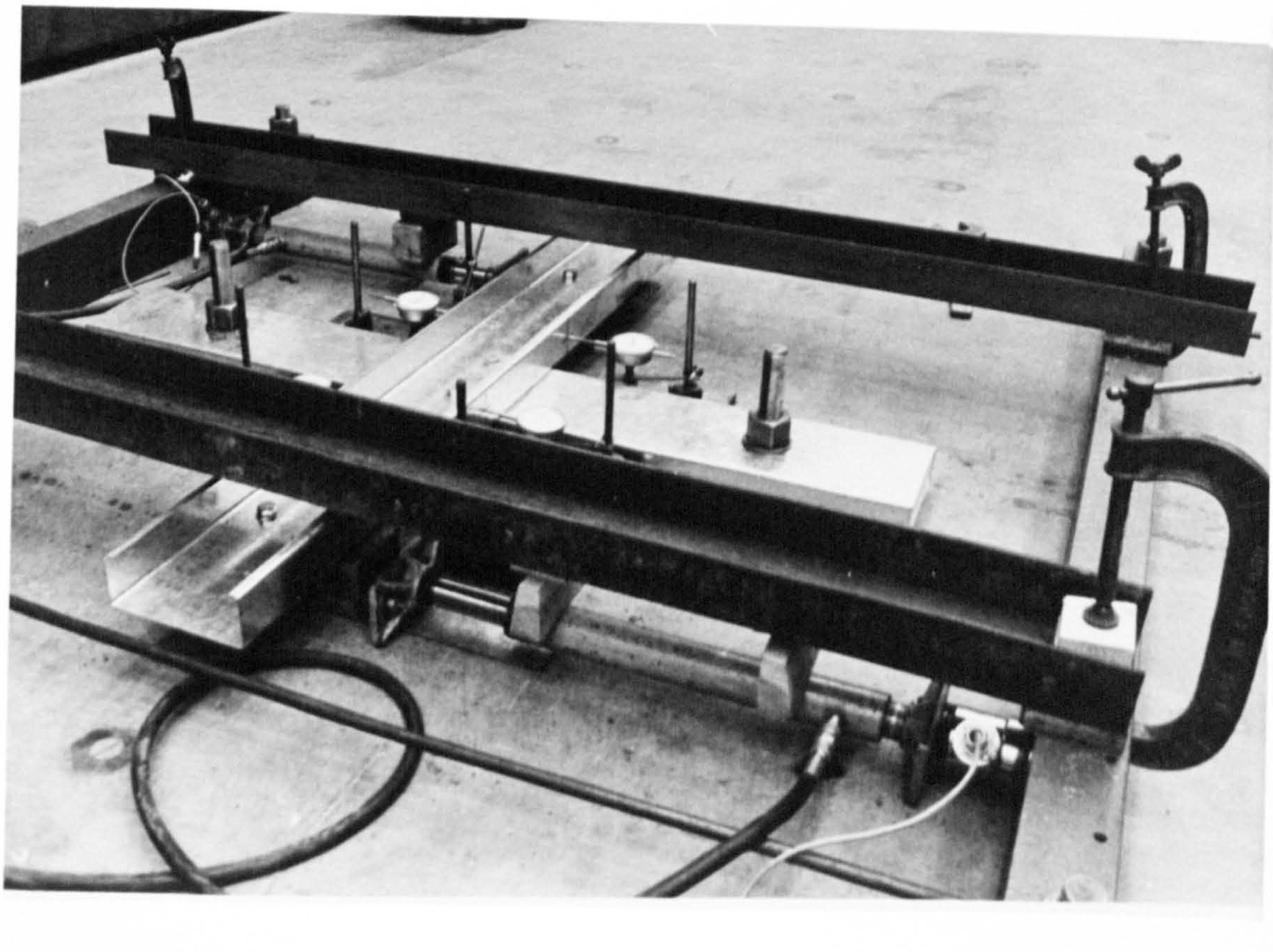


Fig. 8-2. Restraints positioned above the loading jacks to prevent them from buckling out of plane

Applied load was transferred to the plane of web of the top channel by angle cleats bolted to it. At first it was attempted to direct the jacks at the shear centre of the section, i.e 33 mm away from the face of the web. Thereby eliminating the element of torsion.

In retrospect it was realised that this could only have an adverse effect on the twisting of the section. Since the load in any case would still be transmitted through the plane of the web. Therefore by directing the jacks 33 mm away from it, all that happened would be to induce an extra moment into the section. Therefore to keep the twisting of the sections to a minimum the jacks were directed as close to the web as possible.

The loading system is outlined in Fig. 8-3.

A connection under test would therefore be subjected to a pure moment, applied by a couple with a lever arm of one metre. That is :

$$M = P \,.\, 1 = P \qquad\qquad [\text{kN.m}]$$

Six dial gauges were positioned, at a preset positions of 150 mm from the cross centre of the two sections to monitor the vertical element of connection deflection.

A load was then applied in increments and deflection readings were noted. The deflection on either side of the connection would then be the average of the two relevant dial gauges.

From Fig. 8-3, it is evident that dial gauges 5 and 6 were only used to check that the bottom channel was firmly held in position. It was found that the anchorage system performed very well and the total movements of the bottom channels throughout the tests were of an order of less than a fraction of a millimetre.

The arrangement described and depicted above was used to test four, two and three bolt connections.

With four and two bolt groups where the connections were symmetrical, dial gauges 1 to 4 gave almost identical readings. The vertical element of deflection was therefore simply taken as the average of the four readings.

The connection rotation would therefore be :

$$\phi = \frac{\delta_v}{150} \quad (\textit{Rads.}) \qquad\qquad (\text{Where } \delta_v = \text{Vertical component of deflection})$$

The equation above would obviously not apply to the case of three bolt connections where the centre of rotation did not coincide with the cross centres of the two channels. The results obtained for two and four bolt connections will be discussed in Chapter Nine, and that for three bolts connection in Chapter Ten.

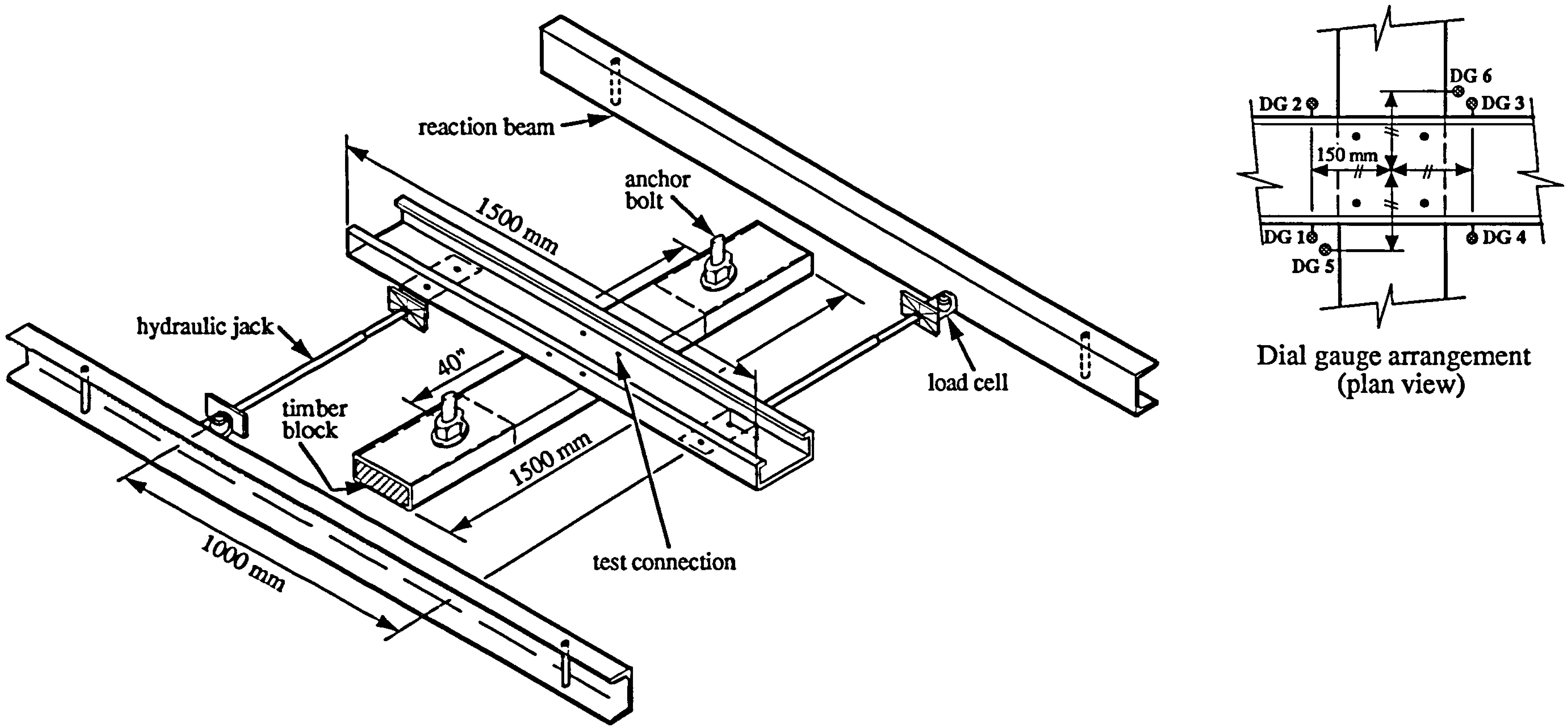


Fig. 8 - 3. Loading diagram for the floor set-up.

8.4.2. The beam, four point loading, test arrangement

Fig. 8-4 shows a complete set up prior to testing.

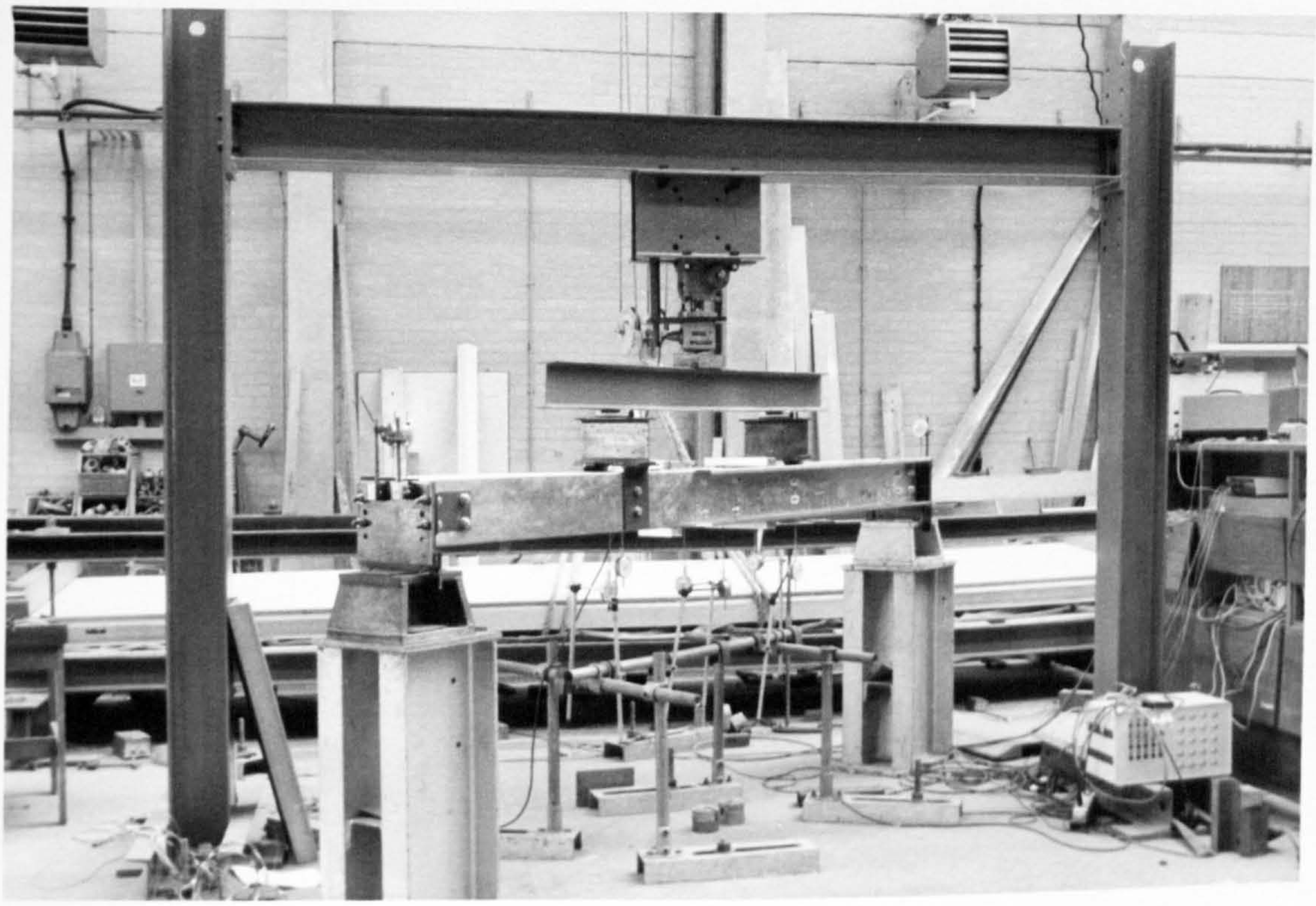


Fig. 8-4 : Four point loading test arrangement

A loading frame, made up of Universal Column sections, was specially made for these tests. A transverse beam with bolted end plates, was supported by two columns. The column flanges were drilled with a series of holes to allow the beam some degree of height flexibility. The column bases where designed to hold two anchor bolts, symmetrical about the column base, to avoid any eccentricity of the reacting forces.

Load was applied by means of a screw jack (and measured with a load cell) supported by the frame. Using a screw jack made it possible to measure the load even after the maximum value had been passed. The screw jack was mounted on a special rig designed to prevent it twisting and hence made sure that it could only apply a force in the vertical plane.

The sections were loaded through a system of steel spreader beams at the one third points of the span. The weight of the spreader beams has been ignored in subsequent analysis. That is a load of zero [kN] was that with the loading system already mounted on the beams.

The load was transferred to each section by a Tee cleat bolted to its web. The span was constant at 3 metres throughout the tests.

To keep the loading symmetrical and eliminate torsion; two connections, i.e four channels, were loaded side by side. Fig. 8-5 shows the outline of the loading diagram.

In retrospect it is arguable whether this symmetrical system of loading would eliminate torsion in the sections. Since the load would still be transferred through the web of each section and not its shear centre, this would in turn twist the section.

The loading system adopted was however essential to provide lateral stability to the beams. The lateral restraints provided would also serve as torsional restraints, see Fig. 8-5.

The sections were supported at each end in pairs, by knife edge steel end plates. Each section was bolted to the end plates by an angle cleat.

Obviously the advantage of the four point loading system was that the moment gradient in between the loading points was zero. Therefore as with the previous set up, the connection would be subjected to a constant moment.

Load was then applied in increments and deflection readings noted at the supports, loading points and mid spans of the two beams. Therefore ten dial gauge readings were taken for each load increment, five for each beam. Dial gauges were placed as close to the webs of the sections as possible, so that any out of plane twisting of the flanges would not affect the deflection readings. At mid span where the two sections overlapped, Tee extensions were put on the middle two dial gauges to record the overall deflection of both channels.

Loading was carried out in two stages. First the load was increased in increments and deflection readings noted until the mid span moment reached that of the calculated slip value - or until such time that the clearance slip was taken up. The load was then removed in stages to zero. The connection was therefore "Bedded-in". The loading was then repeated, noting the deflections at every increment, until failure occurred.

The repeat loading is considered to be the important one in practice, since dead loads, wind loads etc. will soon "shake down" the initial case to the repeat case.

Predicted moment-rotation characteristics will therefore be based on the repeat case.

The deflection readings thus taken would obviously include a component of beam flexure as well as that due to connection rotation.

In order to isolate the latter component; once the bolted sections had failed the load was removed and the beams were dismantled. Any local deformation round the bolt holes or any other local deformation, if any, was hammered back into shape.

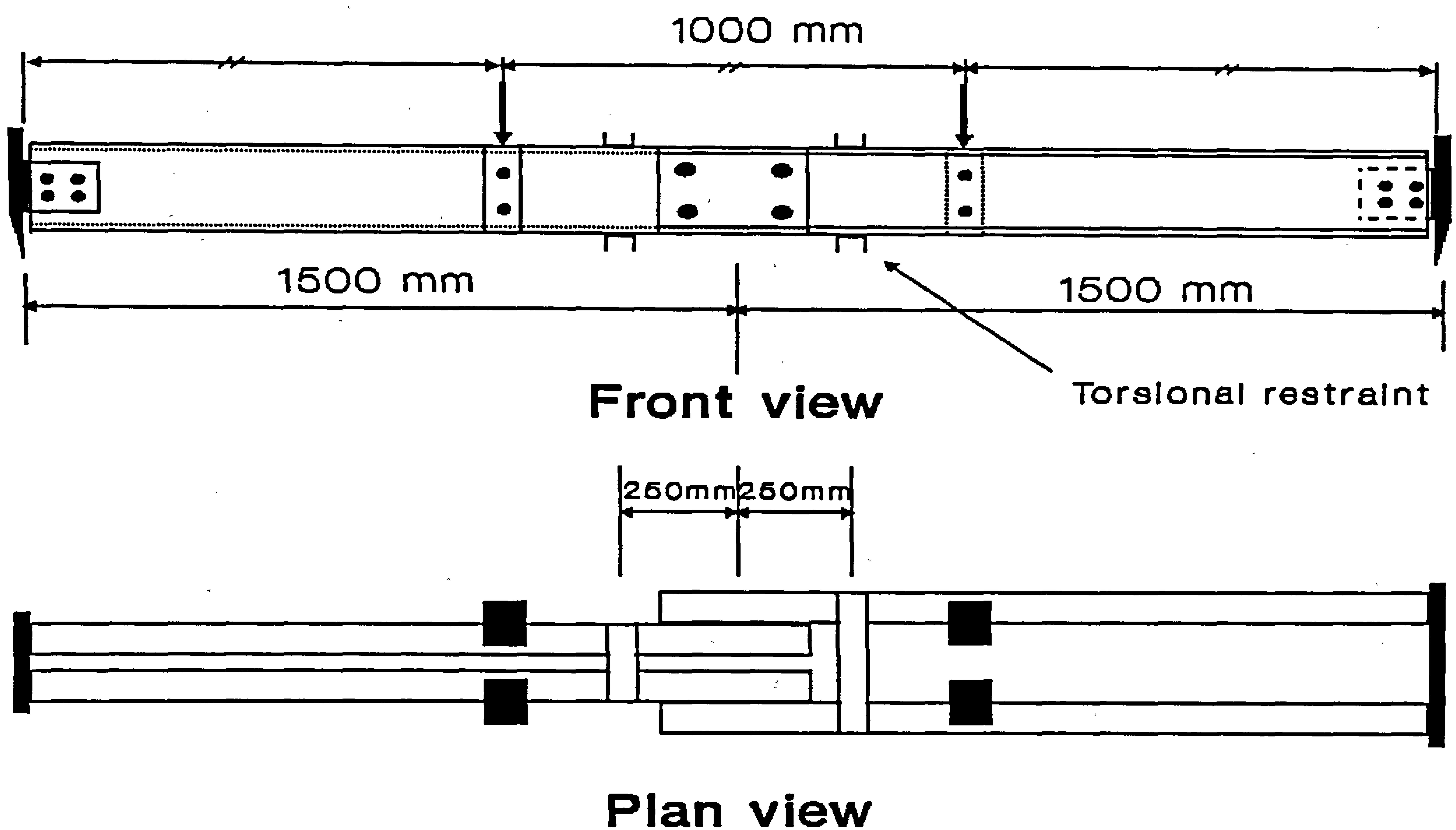


Fig. 8-5 : Loading diagram for the beam set up.

The beams were then realigned and the whole of the connection area, where the channels overlapped, was fillet welded. Test results indicate that this process in no way impaired the characteristics of the sections.

The whole arrangement was then re-assembled and the sections, which could now be regarded as uniform beams, were loaded in increments to failure. Deflection readings were noted at the same positions as previously.

The deflection due to connection rotation could then be differentiated as follows : (Fig. 8-6)

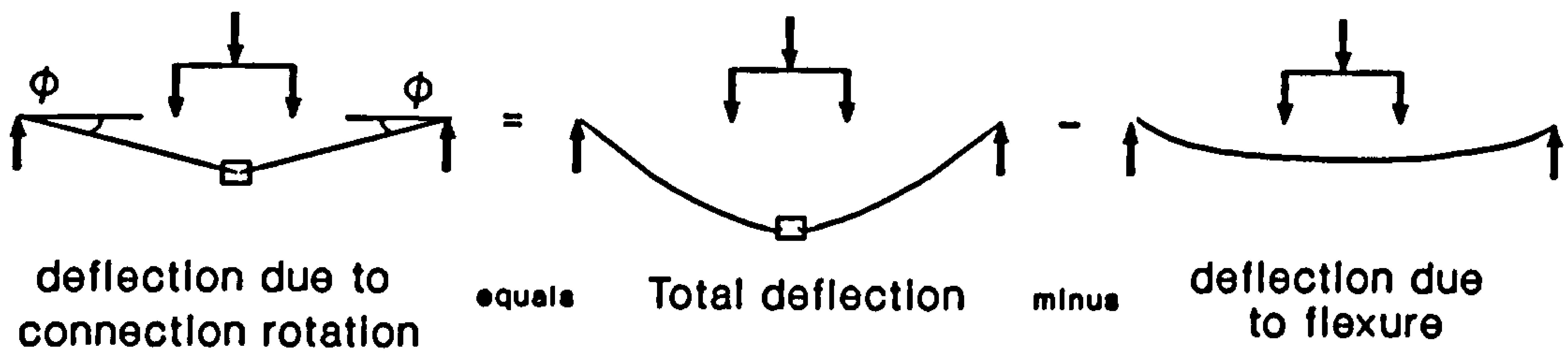


Fig. 8-6 : Deflection due to the connection rotation

The moment-rotation characteristics of the connection could thus be determined.

Deflection readings at the supports in all tests were found to be very small and hence insignificant. These have therefore been ignored in subsequent calculations.

The test readings obtained were satisfactory and the deflection readings of the two beams, i.e the two connections loaded side by side, agreed quite closely. The deflection readings at the loading points were also in good agreement, indicating that the applied load was distributed evenly at each point.

Load-deflection readings and moment-rotation characteristics are plotted for each test in the following chapters.

In load-deflection plots, the total applied load has been plotted against the average of the four dial gauge readings at loading points, and the average of the two gauges at mid spans.

In moment-rotation characteristics plots the applied moment per connection has been plotted against rotation.

The applied moment, at mid span and loading points, was therefore equal to :

$$M = (\frac{P}{4})(\frac{L}{3}) = \frac{PL}{12} \qquad [kN.m]$$

The connection rotation was calculated as follows :

Using mid span data :

$$\phi = \frac{\delta}{(\frac{L}{2})} \qquad [Rads.]$$

Using loading points data :

$$\phi = \frac{\delta}{(\frac{L}{3})} \qquad [Rads.]$$

Where δ is the average of the relevant deflection readings, calculated as it was shown in Fig. 8-6.

The actual moment-rotation characteristics obtained from mid span or loading points data were in a remarkably good agreement.

The actual and predicted moment-rotation characteristics during the bedding-in stage are also given as small insert plots within the characteristic plots of the main test to failure (i.e. the repeat test).

The tests carried out using the beam set up are summarized below :

Channel sections:

Four, two and three bolt connections

The results obtained for the two and four bolt connections will be discussed in Chapter Nine, and that for the three bolt connection in Chapter Ten.

In addition, the following tests were carried out on zed sections, with three bolt connections, in the beam set up :

Sections not nesting
Sections nesting

These will be described later.

With all three bolt connection tests, in the beam arrangement, a system of theodolites was set up to see whether it was possible to follow the rotation of the bolt group; and hence determine the location of the centre of rotation. This will be described later in Chapter Ten.

It should be noted that with both test arrangements, described in § 8.4.1 and 8.4.2, the system of taking data was such that the readings double checked themselves as far as possible. All the tests data have been analysed using in-house written computer programmes by the author.

Chapter Nine

Full moment connections test results, four and two bolt connections

9. Full moment connections test results, four and two bolt connections

Summary

Having defined the load-extension characteristics of bolted connections and the test procedures for moment connections, the results of four and two bolt connections in both beam and floor arrangements, are described and discussed in this chapter.

Dimensions common to different bolt configurations, i.e four, two and three bolts, were kept to equal values to be able to make a straight comparison between them.

For each bolt configuration the floor set up is discussed first followed by the results of the beam set up.

9.1. Four bolt connections

With reference to Fig. 9-1 (page 171), the ultimate moment of a four bolt connection is given as :

$$M = 4\,P_{bs}\,r$$

$$r = \sqrt{\left(\frac{a}{2}\right)^2 + \left(\frac{b}{2}\right)^2}$$

$$\therefore \quad M = 2\,P_{bs}\sqrt{a^2 + b^2}$$

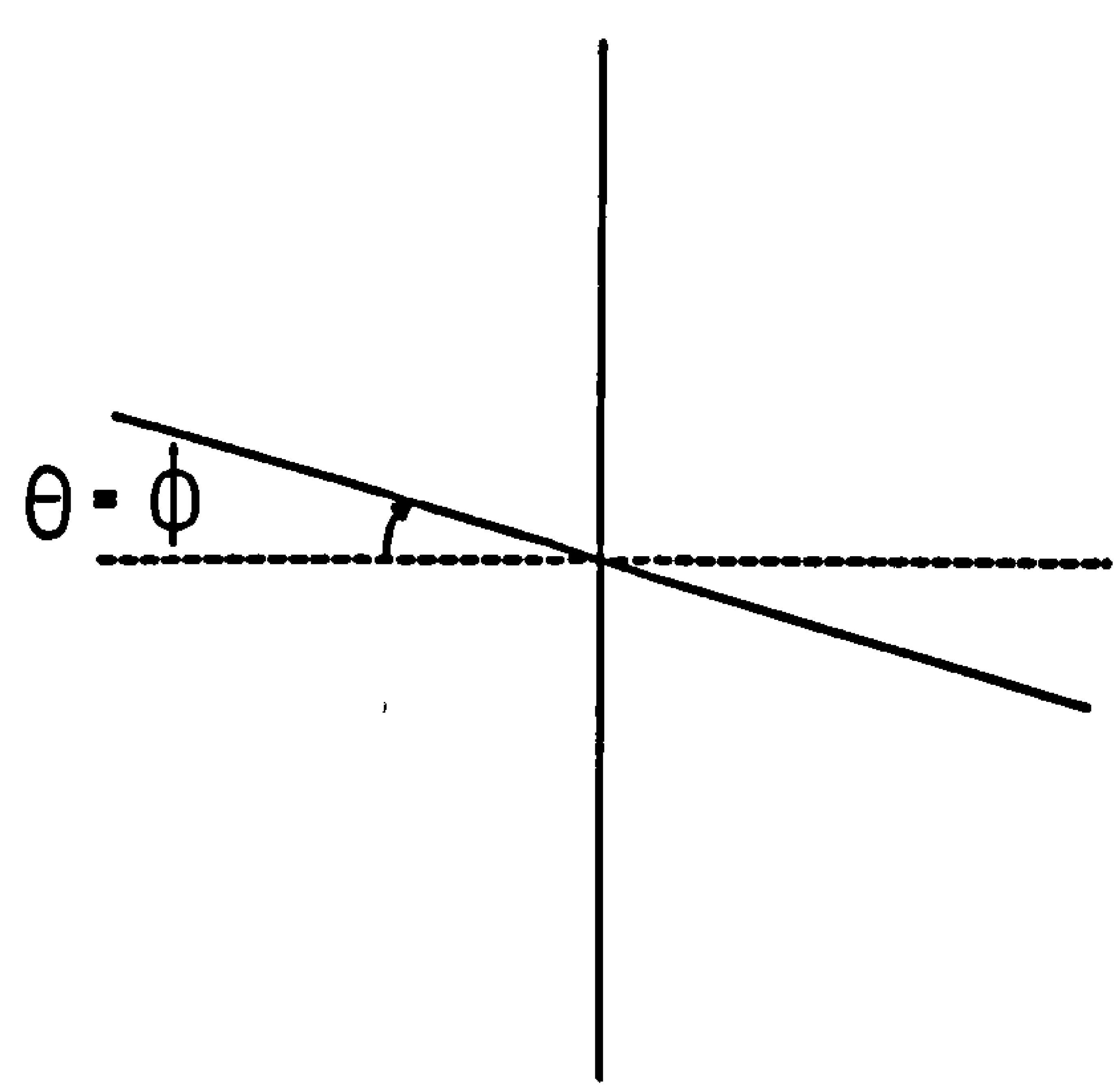
If the joint rotation is defined as the total movement of one member with respect to the other, θ (say) - then the connection rotation denoted as ϕ in Chapter Eight will represent by two different physical quantities, differing by a factor of 2, in the floor and beam set ups.

In the floor set up connection rotation at failure is : (see Fig. 9-1)

$$\phi = \frac{\delta}{r}$$

$$= \frac{P_{bs}\,.\,c}{\sqrt{\left(\frac{a}{2}\right)^2 + \left(\frac{b}{2}\right)^2}}$$

$$= \frac{2\,P_{bs}\,.\,c}{\sqrt{a^2 + b^2}}$$



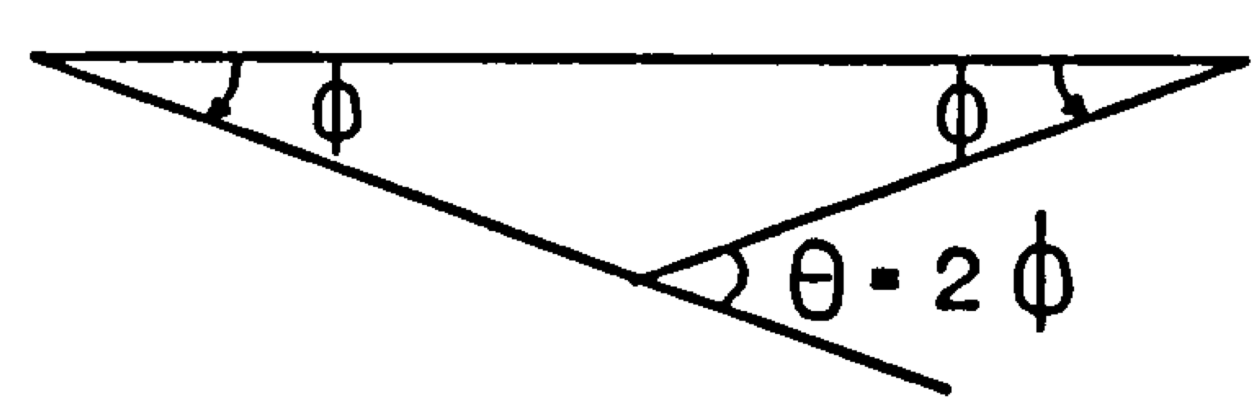
$$\therefore \quad \frac{M}{\phi} \left(= \frac{M}{\theta} \right) = \frac{a^2 + b^2}{c}$$

That is, M/ϕ is the same as M/θ (i.e. the joint rotation) in the floor set up.

With the beam set up however, M/ϕ connection rotation, only represents one half of the joint rotation, as depicted below:

$$\frac{M}{\phi} = \frac{M}{(\theta/2)} = 2\frac{M}{\theta} = 2\left(\frac{a^2+b^2}{2c}\right)$$

$$= \frac{a^2+b^2}{c}$$



Obviously in practice the latter relationship (i.e. beam set up) always applies.

In the following sections all the test and predicted characteristics are based on M/ϕ and its physical significance with respect to either test arrangement is as defined, for each set up, above.

9.1.1. The floor set up, four bolt connections

The connection details were as given in figure below :

Section properties were as follows :

t = 1.43[‡] mm

σ_y = 319.4 N/mm^2

σ_{ult} = 408.8 N/mm^2

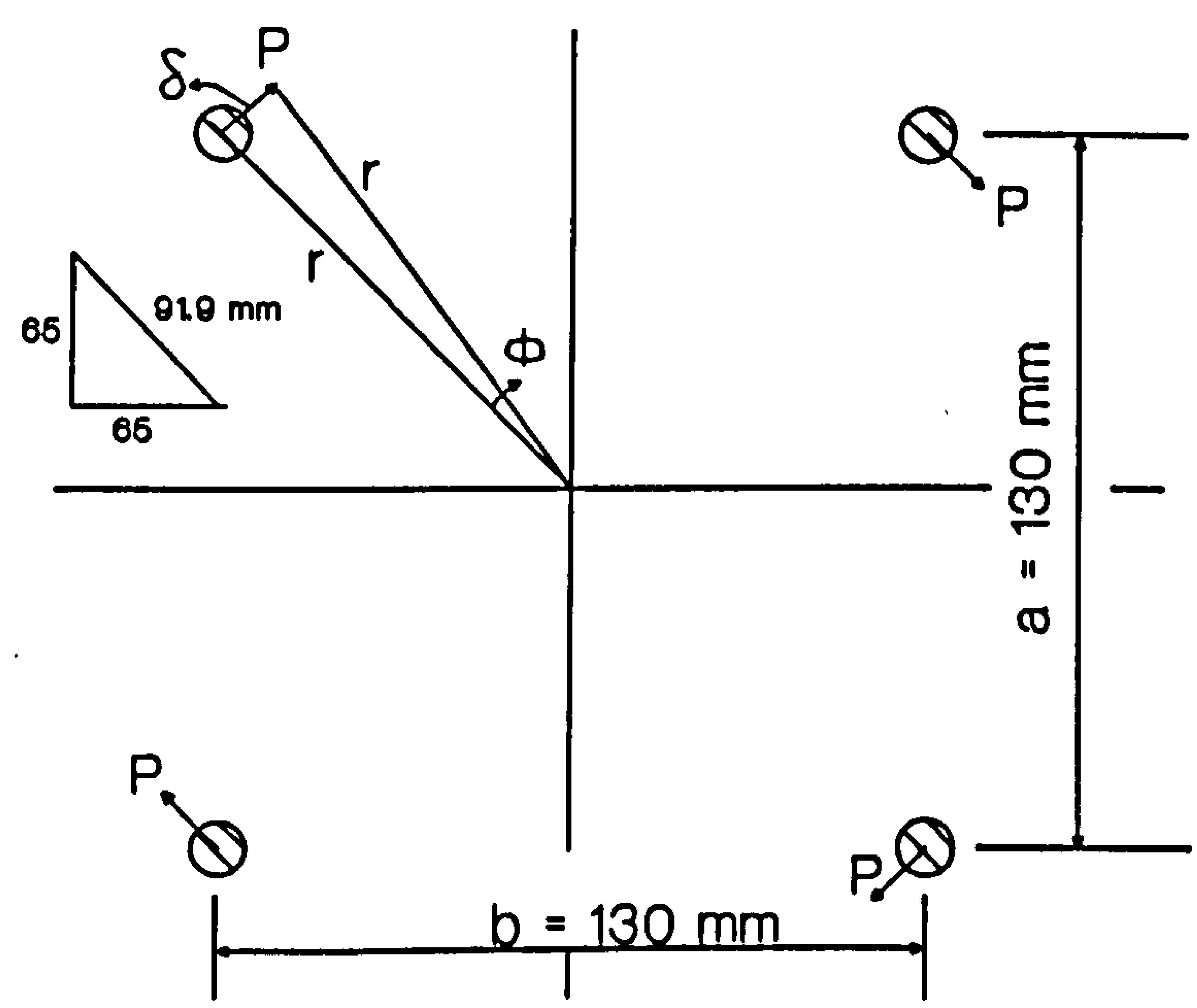


Fig. 9-1: Connection details, four bolt floor set up.

The predicted P-δ may be calculated as below; (see Fig. 7-20, chapter 7)

‡ All sheet thicknesses have been corrected for galvanising.

$P_{bs}' = \alpha \,.\, d.\, t.\, \sigma_{ult} \qquad = \alpha \times 16 \times 1.43 \times 0.4088 = \qquad 9.35\,\alpha$

$\alpha = k_1.k_2.k_3.k_4.k_5.k_6.k_7 = k_2 \qquad (k_1 \text{ and } k_3 \text{ to } k_7 \text{ all being equal to } 1)$
$= (1.9 + 0.2 \times 1.43) = 2.186$

$\therefore \quad P_{bs} = 20.4 \text{ kN}$

$$c = 5n\left(\frac{10}{t_1} + \frac{10}{t_2} - 2\right) \times 10^{-3} \qquad \left(\frac{mm}{kN}\right)$$

With reference to § 7.8 (page 154) the factor n for moment connections, with threads occurring in the shear plane, is equal to 3. i.e.;

$$c = 5 \times 3 \left(\frac{10}{1.43} \times 2 - 2\right) \times 10^{-3} = 0.180 \qquad \left(\frac{mm}{kN}\right)$$

Note that with the floor test arrangement the connection was tested directly to failure; i.e. it was not bedded-in. Hence referring back to Fig. 7-20 (page 155) the predicted load-extension characteristics is given as :

		(δ [mm], P [kN])
A	(0, 0)	(0.0, 0.0)
B	(4c, 4)	(0.72, 4)
C	[(4c+ hole clearance), 4]	(2.72, 4)
D	[(P_{bs} c + hole clearance), P_{bs}]	(5.67, 20.4)

The equivalent moment-rotation is given as :

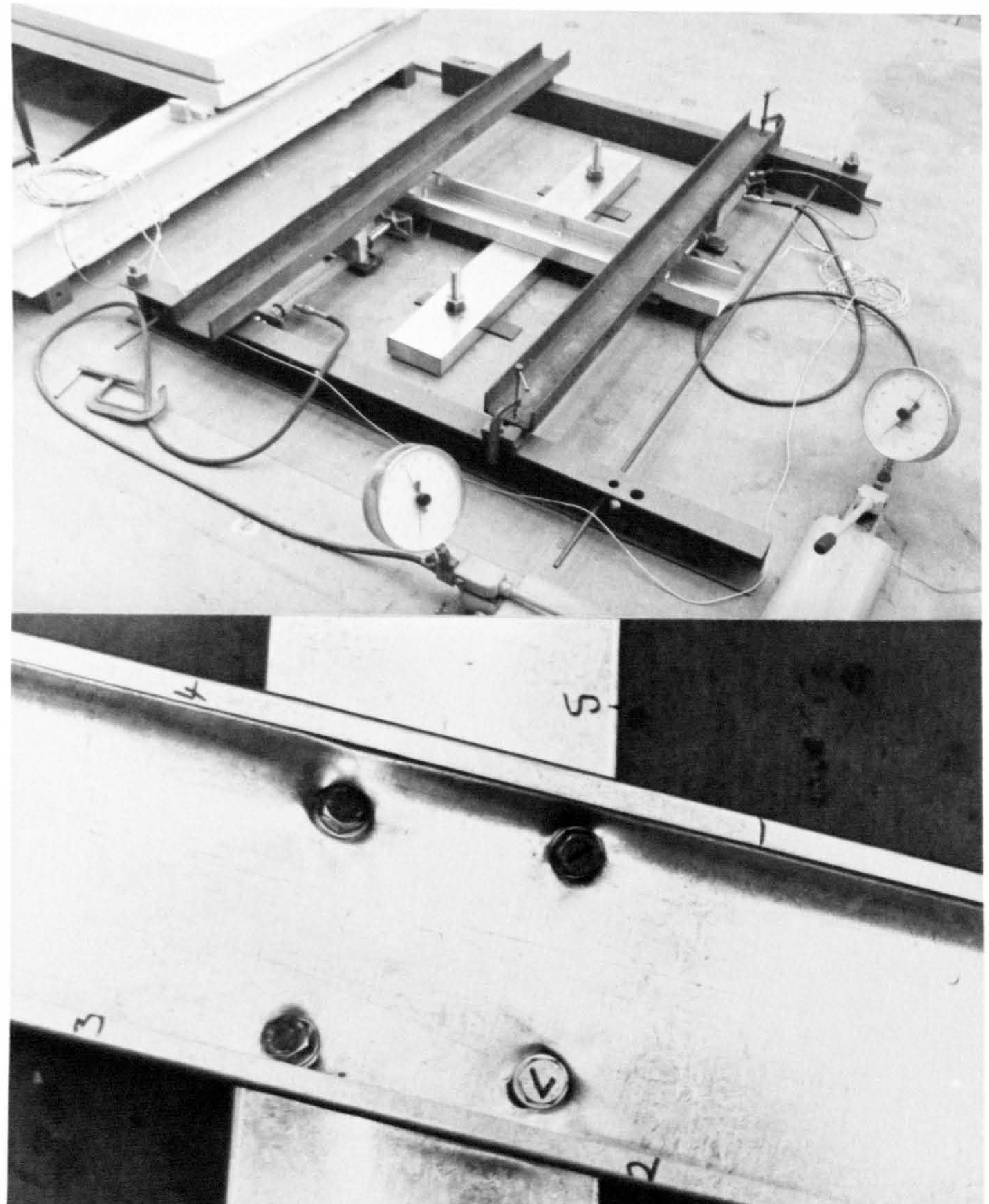
$$\phi = \frac{\delta}{r}, \qquad M = 4Pr \qquad\qquad where \;\; r = 91.9 \; mm$$

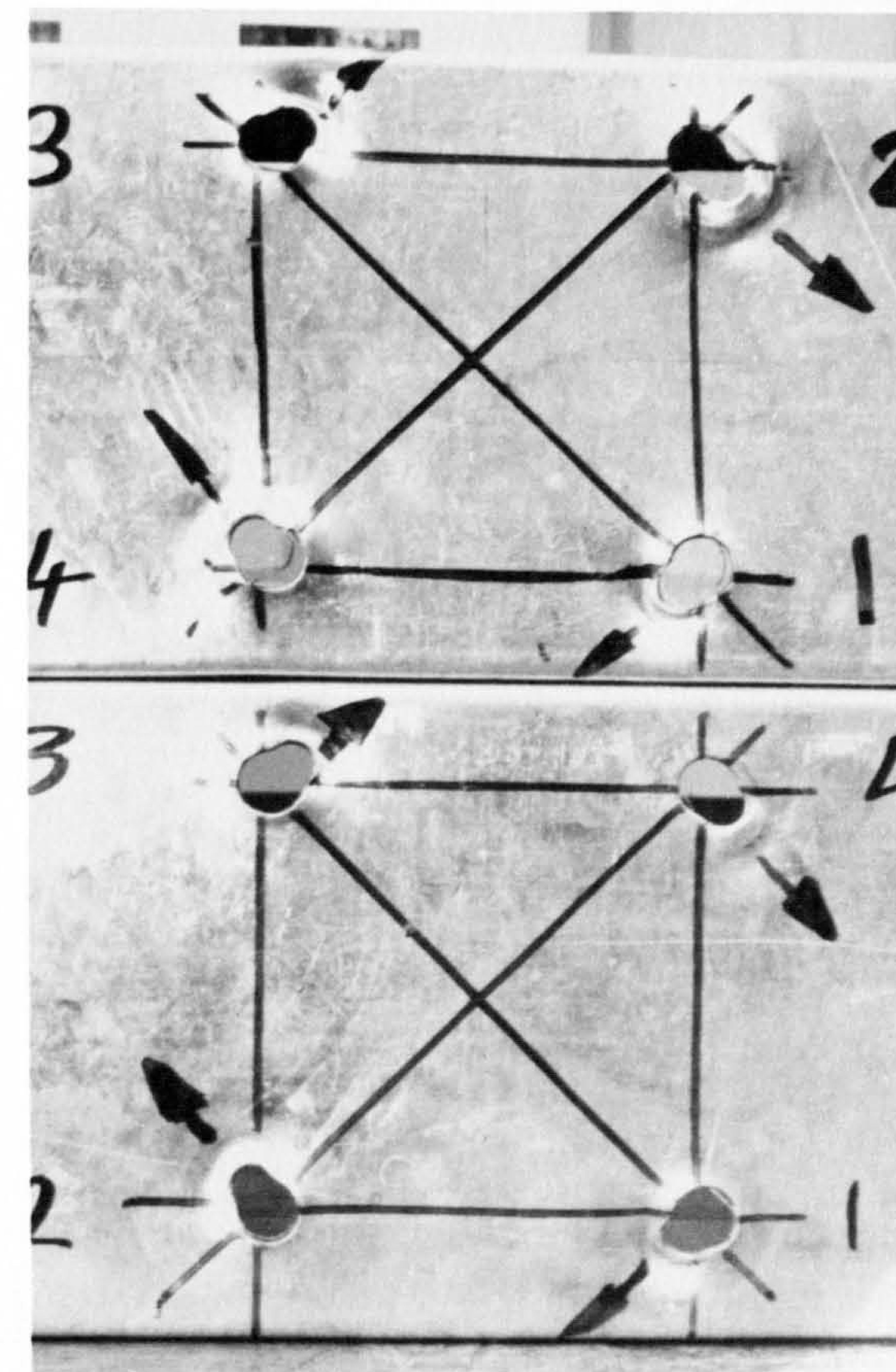
($\phi \times 10^{-3}$ [Rads.], M [kN.m])

A	(0.0, 0.0)
B	(7.83, 1.47)
C	(29.60, 1.47)
D	(61.70, 7.50)

The above coordinates essentially define the predicted moment-rotation characteristics of the connection. The actual connection failed at an ultimate moment of 7.63 kNm (cf. to 7.50 kNm predicted above).

Fig. 9-2 shows the connection before and after failure. Notice that bolt tilting is quite evident at failure. Also in this figure, the final distortion of the bolt holes clearly proves that the bolts were rotating about the centre of the bolt group, as assumed in Fig. 9-1.





Distortion of holes after failure

Fig. 9-2. Four bolt connection in the floor set up

Using the design equation for the bearing strength in Annex A, the predicted failure is given as :

$$M_{ult} = 4\ (2.5 \times 16 \times 1.43 \times 0.4088)\ 91.9 \times 10^{-3} = 8.6\ \text{kNm}$$

The test, predicted and that of the converted lap data moment-rotation characteristics are plotted and compared with each other in Fig. 9-3.

By converted lap data it is meant the (δ,P) relationship of the equivalent lap tests data has been transformed to a (ϕ,M) characteristic, using the same equations as those given above. That is, the equations defining ϕ and M were applied to each and every lap test increment, using a computer programme.

It is seen that the predicted moment-rotation characteristics is very satisfactory, while Annex A design equation overestimates the connection strength.

The slip load of the connection has also been predicted with good accuracy.

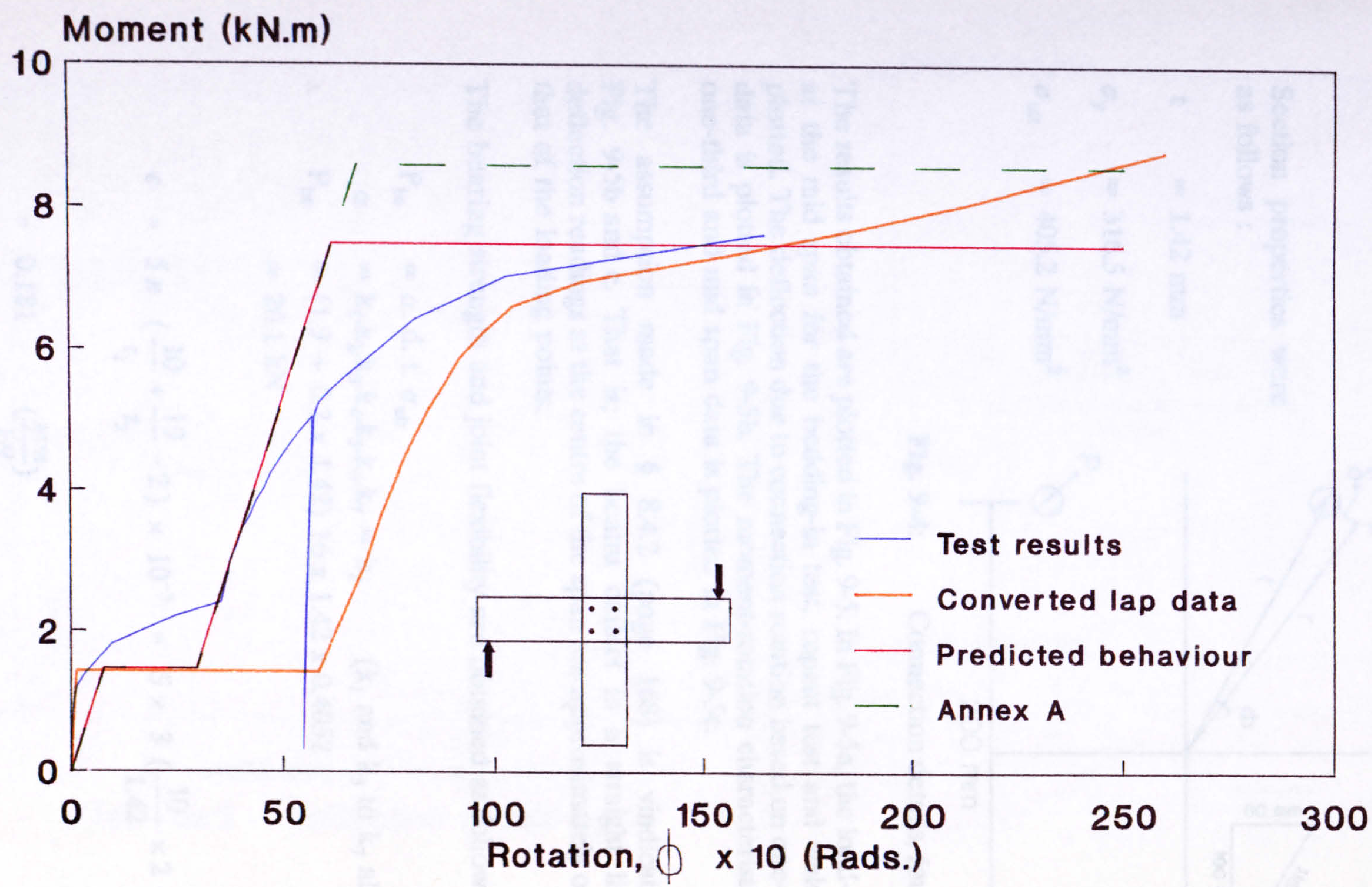


Fig. 9-3

Actual and predicted moment-rotation characereistics of four bolt floor set up

9.1.2. The beam set up, four bolt connections

The connections details were as given in figure below :

Section properties were as follows :

$t \quad = 1.42$ mm

$\sigma_y \quad = 316.5$ N/mm^2

$\sigma_{ult} \quad = 405.2$ N/mm^2

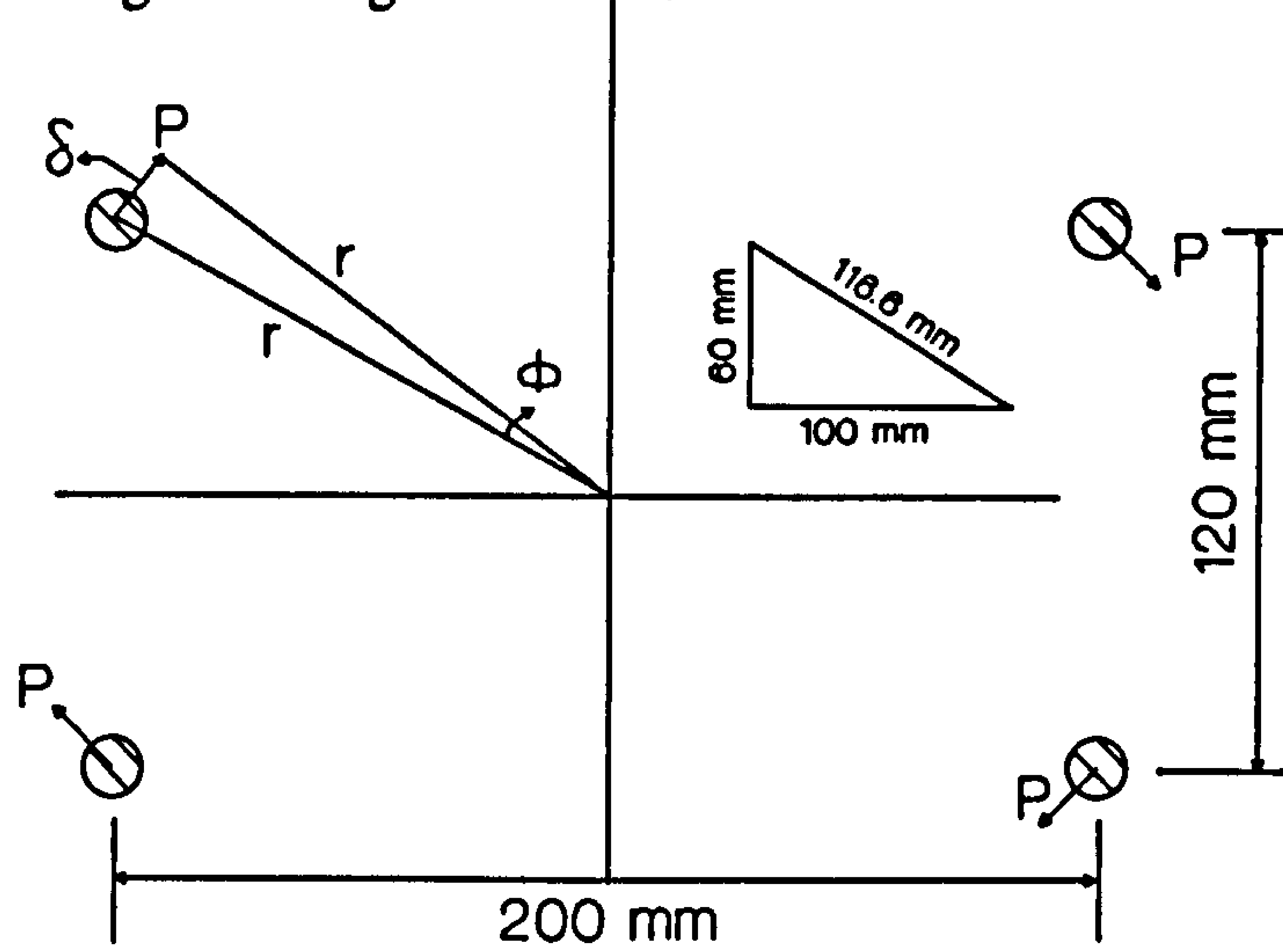


Fig. 9-4: Connection details, four bolt beam set up.

The results obtained are plotted in Fig. 9-5. In Fig. 9-5**a**, the load-deflection readings at the mid span for the bedding-in test, repeat test and welded beam test are plotted. The deflection due to connection rotation based on one-third and mid span data is plotted in Fig. 9-5**b**. The moment-rotation characteristics again based on one-third and mid span data is plotted in Fig. 9-5**c**.

The assumption made in § 8.4.2 (page 168) is vindicated by comparing Fig. 9-5**b** and **c**. That is; the beams deflect in a straight line. Therefore the deflection readings at the centre of the span are approximately one and a half times that of the loading points.

The bearing strength and joint flexibility are obtained as follows :

$$P_{bs} = \alpha .\ d.\ t.\ \sigma_{ult}$$

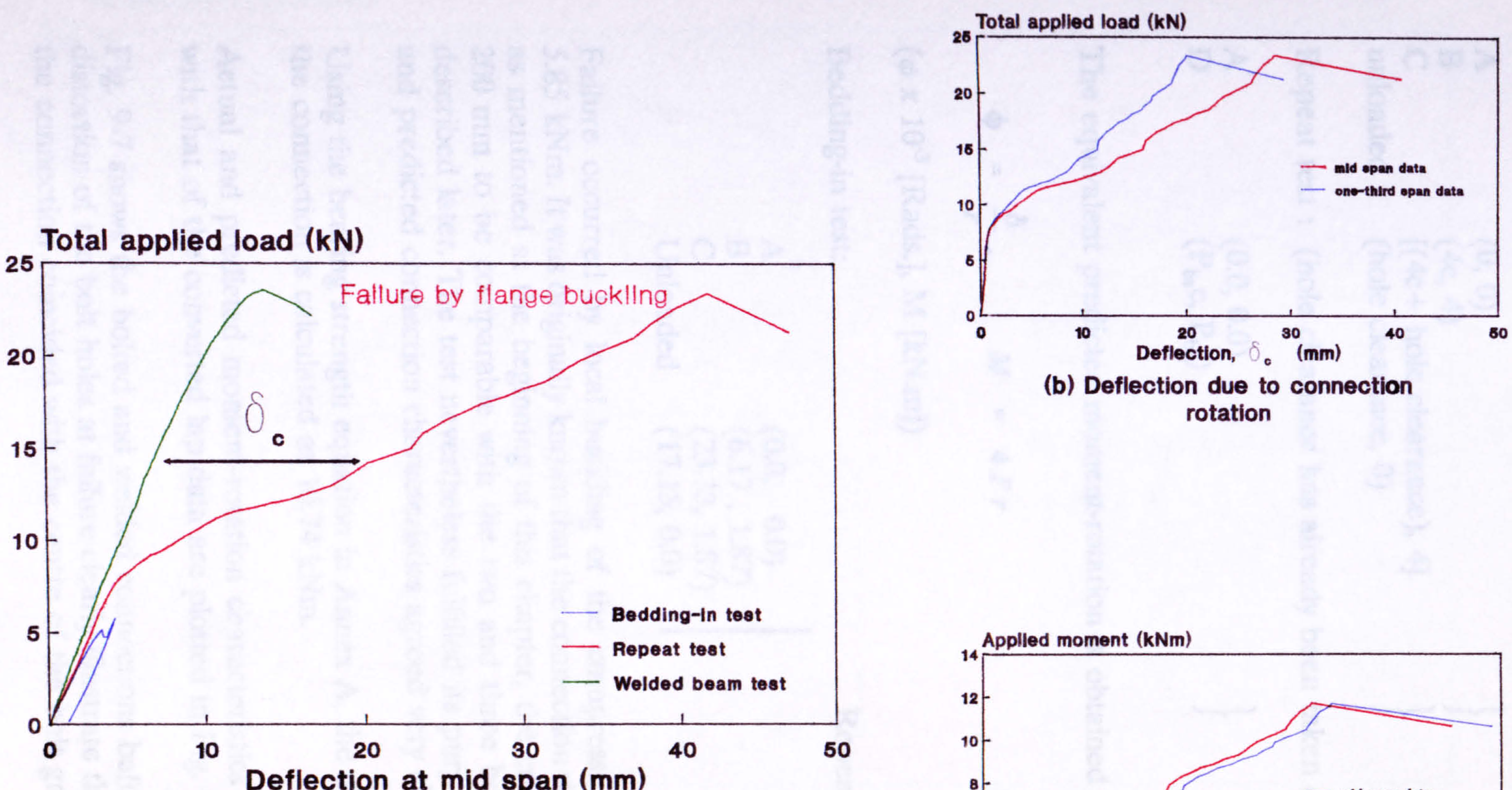
$$\alpha = k_1.k_2.k_3.k_4.k_5.k_6.k_7 = k_2 \qquad (k_1 \text{ and } k_3 \text{ to } k_7 \text{ all being equal to } 1)$$

$$\therefore \quad P_{bs} = (1.9 + 0.2 \times 1.42)\ 16 \times 1.42 \times 0.4052$$

$$= 20.1 \text{ kN}$$

$$c = 5n\ \left(\frac{10}{t_1} + \frac{10}{t_2} - 2\right) \times 10^{-3} = 5 \times 3\ \left(\frac{10}{1.42} \times 2 - 2\right) \times 10^{-3}$$

$$= 0.181 \qquad \left(\frac{mm}{kN}\right)$$



(a) Load v. deflection at mid span

(b) Deflection due to connection rotation

(c) Moment-rotation characteristics

Fig. 9-5
Test results, four bolt connections (beam set up)

With reference to Fig. 7-20, the predicted load-deflection behaviour is as follows :

Bedding-in :			(δ [mm], P [kN])
A	(0, 0)	}	(0.0, 0.0)
B	(4c, 4)		(0.72, 4)
C	[(4c+ hole clearance), 4]	}	(2.72, 4)
unloaded	(hole clearance, 0)		(2.0, 0)

Repeat test : (hole clearance has already been taken out)

A'	(0.0, 0.0)	}	(0.0, 0.0)
D	($P_{bs}c$, P_{bs})		(3.64, 20.1)

The equivalent predicted moment-rotation is obtained as :

$$\phi = \frac{\delta}{r}, \qquad M = 4Pr \qquad \textit{where } r = 116.6 \textit{ mm}$$

(ϕ x 10^{-3} [Rads.], M [kN.m])

Bedding-in test:		Repeat test :
A	(0.0, 0.0)	(0.0, 0.0)
B	(6.17 , 1.87)	(31.22, 9.37)
C	(23.33, 1.87)	
Unloaded	(17.15, 0.0)	

Failure occurred by local buckling of the compression flange, at a moment of 5.85 kNm. It was originally known that the connection was over-designed. However, as mentioned at the beginning of this chapter, the connection width was kept to 200 mm to be comparable with the two and three bolt connections that will be described later. The test nevertheless fulfilled its purpose. That is to say, the test and predicted connection characteristics agreed very closely.

Using the bearing strength equation in Annex A, the ultimate moment capacity of the connection is calculated as 10.74 kNm.

Actual and predicted moment-rotation characteristics (per connection), together with that of the converted lap data are plotted in Fig. 9-6.

Fig. 9-7 shows the bolted and welded connections before and after failure. Again distortion of the bolt holes at failure clearly illustrate that the centre of rotation of the connection coincided with the centre of the bolt group.

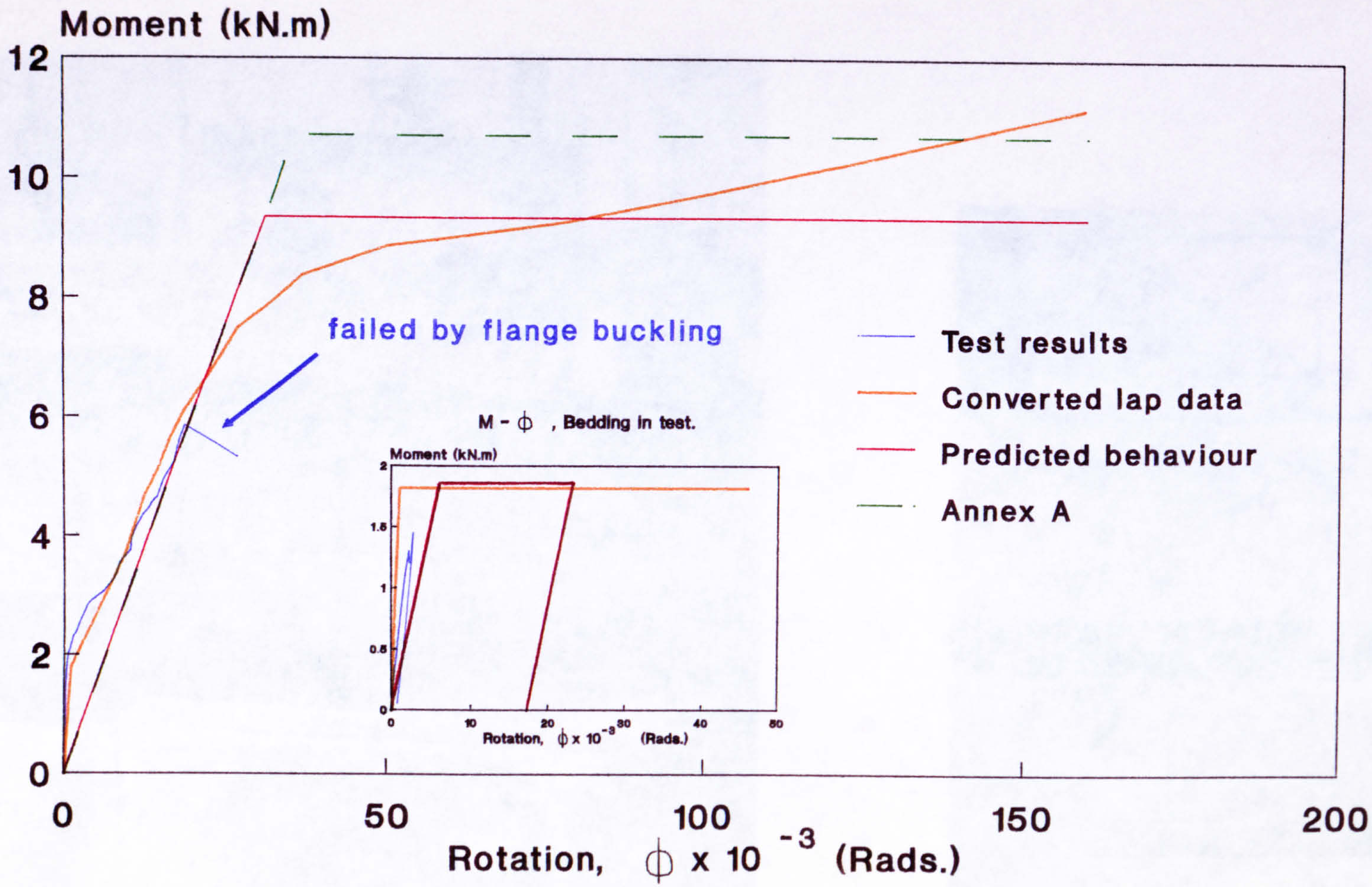
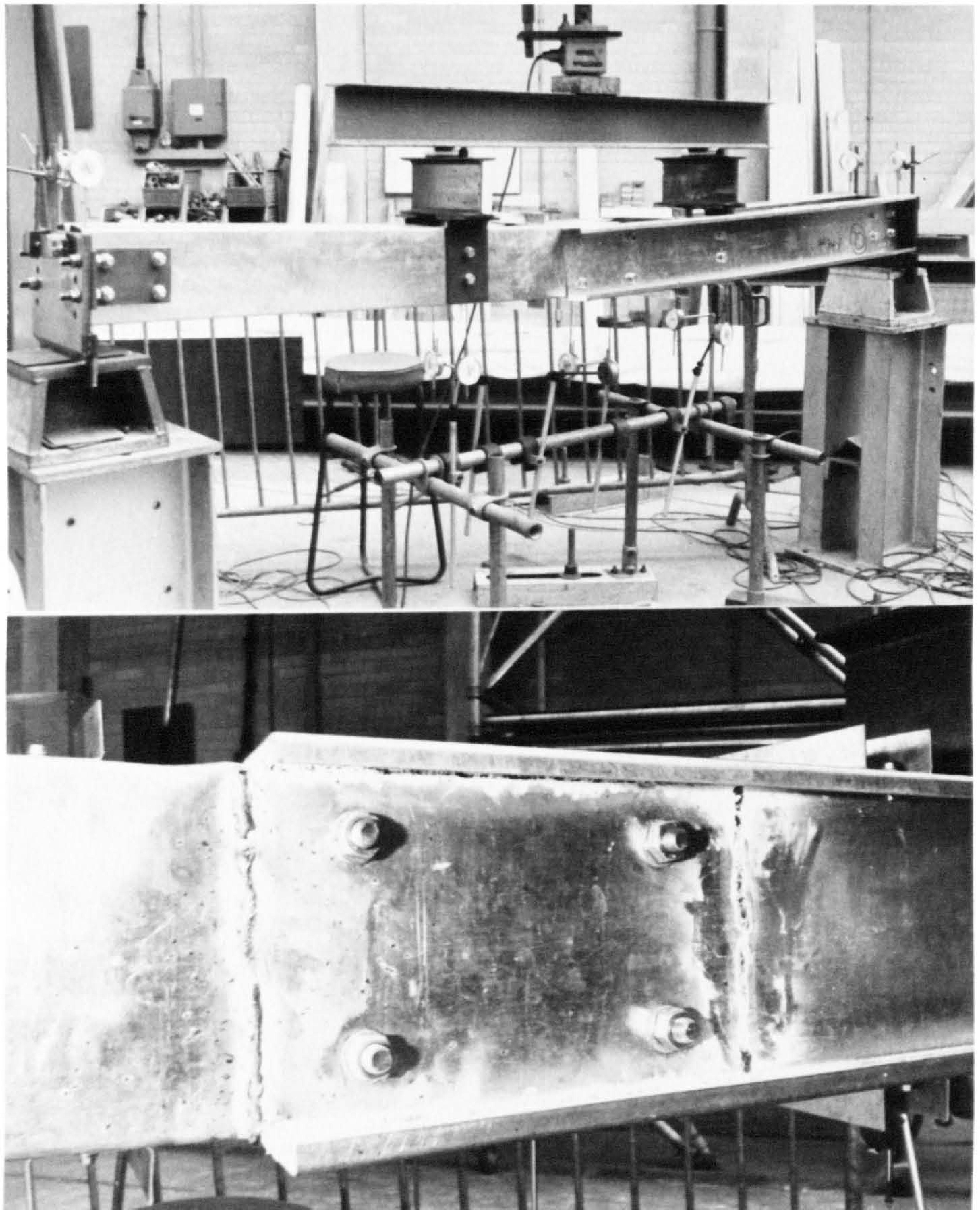
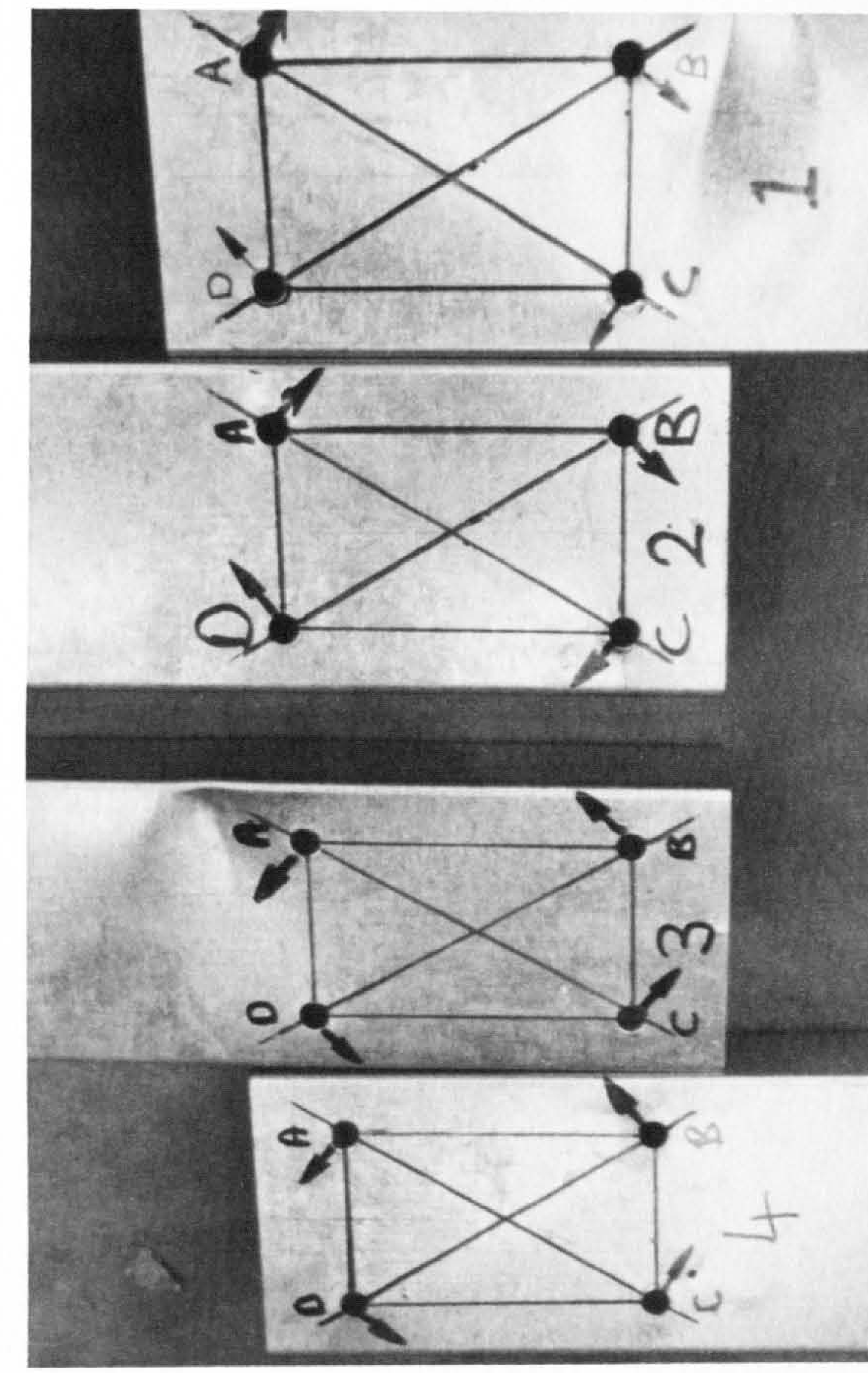


Fig. 9-6

Actual and predicted moment rotation characteristics of four bolt beam set up





Bolt hole distortion at failure

Fig. 9-7. Four bolt connections, bolted and welded beams after failure

9.2. Two bolt connections

With reference to Fig. 9-8 :

$$M = 2P_{bs}r$$
$$= P_{bs}b$$

$$\phi = \frac{\delta}{r} = \frac{P_{bs}c}{\frac{b}{2}} = \frac{2P_{bs}c}{b}$$

$$\therefore \quad \frac{M}{\phi} = \frac{b^2}{2c}$$

As mentioned previously, M/ϕ represents the total joint rotation in the floor set up, and only the connection rotation (i.e. half the joint rotation) in the beam set up.

9.2.1. The floor set up, two bolt connections

The connection details were as given in figure below :

Section properties were as follows :

$t = 1.42$ mm

$\sigma_y = 299.1$ N/mm^2

$\sigma_{ult} = 397.6$ N/mm^2

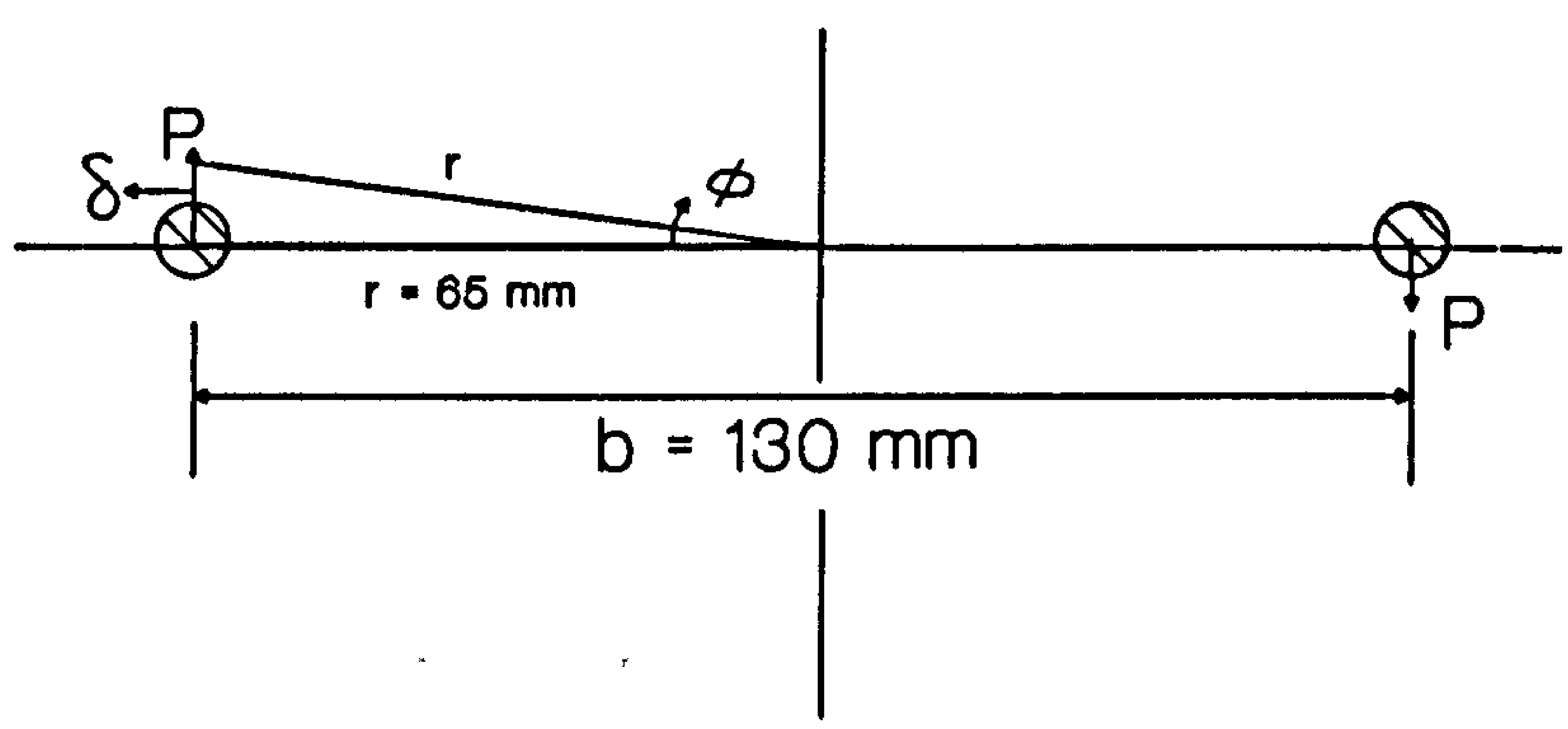


Fig. 9-8: Connection details, two bolt floor set up.

The bearing strength and joint flexibility are equal to :

$P_{bs} = \alpha \, . \, d. \, t. \, \sigma_{ult} = \alpha \times 16 \times 1.42 \times 0.3976 = 9.03\,\alpha$

$\alpha = k_1.k_2.k_3.k_4.k_5.k_6.k_7 = k_2$ (k_1 and k_3 to k_7 all being equal to 1)
$= (1.9 + 0.2 \times 1.42) = 2.184$

$\therefore \quad P_{bs} = 19.7$ kN

$$c = 5n\left(\frac{10}{t_1} + \frac{10}{t_2} - 2\right) \times 10^{-3} = 5 \times 3\left(\frac{10}{1.42} \times 2 - 2\right) \times 10^{-3}$$

$$= 0.181 \qquad \left(\frac{mm}{kN}\right)$$

The ultimate moment is calculated as :

$$M = P_{bs} \, . \, b = 19.7 \times 0.130 = 2.56 \text{ kNm}$$

Taking account of hole clearance in calculating the joint rotation at failure, (since the connection was tested directly to failure) :

$$\phi = \frac{\delta}{r}$$

$$= \frac{P_{bs} c + hole\ clearance}{\frac{b}{2}}$$

$$= \frac{2\,(P_{bs} c + 2)}{b}$$

$$= \frac{2\,(19.7 \times 0.181 + 2)}{130} = 85.63 \times 10^{-3} \quad \textit{Rads.}$$

The predicted slip moment can be calculated similarly, as was done for the four bolt connections.

A comparison of the actual and predicted moment-rotation characteristics, together with that of the converted lap data, is carried out in Fig. 9-9.

Due to test limitations, one of the pumps was starting to leak, the failure plateau of the connection could not be obtained. But the connection had failed at an ultimate load of 2.83 kNm, to all intents and purposes.

The predicted and actual strength and stiffness of the connection are in very close agreement. Using the design bearing equation in Annex A, the predicted ultimate moment would be equal to 2.94 kNm.

Fig. 9-10 shows the connection before and after failure. Note that bolt tilting is quite evident at failure.

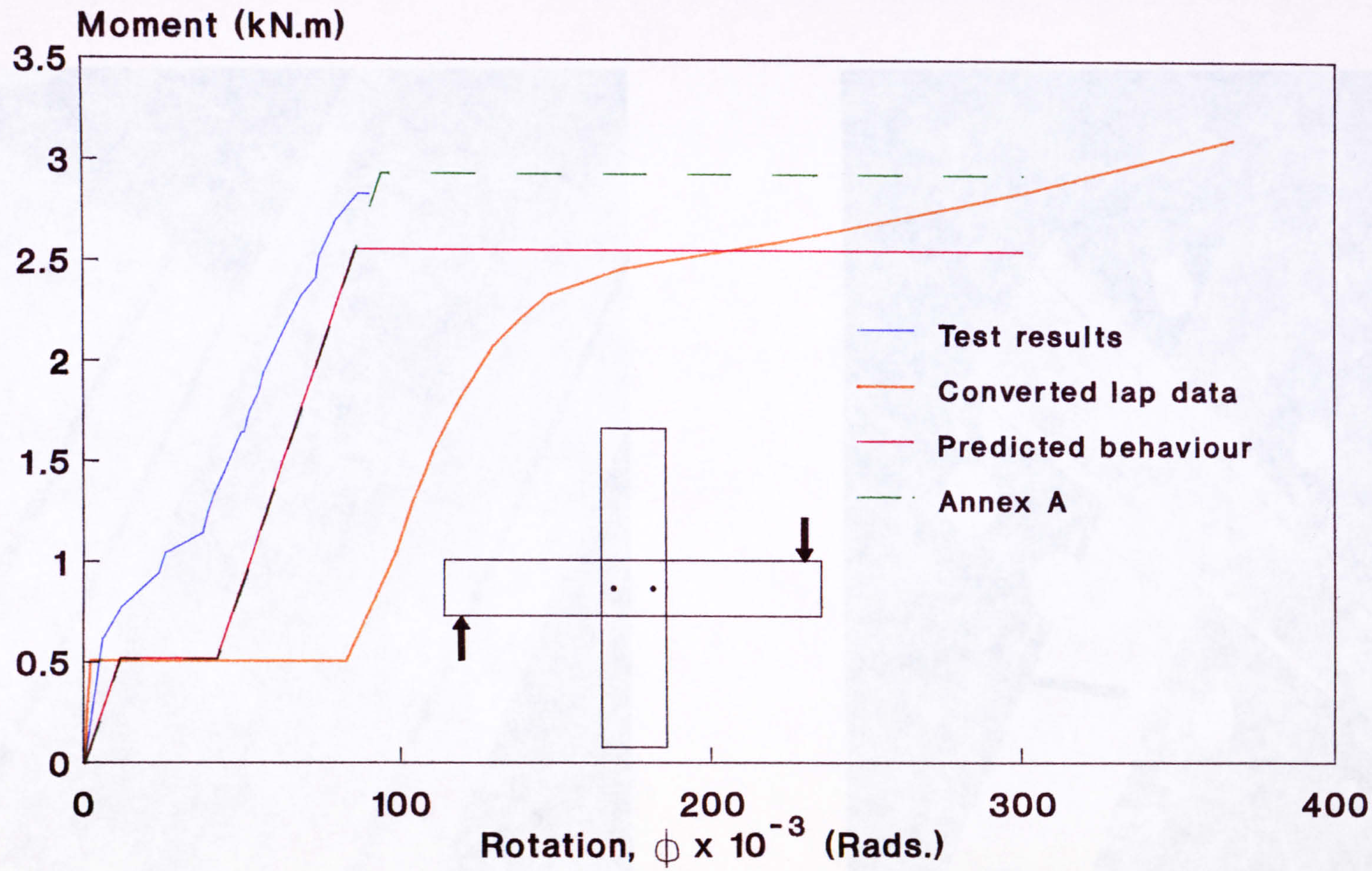
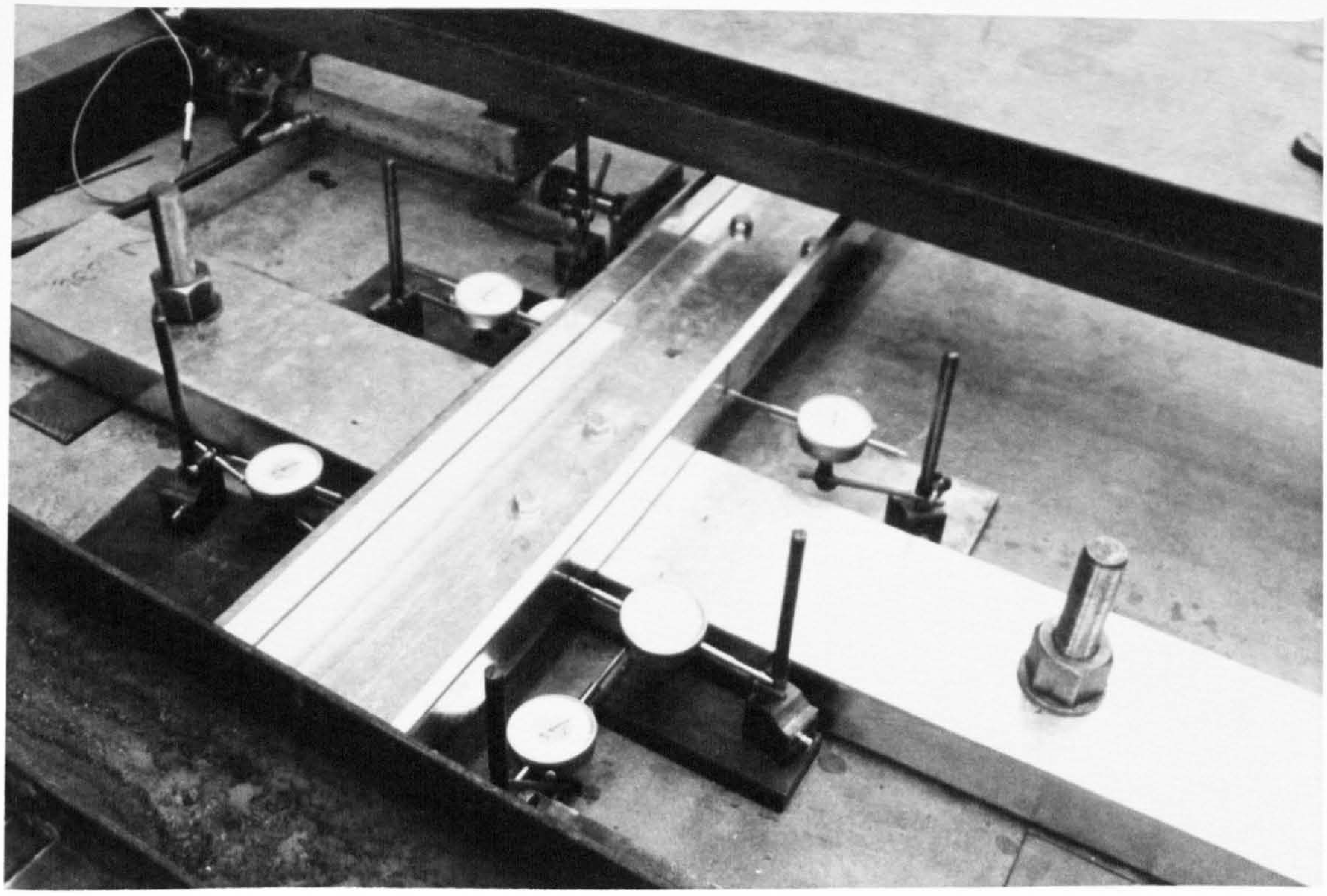


Fig. 9-9
Actual and predicted moment rotation characteristics of two bolt floor set up



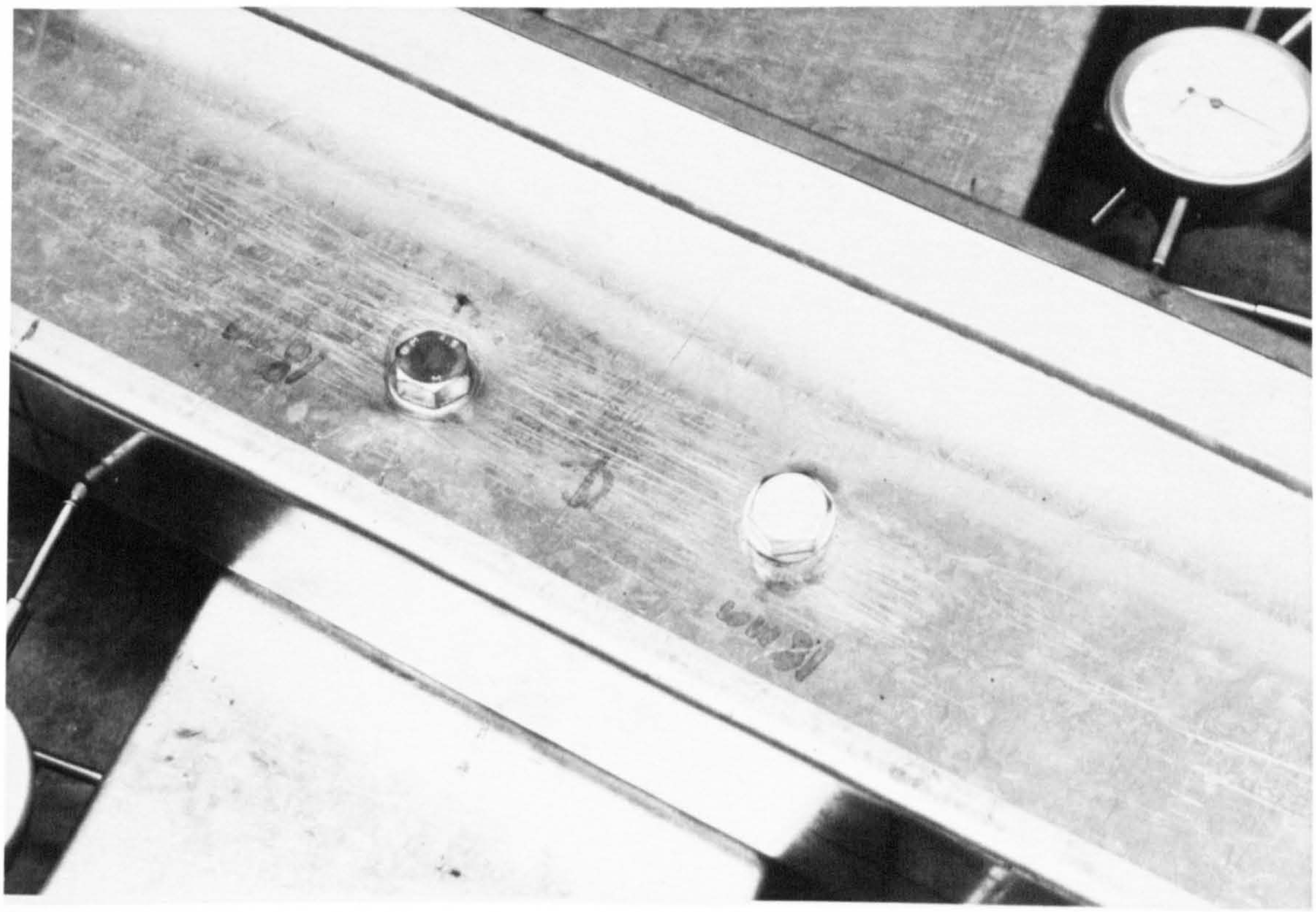


Fig. 9-10. Two bolt connection in the floor set up, before and after failure

9.2.2. The beam set up, two bolt connections

The connections details are given in figure below :

Section properties were as follows :

$t = 1.42$ mm

$\sigma_y = 309.0$ N/mm^2

$\sigma_{ult} = 399.0$ N/mm^2

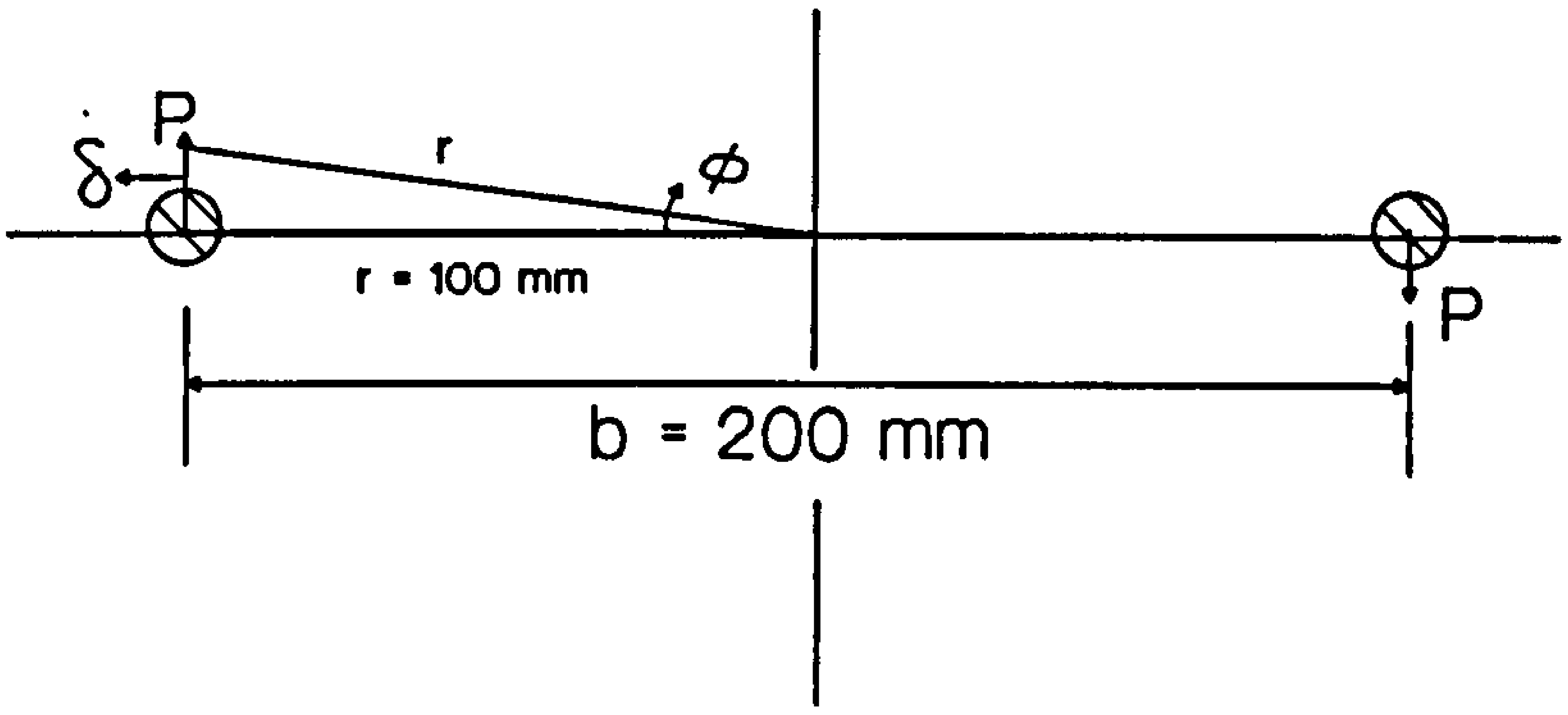


Fig. 9-11: Connection details, two bolt beam set up.

The results obtained are plotted in Fig. 9-12. In Fig. 9-12**a**, the load-deflection readings at the mid span for the bedding-in test, repeat test and welded beam test are plotted. The deflection due to connection rotation based on centre and one-third span data is plotted in Fig. 9-12**b**. The moment-rotation characteristics again based on one-third and mid span data are plotted in Fig. 9-12**c**.

As with the case of four bolt channel beams the assumption made in calculating the connection rotation, as to the beam deflecting in a straight line, is justified by comparing Fig. 9-12**b** and **c**.

The predicted ultimate moment of the connection is calculated as follows :

$$M = P_{bs} \cdot b$$

where

$$\begin{aligned} P_{bs} &= \alpha.\ d.\ t.\ \sigma_{ult} \\ &= (k_1.k_2.k_3.k_4.k_5.k_6.k_7)\ d.\ t.\ \sigma_{ult} \\ &= k_2\ .d.\ t.\ \sigma_{ult} \qquad (k_1 \text{ and } k_3 \text{ to } k_7 \text{ all being equal to } 1) \\ &= (1.9 + 0.2 \times 1.42)\ 16 \times 1.42 \times 0.399 \\ &= 19.8 \quad \text{kN} \end{aligned}$$

$$\therefore M_{ult} = 19.8 \times 0.2 = 3.96 \text{ kNm}$$

Joint flexibility c, is equal to 0.181 mm/kN, just as with the two bolt floor set up. (see § 9.2.1, page 180)

In calculating the connection rotation at failure, it should be noted that the hole clearance has been eliminated in the bedding in test. As explained in the previous chapter, this is also thought to be the case in practice. That is, hole clearance will

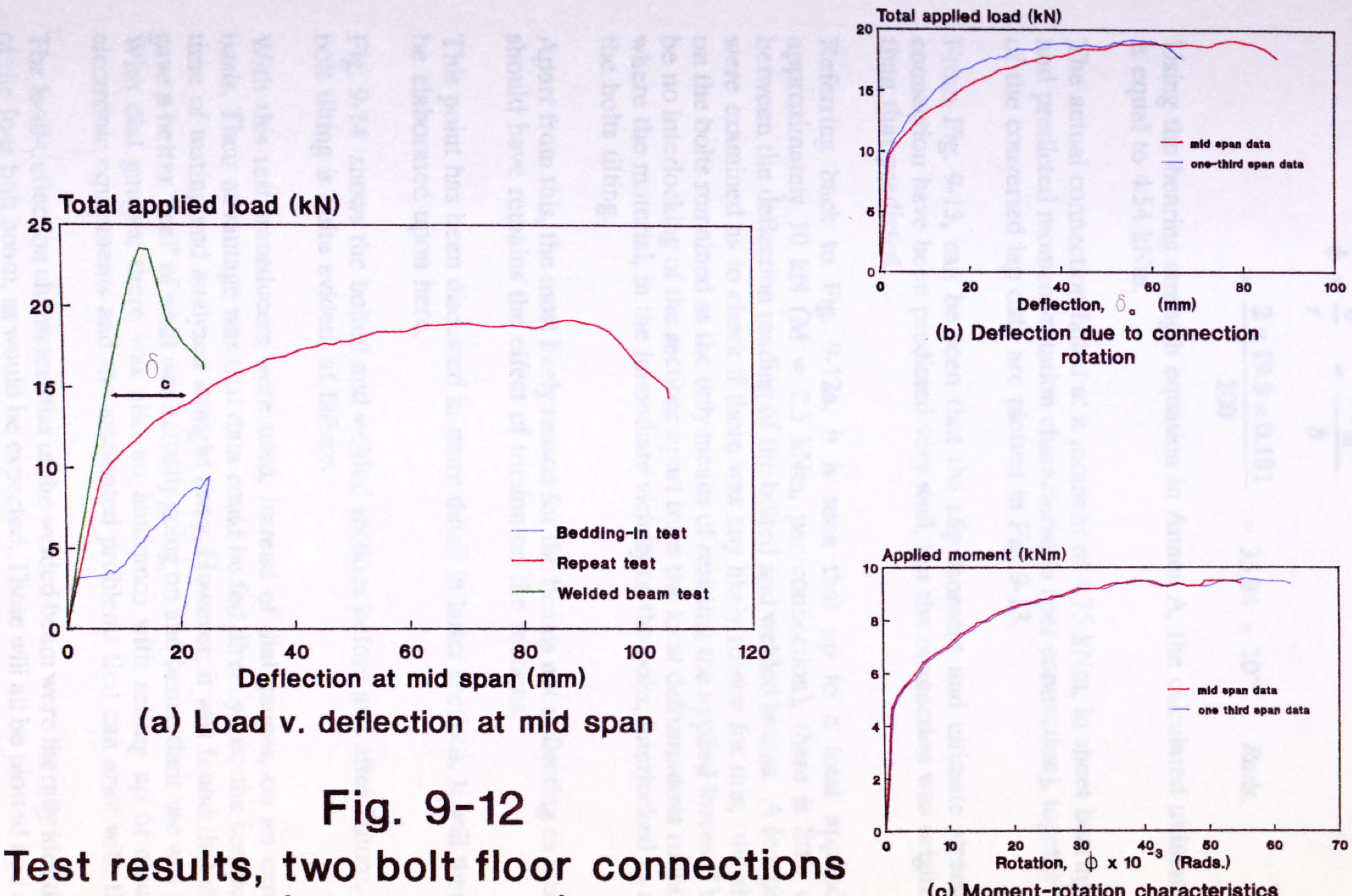


Fig. 9-12

Test results, two bolt floor connections (beam set up)

be taken out by self weight of the structure, wind loads, misalignment of members and other constructional imperfections. The connection rotation at failure is therefore calculated as :

$$\phi = \frac{\delta}{r} = \frac{2P_{bs}c}{b}$$

$$= \frac{2 \times 19.8 \times 0.181}{200} = 35.84 \times 10^{-3} \quad \textit{Rads.}$$

Using the bearing strength equation in Annex A, the calculated ultimate moment is equal to 4.54 kNm.

The actual connection failed at a moment of 4.75 kNm, in sheet bearing. The test and predicted moment-rotation characteristics (per connection), together with that of the converted lap data are plotted in Fig. 9-13.

From Fig. 9-13, can be seen that the slip moment and ultimate strength of the connection have been predicted very well, but the connection was originally stiffer than that predicted.

Referring back to Fig. 9-12a, it is seen that up to a total applied load of approximately 10 kN (M = 2.5 kNm, per connection), there is little difference between the deflection reading of the bolted and welded beams. A few possibilities were examined as to check if there was any likely reason for this; the shear force on the bolts remained as the only means of resisting the applied forces. There could be no interlocking of the sections apart from the local deformations round the bolts where the material, in the immediate vicinity of the bolts, interlocked as a result of the bolts tilting.

Apart from this, the most likely reason for the beams not deflecting as much as they should have remains the effect of torsion on the sections.

This point has been discussed in more detail in latter sections. It will therefore not be elaborated upon here.

Fig. 9-14 shows the bolted and welded sections before and after failure. Note that bolt tilting is quite evident at failure.

With this test transducers were used, instead of dial gauges, on an experimental basis. Their advantage was that data could be fed directly into the computer at the time of testing and analyzed straight away. However, it was found that dial gauges gave a better "feel" of what was actually going on and hence their use was preferred. With dial gauges, there was also no hindrance with setting up of electrical and electronic equipments and the associated problems that can arise with them.

The load-deflection characteristics of the welded beam were literally identical to that of the four bolt beam, as would be expected. These will all be plotted and compared on one set of axes at a later stage.

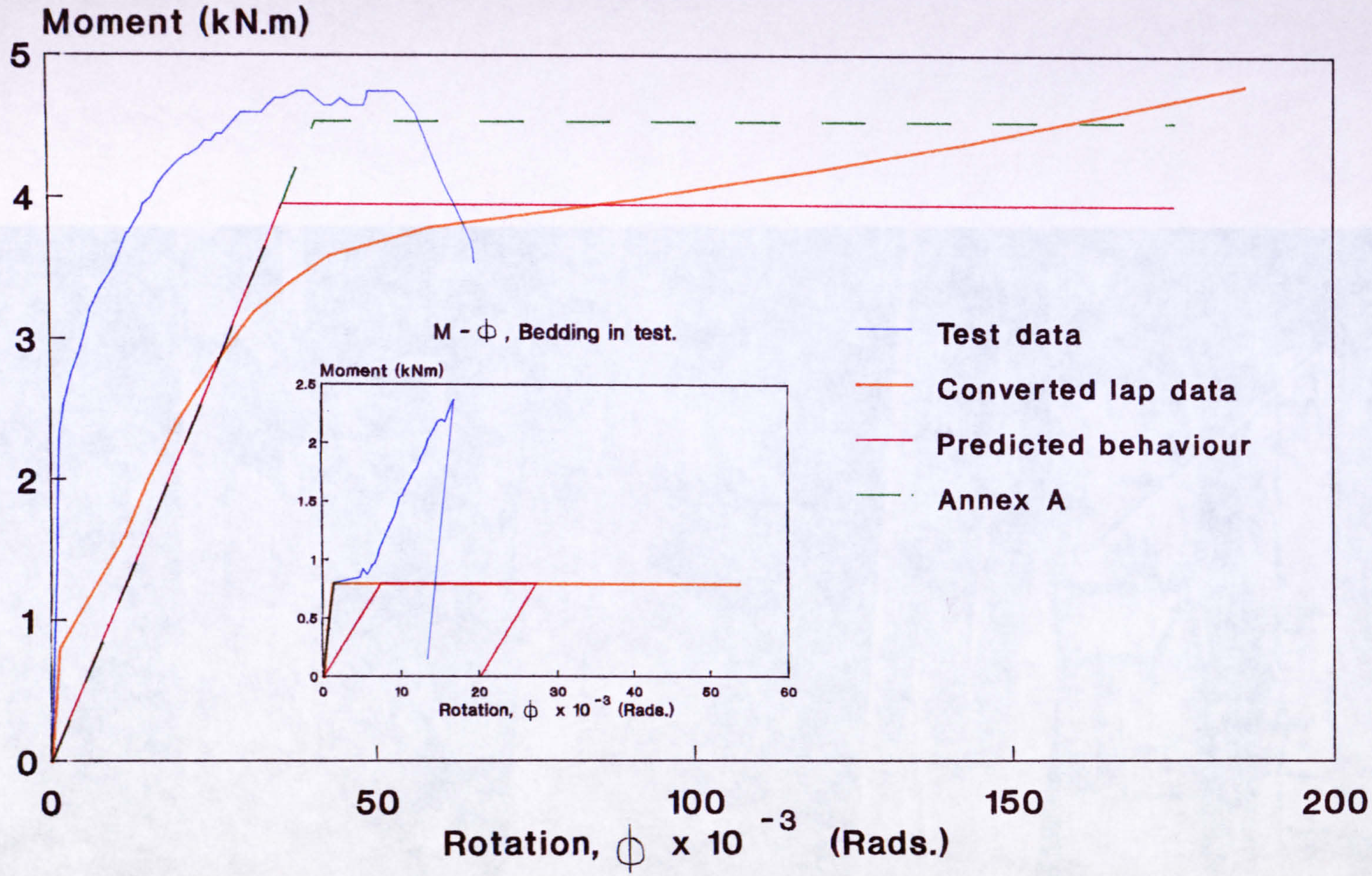


Fig. 9-13
Actual and predicted moment rotation characteristics of two bolt beam set up

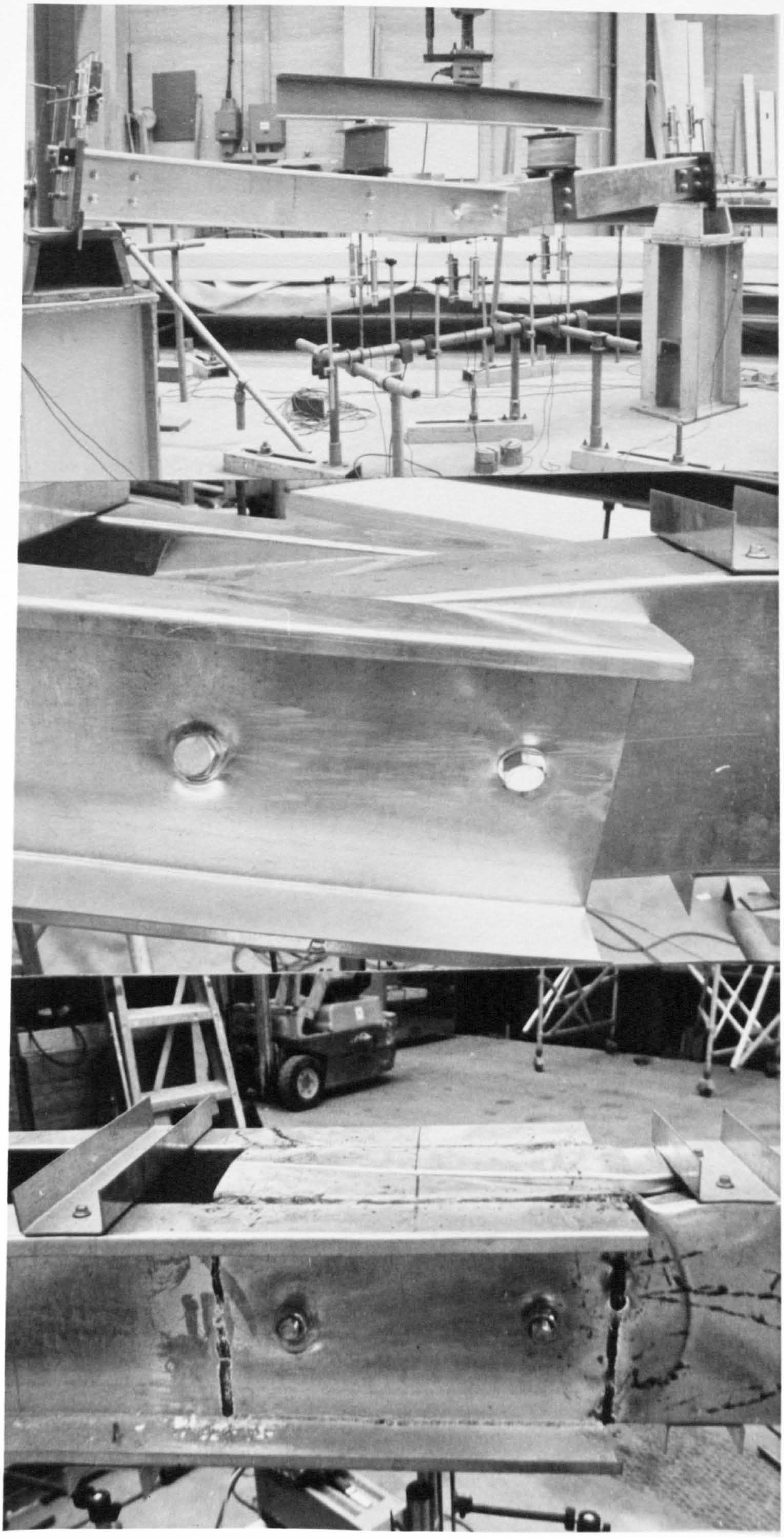


Fig. 9-14. Two bolt beam connection. Bolted and welded beams at failure

9.3. Summary of the test results for four and two bolt connections

The predicted and actual test results obtained so far are tabulated below :

Channel sections

Floor set up

Connection configuration	Ultimate moment (kN.m)		
	Test result	Predicted	Lap data (converted to M-ϕ)
Four bolts	7.63	7.50	8.82
Two bolts	2.83	2.56	3.12

Beam set up

Connection configuration	Ultimate moment (kN.m)		
	Test result	Predicted	Lap data (converted to M-ϕ)
Four bolts	5.85*	9.37	11.20
Two bolts	4.75	3.96	4.80

* Failed by flange buckling.

Chapter Ten

Full moment connections test results, three bolt connections

10. Full moment connections test results, three bolt connections

Summary

With two and four bolt connections, the centre of rotation of the connection coincides with the centre of the bolt group due to symmetry. Therefore all bolts reach their bearing capacity simultaneously. With three bolt connections, this is not the case. In BS 5950 Part 5 (§ 8.2.9) it is specified that for a group of bolts in shear, for material thickness less than 4 mm, the moment capacity of the bolt group may be determined by assuming that each bolt reaches its ultimate capacity. In the case of three bolt connections, this leads to a plastic calculation, as opposed to the elastic moment capacity assumed in bolted connections in hot rolled steel sections.

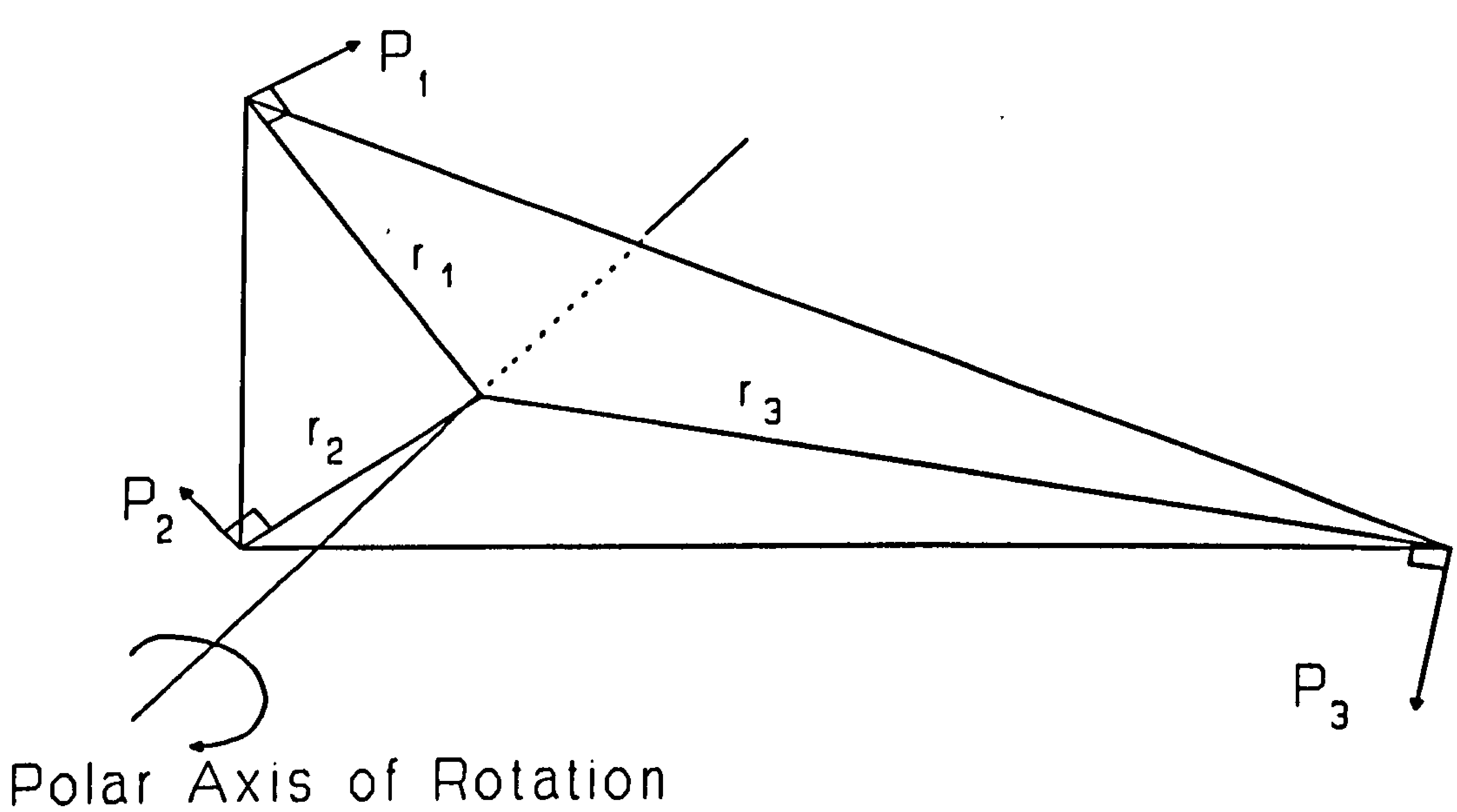
In this chapter the possible locations of centre of rotation of a bolt group are derived both analytically and experimentally, and the results of three bolt moment connections are described and discussed in detail.

10.1. Introduction

Rotation of a bolted connection

When a moment is applied to a bolt group, the whole group will rotate about a common point. This point is known as the Centre Of Rotation (COR) of the bolt group. A line, perpendicular to the plane of the bolt group, passing through the COR is known as the polar axis of rotation of the bolt group.

If a connection is subjected to a pure moment, that is no shear forces acting, then the direction of the resisting force at each bolt will be at right angles to the radius of rotation of that bolt. (Fig. 10-1, below)



With four and two bolt connections described so far, it was assumed that the polar axis of rotation passed through the centre of the bolt group. This is to be expected due the symmetrical nature of these connections. This assumption was also corroborated by results produced in the previous chapter, and photographs of the distorted bolt holes after failure.

With three bolt connections, symmetry about a common point does not exist. The COR is therefore obtained by a rational analysis of the reactant forces satisfying the equilibrium conditions.

Two different solutions are possible, based on Elastic and Plastic assumptions of how forces react in a three bolt connection in cold formed steel.

It is now intended to first develop the possible locations of the COR, and second apply and compare the resulting moment-rotation characteristics with the test results obtained. Hence appropriate recommendations can be made.

10.2. Resolution of the Centre Of Rotation

10.2.1. Elastic analysis

10.2.1.1. Elastic centre of rotation

Assumption :

The magnitude of the force acting on each bolt is proportional to its lever arm from the COR.

With reference to Fig. 10-1:

$$P_i \propto r_i \qquad (Where\ i = 1\ to\ 3)$$

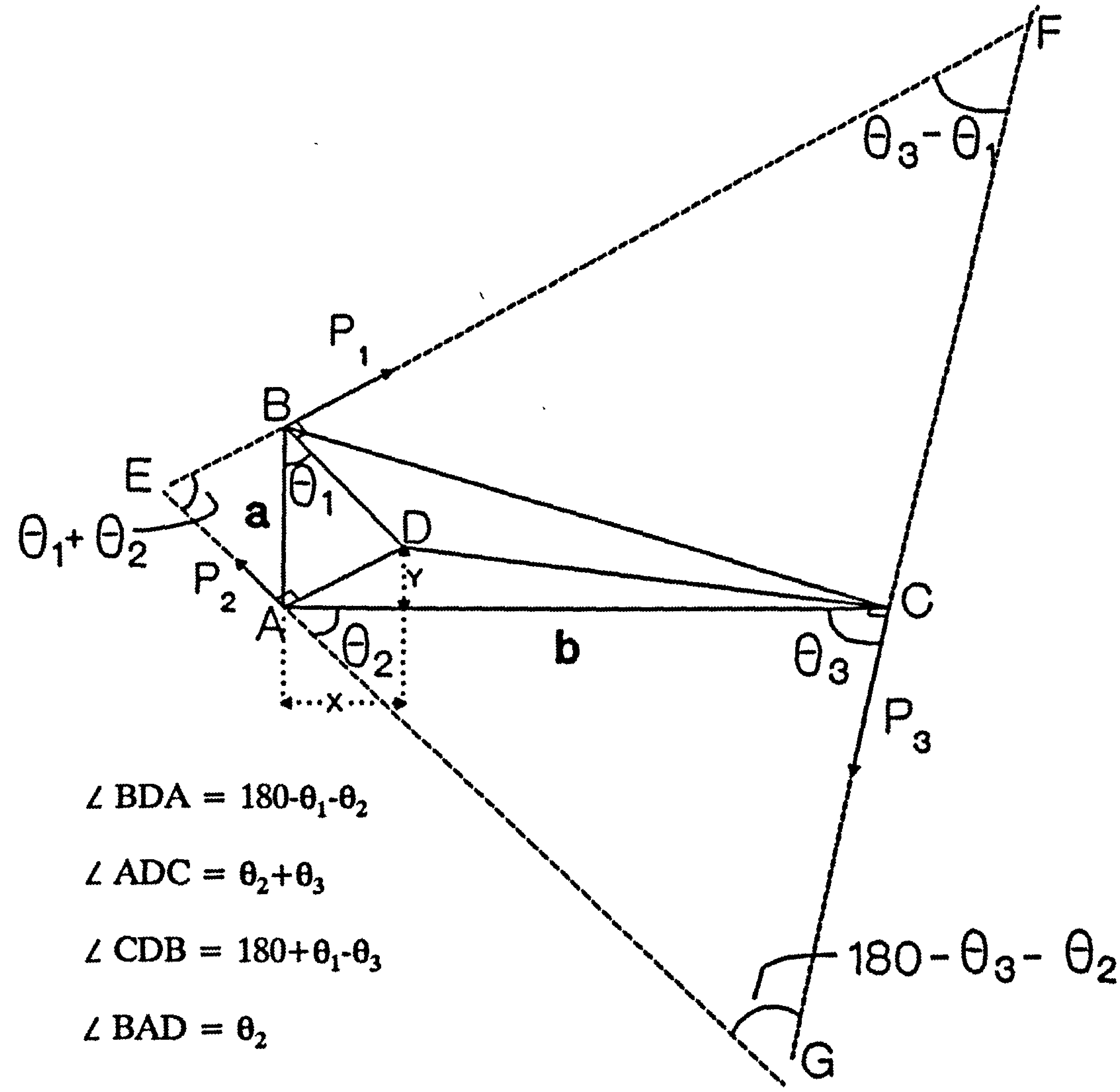
$$i.e: \quad \frac{P_i}{r_i} = K \qquad (Where\ K\ is\ a\ constant)$$

$$\therefore \quad \frac{P_1}{r_1} = \frac{P_2}{r_2} = \frac{P_3}{r_3} = K$$

Fig. 10-2 shows the triangle of forces for the Elastic assumption.

For equilibrium, the sum of the resultant of forces in the direction of each force must add to zero.

Elastic triangle of forces



$\angle$ BDA = $180-\theta_1-\theta_2$

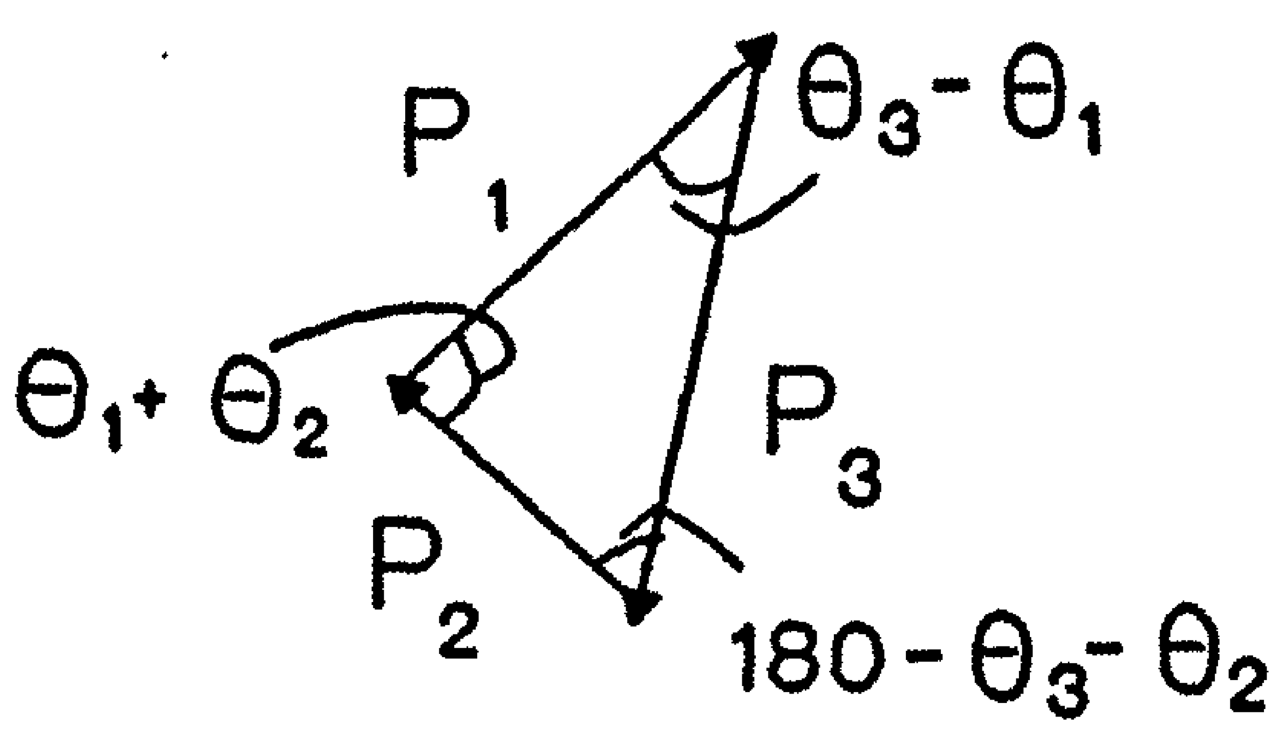
$\angle$ ADC = $\theta_2+\theta_3$

$\angle$ CDB = $180+\theta_1-\theta_3$

$\angle$ BAD = θ_2

Fig. 10-2

Triangle of forces :



r_1 = BD

r_2 = AD

r_3 = CD

Resolving in the direction of P_1 :

$$\begin{aligned} P_1 &= P_3 \cos(\theta_3-\theta_1) - P_2 \cos(180-(\theta_1+\theta_2)) \\ &= P_3 \cos(\theta_3-\theta_1) + P_2 \cos(\theta_1+\theta_2) \end{aligned} \tag{1}$$

Resolving in the direction of P_2 :

$$\begin{aligned} P_2 &= P_1 \cos(\theta_1+\theta_2) + P_3 \cos(180-(\theta_2+\theta_3)) \\ &= P_1 \cos(\theta_1+\theta_2) - P_3 \cos(\theta_2+\theta_3) \end{aligned} \tag{2}$$

Resolving in the direction of P_3:

$$P_3 = P_1 \cos(\theta_3-\theta_1) - P_2 \cos(\theta_2+\theta_3) \tag{3}$$

From the cosine rule :

$$\left.\begin{aligned} &\text{in } \triangle\, ADB : \\ &\qquad a^2 = r_1^2 + r_2^2 - 2r_1r_2\cos(180-(\theta_1+\theta_2)) \\ &\text{in } \triangle\, ADC : \\ &\qquad b^2 = r_2^2 + r_3^2 - 2r_2r_3\cos(\theta_2+\theta_3) \\ &\text{in } \triangle\, BDC : \\ &\qquad a^2 + b^2 = r_1^2 + r_3^2 - 2r_1r_3\cos(180+(\theta_1-\theta_3)) \end{aligned}\right\} \rightarrow$$

Re-arranging in terms of cosines gives:

$$\left\{\begin{aligned} \cos(\theta_1+\theta_2) &= \frac{-r_1^2-r_2^2+a^2}{2r_1r_2} && (4) \\ \cos(\theta_2+\theta_3) &= \frac{r_2^2+r_3^2-b^2}{2r_2r_3} && (5) \\ \cos(\theta_3-\theta_1) &= \frac{-r_1^2-r_3^2+(a^2+b^2)}{2r_1r_3} && (6) \end{aligned}\right.$$

From the elastic assumption it is given that :

$$\frac{P_1}{r_1} = \frac{P_2}{r_2} = \frac{P_3}{r_3} \tag{7}$$

Substituting (7) in (1) gives $\rightarrow$

$$P_1 = \frac{P_1}{r_1} r_2 \cos(\theta_1 + \theta_2) + \frac{P_1}{r_1} r_3 \cos(\theta_3 - \theta_1)$$

Substituting (4) and (6) in the equation above and re-arranging :

$$r_1 = r_2 \frac{a^2 - r_1^2 - r_2^2}{2 r_1 r_2} + r_3 \frac{a^2 + b^2 - r_1^2 - r_3^2}{2 r_1 r_3}$$

$$2a^2 + b^2 = 4r_1^2 + r_2^2 + r_3^2 \tag{8}$$

Similarly, substituting (7), (4) & (5) in (2) $\rightarrow$

$$a^2 + b^2 = r_1^2 + 4r_2^2 + r_3^2 \tag{9}$$

Substituting (7), (5) & (6) in (3) $\rightarrow$

$$a^2 + 2b^2 = r_1^2 + r_2^2 + 4r_3^2 \tag{10}$$

Three equations (8), (9) and (10), and three unknowns (r_1 ,r_2 ,r_3)

$$\begin{pmatrix} 2 & 1 \\ 1 & 1 \\ 1 & 2 \end{pmatrix} \cdot \begin{pmatrix} a^2 \\ \\ b^2 \end{pmatrix} = \begin{pmatrix} 4 & 1 & 1 \\ 1 & 4 & 1 \\ 1 & 1 & 4 \end{pmatrix} \cdot \begin{pmatrix} r_1^2 \\ r_2^2 \\ r_3^2 \end{pmatrix}$$

Hence:

$$\left.\begin{aligned} r_1 &= \frac{1}{3}\sqrt{4a^2 + b^2} \\ r_2 &= \frac{1}{3}\sqrt{a^2 + b^2} \\ r_3 &= \frac{1}{3}\sqrt{a^2 + 4b^2} \end{aligned}\right\} \qquad (11)$$

The coordinates of the COR are given as :

$$x = r_2 \sin(\theta_2)$$

$$y = r_2 \cos(\theta_2)$$

From Δ ADB, using the cosine rule, θ_2 is easily calculated :

$$r_1^2 = a^2 + r_2^2 - 2 a r_2 \cos\theta_2$$

Re-arranging the equation above in terms of θ_2 and substituting for r_1 and r_2 gives :

$$\theta_2 = \cos^{-1}\left(\frac{a}{\sqrt{a^2+b^2}}\right)$$

$$\therefore \ \theta_2 = \angle A\hat{B}C$$

The coordinates of the COR are therefore simply given as :

$$\boxed{\left.\begin{aligned} x &= \frac{b}{3} \\ y &= \frac{a}{3} \end{aligned}\right\}}$$

Therefore for Elastic behaviour the Centre Of Rotation lies at the centroid of the bolt group.

10.2.1.2. Elastic moment of resistance of a bolt group

The elastic moment of resistance is given as :

$$M = P_1 r_1 + P_2 r_2 + P_3 r_3$$

$$= \sum_{i=1}^{3} P_i r_i \qquad (12)$$

It has been clearly stated that according to the elastic assumption, the further a bolt is from the COR the greater will be the force acting on it.

Since $r_3 > r_1 > r_2$ then $P_3 > P_1 > P_2$. (See Fig. 10-2)

It follows that P_3 will be the critical force in the connection.

Let : $P_3 = P$

Using the elastic assumption the other two forces are also expressed in terms of P :

$$P_1 = (P_3 / r_3)\, r_1 \qquad\qquad P_2 = (P_3 / r_3)\, r$$

Substituting into equation (12) ;

$$M = P\frac{r_1^2}{r_3} + P\frac{r_2^2}{r_3} + P r_3$$

$$= \frac{P}{r_3}(r_1^2 + r_2^2 + r_3^2)$$

$$= \frac{P}{r_3}\sum_{i=1}^{3} r_i^2$$

Substituting for the radii of rotation, from (11), the moment of resistance is defined as :

$$\boxed{M = \frac{2P}{\sqrt{a^2+4b^2}}(a^2+b^2)}$$

Thus by knowing the critical sheet bearing strength of a connection, its moment of resistance is easily obtained.

10.2.2. Plastic analysis

10.2.2.1. Plastic centre of rotation

Assumption :

Due to the inherent ductility of the individual fastenings, it is permissible to assume that each bolt carries the ultimate sheet bearing capacity at failure.

i.e : $P_1 = P_2 = P_3 = P$

Fig. 10-3 shows the triangle of forces for the Plastic assumption. With reference to this figure :

Resolving along FG :

$$P = P\cos\alpha + P\cos\beta$$
$$1 = \cos\alpha + \cos\beta \qquad (13)$$

Resolving along EF :

$$P = P\cos\beta + P\cos\gamma$$
$$1 = \cos\beta + \cos\gamma \qquad (14)$$

Resolving along GE :

$$P = P\cos\gamma + P\cos\alpha$$
$$1 = \cos\gamma + \cos\alpha \qquad (15)$$

Three equations and three unknowns :

$$\left.\begin{aligned} \cos\alpha + \cos\beta &= 1 \\ \cos\beta + \cos\gamma &= 1 \\ \cos\gamma + \cos\alpha &= 1 \end{aligned}\right\}$$

solving for α, β and γ gives :

$$\alpha = \beta = \gamma = 60^\circ$$

Plastic triangle of forces

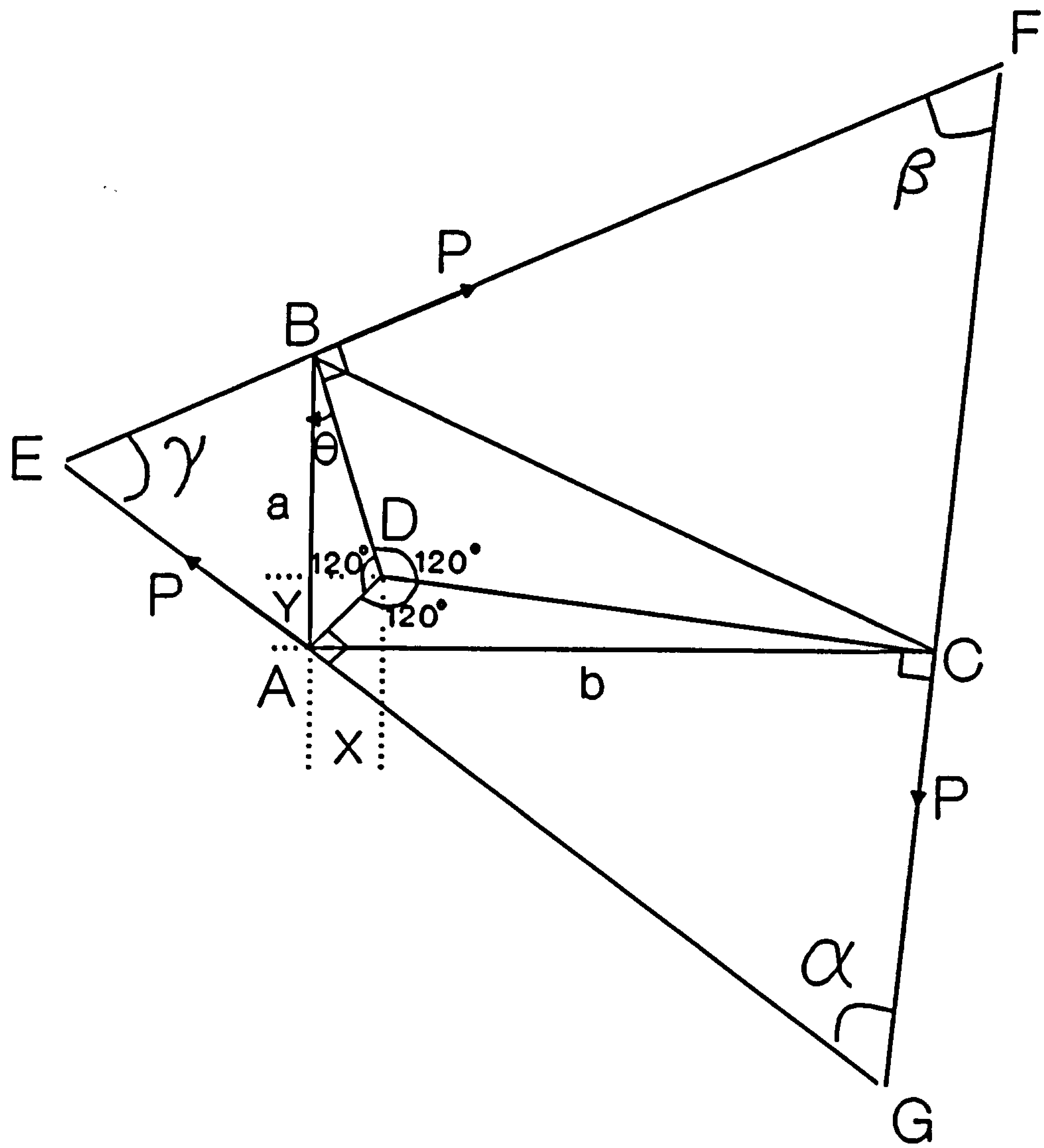


Fig. 10-3

r_1 = BD

r_2 = AD

r_3 = CD

∠ BAD = 60 - θ

∠ DAC = 30 + θ

It follows that as there are no horizontal and vertical force resultants, the directions of the bolt forces must therefore, form an equilateral force triangle.

Since the radii of rotation are perpendicular to the bolt forces, the position of the plastic COR must be at the point where lines from the COR to the bolt positions form angles of 120° to one another. (Because in quadrilaterals ADCG, ADBE & FBDC opposite angles must be supplementary)

If we can define angle θ (∠ ABD) in terms of a & b, the radii of rotation can then be simply obtained using the sine rule.

in $\triangle ADC$:

$$\frac{b}{\sin 120} = \frac{r_2}{\sin(30-\theta)}$$

in $\triangle ADB$:

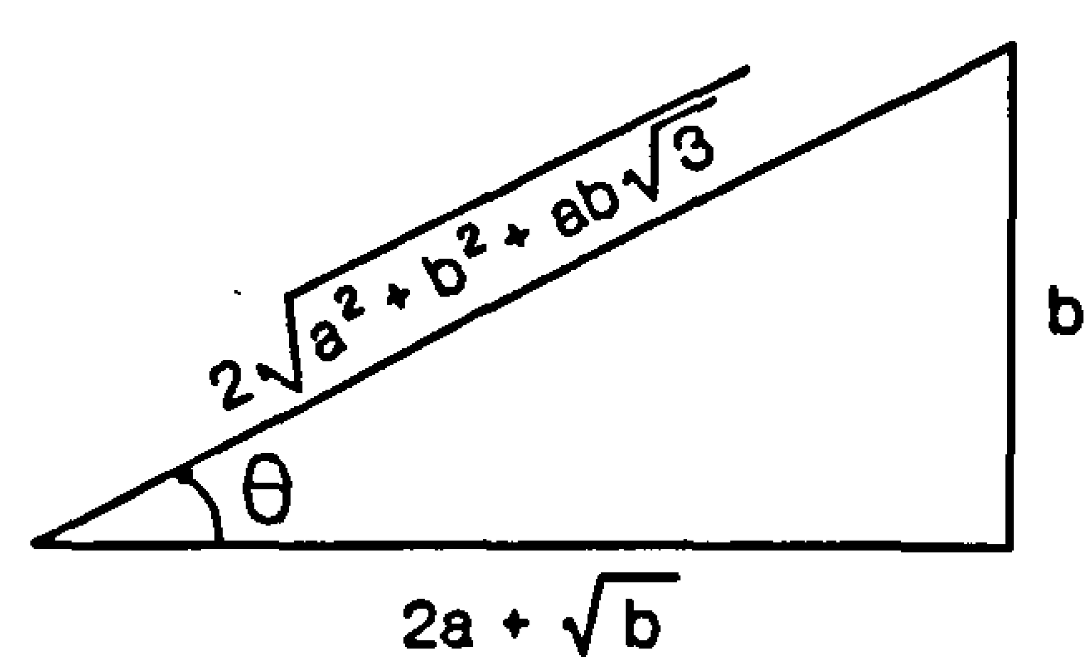
$$\frac{a}{\sin 120} = \frac{r_2}{\sin\theta}$$

Divide the two equations above to eliminate r_2 and re-arrange :

$$\begin{aligned} a\sin\theta &= b\sin(30-\theta) \\ &= b\sin 30\cos\theta - b\sin\theta\cos 30 \end{aligned}$$

$$a\tan\theta = b\sin 30 - b\tan\theta\cos 30$$

$$\therefore \tan\theta = \frac{b}{2a+b\sqrt{3}}$$



Using the sine rule :

in $\triangle$ *ADB* :

$$\frac{r_1}{\sin(60-\theta)} = \frac{a}{\sin 120}$$

$$\frac{r_2}{\sin\theta} = \frac{a}{\sin 120}$$

in $\triangle$ *ADC* :

$$\frac{r_3}{\sin(30+\theta)} = \frac{b}{\sin 120}$$

Each of the equations above define the radius of rotation of each bolt as follows :

$$\left.\begin{aligned} r_1 &= \frac{a(a + b/\sqrt{3})}{\sqrt{a^2+b^2+ab\sqrt{3}}} \\ r_2 &= \frac{ab/\sqrt{3}}{\sqrt{a^2+b^2+ab\sqrt{3}}} \\ r_3 &= \frac{b(a/\sqrt{3} + b)}{\sqrt{a^2+b^2+ab\sqrt{3}}} \end{aligned}\right\}$$

The coordinates of the COR are given as :

$$\left.\begin{aligned} x &= r_2 \cos(30+\theta) \\ y &= r_2 \cos(60-\theta) \end{aligned}\right\} \rightarrow \left.\begin{aligned} x &= \frac{r_2}{2}(\sqrt{3}\cos\theta - \sin\theta) \\ y &= \frac{r_2}{2}(\cos\theta + \sqrt{3}\sin\theta) \end{aligned}\right\}$$

Substituting for r_2, cos θ and sin θ gives :

$$\boxed{\left.\begin{aligned} x &= \frac{ab(a + b/\sqrt{3})}{2(a^2+b^2+ab\sqrt{3})} \\ y &= \frac{ab(a/\sqrt{3}+b)}{2(a^2+b^2+ab\sqrt{3})} \end{aligned}\right\}}$$

10.2.2.2. Plastic moment of resistance of a bolt group

The plastic moment of resistance is given as :

$$M = P\,(r_1 + r_2 + r_3)$$

$$= P.\sum_{i=1}^{3} r_i$$

Substituting for the radii of rotation gives :

$$\boxed{M = P\,\sqrt{a^2 + b^2 + ab\sqrt{3}}}$$

10.3. Test results

Having defined the COR and moment of resistance of a three bolt connection based on elastic and plastic assumptions; the equations derived are now used to predict the characteristics of such connections. The predicted behaviours are then compared with that of the actual test results.

The tests arrangements and procedures were as described previously in Chapter Eight.

10.3.1. The floor set up, three bolt connections

The connection details were as given in figure below :

Section properties were as follows :

$t \;= 1.43$ mm

$\sigma_y \;= 321.2$ N/mm^2

$\sigma_{ult} = 408.0$ N/mm^2

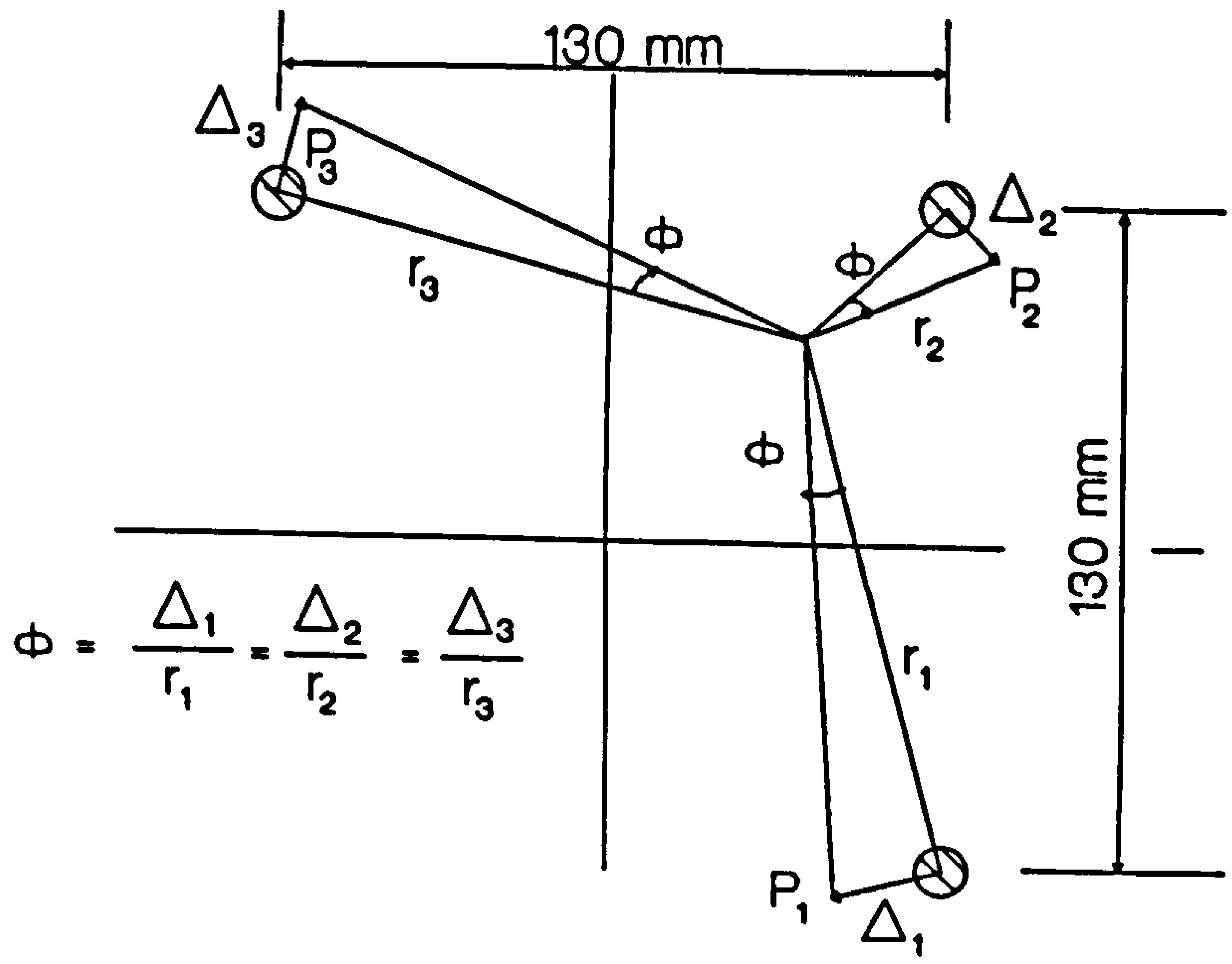


Fig. 10-4: Connection details, three bolt floor set up.

The ultimate bearing strength and flexibility of each fastening is calculated as :

$$P_{bs} = \alpha \,.\, d.\, t.\, \sigma_{ult} \qquad = \alpha \times 16 \times 1.43 \times 0.408 \qquad = 9.36\,\alpha$$

$$\alpha = k_1.k_2.k_3.k_3.k_4.k_5.k_6.k_7$$

$$= k_2 \qquad (k_1 \text{ and } k_3 \text{ to } k_7 \text{ all being equal to 1})$$

$$\therefore \quad P_{bs} = 9.36\,(1.9 + 0.2\,t) \qquad = 20.5 \text{ kN}$$

$$c = 5n\,\left(\frac{10}{t_1} + \frac{10}{t_2} - 2\right) \times 10^{-3}$$

$$= 5 \times 3\,\left(\frac{10}{1.43} \times 2 - 2\right) \times 10^{-3} = 0.180 \qquad \left(\frac{mm}{kN}\right)$$

The predicted moment-rotation characteristics, will obviously hinge on the location of the centre of rotation, i.e. it depends on which assumption (elastic or plastic) is taken.

In the following sections the predicted moment-rotation characteristics based on the elastic and plastic centres of rotation are derived, and compared with what considered to be the actual COR, based on the actual test readings.

10.3.1.1. Elastic moment-rotation characteristics, three bolt floor set up

For a = b = 130 mm (Fig. 10-4), the radii of rotation are obtained as :

$$\left.\begin{aligned} r_1 &= \frac{1}{3}\sqrt{4a^2 + b^2} \\ r_2 &= \frac{1}{3}\sqrt{a^2 + b^2} \\ r_3 &= \frac{1}{3}\sqrt{a^2 + 4b^2} \end{aligned}\right\} \rightarrow \left.\begin{aligned} r_1 &= r_3 = 96.9\ mm \\ r_2 &= 61.3\ mm \end{aligned}\right\}$$

The COR is at :

$$\left.\begin{aligned} x &= \frac{b}{3} \\ y &= \frac{a}{3} \end{aligned}\right\} \rightarrow \left.\begin{aligned} x &= 43.3 \\ y &= 43.3 \end{aligned}\right\}$$

r_3 (or r_1) is the largest radius. Hence P_3 is the critical force and failure by sheet bearing should first occur at bolt 3.

Let $P_3 = P$ then the moment of resistance is given as :

$$M = \frac{2P}{\sqrt{a^2+4b^2}}(a^2+b^2)$$

$$\therefore \qquad M = 0.233\ P$$

For obvious reasons the critical connection rotation is also based on r_3.

$$\phi = \frac{\Delta}{r_3} \quad [Rads.] \qquad and \qquad M = 0.233\,P \quad [kNm]$$

(Note that hole deformation is denoted by Δ, in this chapter)

With reference to Fig. 7-20 (page 155), the coordinates of the predicted elastic moment-rotation characteristic are calculated as :

A (0, 0)
B ($4c/r_3$, 0.233 x 4)
C [($4c$+ hole clearance)/r_3 , 0.233 x 4]
D [(P_{bs} c + hole clearance)/r_3 , 0.233 x P_{bs}]

c = 0.180 N/mm^2
r_3 = 96.9 mm
P_{bs} = 20.5 kN
hole clearance = 2mm.

i.e. (ϕ x 10^{-3} [Rads.], M [kN.m])

(0.0, 0.0)
(7.43, 0.93)
(28.07, 0.93)
(58.72, 4.78)

10.3.1.2. Plastic moment-rotation characteristics, three bolt floor set up

For the given connection dimensions, with plastic assumption, the radii of rotation are :

$$r_1 = \frac{a(a + b/\sqrt{3})}{\sqrt{a^2+b^2+ab\sqrt{3}}}$$

$$r_2 = \frac{ab/\sqrt{3}}{\sqrt{a^2+b^2+ab\sqrt{3}}}$$

$$r_3 = \frac{b(a/\sqrt{3} + b)}{\sqrt{a^2+b^2+ab\sqrt{3}}}$$

$\rightarrow$

$$r_1 = r_3 = 106.2 \quad mm$$

$$r_2 = 39.7 \quad mm$$

The COR is at :

$$x = \frac{ab(a + b/\sqrt{3})}{2(a^2+b^2+ab\sqrt{3})}$$

$$y = \frac{ab(a/\sqrt{3}+b)}{2(a^2+b^2+ab\sqrt{3})}$$

$\rightarrow$

$$x = 27.4$$

$$y = 27.4$$

$P_1 = P_2 = P_3 = P$. Hence the moment of resistance is given as :

$$M = P\sqrt{a^2+b^2+ab\sqrt{3}} \qquad = 0.251\ P$$

From the given values for the radii of rotation above, it is evident that first the connection rotates elastically until yield is reached at bolts 3 and 1 . The forces are then redistributed until they are all equal to P at each bolt, when failure occurs. The COR in turn shifts from its elastic to plastic location.

Hence at failure the load-extension characteristics of each fastening should appear as shown in Fig. 10-5.

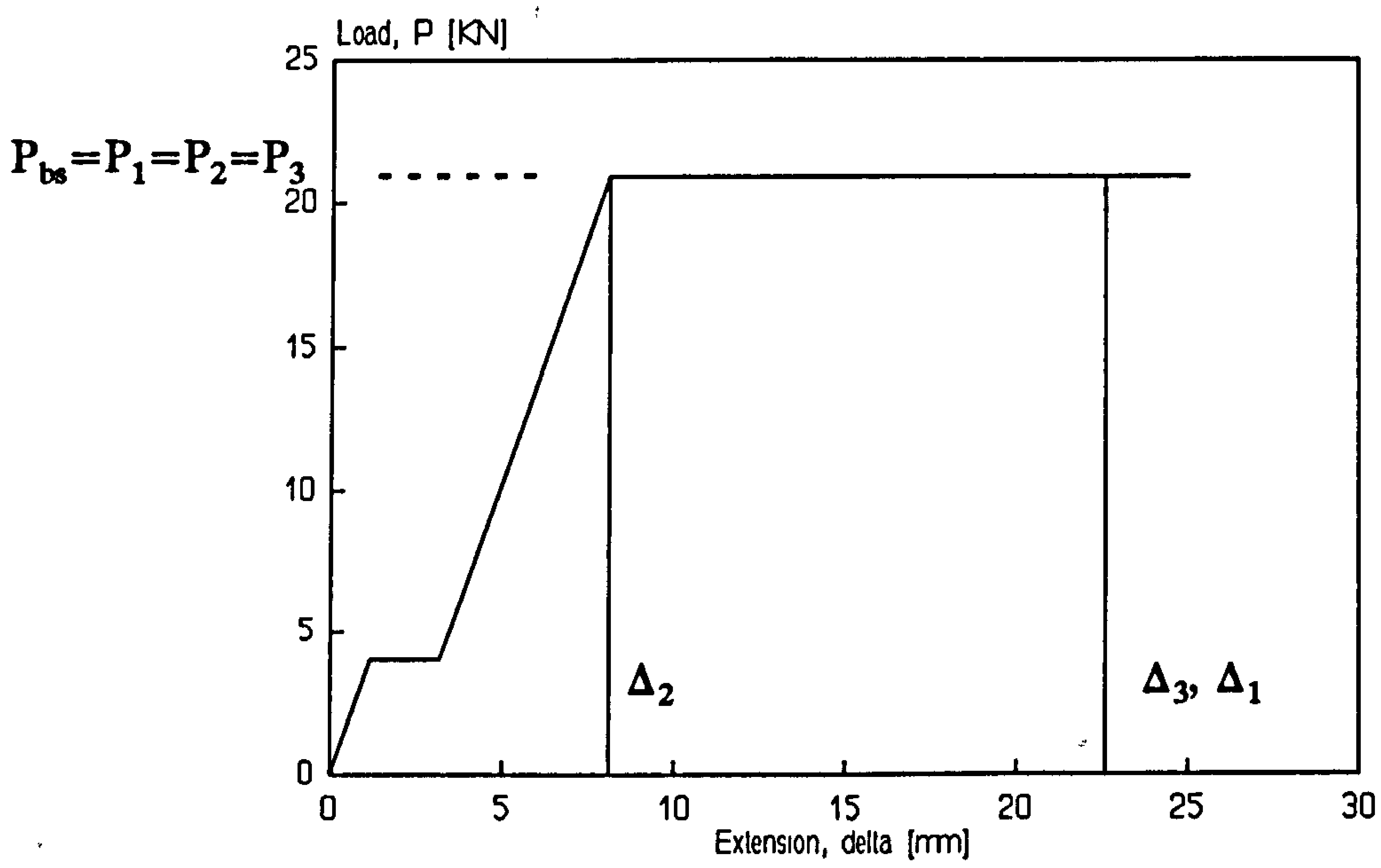


Fig. 10-5 : Load-extension characteristics

The connection rotation ϕ is equal to :

$$\phi = \frac{\Delta_1}{r_1} = \frac{\Delta_2}{r_2} = \frac{\Delta_3}{r_3}$$

Where $\Delta_2 = (r_2 / r_3)\ \Delta_3 = 0.35\ \Delta_3$

Therefore Δ_3 and Δ_1 are well into the plastic plateau when Δ_2 reaches its threshold and subsequently the connection fails.

It follows that the predicted extension at failure should correspond to that of Δ_2.

However if this was to be the case, the predicted connection rotation at failure will be much more flexible compared to its equivalent elastic status. Simply because the predicted hole deformation at failure (i.e. $P_{bs}c$ + hole clearance) will be divided by a much smaller lever arm in the plastic case, compared to the elastic case ($r_{3\,elastic} >> r_{2\,plastic}$). Obviously this can not be.

It therefore appears that the Plastic stipulation of connection failure, as laid out above, contradicts its own definition. This in turn suggests that **the plastic assumption is not valid.**

However, in order to obtain a comparable moment-rotation characteristics with that of the elastic behaviour, $r_{3\ plastic}$ is used to calculate the predicted connection rotation, in the plastic case.

$$\phi = \frac{\Delta}{r_{3\ (plastic)}} \quad [Rads.] \qquad and \qquad M = 0.251\,P \quad [kNm]$$

As was shown in the elastic case, the coordinates of the predicted moment rotation characteristic are given as :

(ϕ x 10^{-3} [Rads.], M [kN.m])

(0.0 ,	0.0)
(6.78,	1.00)
(25.61,	1.00)
(53.58,	5.15)

10.3.1.3. Actual moment-rotation characteristics, three bolt floor set up

The connection failed prematurely by web twisting out of plane at an applied moment of 2.15 kNm. (Compare to predicted connection failure of 4.78 kNm Elastic, and 5.15 kNm Plastic)

However the test results indicated the following before and after failure:

- Distortions of the bolt holes were such that the COR had to be within the connection area, i.e the triangle with the centre of each bolt at its vertices. Fig. 10-6 shows the hole distortion after failure.

- With reference to Fig. 10-7, the deflection readings noted from dial gauges 3 and 4 were approximately twice that of dial gauges 1 and 2.

  i.e. $\delta_{34} \approx 2 \cdot \delta_{12}$ (see Fig. 10-7)

  Where δ_{12} denotes the average deflection readings of dial gauges 1 and 2.

  δ_{34} ditto, for dial gauges 3 and 4.

- The hole distortions at failure were as follows :

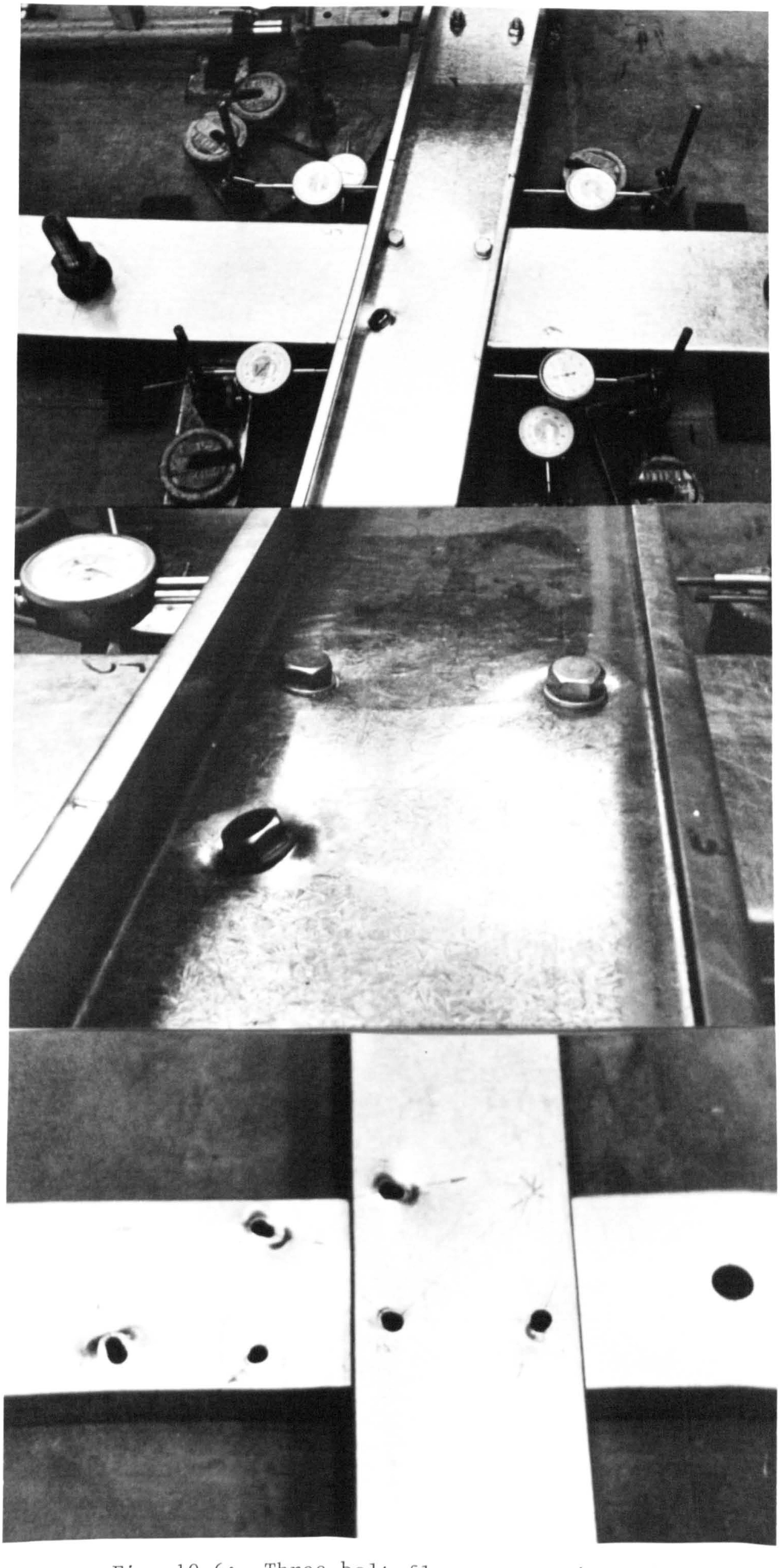


Fig. 10-6: Three bolt floor connection, at failure

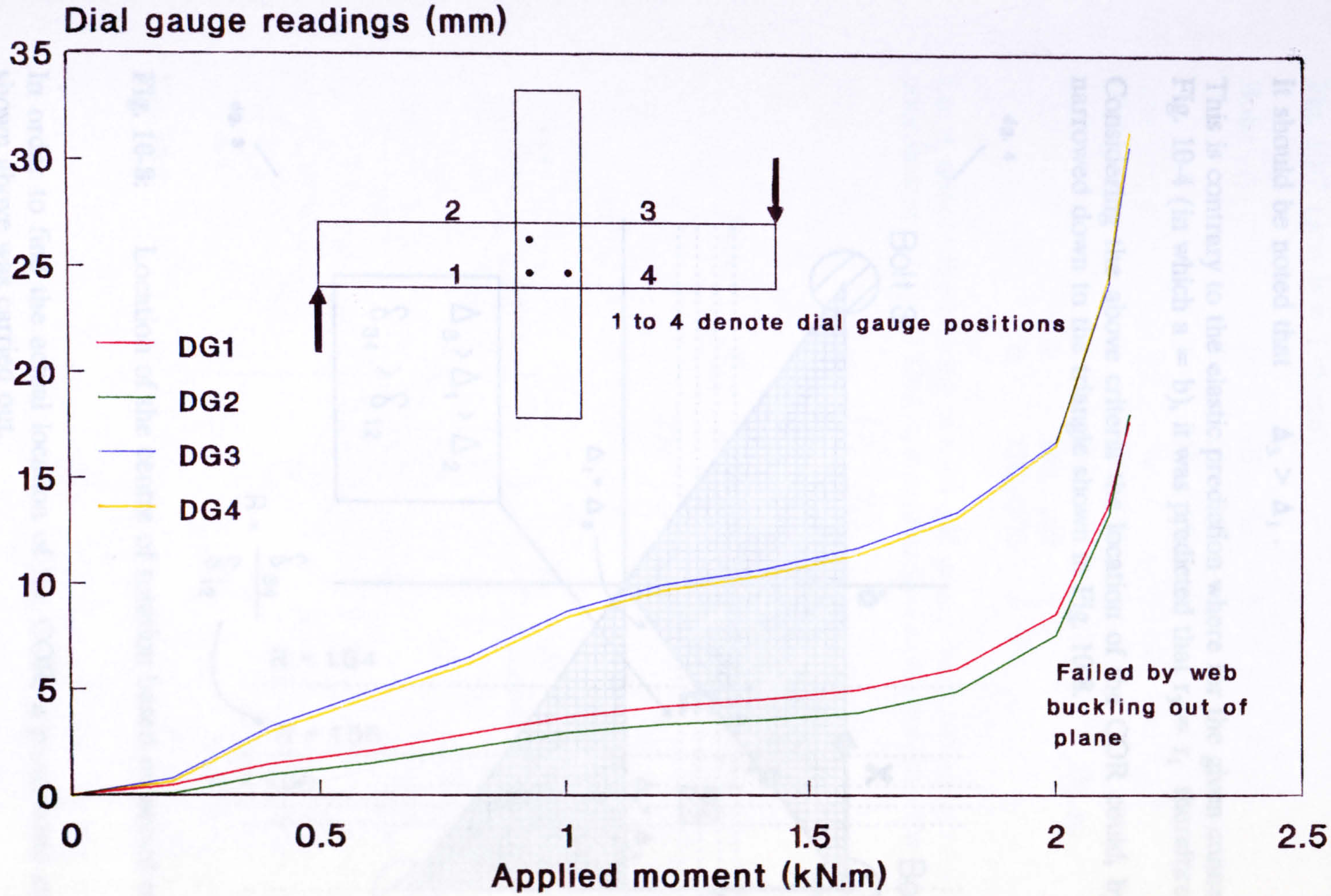


Fig. 10-7
Dial gauge readings v. applied moment.

$\Delta_1 = 5.3$ mm
$\Delta_2 = 3.5$ mm　　Note:　　$\Delta_3 > \Delta_1 > \Delta_2$
$\Delta_3 = 6.0$ mm

The above values each include 1 mm bolt clearance all round.

It should be noted that $\Delta_3 > \Delta_1$.

This is contrary to the elastic prediction where for the given connection geometry, Fig. 10-4 (in which a = b), it was predicted that $r_3 = r_1$ therefore $\Delta_3 = \Delta_1$.

Considering the above criteria the location of the COR could, by inspection, be narrowed down to the triangle shown in Fig. 10-8.

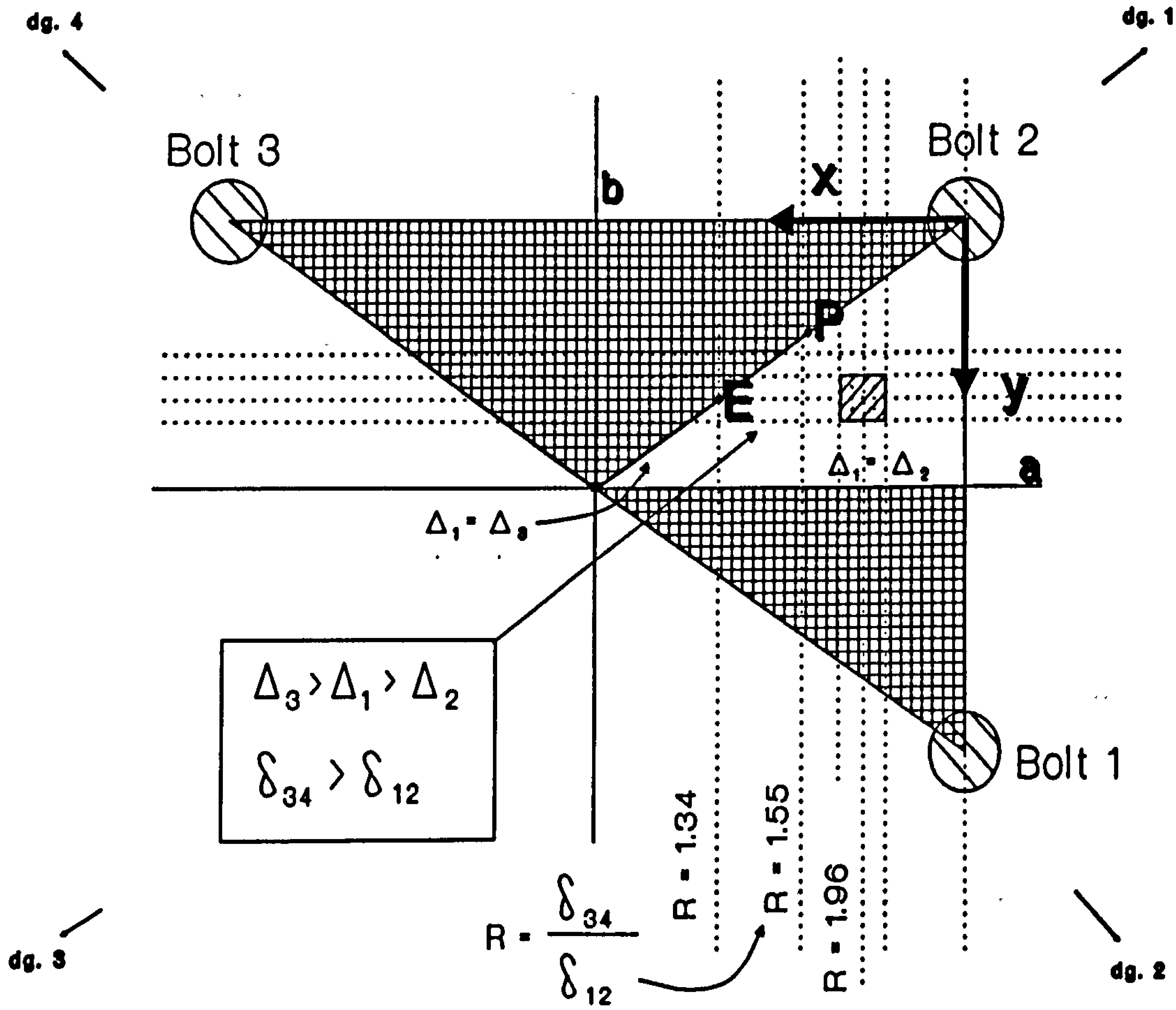


Fig. 10-8: Location of the centre of rotation based on actual test readings.

In order to find the actual location of the COR, a parametric study of the area shown above was carried out.

The triangular area, depicted in Fig. 10-8, was divided into a mesh of squares. The corner of each square being a potential centre of rotation.

A computer programme was written to perform the following algorithm on any given COR.

Taking the cross centres of the two channels as the origin, bolts 1 to 3 and dial gauges 1 to 4 would have the following fixed coordinates. See the test diagram in Fig. 10-7. (a = b = 130 mm)

Bolts	1:	65 , -65	(x_i, y_i)
	2:	65 , 65	
	3:	-65 , 65	
Dial gauges	1:	-150, -100	(x_j, y_j)
	2:	-150, 100	
	3:	150, 100	
	4:	150, -100	

Let θ_j denote the bearing of each dial gauge from any assumed COR, with known coordinates (x_o, y_o). That is, θ is measured clockwise from the vertical. (Fig. 10-9)

$$\textit{If} \qquad m_j = \left(\frac{y_j - y_o}{x_j - x_o}\right)$$

$$\textit{Then} \qquad \theta_j = 90 - \tan^{-1} m_i \qquad \textit{for } j = 1 \textit{ to } 2$$

$$= 270 - \tan^{-1} m_i \qquad = 3 \textit{ to } 4$$

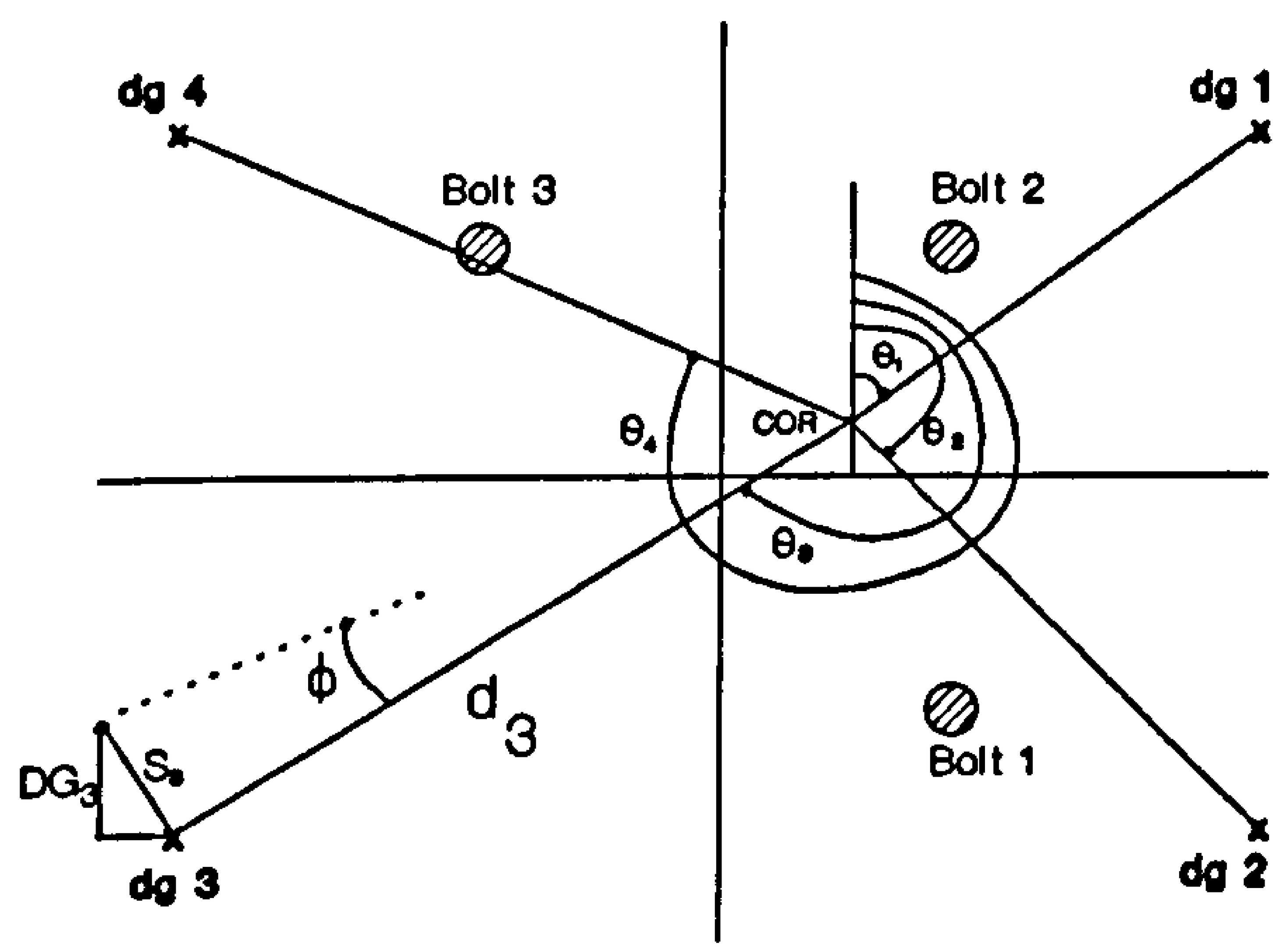


Fig. 10-9

At any assumed COR, the connection was then :

i) subjected to an arbitrary rotation ϕ, as shown in Fig. 10-9.

ii) The applied rotation ϕ would cause dial gauge positions 1 to 4 to move by a distance s in a direction normal to their radius of rotation.

Therefore $s_j = \phi\, d_j$ $(j = 1 \text{ to } 4)$

Where d_j = *the distance between each dial gauge position and the COR.*

$$= \sqrt{(x_j - x_o)^2 + (y_j - y_o)^2}$$

The recorded dial gauge readings would therefore be equal to :

$$DG_j = s_j \operatorname{Sin} \theta_j \qquad (j = 1 \text{ to } 2)$$

$$DG_j = - s_j \operatorname{Sin} \theta_j \qquad (= 3 \text{ to } 4)$$

(-ve) sign indicates movement in opposite direction to dial gauges 1 and 2

iii) The hole distortion at each bolt as a result of the applied rotation could also be simply calculated as :

$$\Delta_i = \phi\, d_i \qquad (i = 1 \text{ to } 3)$$

iv) The corresponding force P_i, on each bolt for a sheet extension of Δ_i was interpolated from the average load-extension characteristics of the equivalent lap joint tests with the same sheet properties as that of the tested channels.

The amount of slip of this equivalent lap data was reduced to 2 mm to be comparable with that of the tested channels as depicted in Fig. 10-10.

v) The induced moment, as a result of the applied rotation, was then calculated as :

$$M_i = \sum_{i=1}^{3} P_i \cdot r_i$$

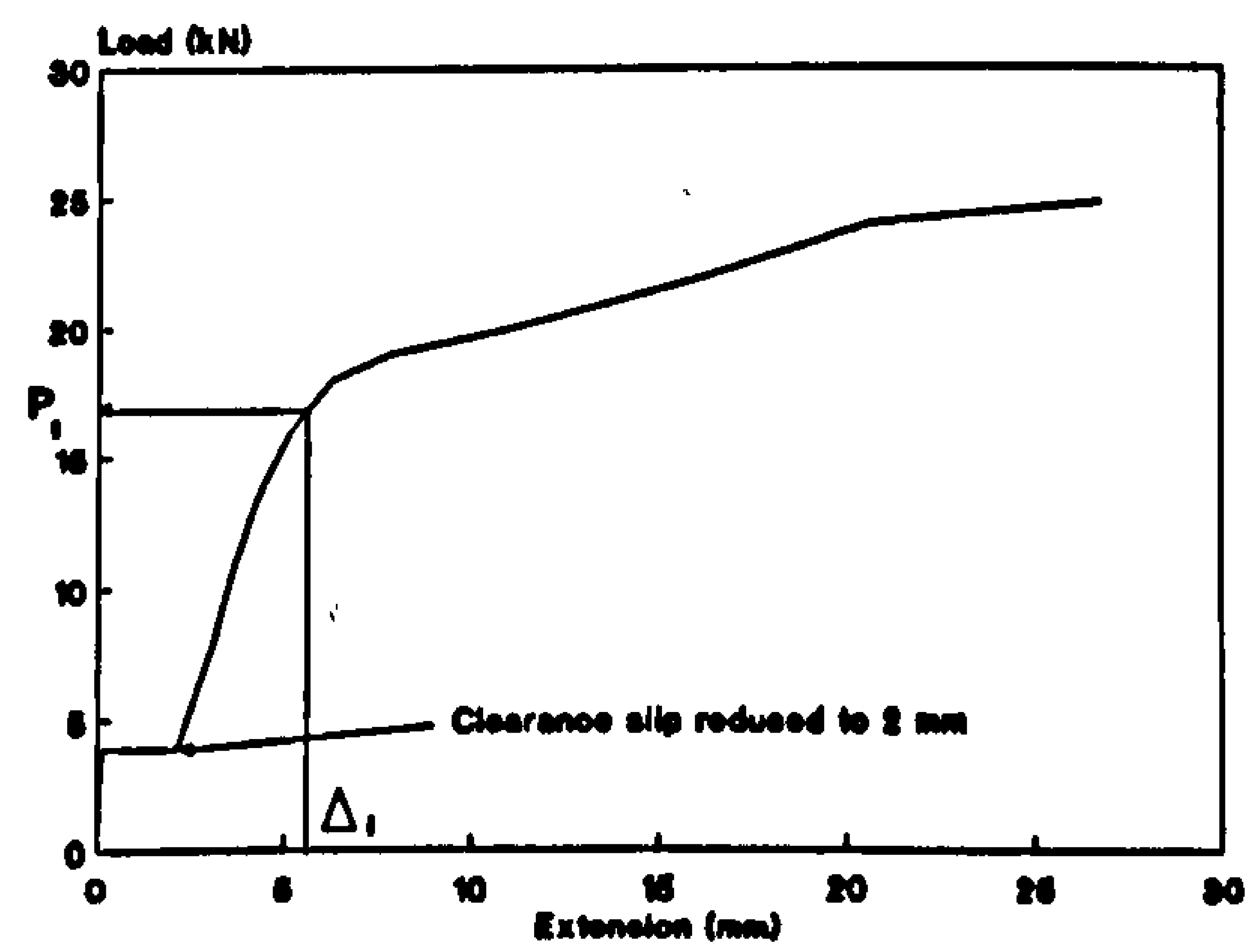


Fig. 10-10 : Equivalent lap data

vi) The calculated moment was then plotted against the calculated dial gauge movements, and the ratio of $\Delta_3/\Delta_1/\Delta_2$ was also noted in a separate data file.

vii) The rotation ϕ was then increased by an increment and steps (i) to (vi) were repeated.

ϕ was again increased by another increment and so on ... until such time that failure occurred or the whole cycles of the required increments were completed, whichever occurred first.

viii) The whole process (i to vii) was then repeated for a new COR.

The iterative procedure outlined above was carried out for a few number of points on the mesh until the results converged to that of the given criteria.

It was found that the given boundary conditions were best satisfied when the COR lay within the marked square shown in Fig. 10-8.

The coordinates of the centre of the square are :

(b/8 , a/3) with sides equal to b/12 .

Note that the ordinate of the centre of the square is the same as that of the elastic centre of rotation, while its abscissa is offset by 5b/24.

The radii of rotation, based on the actual test observations, are therefore given as:

$$\left.\begin{aligned} r_1 &= \sqrt{\left(\frac{2a}{3}\right)^2 + \left(\frac{b}{8}\right)^2} \\ r_2 &= \sqrt{\left(\frac{a}{3}\right)^2 + \left(\frac{b}{8}\right)^2} \\ r_3 &= \sqrt{\left(\frac{7b}{8}\right)^2 + \left(\frac{a}{3}\right)^2} \end{aligned}\right\} \rightarrow \left.\begin{aligned} r_1 &= 88.0\ mm \\ r_2 &= 46.3\ mm \\ r_3 &= 114.0\ mm \end{aligned}\right\}$$

The coordinates of the COR are : (with bolt 2 as the origin, see Fig. 10-8)

$$\left.\begin{aligned} x &= \frac{b}{8} \\ y &= \frac{a}{3} \end{aligned}\right\} \rightarrow \left.\begin{aligned} x &= 16.25 \\ y &= 43.3 \end{aligned}\right\}$$

Obviously P_3 is the critical force. Let $P = P_3$ then as shown for the elastic case :

$$M = \frac{P}{r_3}\sum_{i=1}^{3} r_i^2 \qquad = 0.201\,P$$

The predicted moment-rotation characteristics, is given as :

$$\phi = \frac{\Delta}{r_3} = \frac{\Delta}{114.0} \quad [Rads.] \qquad and \qquad M = 0.201\,P \quad [kNm]$$

(ϕ x 10^{-3} [Rads.], M [kN.m])

A	(0.0, 0.0)		A	(0.0, 0.0)
B	(4c/114.0, 0.201x4)		B	(6.32, 0.80)
C	[(4c + hole clearance)/114.0, 0.201x4]		C	(23.86, 0.80)
D	[(P_{bs}c + hole clearance)/114.0, 0.201xP_{bs}]		D	(49.91, 4.12)

It should be noted that by assuming the centre of rotation to be at the centre of the predicted square and then effectively applying the elastic criterion to calculate the moment of the resistance of the connection - the equilibrium conditions are not satisfied.

This may be contributed to the out of plane twisting of the top channel causing a shift of COR, opposite to the x - direction in Fig. 10-8.

It may also be of interest to mention that, rough estimates of the location of the COR at the time of failure fell within the triangular area confined by the elastic, plastic and predicted COR's (Fig. 10-8). That is for each bolt the direction of the hole distortion was noted. Using a set square, a line normal to this direction was drawn. The three lines met, or nearly met, within the triangular area mentioned above. This was obviously a very crude method, nevertheless an indication of the validity of the more complex procedures and calculations described so far.

Predicted moment-rotation characteristics based on the elastic, plastic and predicted centre of rotation based on test observations, are plotted in Fig. 10-11. It is interesting to note that the least moment capacity is that based on the predicted centre of rotation.

Although the actual test failed prematurely, but the results obtained are in good agreement with that of the converted lap data, also based on different centres of rotation.

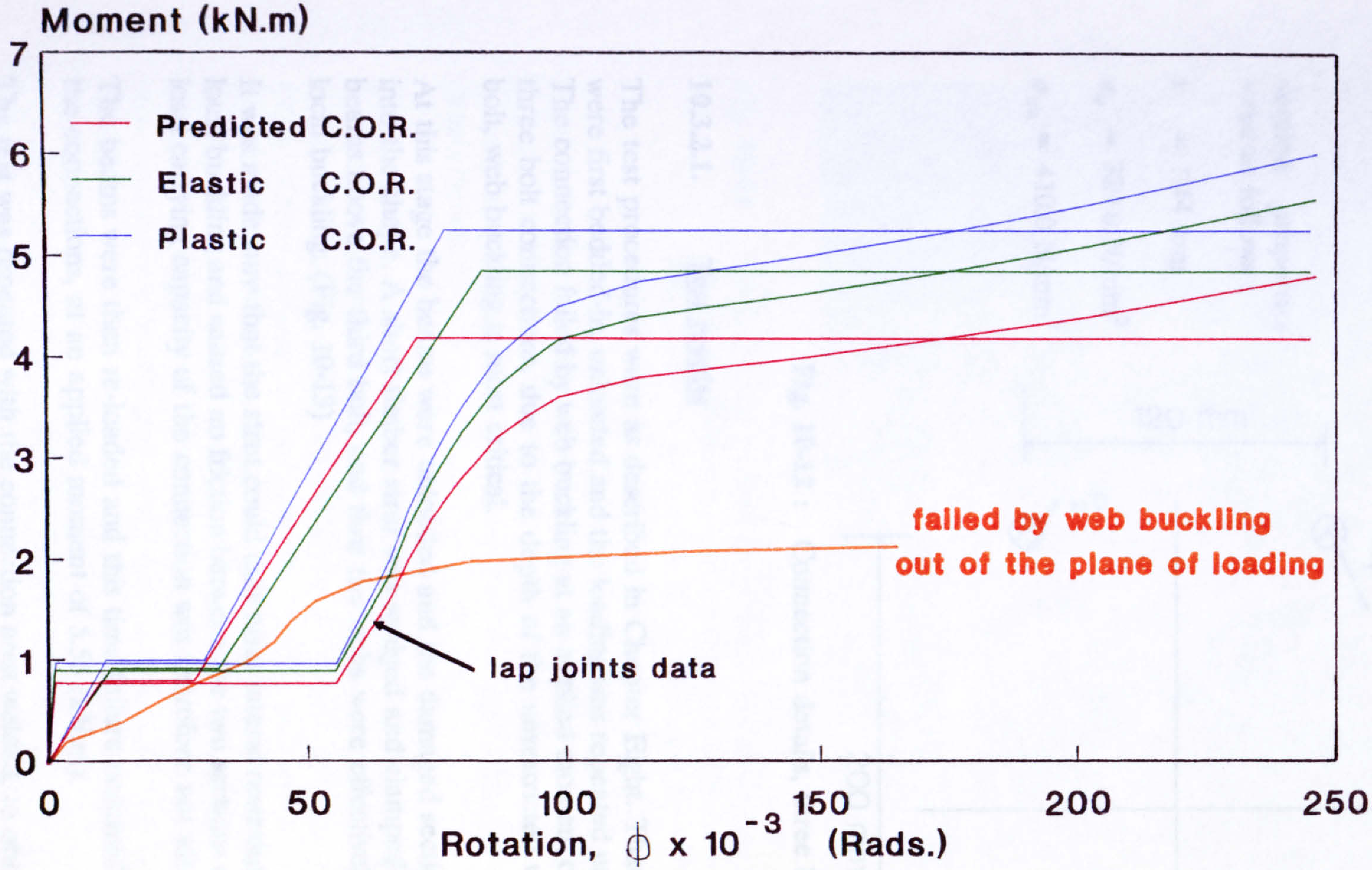


Fig. 10-11
Predicted moment-rotation characteristics based on different centres of rotation

10.3.2. The beam set up, three bolt connections

The connections details were as given in figure below :

Section properties were as follows :

$t \;\; = 1.44$ mm

$\sigma_y \; = 329.0$ N/mm^2

$\sigma_{ult} = 410.0$ N/mm^2

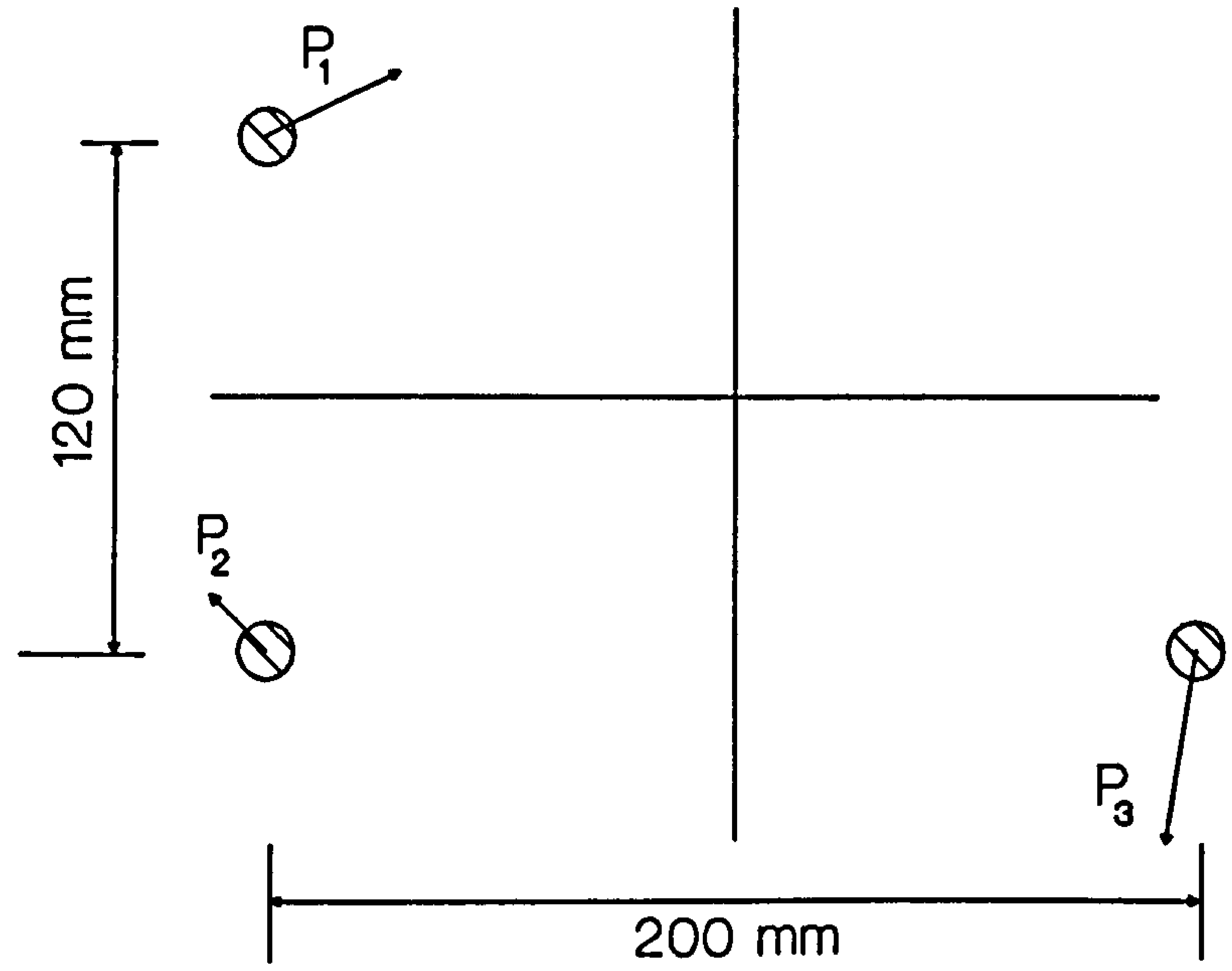


Fig. 10-12 : Connection details, three bolt beam set up.

10.3.2.1. Test results

The test procedures were as described in Chapter Eight. That is, the connections were first bedded-in, unloaded and the loading was repeated until failure occurred. The connection failed by web buckling at an applied moment of 4.50 (kN.m). With three bolt connections, due to the depth of the unrestricted web above the third bolt, web buckling is most critical.

At this stage the beams were unloaded and the damaged section was formed back into the shape. A short timber strut was wedged and clamped in between the two beams above the third bolt, and thus the webs were effectively restrained against local buckling. (Fig. 10-13)

It was made sure that the strut could only offer lateral restraint to the webs against local buckling and caused no friction between the two sections of a connection. The load carrying capacity of the connection was therefore not affected in any way.

The beams were then re-loaded and this time failure occurred by sheet bearing in the connections, at an applied moment of 5.55 (kN.m).

The test was repeated with the connection area welded, to obtain the deflection of the uniform beam and differentiate the deflection due to connection rotation, as was described in Chapter Eight.

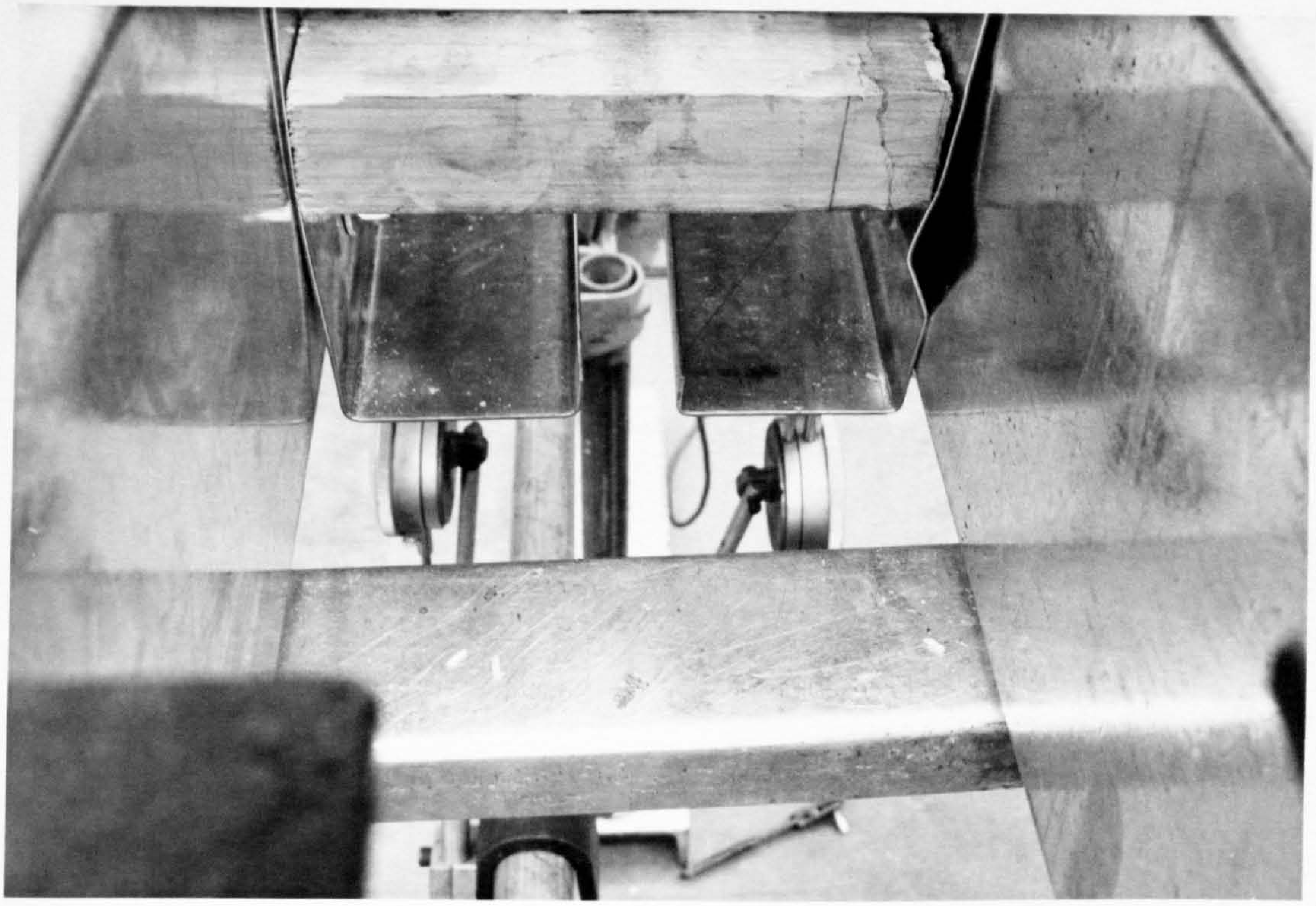


Fig. 10-13 : Timber strut wedged in between the two beams, to avoid local web buckling. (View over and into the space between the two beams, see Fig. 10-15 for an overall view of the test set up)

Load-deflection readings at the mid span, are plotted in Fig. 10-14. As with the four and two bolt beam connections, the mid and one third span data were in remarkably good agreement. Different components of beam deflection are noted in this figure.

Earlier in Chapter Eight, it was mentioned that with all three bolt connection tests, in the beam arrangement, a system of theodolites was set up to see if it was possible to trace the rotation of such connections - and hence determine the location of their COR. Other schemes to measure the relative movements of the two channels, bolted back to back, with respect to each other were also studied. But none were considered to be practical. One such scheme had already been tried. That is, inextensible wires were attached to each plate of the connection and their movements were monitored using dial gauges. But this did not prove to be successful.

A remote system of measurement such as theodolites was thought to be the best solution possible. The scheme adopted is described here.

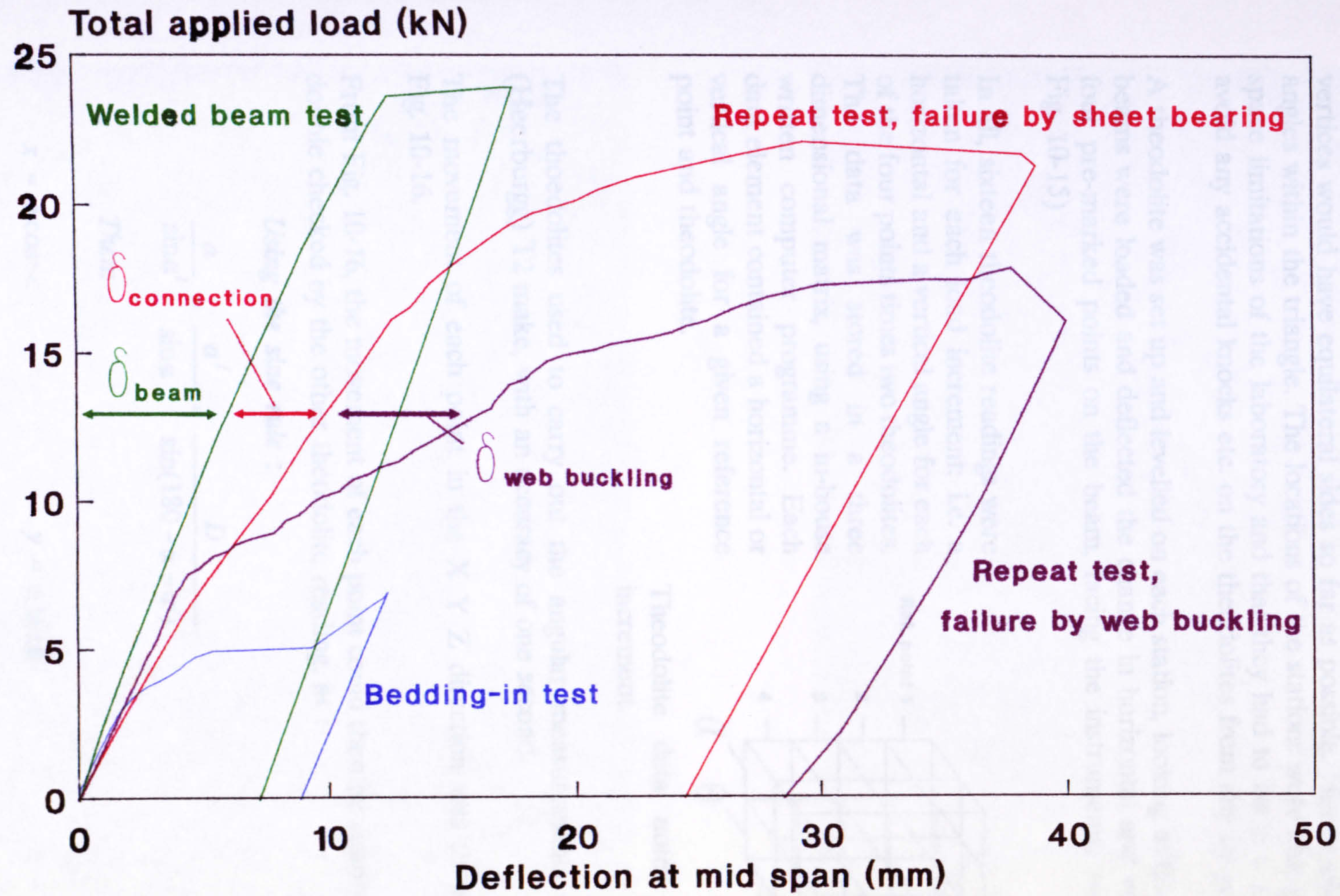


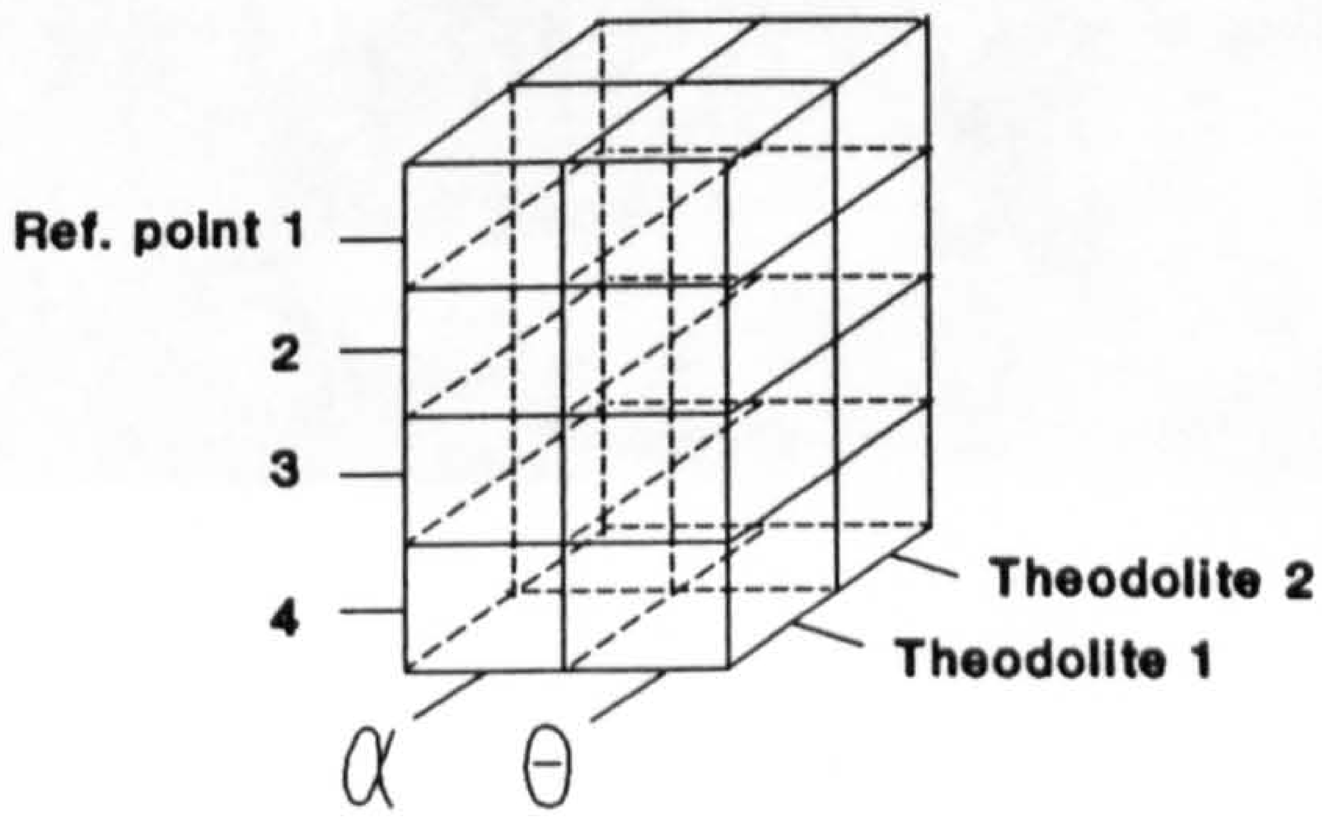
Fig. 10-14
Load v. deflection at mid span

Theodolite set up

Two theodolite stations were set up at a distance away from the test rig. The locations of the stations were chosen such that an imaginary triangle in space with the two stations and the centre point of the beam facing the theodolites, at its vertices would have equilateral sides so far as possible. Hence evading any sharp angles within the triangle. The locations of the stations were also governed by the space limitations of the laboratory and that they had to be in a secluded area to avoid any accidental knocks etc. on the theodolites from any by-passers.

A theodolite was set up and levelled on each station, looking at the test rig. As the beams were loaded and deflected the change in horizontal and vertical angles of four pre-marked points on the beam, facing the instruments, were noted. (See Fig. 10-15)

In all, sixteen theodolite readings were taken for each load increment; i.e. a horizontal and a vertical angle for each of the four points times two theodolites. The data was stored in a three dimensional matrix, using a in-house written computer programme. Each data element contained a horizontal or vertical angle for a given reference point and theodolite.



Theodolite data noted for one load increment.

The thoedolites used to carry out the angular measurements were of WILD (Heerburgg) T2 make, with an accuracy of one second.

The movement of each point in the X Y Z direction was then as depicted in Fig. 10-16.

From Fig. 10-16, the movement of each point could then be simply calculated, and double checked by the other theodolite reading, as :

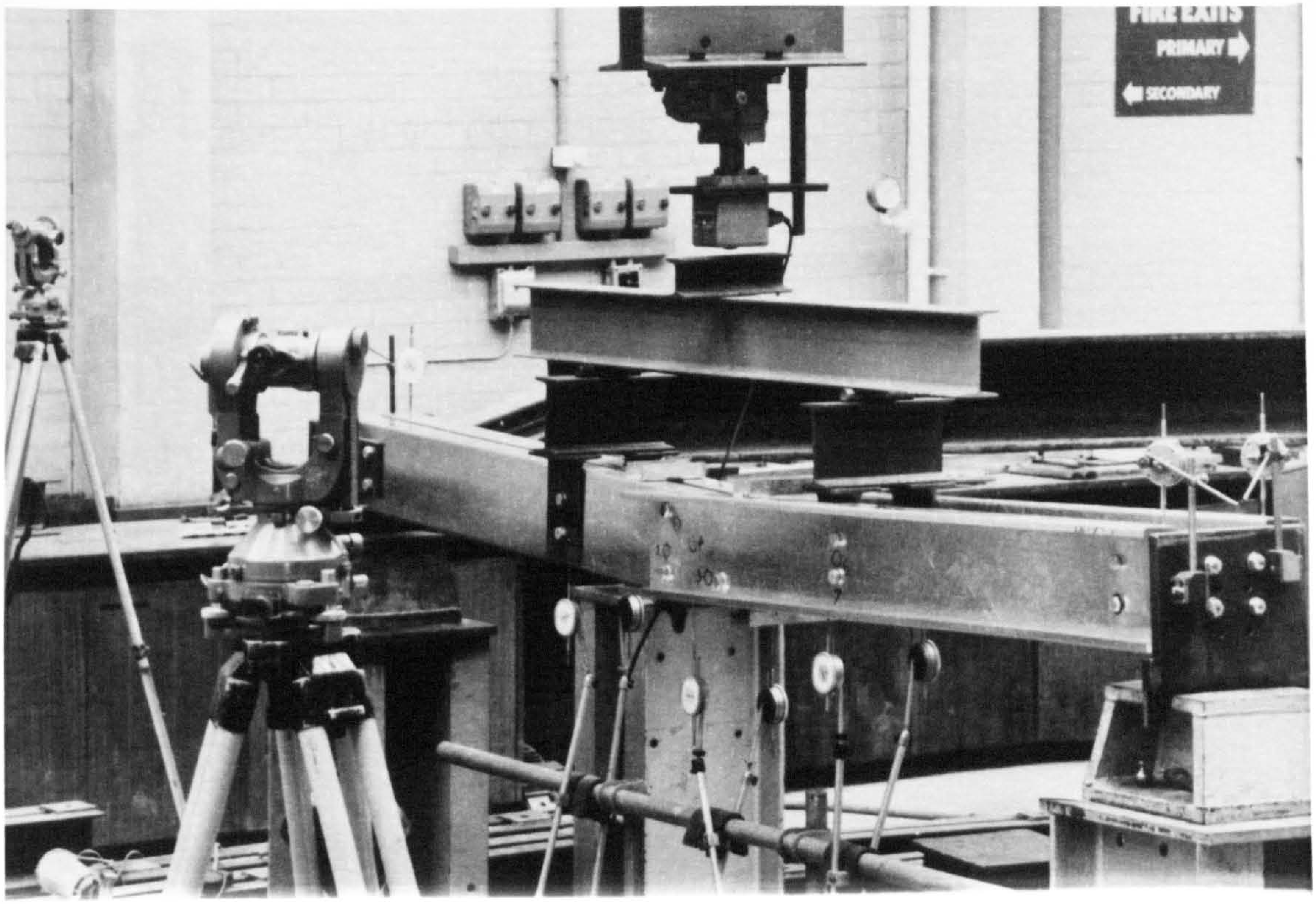
Using the sine rule :

$$\frac{a}{\sin\alpha'} = \frac{a'}{\sin\alpha} = \frac{D}{\sin(180-\alpha-\alpha')}$$

Then:

$$x = a\cos\alpha \qquad y = a\tan\theta \qquad z = a\sin\alpha$$

$$= D - a'\cos\alpha' \qquad = a'\tan\theta' \qquad = a'\sin\alpha'$$



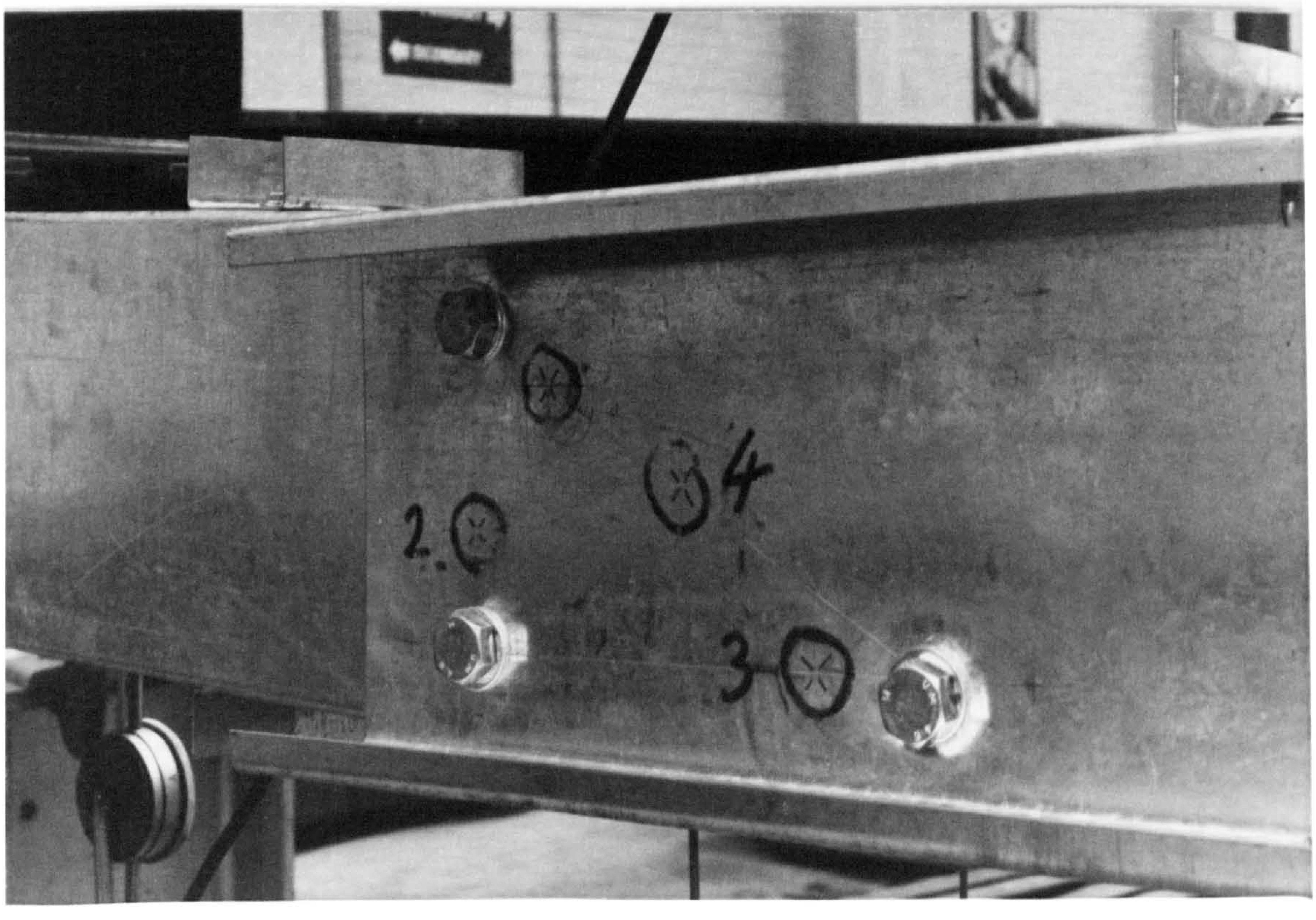


Fig. 10-15: Three bolt channel beams with the theodolite set up

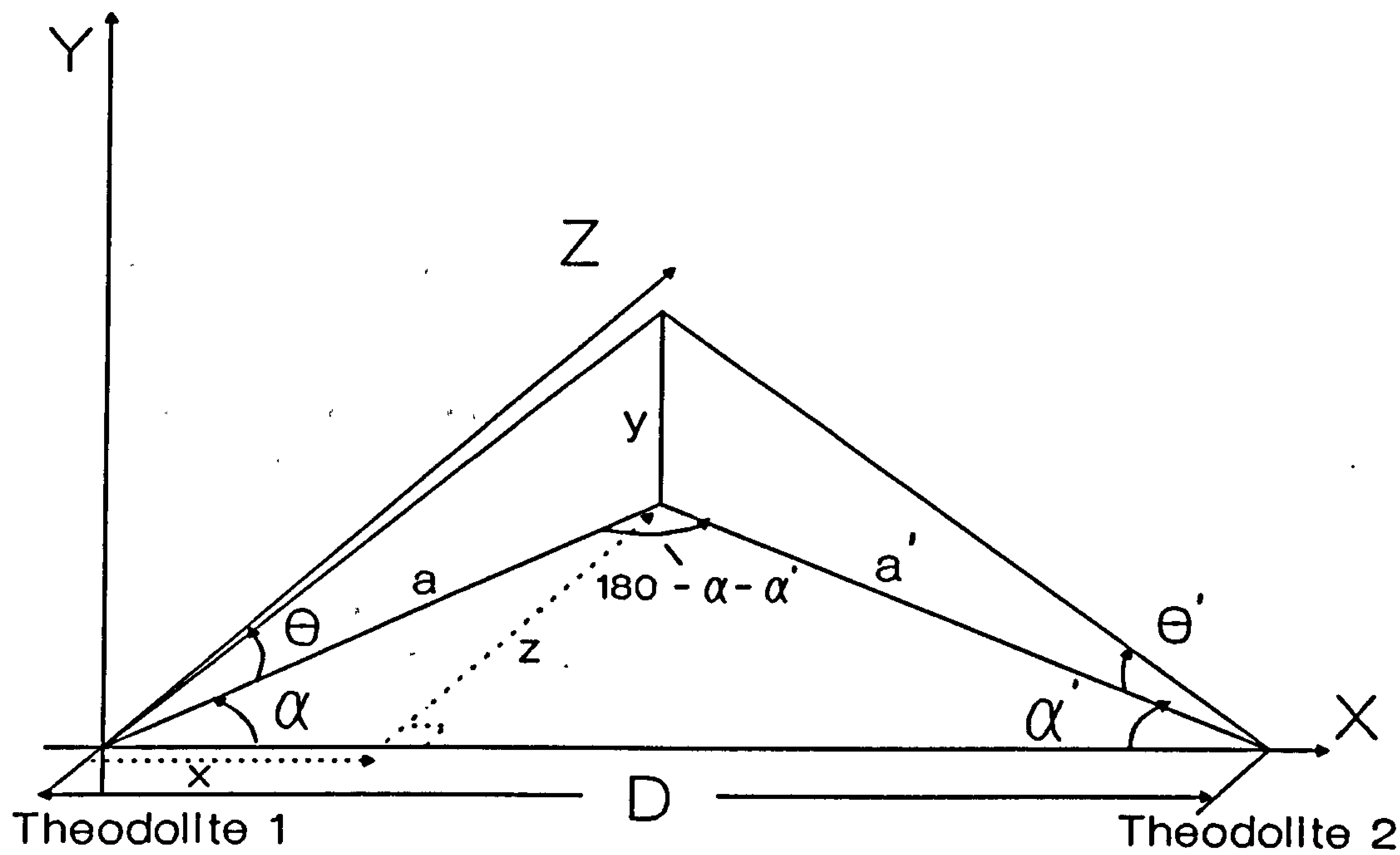


Fig. 10-16 : Defining a point in 3-D cartesian coordinates.

The distance between the two theodolites D, was measured accurately using a steel tape. The recorded reading was adjusted for temperature difference between the laboratory and that specified for the tape.

The deflections recorded in the X-Y plane for the bolted connection (repeat loading, failure in sheet bearing) were as shown in Fig. 10-17, below.

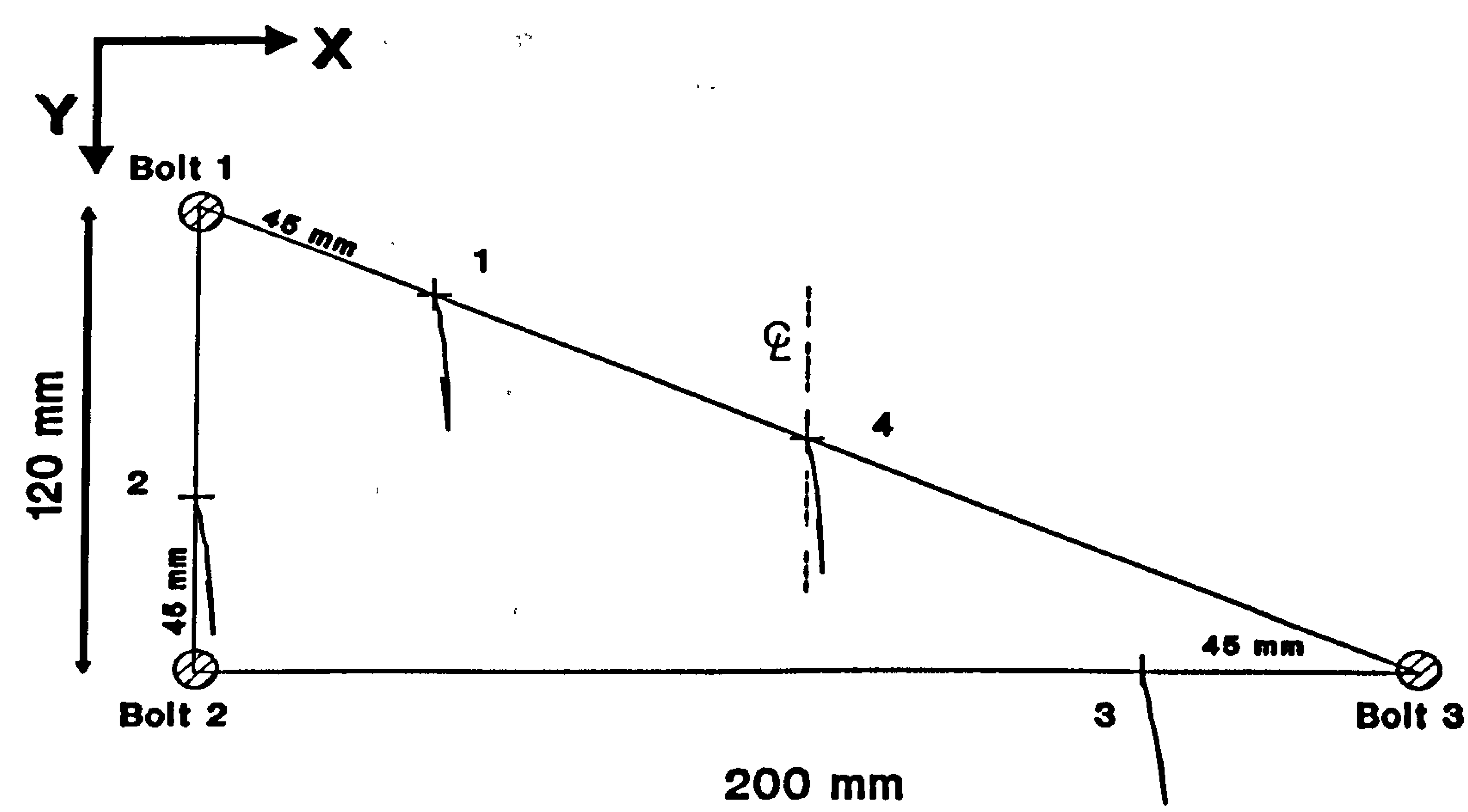


Fig. 10-17 : Deflection patterns, for the four pre-marked points, in the X-Y plane.

It can be seen that all the four points moved along parallel lines.

There is no negative element of movement in the x direction, which would have to occur for some, or at least one of the four points, if the COR was to be within the triangle of bolts as expected.

At first instance this would suggest that the connection rotated about a distant point, i.e. the supports.

A closer examination of the results however, revealed that the theodolites had followed the overall deflection of the beams and not the rotation of the connection. Hence the four parallel lines of deflection in the X-Y plane.

Although the system used proved to be very accurate in monitoring the movements of the pre-set points in the connection area, the results obtained were not what was originally intended.

Theodolite point 4 was marked precisely at the mid span of the beams (Fig. 10-15). The vertical movement, i.e. in the Y-direction, recorded for this point is compared with that of the dial gauge readings at the mid span in Fig. 10-18.

It can be seen that two absolutely independent systems (i.e. dial gauges and theodolites) gave literally identical results, which is an indication of the accuracy of the test results.

10.3.2.2. Predicted moment-rotation characteristics

In this section the predicted moment-rotation characteristics, depending on different assumptions (i.e. elastic and plastic), are obtained and then used to compare with the actual characteristics due to connection rotation.

Only the repeat loading case will be considered, since the bedding-in test is not of practical importance.

The bearing strength and joint flexibility of the connections are obtained as follows :

$$P_{bs} = \alpha \,.\, d.\, t.\, \sigma_{ult} = \alpha \times 16 \times 1.44 \times 0.410 = 9.45\,\alpha$$

$$\alpha = k_1.k_2.k_3.k_3.k_4.k_5.k_6.k_7$$

$$= k_2 \qquad (k_1 \text{ and } k_3 \text{ to } k_7 \text{ all being equal to 1})$$

$$\therefore \quad P_{bs} = 9.45\,(1.9 + 0.2\,t) = 20.7 \text{ kN}$$

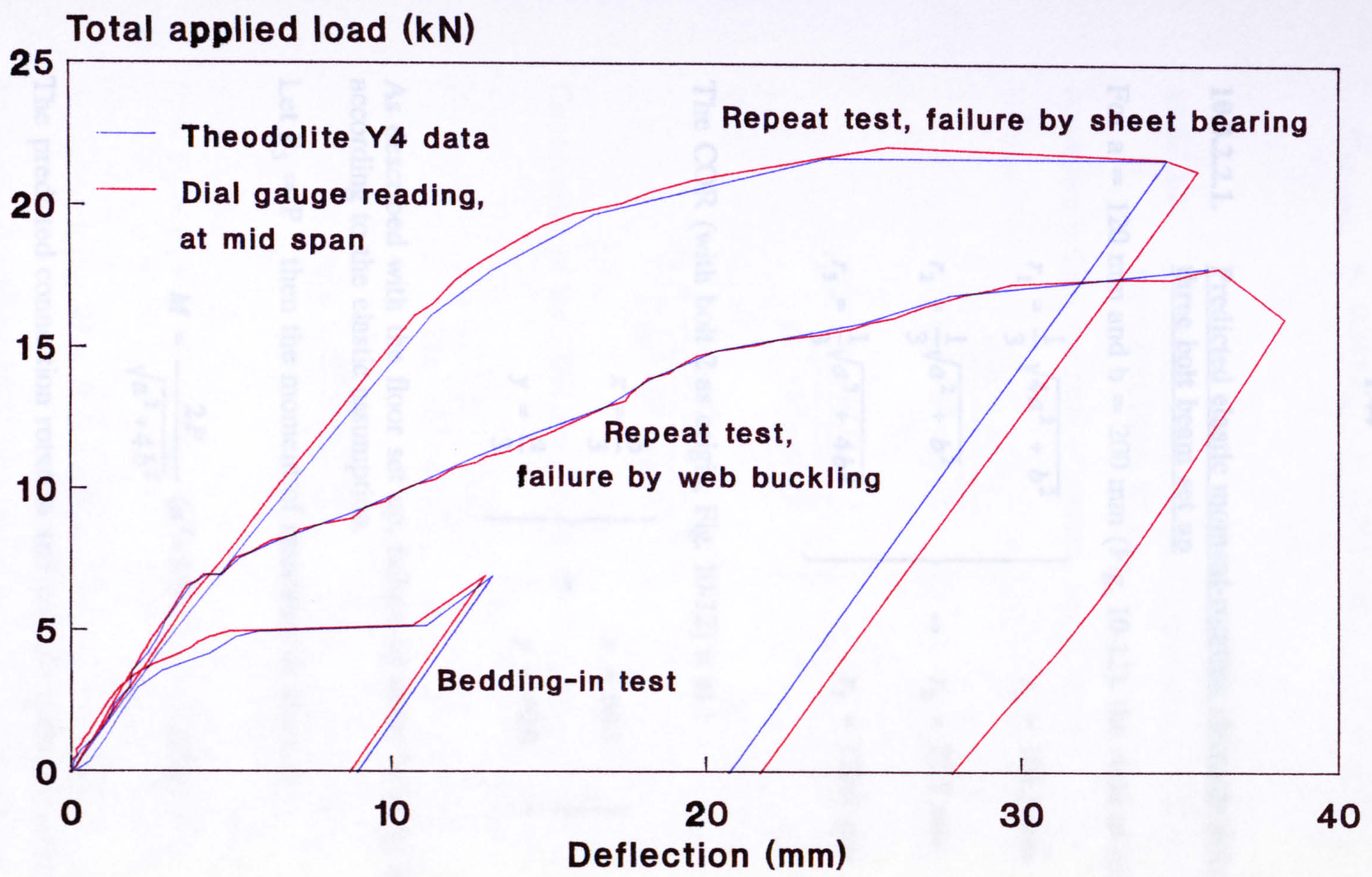


Fig. 10-18

Dial gauge reading (beam facing the theodolites) compared to vertical component of theodolite point 4

$$c = 5n\left(\frac{10}{t_1} + \frac{10}{t_2} - 2\right) \times 10^{-3}$$

$$= 5 \times 3\left(\frac{10}{1.44} \times 2 - 2\right) \times 10^{-3} = 0.178 \qquad \left(\frac{mm}{kN}\right)$$

10.3.2.2.1. <u>Predicted elastic moment-rotation characteristics, three bolt beam set up</u>

For a = 120 mm and b = 200 mm (Fig. 10-12), the radii of rotation are given as :

$$\left.\begin{aligned} r_1 &= \frac{1}{3}\sqrt{4a^2 + b^2} \\ r_2 &= \frac{1}{3}\sqrt{a^2 + b^2} \\ r_3 &= \frac{1}{3}\sqrt{a^2 + 4b^2} \end{aligned}\right\} \rightarrow \left.\begin{aligned} r_1 &= 104.1\ mm \\ r_2 &= 77.7\ mm \\ r_3 &= 139.2\ mm \end{aligned}\right\}$$

The COR (with bolt 2 as origin, Fig. 10-12) is at :

$$\left.\begin{aligned} x &= \frac{b}{3} \\ y &= \frac{a}{3} \end{aligned}\right\} \rightarrow \left.\begin{aligned} x &= 66.7 \\ y &= 40.0 \end{aligned}\right\}$$

As described with the floor set up, failure by sheet bearing first occurs at bolt 3 according to the elastic assumption.

Let $P_3 = P$ then the moment of resistance is given as :

$$M = \frac{2P}{\sqrt{a^2 + 4b^2}}(a^2 + b^2) = 0.261\ P$$

The predicted connection rotation and moment capacity at failure are therefore :

$$\phi = \frac{P_{bs}\, c}{r_3} = \frac{20.7 \times 0.178}{139.2} = 26.50 \times 10^{-3} \qquad (Rads.)$$

$$M_{ult.\,elastic} = 0.261\, P_{bs} = 0.261 \times 20.7 = 5.40 \qquad (kNm)$$

10.3.2.2.2. Predicted plastic moment-rotation characteristics, three bolt beam set up

As shown before, the radii of rotation are given as :

$$\left.\begin{aligned} r_1 &= \frac{a\,(a + b/\sqrt{3})}{\sqrt{a^2+b^2+ab\sqrt{3}}} \\ r_2 &= \frac{ab/\sqrt{3}}{\sqrt{a^2+b^2+ab\sqrt{3}}} \\ r_3 &= \frac{b\,(a/\sqrt{3} + b)}{\sqrt{a^2+b^2+ab\sqrt{3}}} \end{aligned}\right\} \rightarrow \left.\begin{aligned} r_1 &= 91.2\ mm \\ r_2 &= 44.7\ mm \\ r_3 &= 173.9\ mm \end{aligned}\right\}$$

Coordinates of the COR are : (with bolt 2 as origin, Fig. 10-12)

$$\left.\begin{aligned} x &= \frac{ab\,(a + b/\sqrt{3})}{2(a^2+b^2+ab\sqrt{3})} \\ y &= \frac{ab\,(a/\sqrt{3}+b)}{2(a^2+b^2+ab\sqrt{3})} \end{aligned}\right\} \rightarrow \left.\begin{aligned} x &= 29.4 \\ y &= 33.7 \end{aligned}\right\}$$

$P_1 = P_2 = P_3 = P$. Hence the moment of resistance is given as :

$$M = P\,\sqrt{a^2+b^2+ab\sqrt{3}} \qquad = 0.310\, P$$

As it was described with the three bolt floor set up, according to the plastic stipulation for connection failure, the predicted connection rotation should be based on $r_{2\,plastic}$. Yet if this is heeded, the resulting moment-rotation characteristics will be far too flexible. It is therefore not known which bolt lever arm is critical. Again, it appears that the plastic assumption is not justified. For results to be comparable to the elastic case $r_{3\,plastic}$ is used to predict the connection rotation.

The predicted connection rotation and moment capacity at failure, according to the plastic assumption, are therefore :

$$\phi = \frac{P_{bs}\,c}{r_{3\,plastic}} = \frac{20.7 \times 0.178}{173.9} = 21.19 \times 10^{-3} \qquad (Rads.)$$

$$M_p = 0.310\,P_{bs} = 0.310 \times 20.7 = 6.42 \qquad (kNm)$$

Predicted moment capacity using the bearing strength equation in Annex A

For the given test parameters, the sheet bearing strength as given by Annex A is equal to :

$$P_{ult} = 2.5\ d\ t\ \sigma_{ult} = 2.5 \times 16 \times 1.44 \times 0.410 = 23.6\ \text{kN.}$$

The predicted moment capacities, based on elastic and plastic assumptions, are simply obtained as :

$$M_{elastic} = 0.261\ P_{ult} = 6.16\ \text{kNm}$$

$$M_p = 0.310\ P_{ult} = 7.32\ \text{kNm}$$

That is, using either assumption, the predicted moment capacity at failure is unsafe. (cf. to 5.55 kNm, test result)

10.3.2.2.3. Moment-rotation characteristics based on the predicted COR, three bolt beam set up

With the three bolt floor set up it was suggested that the centre of rotation lies within a square, the centre of which lay at coordinates (b/8, a/3), with sides equal to b/12. This is thought to be particular to the floor test arrangement, due to the lifting of the web of the top channel out of plane. Applying the same criterion to the beam arrangement does not satisfy the conditions of equilibrium. However, just as a matter of interest and for completeness, the predicted moment-rotation characteristics, assuming the COR to be at the centre of the suggested square are as follows :

The radii of rotation are given as:

$$\left.\begin{aligned} r_1 &= \sqrt{\left(\frac{2a}{3}\right)^2 + \left(\frac{b}{8}\right)^2} \\ r_2 &= \sqrt{\left(\frac{a}{3}\right)^2 + \left(\frac{b}{8}\right)^2} \\ r_3 &= \sqrt{\left(\frac{7b}{8}\right)^2 + \left(\frac{a}{3}\right)^2} \end{aligned}\right\} \rightarrow \left.\begin{aligned} r_1 &= 83.8\ mm \\ r_2 &= 47.2\ mm \\ r_3 &= 179.5\ mm \end{aligned}\right\}$$

The coordinates of the COR are :

$$\left.\begin{aligned} x &= \frac{b}{8} \\ y &= \frac{a}{3} \end{aligned}\right\} \rightarrow \left.\begin{aligned} x &= 25 \\ y &= 40 \end{aligned}\right\}$$

P_3 is the critical force. So let $P = P_3$ then :

$$M = \frac{P}{r_3} \sum_{i=1}^{3} r_i^2 \qquad = 0.231\, P$$

The predicted connection rotation and moment capacity at failure, are therefore :

$$\phi = \frac{P_{bs}\, c}{r_3} = \frac{20.7 \times 0.178}{179.5} = 20.53 \times 10^{-3} \qquad (Rads.)$$

$$M_p = 0.231\ P_{bs} = 0.231 \times 20.7 = 4.78 \qquad (kNm)$$

10.3.2.3. Comparison of the actual and predicted moment-rotation characteristics

The moment-rotation characteristics due to connection rotation is plotted and compared against the predicted behaviours according to the elastic and plastic assumptions, in Fig. 10-19.

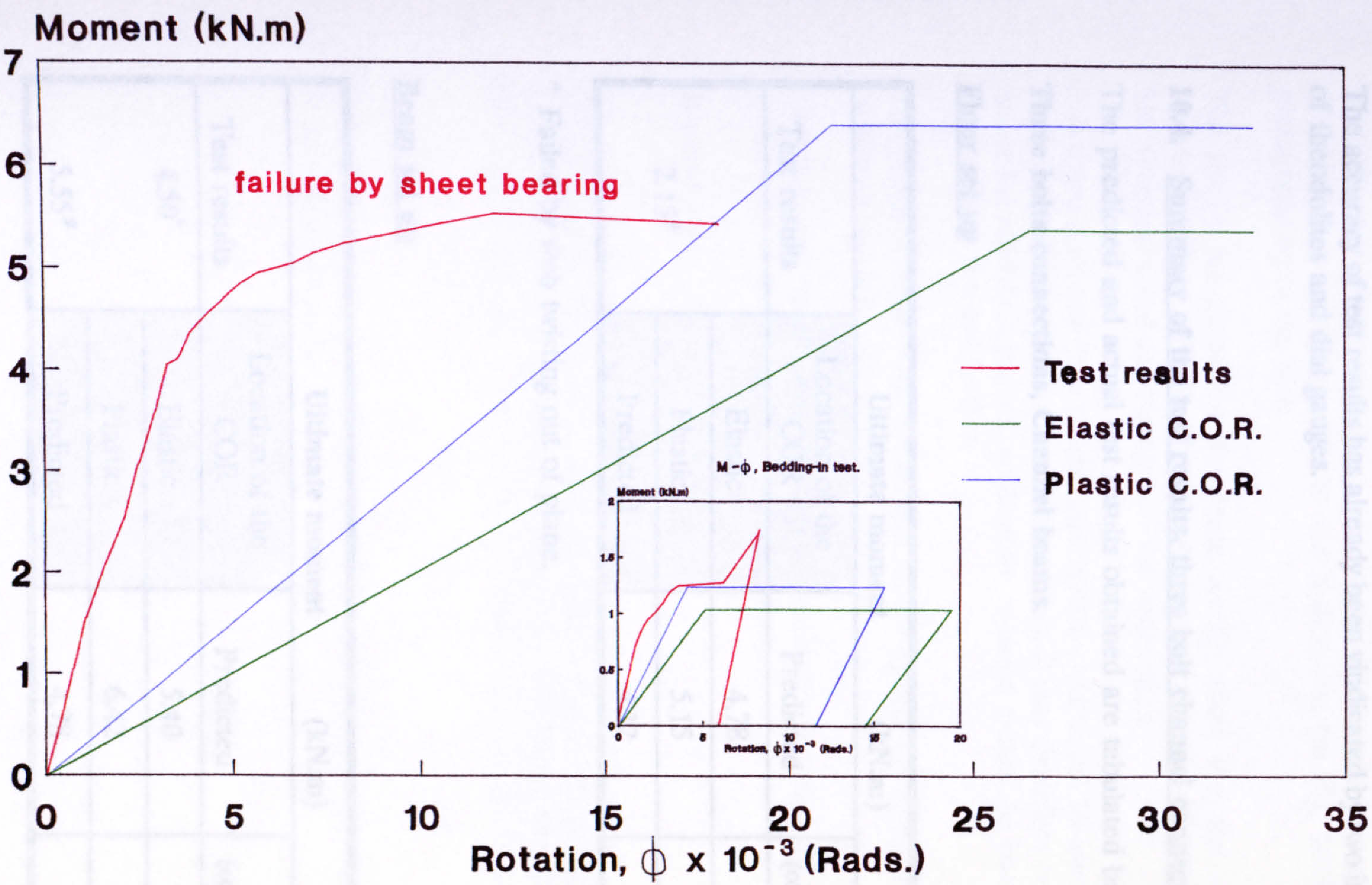


Fig. 10-19

Actual and predicted moment-rotation characteristics of three bolt beam set

The ultimate and slip loads have been predicted extremely well, while the connection appears to be stiffer than that predicted. This will be discussed in more detail in Chapter Twelve. The stiffness of the converted lap data is nevertheless in good agreement with that predicted.

The accuracy of test results has already been vindicated by two independent systems of theodolites and dial gauges.

10.4. Summary of the test results, three bolt channel connections

The predicted and actual test results obtained are tabulated below.

Three bolts connections, Channel beams.

Floor set up

Ultimate moment (kN.m)			
Test results	Location of the COR	Predicted	Lap data (converted to M-ϕ)
2.15^{+}	Elastic	4.78	5.58
	Plastic	5.15	6.01
	Predicted	4.12	4.82

$^{+}$ Failed by web twisting out of plane.

Beam set up

Ultimate moment (kN.m)			
Test results	Location of the COR	Predicted	Lap data (converted to M-ϕ)
4.50^{*}	Elastic	5.40	6.26
	Plastic	6.42	7.44
$5.55^{\#}$	Predicted	4.78	5.54

. Failed by web buckling.

$^{\#}$ connection failure, by sheet bearing.

10.5. Conclusions

From what has been mentioned and discussed in this chapter, the following conclusions may be drawn:

With the three bolt floor set up, the centre of rotation was offset from the Elastic COR by 5b/24 mm, opposite to the X-direction (in Fig. 10-8) due to the out of plane twisting of the web.

The results tabulated for the beam set up, show that the ultimate moment of resistance is best estimated by the elastic assumption. In fact the plastic assumption has over-estimated the strength of the connection.

It has been proven that plastic assumption contradicts its own definition, when it comes to estimating the connection rotation. It may therefore be concluded that, for unsymmetrical bolt groups, connection rotation should be based upon the largest lever arm in the connection.

In a group of three bolts, connection failure by sheet bearing first occurs at the bolt furthest away from the centre of rotation.

Therefore the plastic assumption is not valid in estimating the strength or stiffness of three bolts connections in cold-form steel.

In summary :

The Strength, Stiffness and location of the Centre Of Rotation of Three bolt connections in cold formed steel, should be based upon the Elastic assumption.

In the following chapters the predicted characteristics of three bolt connections, will be calculated as such.

Chapter Eleven

Comparison of two, three and four bolt connections test results

11. Comparison of two, three and four bolt connections test results

Summary

In this chapter a comparison between the results obtained for two, three and four bolt connections is carried out.

The *bearing strength per bolt* for each of the full moment connection tests is calculated. The values obtained are transformed into *bearing strength per mm sheet thickness*, and thus the characteristic bearing strength of all the moment connection tests, independent of sheet thickness and connection configuration is obtained. This value is then compared to that predicted by various design equations.

First however, the load-deflection behaviour of welded beams for the three beam set up tests are compared.

11.1. Load-deflection characteristics of the welded beams

In Chapter Eight it was explained that, with the beam set up, the beam flexure component of deflection was taken account of by welding round the connection area and re-loading the beams. The beam component of the deflection was then deducted from the total deflection of the bolted connection, and thus the deflection due to connection rotation was obtained, as depicted in Fig. 8-6 (page 167).

The resulted sections, i.e. the welded beams, could essentially be regarded as identical uniform beams irrespective of the bolt arrangement. Hence, it is interesting to compare the load-deflection characteristics of the welded beam tests.

The total applied load is plotted against the mid span deflection for each case on the same axes in Fig. 11-1. It is seen that the characteristics obtained are literally identical.

This is a further indication of the accuracy of tests procedures and results obtained.

11.2. Ultimate bearing strength per bolt, per mm sheet thickness

The ultimate bearing strength of two, three and four bolt groups is defined, in terms of their connection dimensions, as follows :

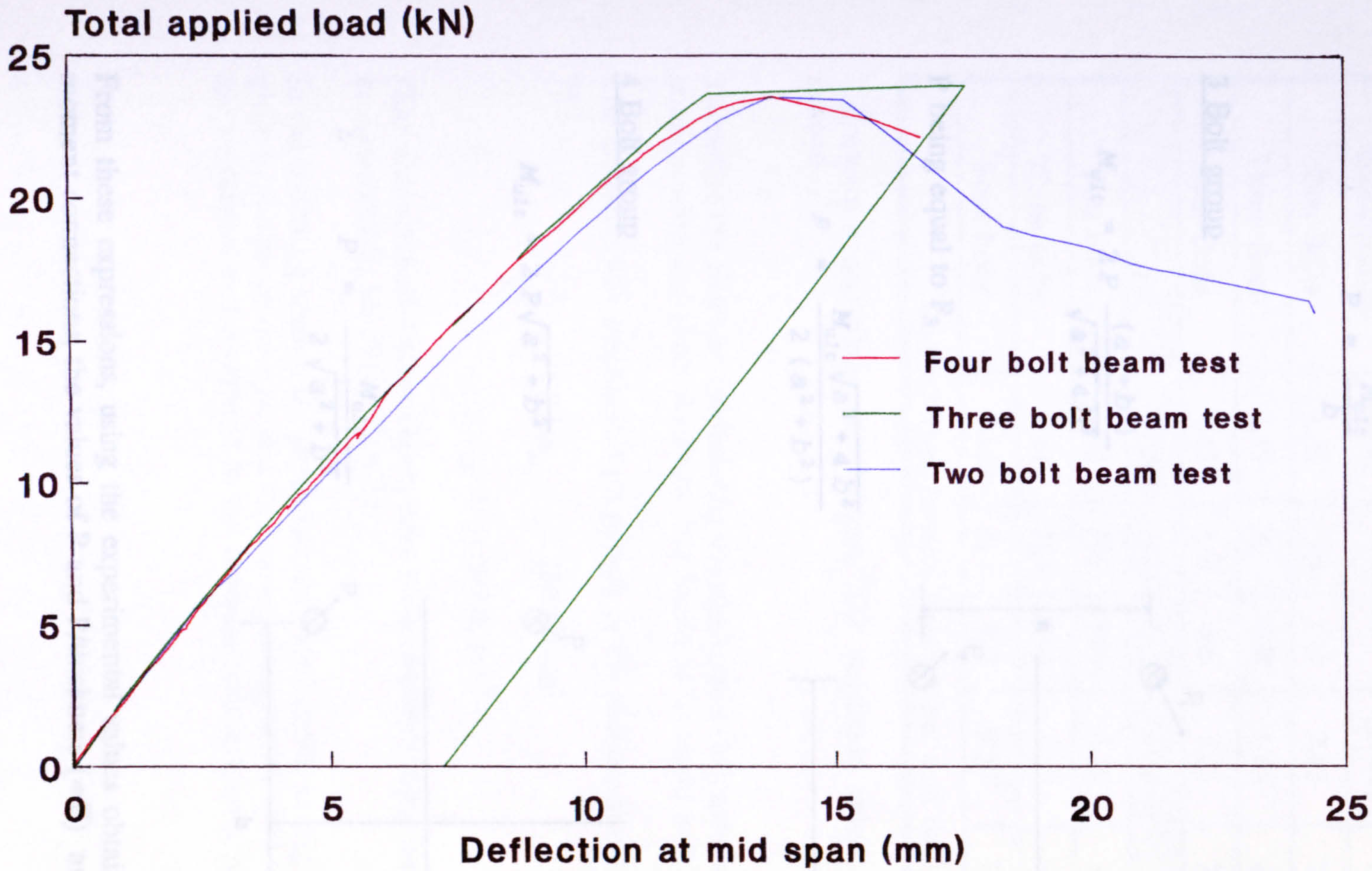
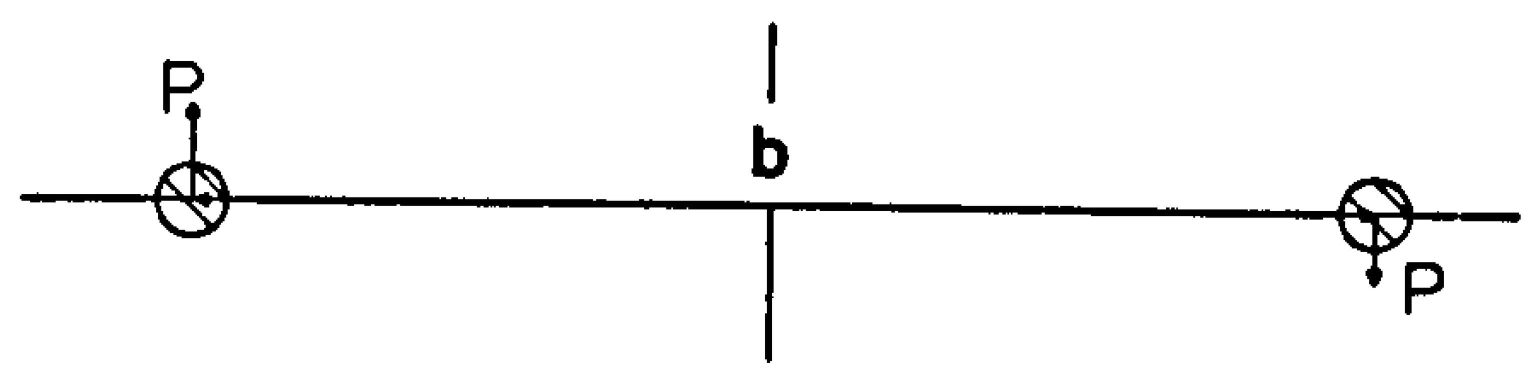


Fig. 11-1
Comparison of welded channel beams, full moment connections, beam set up

2 Bolt group

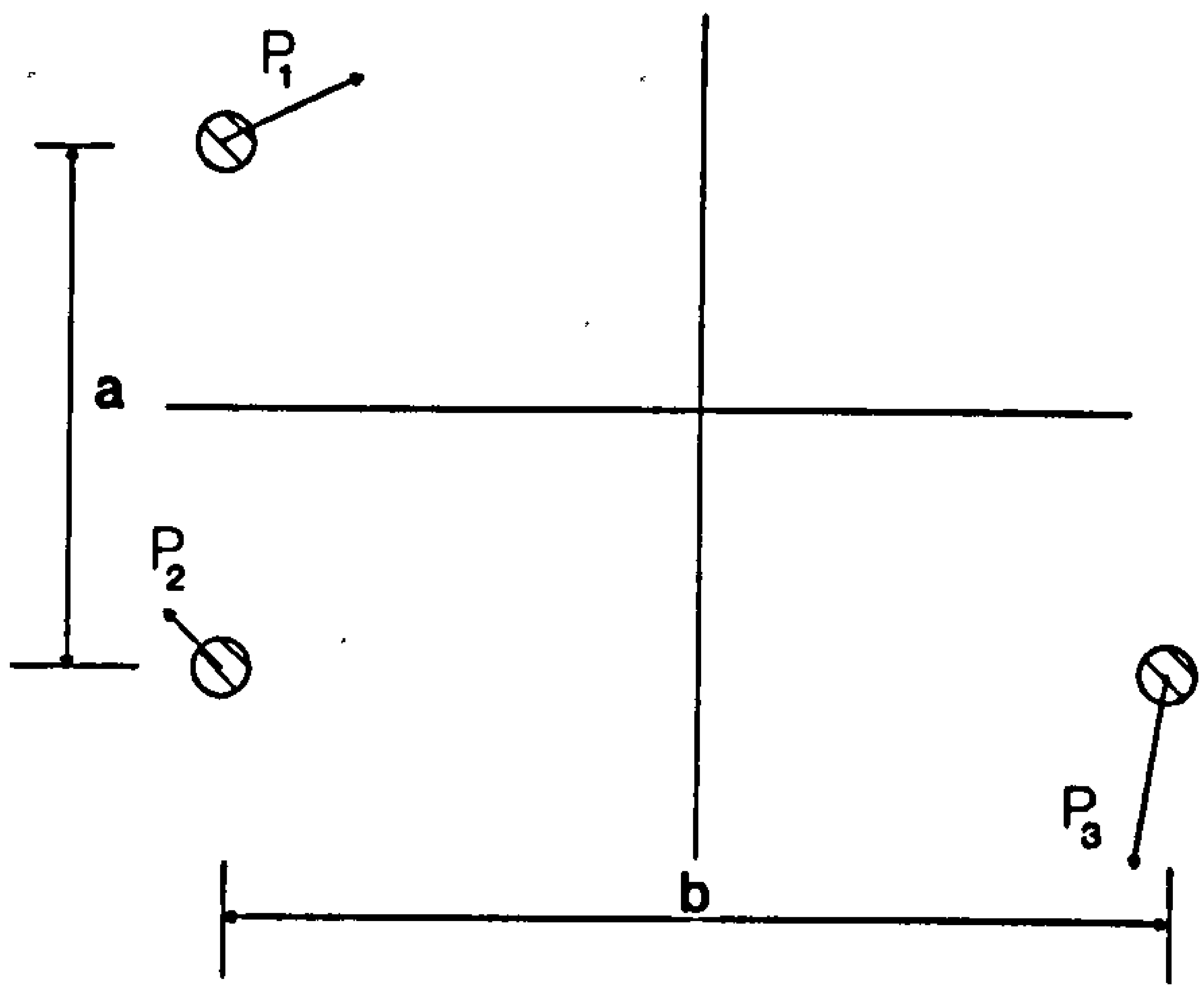
$M_{ult} = P \cdot b$



$$\therefore \quad P = \frac{M_{ult}}{b}$$

3 Bolt group

$$M_{ult} = 2P \frac{(a^2+b^2)}{\sqrt{a^2+4b^2}}$$

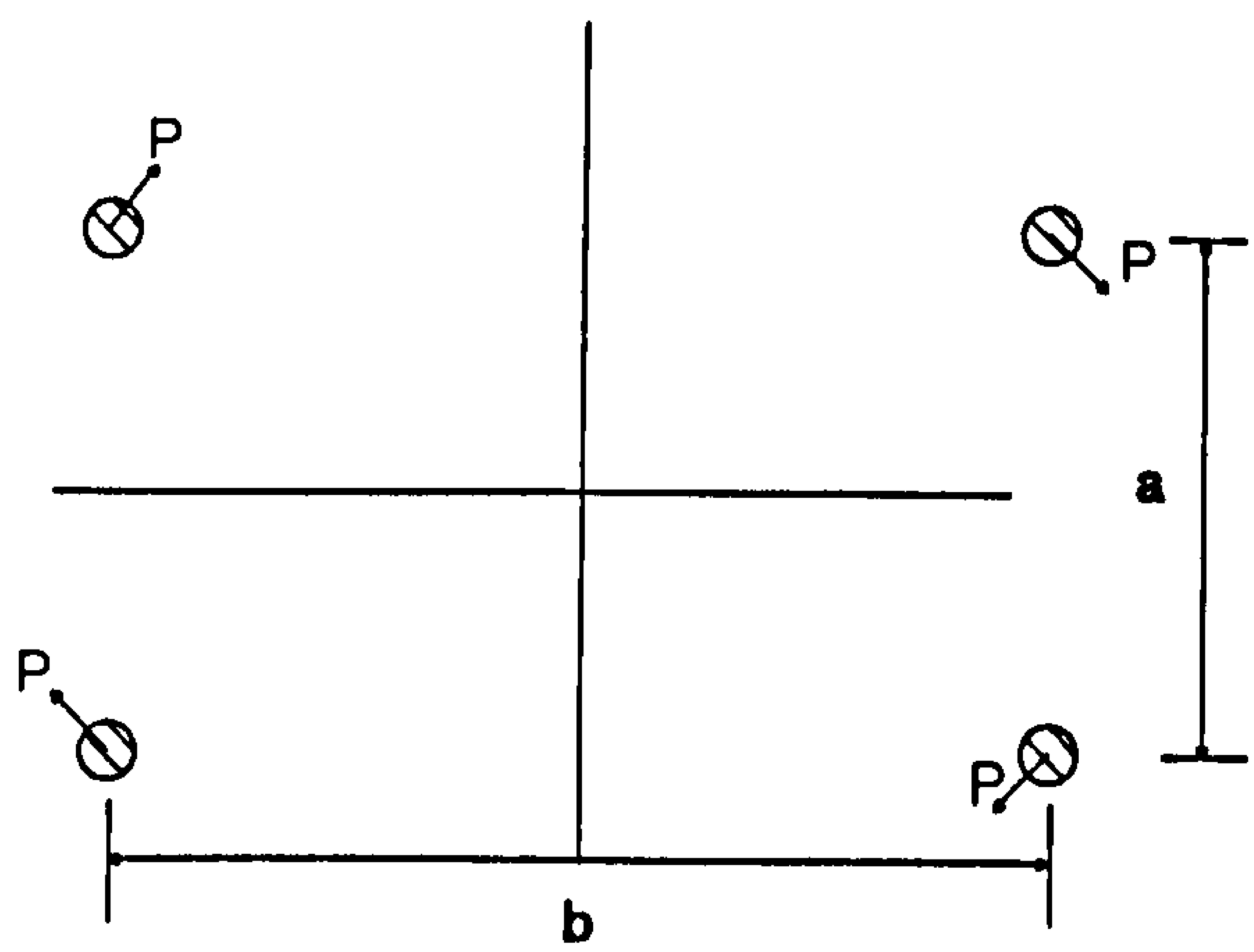


P being equal to P_3

$$\therefore \quad P = \frac{M_{ult}\sqrt{a^2+4b^2}}{2\,(a^2+b^2)}$$

4 Bolt group

$$M_{ult} = 2P\sqrt{a^2+b^2}$$



$$\therefore \quad P = \frac{M_{ult}}{2\sqrt{a^2+b^2}}$$

From these expressions, using the experimental values obtained for M_{ult} for all moment connections, the values of P and P/thickness (=F) are tabulated below.

Connection configuration	Floor set up a = b = 130 mm					
	t	σ_y	σ_{ult}	M_{ult}	P	F
	(mm)	(N/mm^2)		(kNm)	(kN)	(kN/mm)
Two Bolts	1.42	299	398	2.83	21.77	15.33
Three Bolts	1.43	321	408	4.78*	20.55	14.37
Four Bolts	1.43	319	409	7.63	20.75	14.51
	Beam set up a = 120 mm b = 200 mm					
Two bolts	1.42	309	399	4.75	23.75	16.73
Three bolts	1.44	329	410	5.55	21.30	14.79
Four bolts	1.42	317	405	9.37*	20.09	14.15
Characteristic bearing strength per mm thickness :						13.54

* Premature non-connection failures. The predicted value has been tabulated instead.

Therefore the characteristic bearing strength (mean - ks), independent of connection configuration and sheet thickness, has found to be equal to 13.54 kN.

The characteristic mechanical properties of the sections listed above are obtained as :

$$\sigma_y = 300 \text{ N/mm}^2$$

$$\sigma_{ult} = 397 \text{ N/mm}^2$$

(The characteristic values above, have been obtained using the procedure outlined in Appendix E, BS 5950 : Part 5)

In the following section the predicted ultimate bearing strengths for unit thickness given by the proposed equation for bearing strength, BS 5950 : Part 5 and Annex A are calculated and compared to the characteristic test value of 13.54 kN.

11.3. Predicted ultimate bearing strength per bolt, per mm sheet thickness

11.3.1. Proposed design equation

For unit thickness:

$$P_{bs} = \alpha \, . \, d.t.\sigma_{ult}$$

$$\alpha = k_1.k_2.k_3.k_4.k_5.k_6.k_7 = k_2$$ (k_1 and k_3 to k_7 are all equal to 1, for the given tests parameters)

$$\therefore \quad \alpha = k_2 = (1.9 + 0.2 \times 1) = 2.1$$

$$P_{bs} = 2.1 \times 16 \times 1 \times 397 = 13.34 \text{ kN/mm.}$$

(Cf. to the characteristic test value of 13.54 kN, tabulated above)

Proposed design equation based on yield stress

The above equation can be equally expressed in terms of yield stress as follows :

$$P_{bs} = \alpha \, . \, d.t.\sigma_y$$

$$\alpha = k_1.k_2.k_3.k_4.k_5.k_6.k_7$$

Where $k_2 = (2.6 + 0.3\,t)$ for $t \leq 3$ mm
$= 3.5$ for $3\text{mm} \leq t \leq 8$ mm

and $k_3 = (280/\sigma_{y\ design})^{1/2}$.

k_1 and k_4 to k_7 all being the same as before.

As previously, for the given tests parameters :

$$\alpha = k_1.k_2.k_3.k_4.k_5.k_6.k_7 = k_2$$
$$= (2.6 + 0.3 \times 1) = 2.9$$

$$\therefore \quad P_{bs} = 2.9 \times 16 \times 1 \times 300 = 13.92 \text{ kN.}$$

The difference between the two equations above is essentially that when a nominal yield stress of (say) 280 N/mm^2 is specified, the actual yield stress is well above this value. The same however, can not be said about the nominal and actual ultimate stresses. In other words, the actual (σ_{ult} / σ_y) is more often than not, less than the assumed 1.4 ratio.

11.3.2. BS 5950 : Part 5

The equivalent value given by BS 5950 pt. 5, for t = 1 mm, is :

$$P_{ult} = 2.1 \, . \, d.t.\sigma_y$$
$$= 2.1 \times 16 \times 1 \times 300 = 10.1 \text{ kN/mm.}$$

11.3.3. EC3, Annex A

Similarly for Annex A, EC3 ;

$$P_{ult} = 2.5 \, . \, d.t.\sigma_{ult}$$
$$= 2.5 \times 16 \times 1 \times 397 = 15.9 \text{ kN/mm.}$$

11.4. Conclusions

A comparison of the values predicted above with the actual results of two totally independent test arrangements, i.e. floor and beam set up, should leave no doubt that the propounded design expression yields the most reliable design values. In fact, it could hardly be any more accurate.

As was pointed out previously in Chapter Six, the design expression in Annex A overestimates the bearing strength, for the practical range of sections in cold formed steel connections. Also, to rely upon a factor of safety for the bolt material to subsidize over-estimating of strength of steel sheets in bearing, is not warranted.

It is also evident that the bearing strength given in BS 5950 : Part 5 is conservative.

Chapter Twelve

Nesting of sections

12. Nesting of sections

Summary

The effects of nesting of cold formed sections are considered in this chapter and suitable design factors, regarding their strength and flexibilities are proposed.

Moment-rotation characteristics of zeds (not nesting) are then compared with that of the equivalent Channel beams and the effect of loading through the shear centre of the section is investigated.

12.1. Introduction

The main advantage of cold forming is that galvanised steel strip can be formed into virtually any profile. This gives rise to endless possibilities. One such prospect is nesting of the sections.

Zed profiles have been especially developed with unequal flanges to fulfil this purpose. Zeds are generally regarded to be more efficient in terms of material use than any other cold formed steel shapes. Add to this their ability to nest, zeds offer a great scope and versatility since they pack much smaller for transport and hence are much easier to handle.

Special purlin systems have been devised to explore the potentials of sections nesting. These comprise of two principal systems of sleeved and overlapped purlins, designed to carry the sheet cladding materials insulated or otherwise, on roofs of up to 30° pitch.

With the overlap system, the inequality in dimensions of the upper and lower flanges makes it possible for sections to be turned over in alternate spans so that the ends will nest inside one another to give a doubled section over the supports, resulting in improved performance.

Alternatively an off-cut of the same sections can be turned over to accommodate (or "sleeve") the two incoming purlins at the supports.

As a general rule, for buildings of less than six bays, the sleeved system is the more economical. This consists of semi-continuous lines of zed purlins of single or double span, joined together over the rafters with connecting sleeves of the same section, as mentioned above.

The overlap system is usually more advantages for longer buildings and allows a zed section of a given size to carry about 33% more load than when it is sleeved. This is achieved by the overlapping the zeds by requisite amount at the rafters to take care of the high support moments, together with the provision of thicker sections of the same profile for the end bays.

This overlap of the purlins extends each side of the rafter to where the moments are approximately equal (but of the opposite sign) to the maximum internal moment.

The end spans, having higher moments than the maxima in the internal spans, consist of the same profile in a thicker gauge or reinforced sections, with the overlaps adjusted the appropriate amounts.

Despite the greater length of section resulting from the overlaps, and the heavier end spans , the saving in total weight over that of sleeved purlins carrying the same load is usually significant for a six bay building, and can amount to as much as 20 to 25% as the number of bays increase.

The inequality in the dimensions of the upper and lower flanges, of the zeds necessary for nesting, is counter balanced by adjusting the sizes of the lips so that the neutral axis would pass practically through the mid point of the web, thus ensuring the greatest possible value of the section modulus.

However, the main disadvantage of the zed profile is that its principal axes are inclined typically about 13 to 17° to the web. Unless the load is applied in this direction, a zed is subjected to bi-axial bending. With a typical roof slope of about 6°, there is an appreciable angle between the line of action of the load (vertical for snow, normal to the roof for wind) and the major principal axis so that a significant bi-axial bending effect tends to occur. In practice the cladding, as a relatively rigid diaphragm, is often relied upon to absorb the minor axis bending in form of an in-plane load, provided that adequate lateral restraint is given to the top flange, to convey this force into the cladding. Proprietary purlin cleats are often adequate for this purpose. There is also a tendency for the forces from the two halves of a pitched roof to balance at the apex.

A new generation of zed sections, Zeta, well established in the market for some years now, is designed to avoid the major disadvantage of the simple zed by bringing the inclination of principal axes much closer to that of the roof slope. A typical section has now its principal axes inclined at about 7° to a line drawn normal to the flanges, thus improving their bending properties by about 10% .

This is achieved by forming extra bends in the web, which has the added benefit of improving the sections' mechanical properties.

There therefore exists a favourable state of affairs with the section's major principal axis being near to the vertical, counter balancing the roof slope. As such, a section is much less prone to twist and would also be expected to show improved performance when interacting with the cladding in its working configuration.

In another new profile, UltraZed, section properties are enhanced by rolling stiffeners at the critical buckling points of the section, thus making it very efficient in primary bending. Such sections are very resistant to local buckling. However with this section, no attempt has been made to improve the position of the principal axis.

Instead, the designers have placed a greater importance on the ease of fastening to the web of the section.

As was mentioned at the beginning of Chapter One, consideration of theoretical and practical factors are of primary importance, and often a balance has to be struck between the two.

In spite of their complex profiles, the new generation of sections maintain the inherent quality of the zeds to nest and overlap. This is further enhanced by the stiffeners interlocking within each other, maximising the section efficiency even further.

For this part of the research project, it was intended to investigate and quantify the effects of nesting of sections on bolted connections. That is to see to what extent, the moment-rotation characteristics of bolted connections in cold-formed steel sections are influenced when they nest.

To this purpose it was decided to carry out two separate tests. One with zed sections staggered so that they did not nest, the other with sections of the same properties and same connection configuration, nested and bolted as usual. (Fig. 12-1, below)

The resulting moment-rotation characteristics would then be compared against each other.

Fig. 12-2 shows the general test arrangements, zed sections staggered and nesting. A three bolt connection configuration was used, as this is often the case in practice.This figure also shows that the beams were wedged by a timber strut and clamped in the same way described for three bolt channel beams to eliminate the web buckling component of deflection.

The test procedures and arrangements were exactly the same as that described in Chapter Eight for the beam set up. That is, the connections were bedded-in and unloaded. The loading was then repeated until failure occurred. The beams were then dismantled, re-aligned and welded round the connection area and loaded as uniform beams. Deflection readings were noted at the mid span, loading points and supports as described previously.

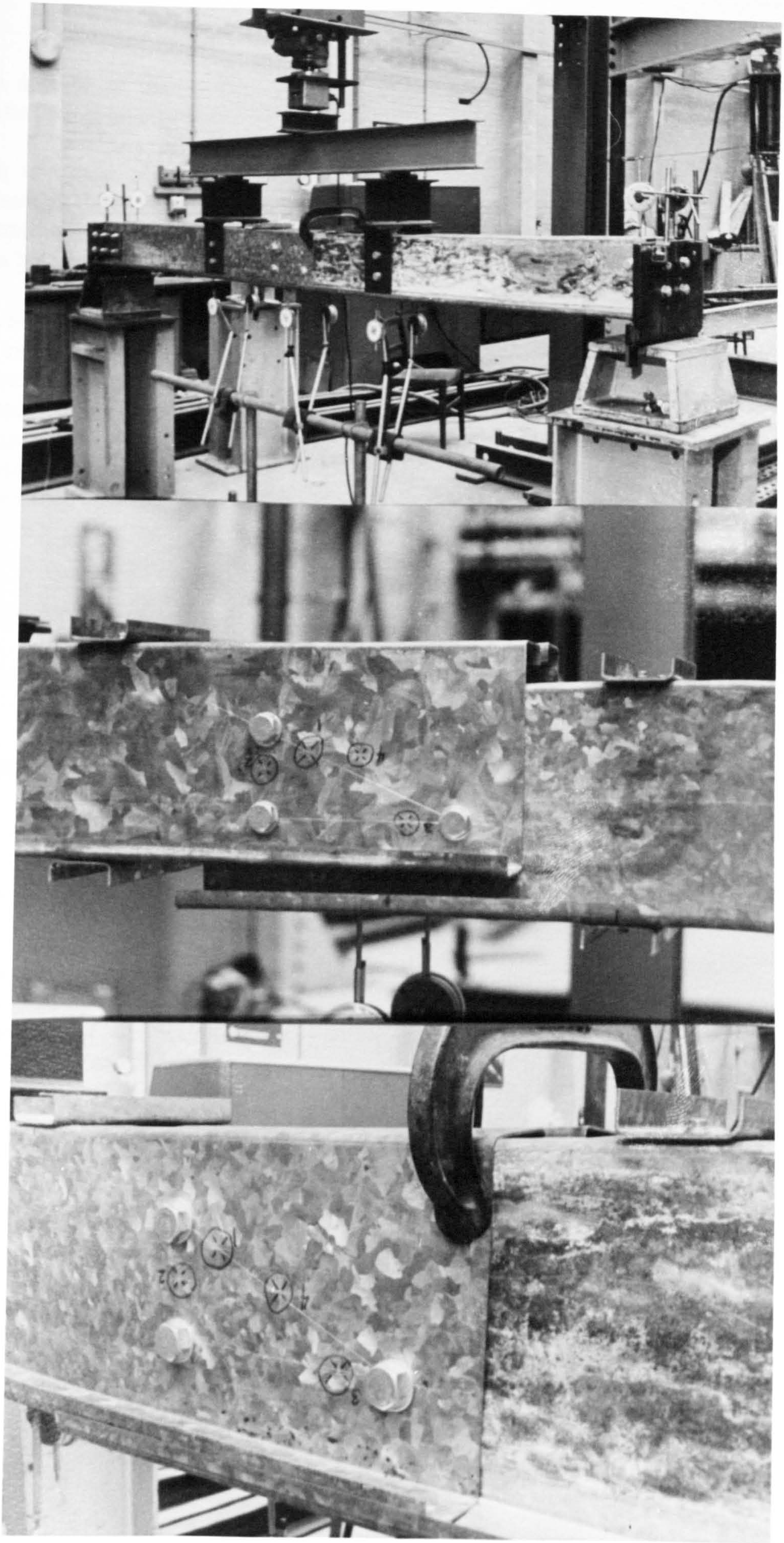


Fig. 12-2: Zed sections shown in general test arrangement, staggered and nesting respectively.

A system of theodolites, as described in Chapter Ten, was also in place to see whether it was possible to trace the connection rotation.

In the case of zeds not nesting, the gap required between the two sections to prevent them establish any bearing against each other during the test was calculated (mainly based on drawings to scale but also on the basis of the previous test results) to be equal to 40 mm. This was found to be just right at the time of testing.

It should be noted that the second moment of area of the zeds not nesting, I_{xx}, was greater than that of the zeds nesting over the connection area. However this would not affect the final moment-rotation characteristics of the connection, since in each case, the uniform (welded) beam test had exactly the same profile as the bolted beam. Therefore the deflection due to beam flexure would be eliminated in any case. What remained would be the component of deflection due to connection rotation, and the connection dimensions were the same for both tests.

12.2. Test parameters

The section profile is given in Fig. 12-3.

The section properties were :

σ_y = 392 N/mm^2

σ_{ult} = 468 N/mm^2

Note that nominal 350 N/mm^2 yield steel was used, with a nominal ultimate strength of 450 N/mm^2. (Z35 table 4 BS 5950 :Part 5)

The connection dimensions were as given in Fig. 12-4.

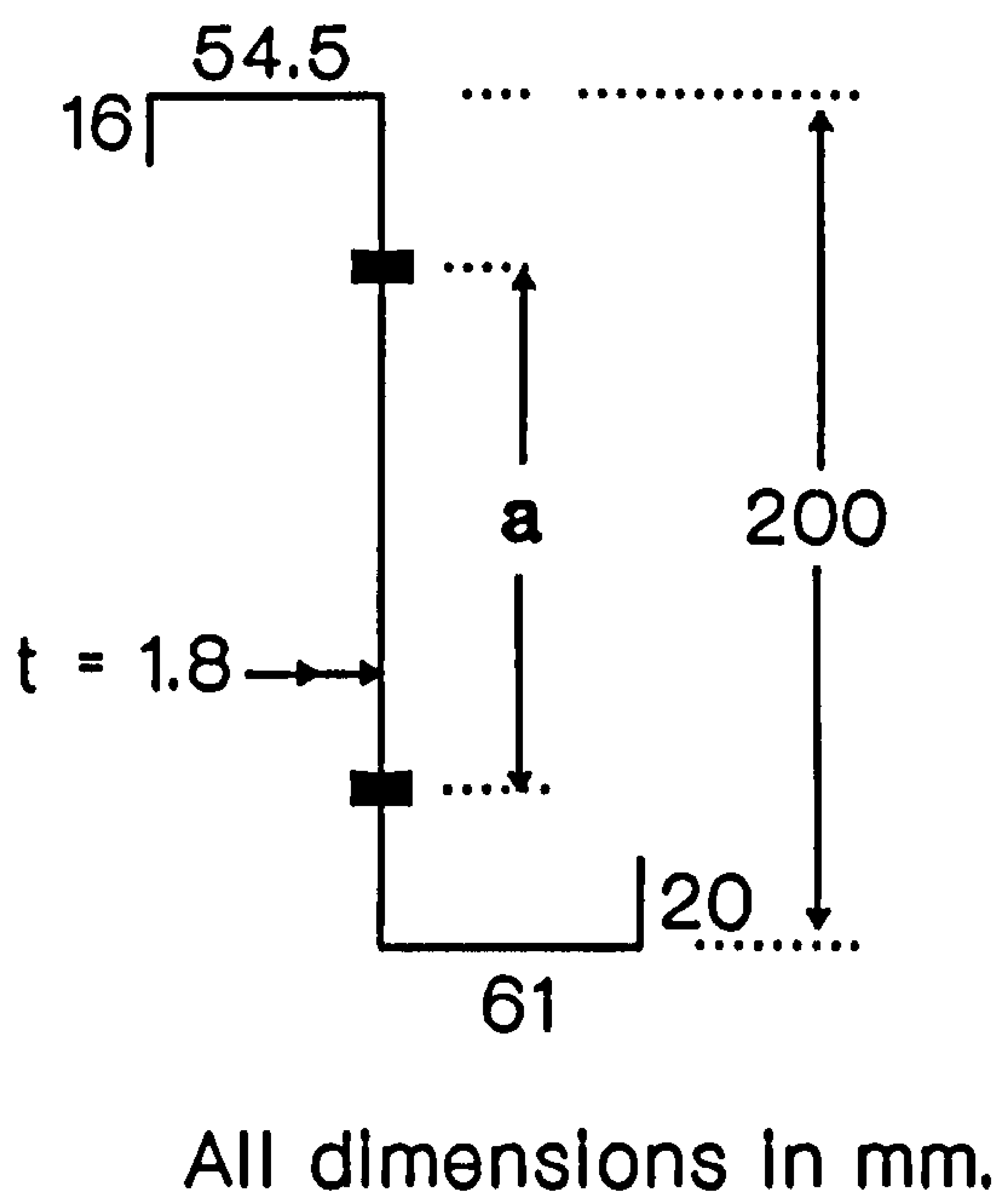


Fig. 12-3 : Section profile.

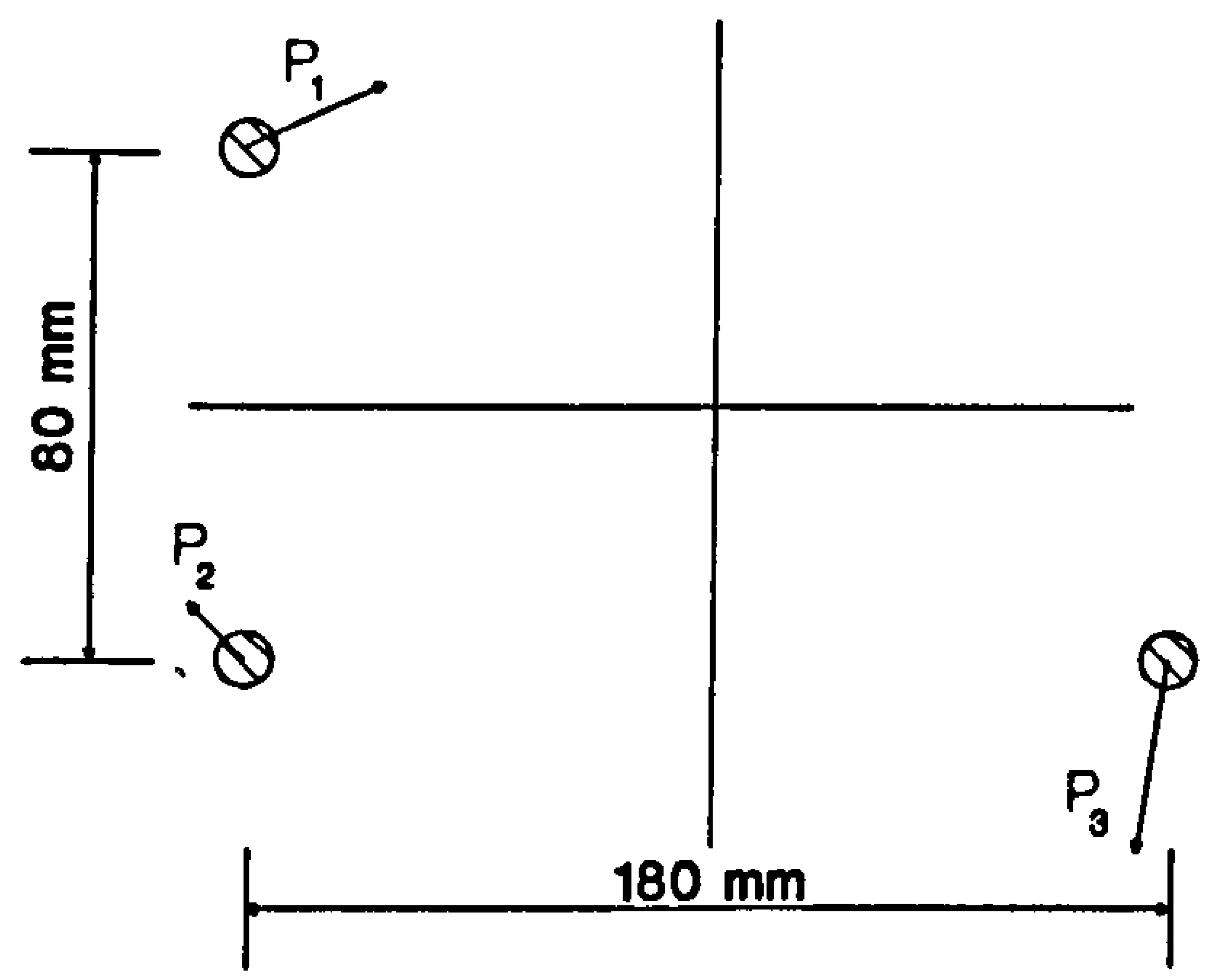


Fig. 12-4 : Connection details

12.3. Predicted behaviour

12.3.1. Predicted connection characteristics

With the given parameters, the ultimate bearing strength and flexibility of each fastening is calculated as :

$P_{bs} = \alpha \,.\, d.t.\sigma_{ult}$

$\alpha = k_1.k_2.k_3.k_4.k_5.k_6.k_7$
$\quad = k_2.k_3$

$k_2 = (1.9 + 0.2\ t)$
$\quad = (1.9 + 0.2 \times 1.8) = 2.26$

$k_3 = (390 / \sigma_{ult.\ nominal})^{1/2}$
$\quad = (390 / 450)^{1/2}$
$\quad = 0.93$

$\alpha = 2.26 \times 0.93 = 2.10$

$\therefore\ P_{bs} = 2.1 \times 16 \times 1.8 \times 0.468 \quad = 28.3\ \text{kN}$

$$c = 5n\left(\frac{10}{t_1} + \frac{10}{t_2} - 2\right) \times 10^{-3}$$

$$= 5 \times 3\left(\frac{10}{1.80} \times 2 - 2\right) \times 10^{-3} = 0.137 \qquad \left(\frac{mm}{kN}\right)$$

For a = 80 mm and b = 180 mm , the radii of rotation (elastic) are given as :

$$\left.\begin{aligned} r_1 &= \frac{1}{3}\sqrt{4a^2 + b^2} \\ r_2 &= \frac{1}{3}\sqrt{a^2 + b^2} \\ r_3 &= \frac{1}{3}\sqrt{a^2 + 4b^2} \end{aligned}\right\} \rightarrow \left.\begin{aligned} r_1 &= 80.3\ mm \\ r_2 &= 65.6\ mm \\ r_3 &= 122.9\ mm \end{aligned}\right\}$$

The COR is at (with bolt 2 as the origin):

$$\left.\begin{aligned} x &= \frac{b}{3} \\ y &= \frac{a}{3} \end{aligned}\right\} \rightarrow \left.\begin{aligned} x &= 60.0 \\ y &= 26.7 \end{aligned}\right\}$$

Therefore failure first occurs by sheet bearing at bolt 3. Let $P_3 = P$ the elastic moment of resistance is then given as :

$$M = \frac{2P}{\sqrt{a^2+4b^2}}(a^2+b^2) = 0.210\ P$$

The predicted connection rotation and moment capacity at failure are therefore:

$$\phi = \frac{P_{bs}\,c}{r_3} = \frac{28.3 \times 0.137}{122.9} = 31.55 \times 10^{-3} \quad (Rads.)$$

$$M_{ult.\,elastic} = 0.210\,P_{bs} = 0.210 \times 28.3 = 5.94 \quad (kNm)$$

∴ Predicted ultimate connection strength 5.94 kN.m (per beam)

12.3.2. <u>Predicted section capacity</u>

BS 5950 : Part 5

§ 5.2.2.2

$$p_o = \left\{1.13 - 0.0019\frac{D}{t}\left(\frac{\sigma_y}{280}\right)^{1/2}\right\} p_y$$

$$= \left\{1.13 - 0.0019 x \frac{200}{1.8}\left(\frac{392}{280}\right)^{1/2}\right\} p_y$$

$$= 0.880\ p_y$$

$$= 0.880 x 392 = 345 \qquad \frac{N}{mm^2}$$

$M_c = p_o\ Z_{xx}$

All inward lips should be included in calculation of section modulus. § 5.2.2.7

Effective width of the compression flange : § 5.2.2.4

$b/t = 54.5 / 1.8 = 30.3$ § 4.4

$(b_{eff} / b) = 0.992$ Table 5

$\therefore$ section fully effective.

$Z_{xx} = 35.94 \times 10^{-3}$

$M_c = 345 \times 35.94 \times 10^{-3} = 12.4$ kN.m (per beam)

Therefore failure should first occur by sheet bearing round the bolt furthest away from the centre of rotation.

12.4. Test results

The results obtained are described here.

12.4.1. Zeds not nesting

The connection failed at an applied moment of 6.80 kN.m in the manner predicted above. (Cf. to 5.94 kN.m predicted)

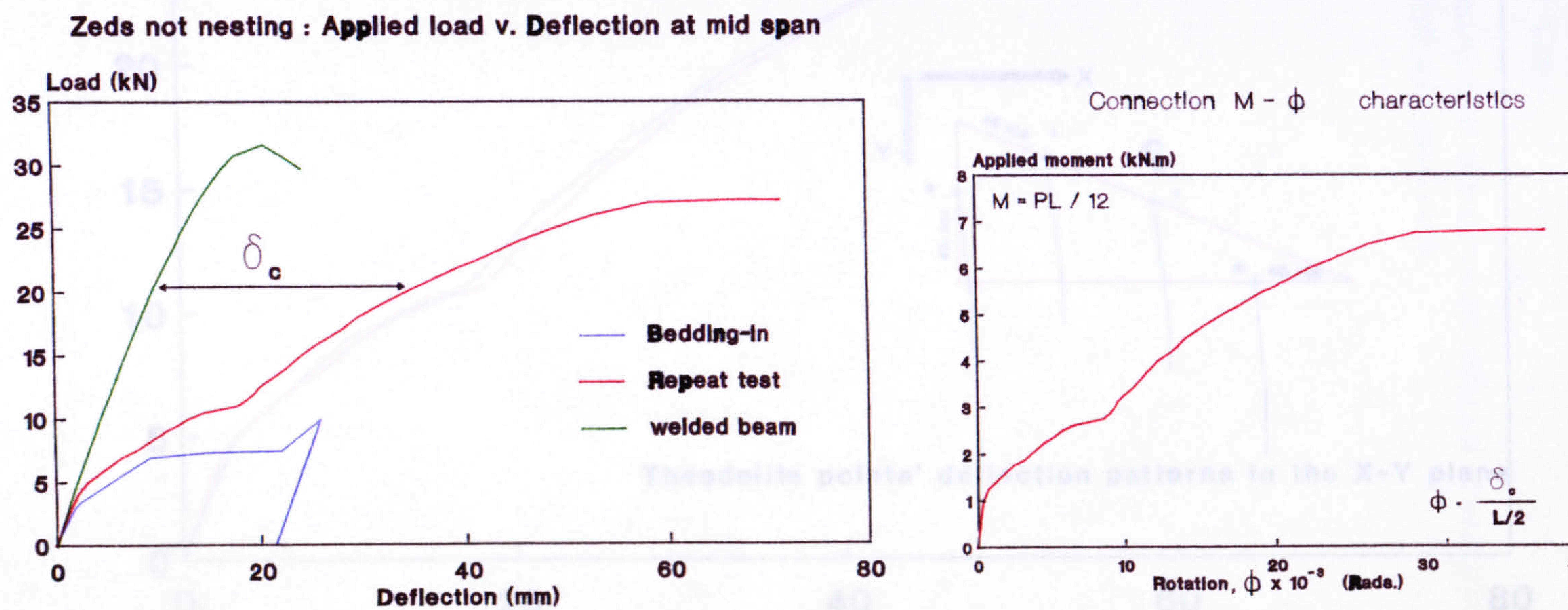
As mentioned previously, no bearing was established between the sections throughout the tests. Fig. 12-5 shows the bolted connection after failure. The welded beams, before and after failure, are also shown in this figure. It can be seen that in the latter case failure occurred by web buckling just outside the overlapped area.

The load-deflection readings, at the centre span, for the bedding in test, repeat test and welded beam test are plotted in Fig. 12-6a. The connection moment-rotation characteristics (based on mid span data) are plotted in Fig. 12-6b. As with the Channel tests the one-third span (loading points) data yielded identical results.

The vertical component of the theodolite point at the mid span has been compared with that of the relevant dial gauge readings in Fig. 12-7. Again it can be seen that, the two totally independent systems have given literally identical results, which is an indication of the accuracy of the test results.

The deflection patterns in the X-Y plane of the four pre-set theodolite points are also depicted in the above figure. As with the three bolt channel beams, it can be seen that the overall deflection of the beams, rather than the rotation of the connection, has been reflected in the four parallel lines.

Fig. 12-5: Bolted and welded Zed sections (not nesting) at failure



Zeds not nesting : Applied load v. Deflection at mid span
Load (kN)
35
30
25
20
15
10
5
0
0
20
40
60
80
Deflection (mm)
δ_c
Bedding-in
Repeat test
welded beam
Connection M - ϕ characteristics
Applied moment (kN.m)
M = PL / 12
8
7
6
5
4
3
2
1
0
10
20
30
40
Rotation, $\phi \times 10^{-3}$ (Rads.)
$\phi = \frac{\delta_c}{L/2}$

Fig. 12-6

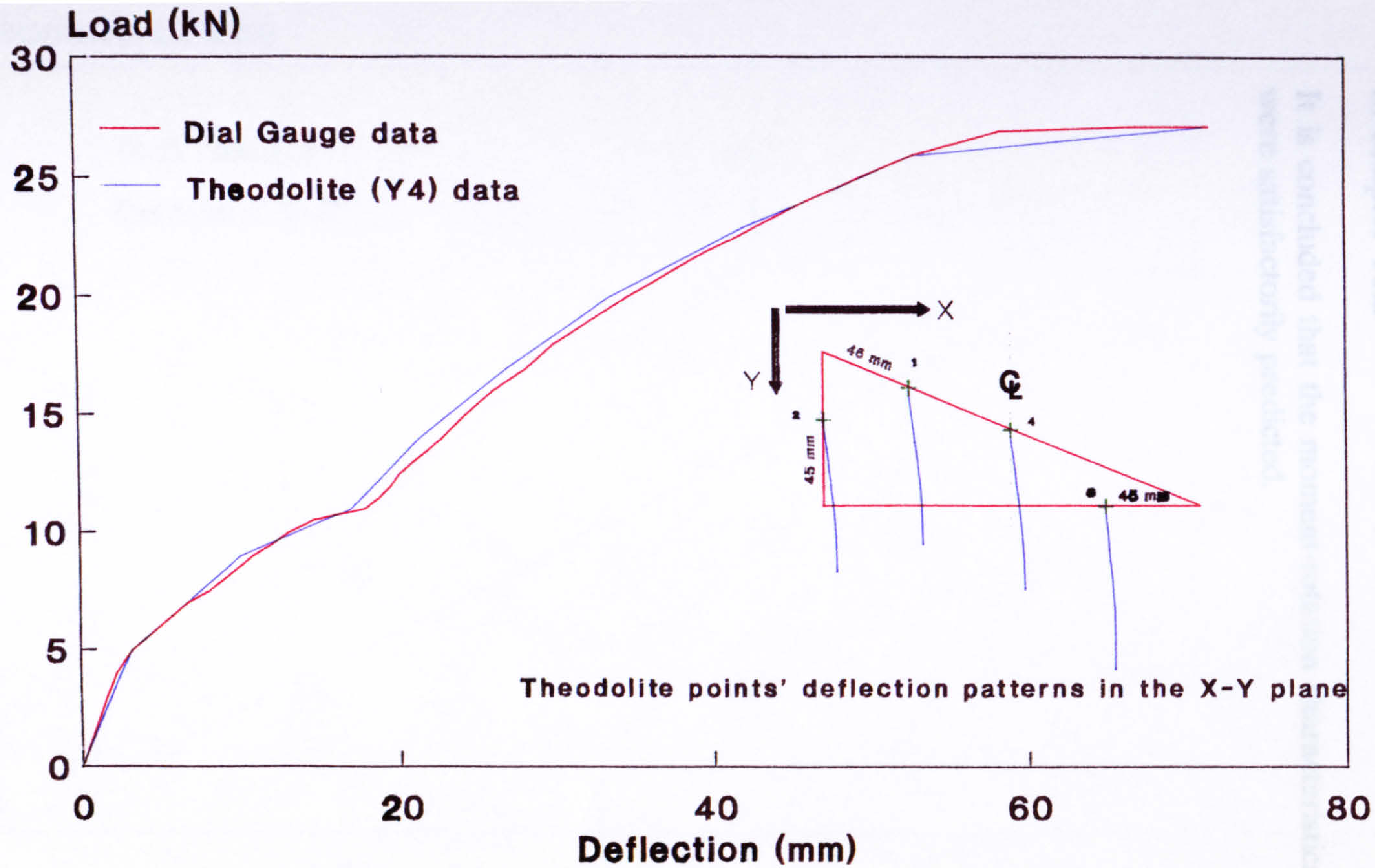


Fig. 12-7 : Zeds not nesting
Dial gauge readings compared with the vertical component of theodolite point 4 at mid span

The actual and predicted moment-rotation characteristics are plotted in Fig. 12-8.

The equivalent plastic moment capacity of the connection, for the given test parameters, would be equal to 7.13 kN.m. Therefore the connection strength would be over-estimated. This is further evidence that the correct conclusions were drawn in Chapter Ten.

It is concluded that the moment-rotation characteristics of the tested connection were satisfactorily predicted.

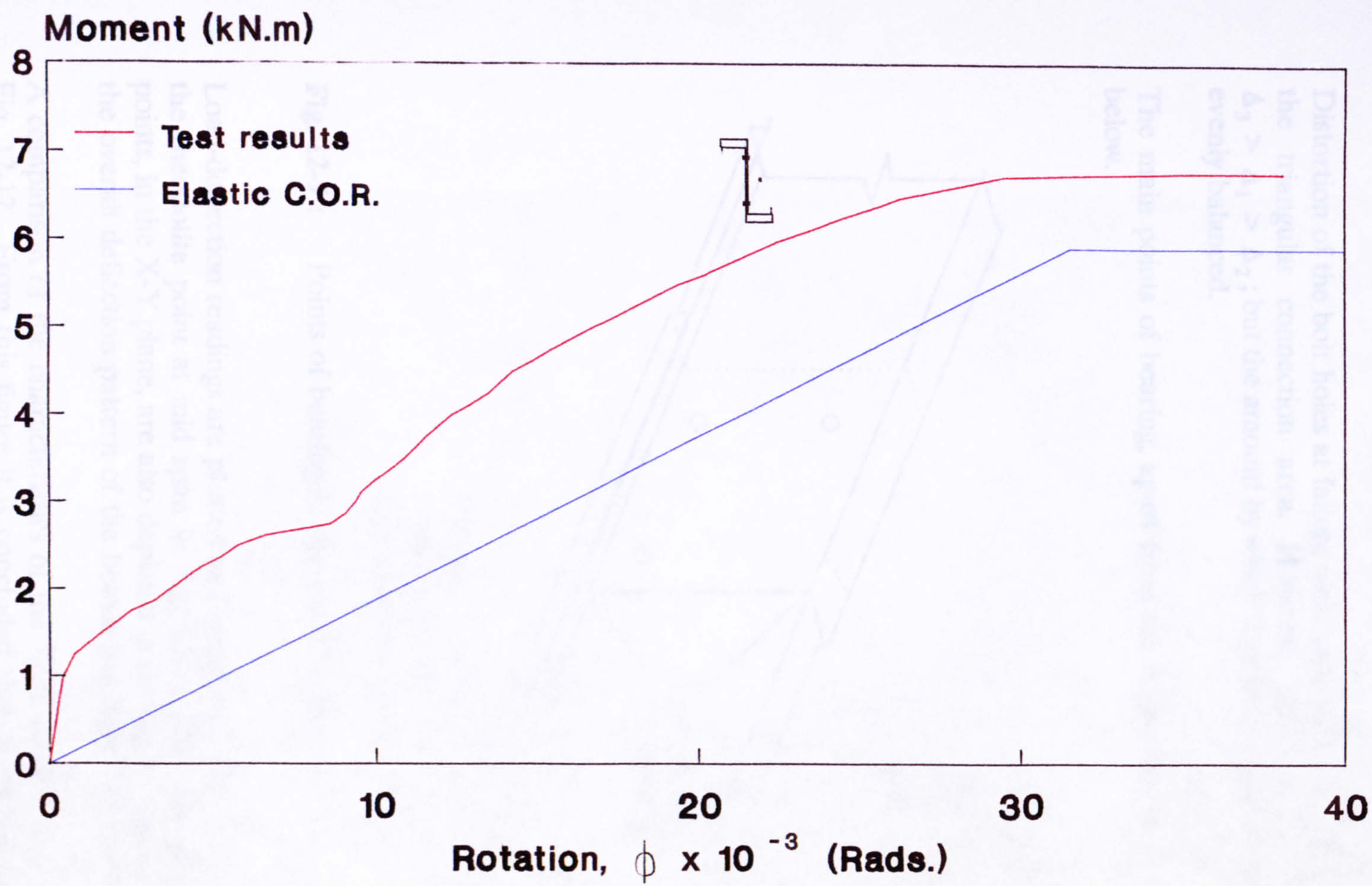


Fig. 12-8

Zeds not nesting : Actual and predicted moment-rotation characteristics

12.4.2. Zeds nesting

The beams failed at an applied moment of 7.5 kN.m by local flange buckling under one of the loading points, see Fig. 12-10. The failed section was later replaced, with an identical one, for the welded beam test.

Distortion of the bolt holes at failure were such that the COR would have to be in the triangular connection area. However, although as with previous cases $\Delta_3 > \Delta_1 > \Delta_2$; but the amount by which they had deformed was, in this case, more evenly balanced.

The main points of bearing, apart from the bolts, were as depicted in the figure below.

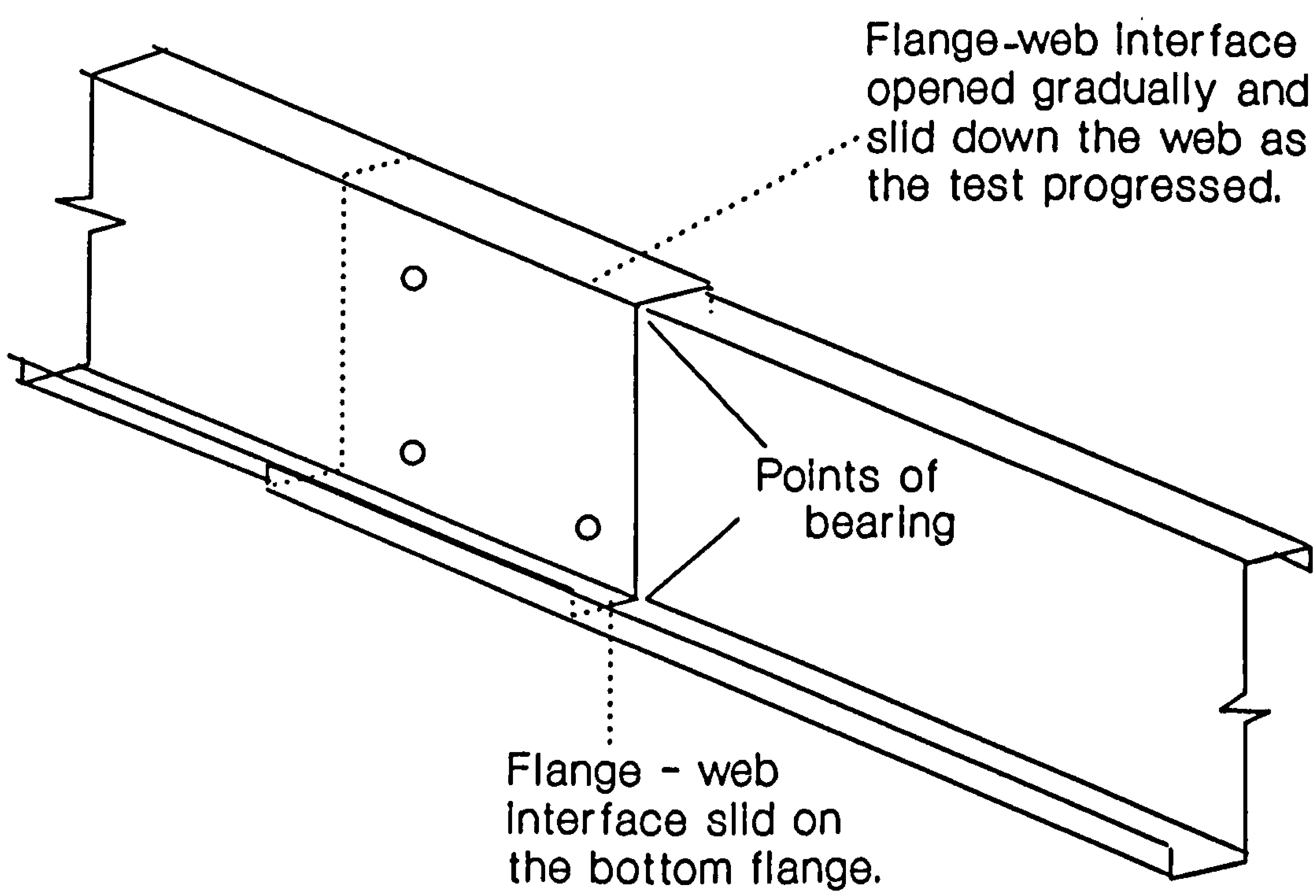
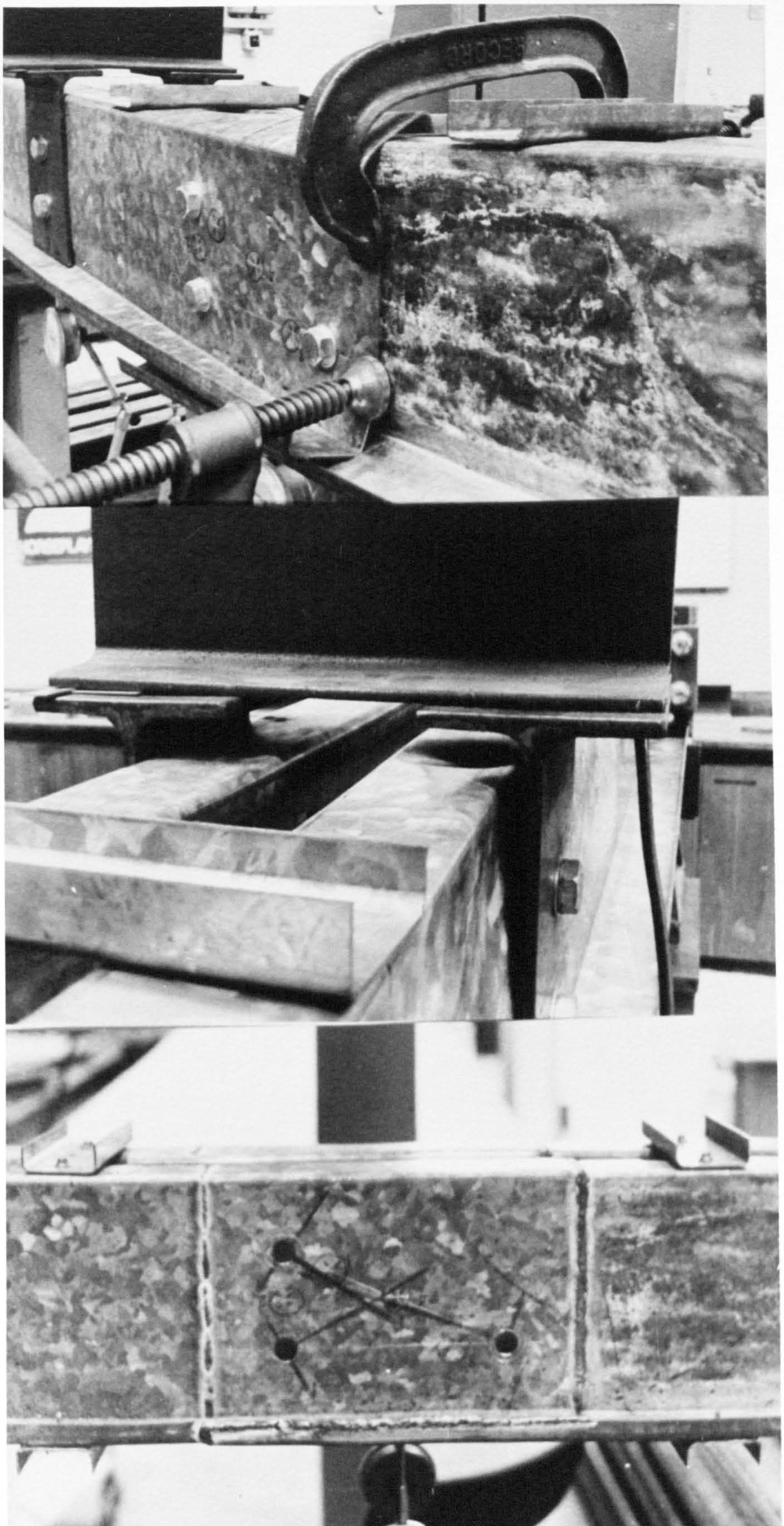


Fig. 12-9 : Points of bearing in the nested sections

Load-deflection readings are plotted and compared with the vertical component of the theodolite point at mid span in Fig. 12-11. The deflection of the theodolite points, in the X-Y plane, are also depicted in this figure. As with the previous cases the overall deflection pattern of the beams has been reflected in the readings.

A comparison of the characteristics of the Zeds nesting and not is carried out in Fig. 12-12. From this figure it is concluded that **the stiffness of a connection is increased by about 20% as a result of sections nesting.**

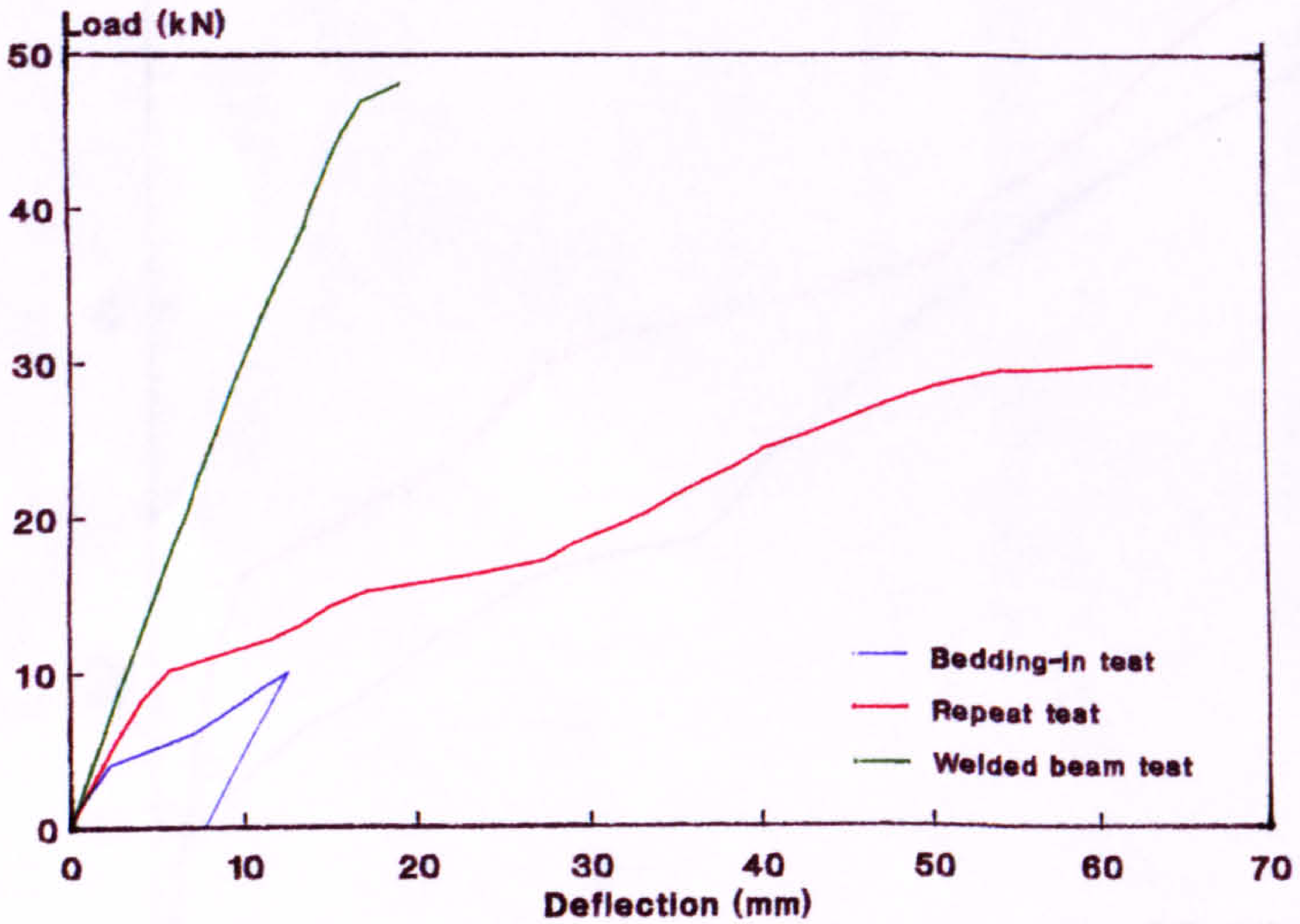
Possible locations of the COR are investigated in the following section.



Local buckling in the compression flange

Fig. 12-10. Zed sections nesting

Applied load v. mid span deflection



Theodolite data compared to that of the dial gauge at mid span

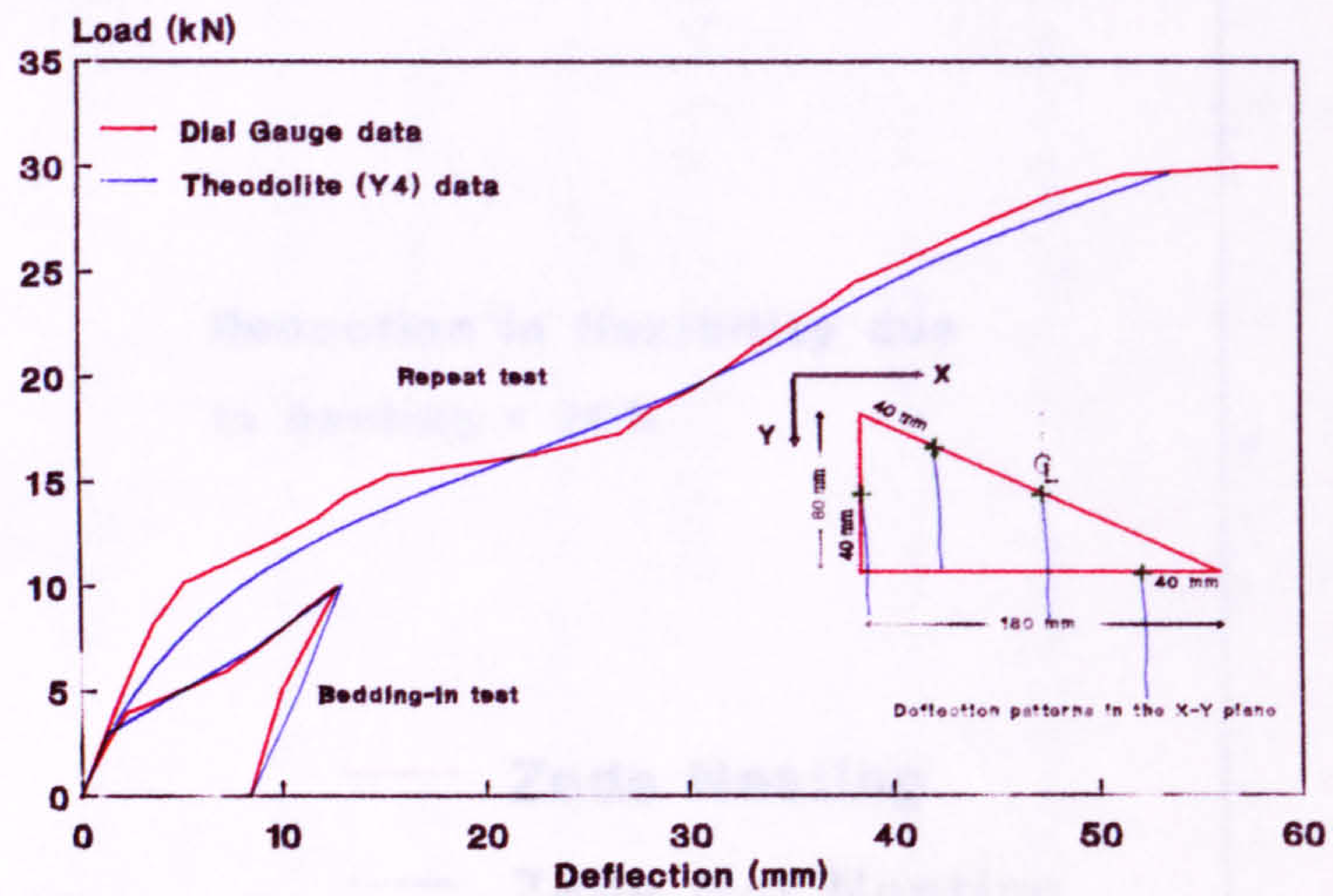


Fig. 12-11

Zeds nesting : test data

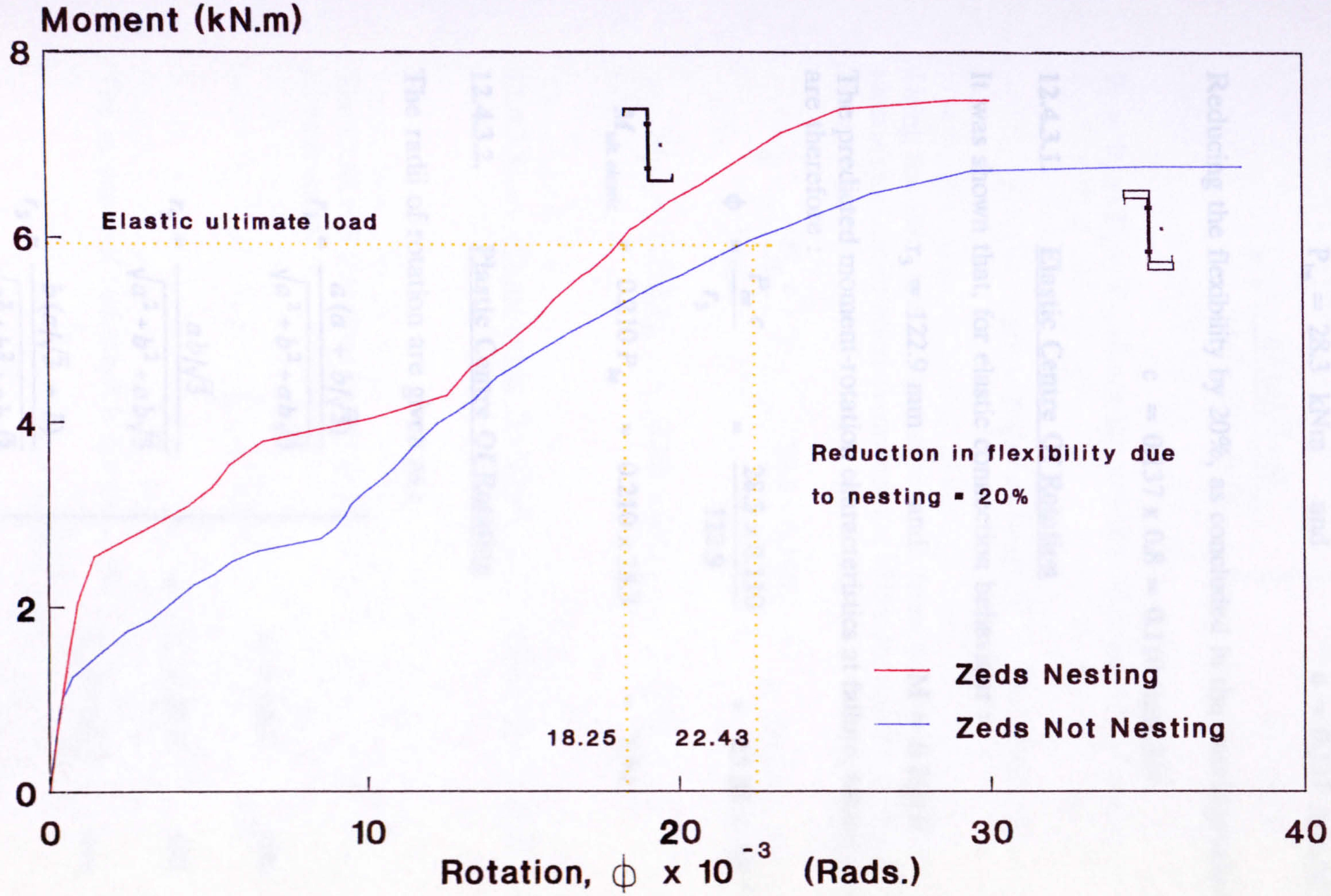


Fig. 12-12
Moment-rotation characteristics of Zeds

12.4.3. Location of the COR when sections nest

For the given section and connection particulars, it has already been shown that :

P_{bs} = 28.3 kNm and c = 0.137 mm/kN.

Reducing the flexibility by 20%, as concluded in the previous section, gives:

c = 0.137 x 0.8 = 0.110 mm/kN.

12.4.3.1. Elastic Centre Of Rotation

It was shown that, for elastic connection behaviour :

r_3 = 122.9 mm and M = 0.210 P

The predicted moment-rotation characteristics at failure, taking account of nesting, are therefore :

$$\phi = \frac{P_{bs}\, c}{r_3} = \frac{28.3 \times 0.110}{122.9} = 25.33 \times 10^{-3} \qquad (Rads.)$$

$$M_{ult.\,elastic} = 0.210\, P_{bs} = 0.210 \times 28.3 = 5.94 \qquad (kNm)$$

12.4.3.2. Plastic Centre Of Rotation

The radii of rotation are given as :

$$\left.\begin{aligned} r_1 &= \frac{a\,(a + b/\sqrt{3})}{\sqrt{a^2+b^2+ab\sqrt{3}}} \\ r_2 &= \frac{ab/\sqrt{3}}{\sqrt{a^2+b^2+ab\sqrt{3}}} \\ r_3 &= \frac{b\,(a/\sqrt{3} + b)}{\sqrt{a^2+b^2+ab\sqrt{3}}} \end{aligned}\right\} \rightarrow \left.\begin{aligned} r_1 &= 58.3 \quad mm \\ r_2 &= 32.9 \quad mm \\ r_3 &= 161.3 \quad mm \end{aligned}\right\}$$

The COR is at :

$$\left.\begin{aligned} x &= \frac{ab(a + b/\sqrt{3})}{2(a^2+b^2+ab\sqrt{3})} \\ \\ y &= \frac{ab(a/\sqrt{3}+b)}{2(a^2+b^2+ab\sqrt{3})} \end{aligned}\right\} \rightarrow \left.\begin{aligned} x &= 20.8 \\ \\ y &= 25.5 \end{aligned}\right\}$$

$P_1 = P_2 = P_3 = P$. Hence the moment of resistance is given as :

$$M = P\sqrt{a^2+b^2+ab\sqrt{3}} \qquad = 0.252\ P$$

Using the lever arm of the furthest bolt from the centre of rotation, the predicted plastic moment-rotation characteristics of the connection, taking account of nesting, are as follows :

$$\phi = \frac{P_{bs}\,c}{r_{3\ plastic}} = \frac{28.3 \times 0.110}{161.3} = 19.30 \times 10^{-3} \qquad (Rads.)$$

$$M_p = 0.252\,P_{bs} = 0.252 \times 28.3 = 7.13 \qquad (kNm)$$

12.4.3.3. Considering the Centre Of Rotation to be at a point equi-distant from the three bolts

i.e $\qquad r = r_1 = r_2 = r_3$

The COR will therefore be at the centre point of a rectangle with the bolts as three of its corners.

$$r = \frac{1}{2}\sqrt{a^2+b^2} \qquad = 98.5 \qquad mm$$

The moment of resistance is obtained as :

$$M = 3Pr = 0.296\ P$$

The predicted plastic moment-rotation characteristics at failure, taking account of nesting, are therefore :

$$\phi = \frac{P_{bs}\,c}{r} = \frac{28.3 \times 0.110}{98.5} = 31.60 \times 10^{-3} \qquad (Rads.)$$

$$M = 0.296\,P_{bs} = 0.296 \times 28.3 = 8.38 \qquad (kNm)$$

A comparison of the actual test results with the predicted behaviours, described above, is made in Fig. 12-13.

In view of the moment-rotation characteristics plotted in Fig. 12-13 and observations made during and after the test - it is evident that nesting of the sections will in effect try to hold the sections together as a uniform beam and maintain a gradient continuity. Under such circumstances it may be reasonable to assume that the bolt forces are equal at failure. It is further evident that the actual test characteristics are best predicted by the plastic assumption, when sections nest.

The ratio of the predicted moment capacity of the connection, based on the plastic and elastic assumptions is :

$$\frac{M_p}{M_{ult.\,elastic}} = \frac{7.13}{5.94} = 1.20$$

This ratio is believed to apply to the common range of sleeve dimensions of zed purlins.

Therefore for design purposes, **the moment capacity of two, three and four bolt connections shall be based on the elastic assumption and increased by 20% due to sections nesting.**

Also as described previously, **connection flexibility shall also be based on the elastic assumption, with the joint flexibility reduced by 20%, due to the sections nesting.**

Note that, the strength of the connection as whole should be increased as a result of nesting and not the sheet bearing strength of each fastening. Otherwise this would affect the flexibility of the connection.

As an example for the zed sections tested, the predicted connection strength and flexibility as a result of the sections nesting would be : (Page 258)

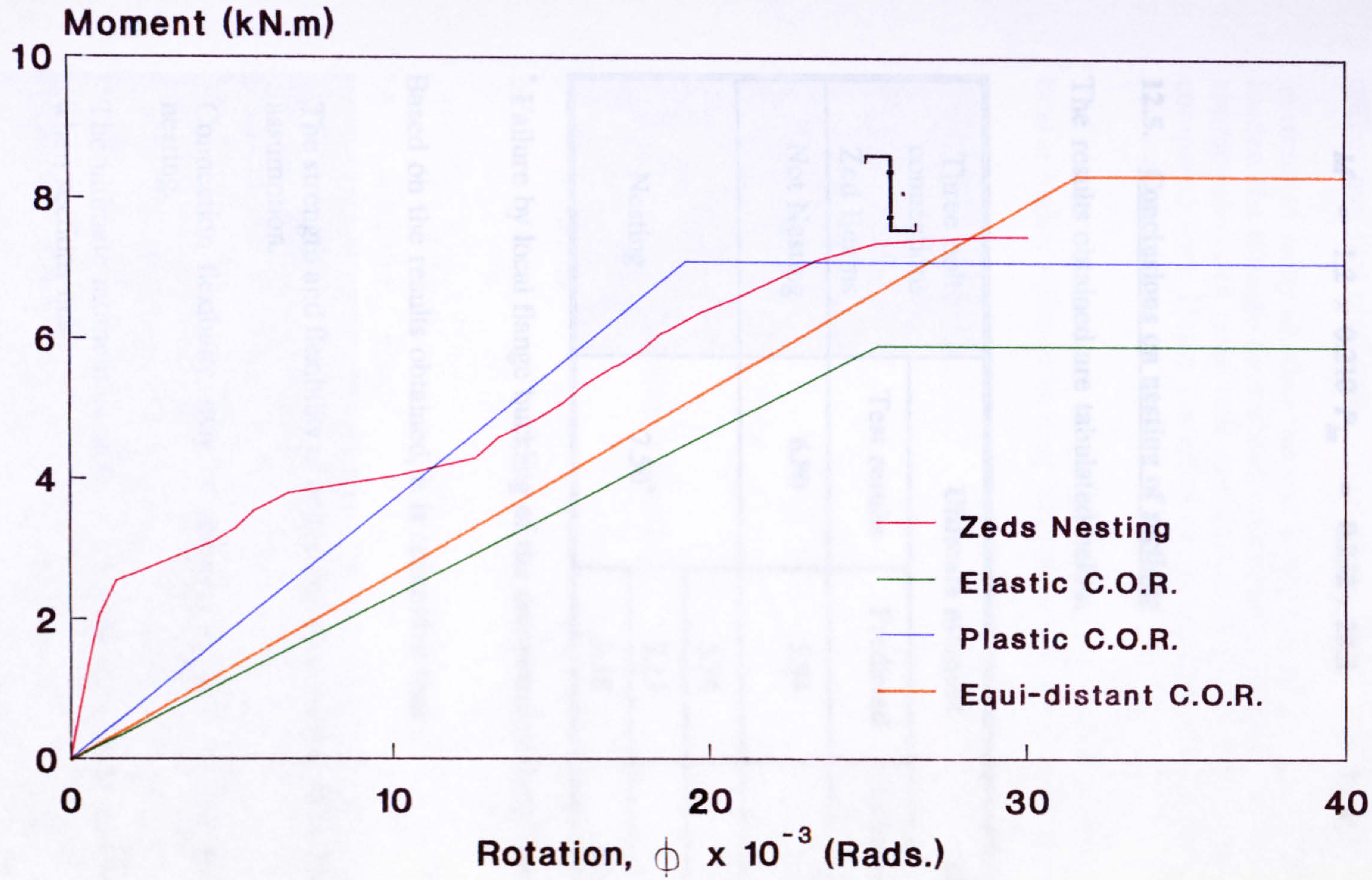


Fig. 12-13

Zeds nesting : Actual and predicted $M - \phi$
(depending on the location of C.O.R.)

$$\phi = \frac{P_{bs}\, c}{r} = \frac{28.3 \times 0.110}{98.5} = 31.60 \times 10^{-3} \qquad (Rads.)$$

(*as before*)

$$M = 1.2 \times 0.210\, P_{bs} = 0.252 \times 28.3 = 7.13 \qquad (kNm)$$

12.5. Conclusions on nesting of sections

The results obtained are tabulated below.

Three bolt connections Zed Beams	Ultimate moment		(kN.m)
	Test results	Predicted	Location of the COR
Not Nesting	6.80	5.94	Elastic
Nesting	7.50*	5.94	Elastic
		7.13	Plastic
		8.38	Equi-distant

* Failure by local flange buckling of the compression flange under a loading point.

Based on the results obtained, it is concluded that :

The strength and flexibility of bolted connections should be based on the elastic assumption.

Connection flexibility, may be reduced by 20% as a result of the sections nesting.

The ultimate moment capacity of a bolted connection may increased by 20%, when sections nest.

12.6. Comparison of the three bolt connections in Zed and Channel sections

The strength and flexibility of full moment connections, described so far have been predicted with remarkably good accuracy. With two, and particularly, three bolt channel connections in the beam set up however, the actual test characteristics were stiffer than that predicted.

In order to verify whether this was in any way related to channel sections not being loaded the through their shear centre, the predicted and actual moment-rotation characteristics of three bolt connections of Zeds (not nesting) and Channels are compared here. Hence the effect of loading through the shear centre of a section is investigated.

The zed sections were of a slightly thicker gauge (1.8 cf. to 1.5 mm channels) hence bear a higher ultimate bearing strength and a lower sheet flexibility.

Zeds (not nesting)		Channels : (three bolt)	
t	$= 1.8$ mm	t	$= 1.5$ mm
σ_y	$= 350$ N/mm^2	σ_y	$= 280$ N/mm^2
P_{bs}	$= 28.3$ kN	P_{bs}	$= 20.7$ kN
c	$= 0.137$ mm/kN	c	$= 0.178$ mm/kN

Connection details :

a	$= 80$ mm	a	$= 120$ mm
b	$= 180$ mm	b	$= 200$ mm
r_3	$= 122.9$ mm	r_3	$= 139.2$ mm
M	$= 0.210\ P_{bs}$ kNm	M	$= 0.261\ P_{bs}$ kNm

For the given test parameters in each case, the predicted moment-rotation characteristics are plotted in Fig. 12-14.

The predicted characteristics are intuitively correct. The higher material properties of the zeds, have been offset by the larger connection dimensions of the channel sections, so the final M - ϕ characteristics are more or less equal.

Compare the predicted (ϕ , M) coordinates at failure : (Fig. 12-14)

(31.55×10^{-3} [Rads.] , 5.94 [kN.m]) Zeds

(26.50×10^{-3} [Rads.] , 5.40 [kN.m]) Channels

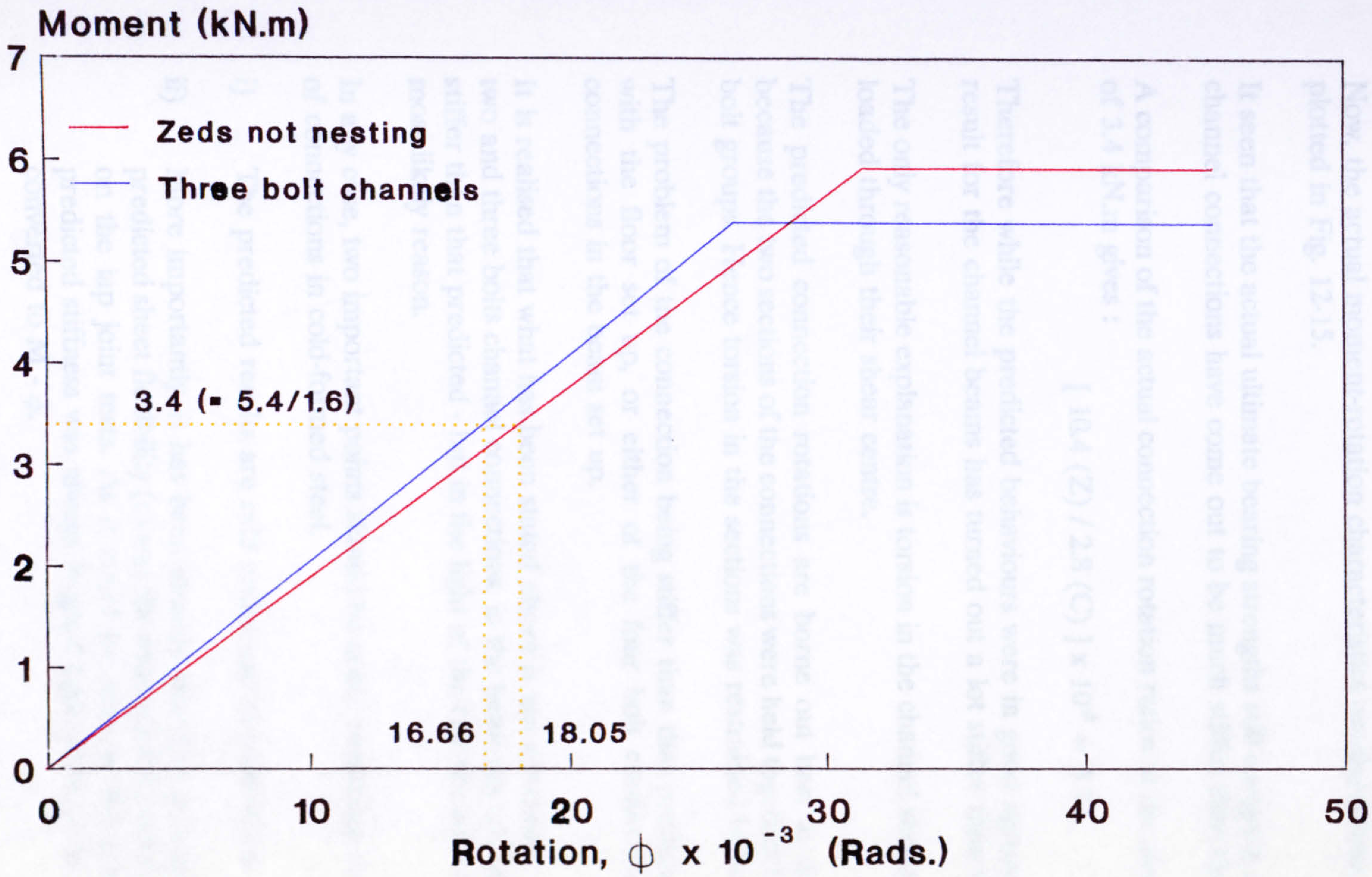


Fig. 12-14

Predicted (elastic) moment-rotation characteristics of Zeds (not nesting) and three bolt channels

With reference to Fig. 12-14, at a working moment of 3.4 kN.m (=5.4/1.6) the ratio of predicted rotation of the zeds connections compared with that of the channels' is ;

$$[\ 18.1\ (Z)\ /\ 16.7\ (C)\ ] \times 10^{-3} = 1.08$$

Now, the actual moment-rotation characteristics obtained from the test results are plotted in Fig. 12-15.

It seen that the actual ultimate bearing strengths still compare quite closely but the channel connections have come out to be much stiffer than that predicted.

A comparison of the actual connection rotation ratios at the same working moment of 3.4 kN.m gives :

$$[\ 10.4\ (Z)\ /\ 2.8\ (C)\ ] \times 10^{-3} = 3.71.$$

Therefore while the predicted behaviours were in good agreement the actual test result for the channel beams has turned out a lot stiffer than that expected.

The only reasonable explanation is torsion in the channel sections, due to not being loaded through their shear centre.

The predicted connection rotations are borne out best in the four bolt groups because the two sections of the connections were held together better than the three bolt groups. Hence torsion in the sections was restrained better.

The problem of the connection being stiffer than that predicted was non existent with the floor set up, or either of the four bolt connection tests, or the zed connections in the beam set up.

It is realised that what has been stated above is not conclusive proof as to why the two and three bolts channel connections, in the beam set up, have turned out to be stiffer than that predicted - but in the light of the test results it does seem to be the most likely reason.

In any case, two important points should be noted regarding the predicted stiffness of connections in cold-formed steel.

i) The predicted results are safe under any circumstances.

ii) More importantly, it has been already stated in previous chapters that the predicted sheet flexibility (c) was the most possible optimistic solution, based on the lap joint tests. As it could be seen in the previous chapters, the predicted stiffness was always in good agreement with that of the lap data converted to M - ϕ.

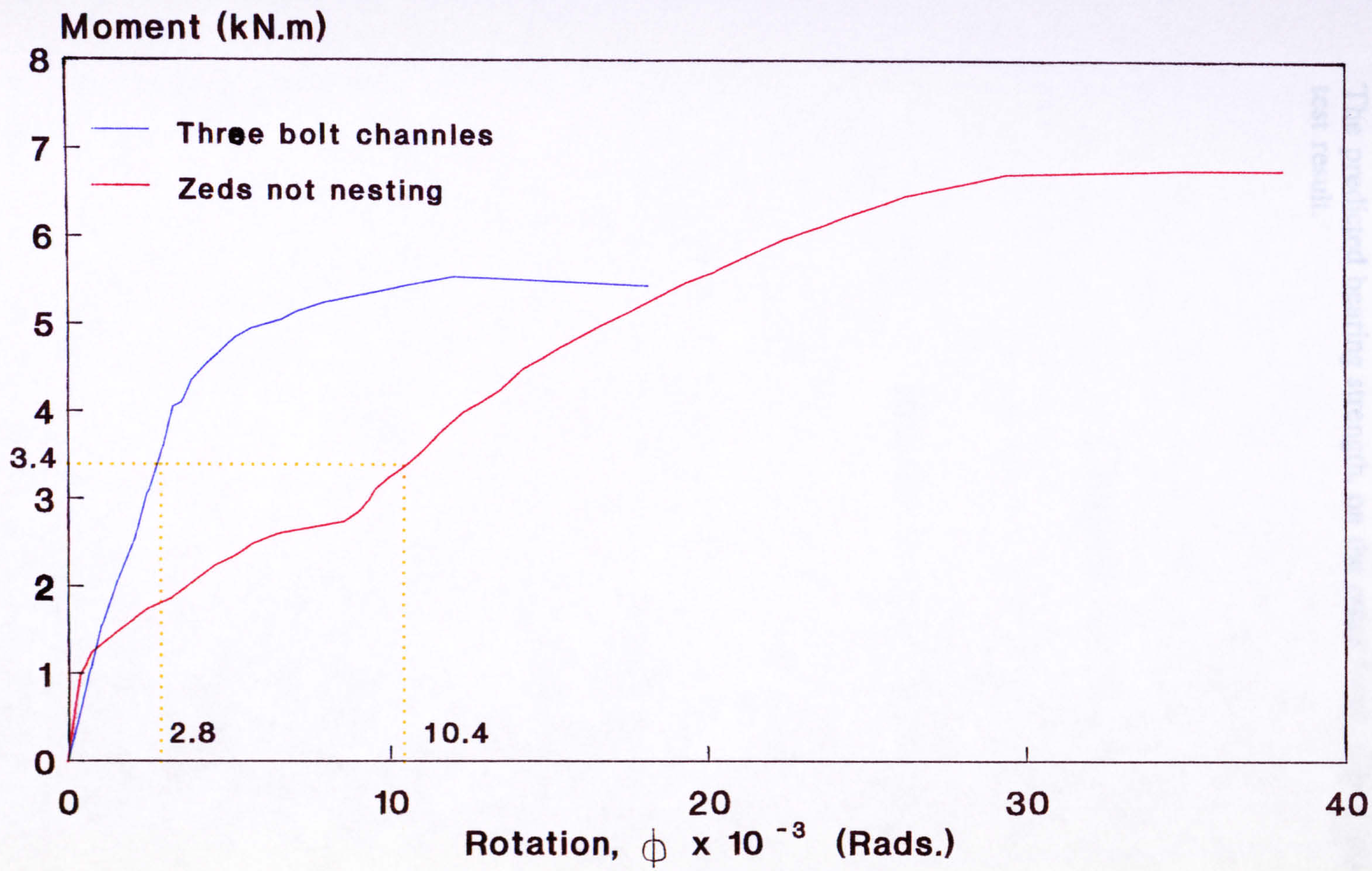


Fig. 12-15

Actual Moment-rotation characteristics of Zeds (not nesting) and 3 bolt channels

Therefore increasing the permissible sheet stiffness (i.e reducing the flexibility, c) any further, can not be based on any sound design justifications. In any case this would be over-ruled by the results obtained for four bolt groups, floor set up tests and the zed connections.

The predicted bearing strength, on the other hand, was vindicated by every single test result.

Chapter Thirteen

Interlocking of sections

13. Interlocking of sections

Summary

The effects of interlocking of cold formed steel sections are briefly considered in this chapter and suitable design factors are introduced.

13.1. Introduction

Apart from nesting of sections, another important inherent quality of cold formed sections, not open to the hot rolled variety, is their ability to interlock.

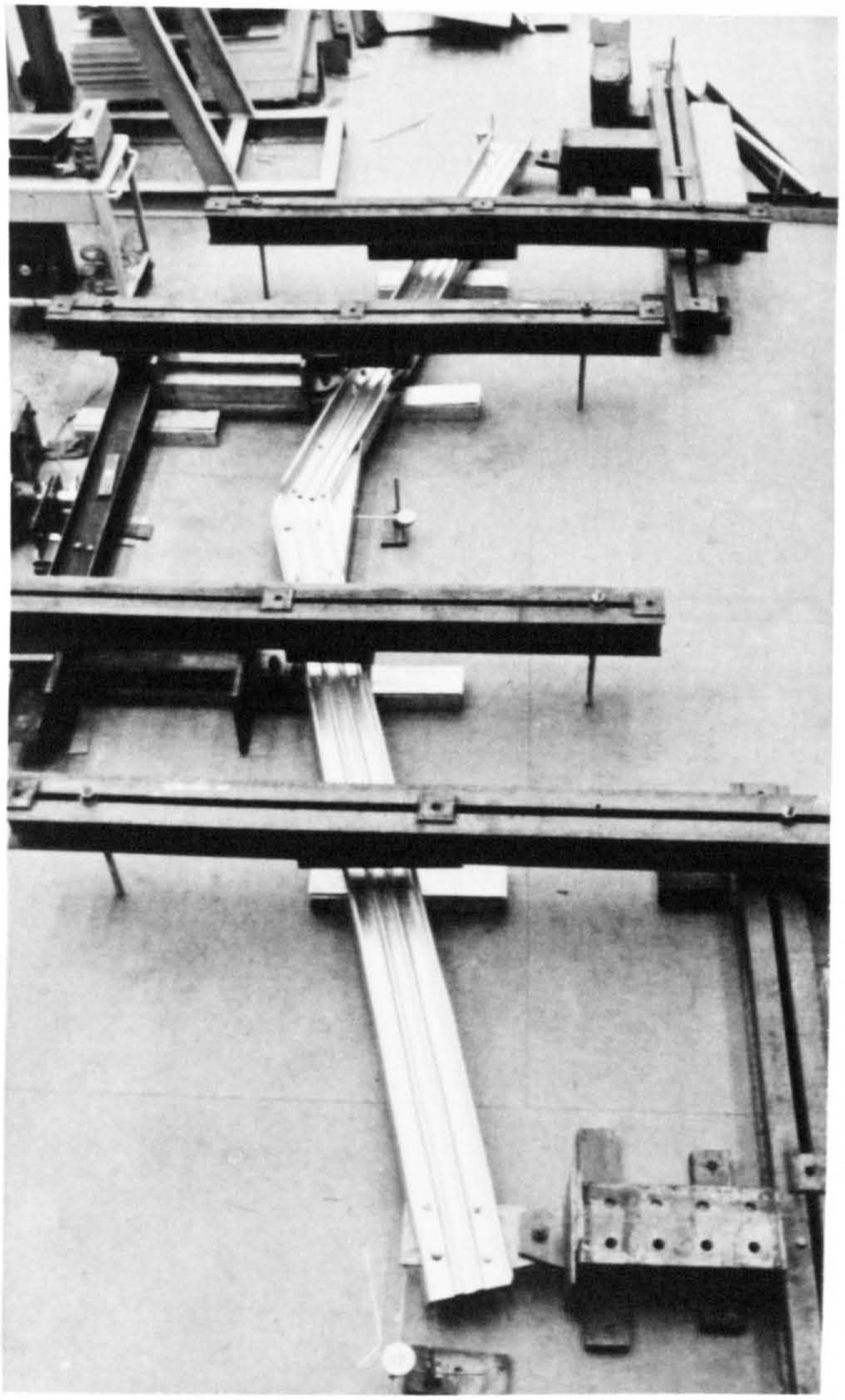
"Grooves" are rolled into the sections, which will enable the members to interlock which will be held together by bolts at the joints. Interlocking adds a new dimension to the efficiency of cold formed sections in the following ways :

- the extra grooves (or swages) rolled in the sections, essentially act as intermediate stiffeners making them very resistant to local buckling, which is a main design consideration in cold formed steel.
- sections locking into each other result in very stiff moment connections which will also provide sway stability to the whole structure.
- interlocking can significantly reduce the frame (or members) deflections. The importance of connection stiffness was demonstrated in Chapter One (Fig. 1-5), where it was shown that frame deflections are very sensitive to connection rigidity.
- combination of the above factors will often reduce the number of bolts required to make a connection.

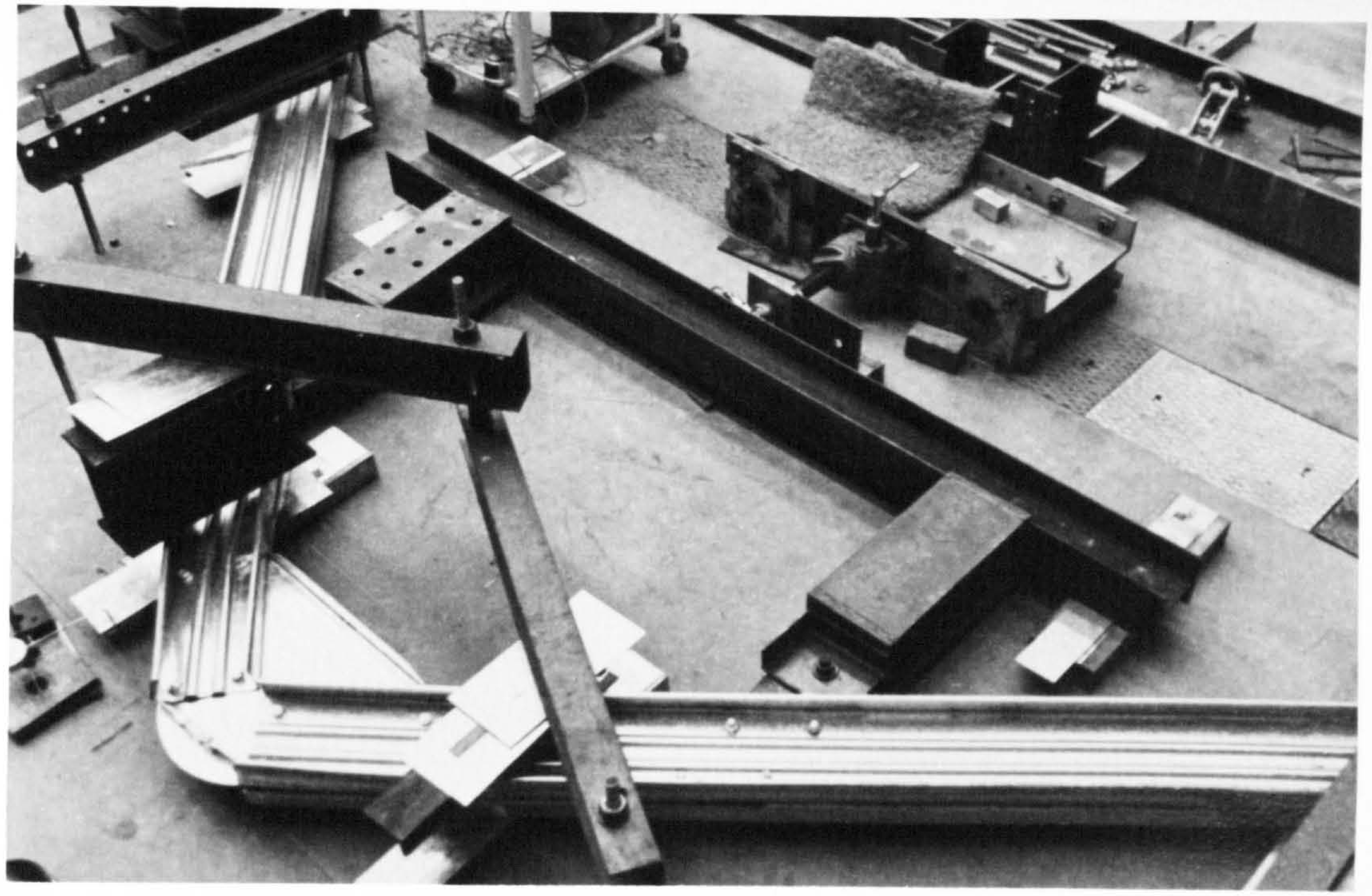
In order to investigate the effect of interlocking on strength and rigidity of bolted connections in cold formed steel sections, a series of tests was carried out on Swagebeams with plain and swaged gussets. Fig. 13-1 shows an Eaves and Apex joint both with plain gussets, tested to failure. It was then intended to compare the results of these tests with exactly similar connections but with swaged gussets. This would then make it possible to determine the effect of interlocking on bolted connections.

However, due to friction in the test set up the results proved inconclusive. These tests will therefore not be elaborated upon here.

Instead, a series of tests previously carried out at Salford[68], on Zeta sleeved purlins, in which sections both nested and interlocked will be considered.



a) Apex joint



b) Eaves joint

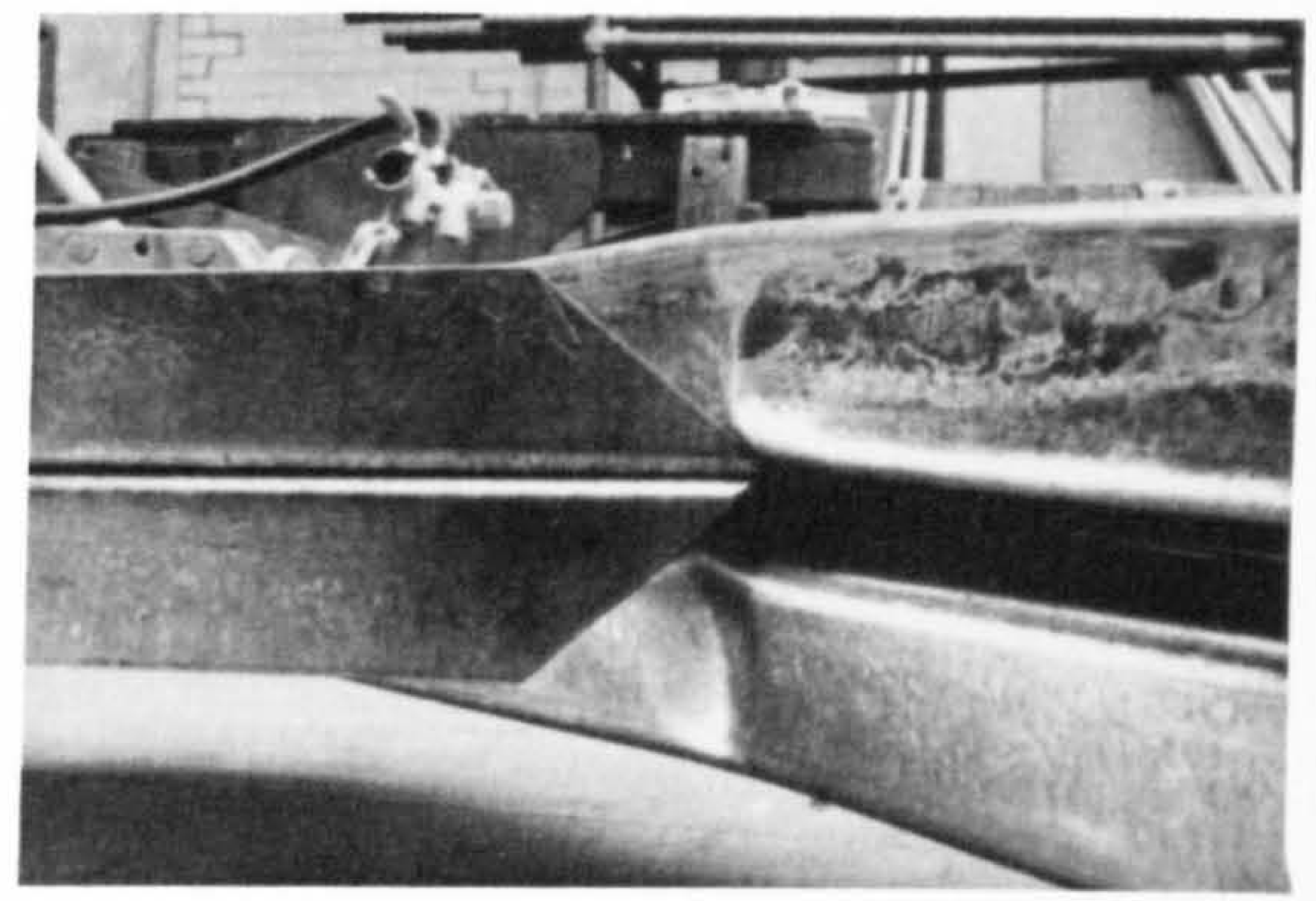


Fig. 13-1 : Tests carried on swage beams with plain gussets

13.2. Tests on Zeta purlins, with sleeves at every rafter

In order to accurately predict the behaviour of sleeved purlins, it is necessary to determine the moment-rotation characteristics of the sleeves.

Tests were carried out on typical sleeved Zeta purlins to obtain design values for moment-rotation characteristics of such sections. The results obtained could then be simply fed into slope-deflection equations to achieve a more even bending moment distribution at the supports and mid spans, hence improving the sections economy. Another purpose of the tests was to obtain the most economical design expression for the section modulus.

13.2.1. Test arrangements

In order to simulate the elastic conditions over a typical support in the laboratory; short span tests (span = 0.4 x full standard span) were carried out. (Fig. 13-2)

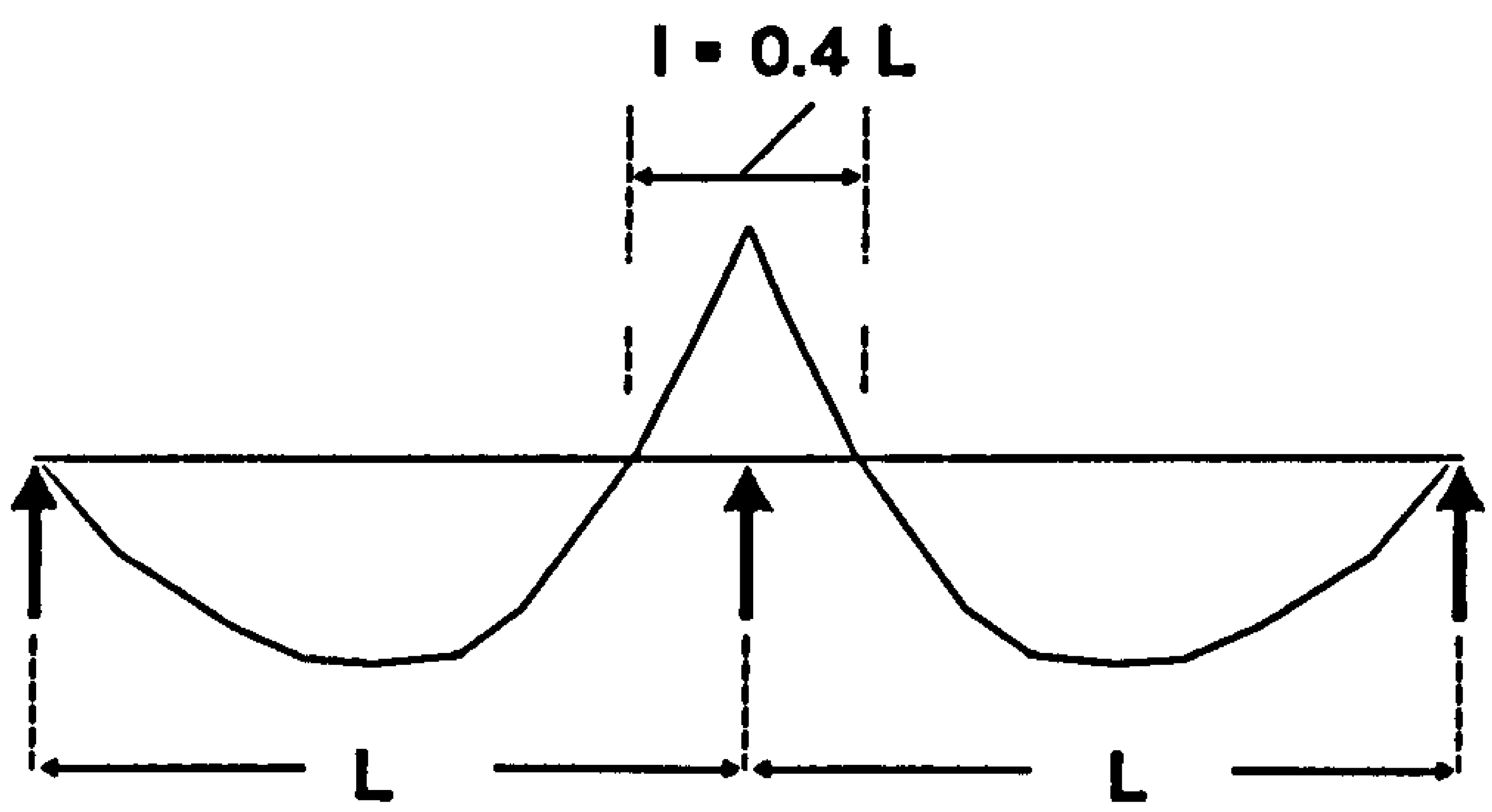


Fig. 13-2 : Simulation of actual elastic support conditions for typical purlin spans.

Therefore the testing procedure adopted was to load the sleeved span to failure with a central point load. (Fig. 13-3)

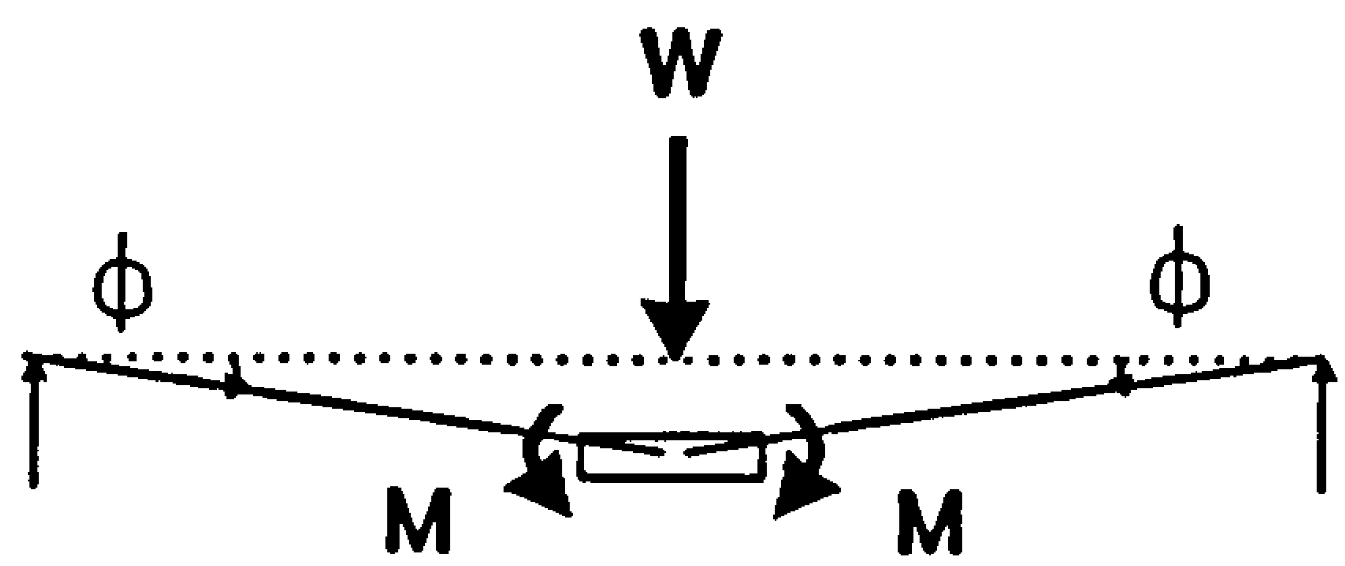


Fig. 13-3 : Three point loading system.

The bending element of deflection would then be differentiated by loading similar sections of uniform length to failure. The difference between the two deflections would therefore be that due to the connection. This principle was described in Chapter Eight (Fig. 8-6).

Hence the moment-rotation characteristics of the bolted sleeved joint could be determined.

13.2.2. Sections tested

Tests were carried out on 125, 150, 175 and 200 mm deep purlins.

With due considerations of typical purlin spans used in practice, for the sizes given above, the test spans adopted were as follows :

Purlin size (mm)	Full size span, L (m)	Test span, l [= 0.4 L] (m)
125	5.5	2.2
150	6.0	2.4
175	6.0	2.4
200	7.5	3.0

With the first three sections two bolt sleeve connections were used. In the case of 200 mm purlins, the section was deep enough to allow two lines of bolts in the web, so a three bolt connection was adopted. Connection dimensions are described in the following section.

13.2.3. Calculation of sleeve dimensions

The lengths of the sleeves were determined by consideration of the spacing required for the bolts to reach the yield moment of the sections at failure.

That is, for two bolt connections :

$$M_y = P_{bs} \cdot b$$

for three bolt connections ;

$$M_y = P_{bs} \sqrt{a^2 + b^2 + ab\sqrt{3}}$$

(Note that the plastic condition was assumed)

The value of P_{bs}, for the given test parameters, was determined in accordance with Addendum No. 1 to BS 449 with a load factor of 1.55 (=1/0.65).

$$P_{bs} = \frac{1.4\,\sigma_{ult}\,d\,t}{0.65}$$

With 16 mm bolt diameters and assuming a nominal ultimate stress of 390 N/mm^2, the ultimate bearing stress could be defined as :

$$P_{bs} = \frac{1.4 \times 0.390 \times 16\,t}{0.65} = 13.44\,t \qquad (kN)$$

Hence in the case of two bolt connections ;

$$280\,Z = 13.44\,t\,b \quad \rightarrow \quad b = 20.83\,Z/t \qquad \text{(mm)}$$

with the three bolt sleeved connection ;

$$280\,Z = 13.44\,t\sqrt{70^2 + b^2 + 70\,b\sqrt{3}}$$

The depth of connection (a = 70 mm) was obviously governed by the section limitations.

Section properties and the calculated sleeve lengths, required to develop the yield moment of the section in bolt bearing at failure, are tabulated below.

Purlin size (mm)	Sheet thickness, t (mm)	Calculated Z (cm^3)	Z / t	b (rounded off)	Total sleeve length (mm)
125		18.34	11.83	250	695
150	1.55	23.41	15.10	315	825
175		28.84	18.61	390	975
200		34.61	22.33	405	1005

The dimensions above are depicted in Fig. 13-4.

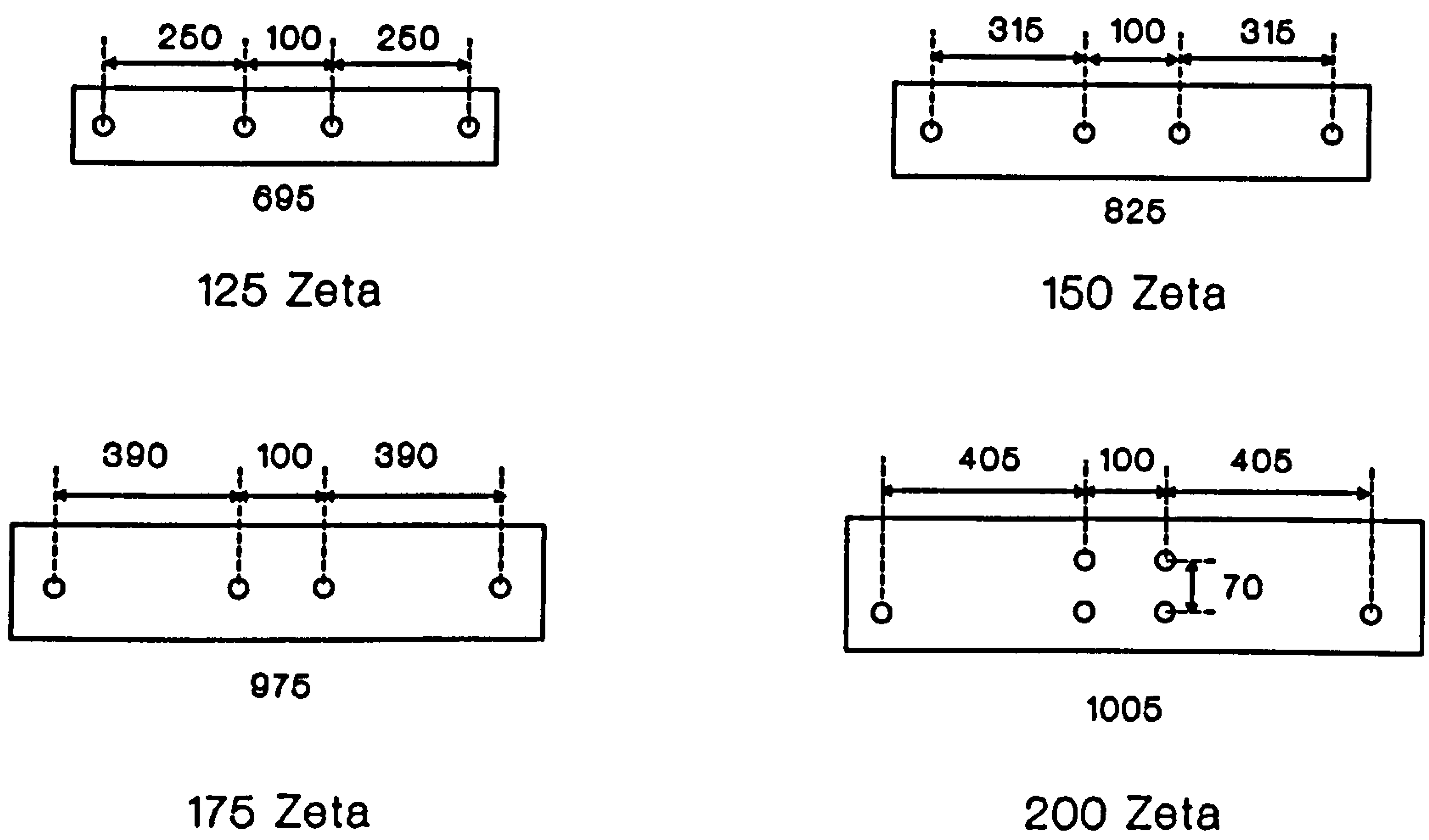


Fig. 13-4 : Sleeve dimensions (All dimensions in mm)

13.2.4. <u>Predicted connection behaviour</u>

The moment-rotation characteristics of the zed sections (nesting) described in the previous chapter are plotted against that of the predicted elastic behaviour, taking account of nesting both in terms of strength and stiffness, in Fig. 13-5. It is seen that the actual and predicted characteristics are in a remarkably good agreement.

The same principle is now applied to Zeta purlins. That is, the predicted elastic M/ϕ at failure is compared to the actual test results obtained.

In the following section subscript c denotes the calculated moment-rotation characteristics of the connection alone, and subscript nest denotes that of connection but taking account of nesting of sections. That is, a 20% increase in moment capacity and 20% reduction in connection flexibility.

Therefore, $\phi_{nest} = 0.8 \times \phi_c$

$M_{nest} = 1.2 \times M_c$

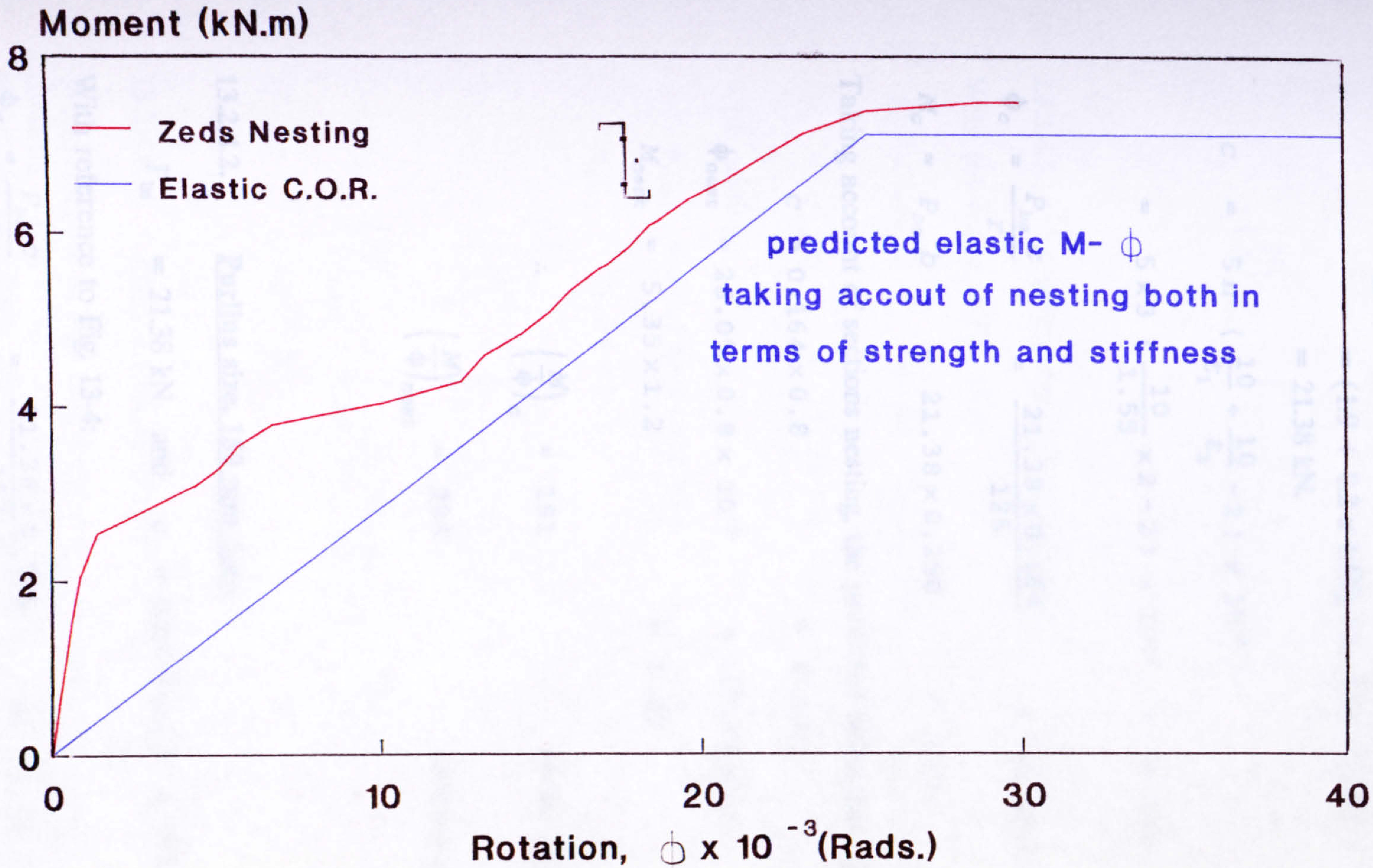


Fig. 13-5

Zeds nesting : Actual and predicted elastic M - ϕ

13.2.4.1. Purlin size, 125 mm deep

With reference to Fig. 13-4 :

$$P_{bs} = \alpha \,.\, d.\, t.\, \sigma_{ult} \qquad = k_2\, d.\, t.\, \sigma_{ult}$$
$$= (1.9 + 0.2 \times 1.55)\ 16 \times 1.55 \times 0.390$$
$$= 21.38 \text{ kN.}$$

$$c = 5n\ \left(\frac{10}{t_1} + \frac{10}{t_2} - 2\right) \times 10^{-3}$$

$$= 5 \times 3\ \left(\frac{10}{1.55} \times 2 - 2\right) \times 10^{-3} = 0.164 \qquad \left(\frac{mm}{kN}\right)$$

$$\phi_c = \frac{P_{bs}\, c}{r} = \frac{21.38 \times 0.164}{125} = 28.05 \times 10^{-3} \qquad (Rads.)$$

$$M_c = P_{bs}\, b = 21.38 \times 0.250 = 5.35 \qquad (kNm)$$

Taking account of sections nesting, the predicted behaviour becomes :

$$c = 0.164 \times 0.8 = 0.131 \qquad (mm/kN)$$

$$\phi_{nest} = 28.05 \times 0.8 \times 10^{-3} = 22.44 \times 10^{-3} \qquad (Rads.)$$

$$M_{nest} = 5.35 \times 1.2 = 6.42 \qquad (kNm)$$

$$\therefore \quad \left(\frac{M}{\phi}\right)_c = 191 \qquad (kNm/Rads.)$$

$$\left(\frac{M}{\phi}\right)_{nest} = 286 \qquad (kNm/Rads.)$$

13.2.4.2. Purlins size, 150 mm deep

P_{bs} = 21.38 kN and c = 0.164 mm/kN as before.

With reference to Fig. 13-4:

$$\phi_c = \frac{P_{bs}\, c}{r} = \frac{21.38 \times 0.164}{157.5} = 22.26 \times 10^{-3} \qquad (Rads.)$$

$$M_c = P_{bs}\, b = 21.38 \times 0.315 = 6.73 \qquad (kNm)$$

Taking account of nesting :

$$\therefore \quad \left(\frac{M}{\Phi}\right)_c = \left(\frac{6.73}{22.26 \times 10^{-3}}\right) = 302 \qquad (kNm/Rads.)$$

$$\left(\frac{M}{\Phi}\right)_{nest} = \left(\frac{8.08}{17.81 \times 10^{-3}}\right) = 454 \qquad (kNm/Rads.)$$

13.2.4.3. Purlins size, 175 mm deep

With reference to Fig. 13-4:

$$\Phi_c = \frac{P_{bs}\,c}{r} = \frac{21.38 \times 0.164}{195} = 17.98 \times 10^{-3} \qquad (Rads.)$$

$$M_c = P_{bs}\,b = 21.38 \times 0.390 = 8.34 \qquad (kNm)$$

With nesting :

$$\Phi_{nest} = 17.98 \times 0.8 \times 10^{-3} = 14.38 \times 10^{-3} \qquad (Rads.)$$

$$M_{nest} = 8.34 \times 1.2 = 10.01 \qquad (kNm)$$

$$\therefore \quad \left(\frac{M}{\Phi}\right)_c = \left(\frac{8.34}{17.98 \times 10^{-3}}\right) = 464 \qquad (kNm/Rads.)$$

$$\left(\frac{M}{\Phi}\right)_{nest} = \left(\frac{10.01}{14.38 \times 10^{-3}}\right) = 696 \qquad (kNm/Rads.)$$

13.2.4.4. Purlins size, 200 mm deep

With reference to Fig. 13-4, for a = 70 mm and b = 405 mm , the critical radius of rotation is that of the bolt furthest away from the centre of rotation of the bolt group. As was shown before, for the elastic assumption, this radius may be obtained as :

$$r_3 = \frac{1}{3}\sqrt{a^2 + 4\,b^2}$$

$$= 271 \qquad mm$$

The moment-rotation of the connection at failure, using the elastic assumption, are therefore obtained as:

$$\phi_c = \frac{P_{bs}\,c}{r} = \frac{21.38 \times 0.164}{271} = 12.94 \times 10^{-3} \qquad (Rads.)$$

$$M_c = \frac{2\,P_{bs}}{\sqrt{a^2 + 4\,b^2}}\,(a^2 + b^2) = 0.416\,P_{bs}$$

$$= 0.416 \times 21.38 = 8.89 \qquad (kNm)$$

With nesting :

$$\phi_{nest} = 12.94 \times 0.8 \times 10^{-3} = 10.35 \times 10^{-3} \qquad (Rads.)$$

$$M_{nest} = 8.89 \times 1.2 = 10.67 \qquad (kNm)$$

$$\therefore \quad \left(\frac{M}{\phi}\right)_c = \left(\frac{8.89}{12.94 \times 10^{-3}}\right) = 687 \qquad (kNm/Rads.)$$

$$\left(\frac{M}{\phi}\right)_{nest} = \left(\frac{10.68}{10.35 \times 10^{-3}}\right) = 1032 \qquad (kNm/Rads.)$$

13.2.5. Test results

The observed behaviour of sleeves and collapse moments for all the tests are tabulated and compared to the predicted values calculated in the previous section :

Purlin size (mm)	Ultimate moment (kNm)			(M/ϕ)		
	Test	M_c	M_{nest}	Test	$(M/\phi)_c$	$(M/\phi)_{nest}$
125	4.95	5.35	6.42	263	191	286
150	7.05	6.73	8.08	685	302	454
175	7.50	8.34	10.01	847	464	696
200	12.75	8.89	10.67	1964	687	1032

13.2.6. Conclusions

From the results tabulated above, the main conclusions drawn are as follows :

i) It is apparent that a minimum of three bolts are required to develop effective nesting of the sections. That is, with two bolt sleeved connections described above the whole connection assembly was not held together tightly enough to develop the full nesting moment capacity M_{nest} (= M_c x 1.2) that was observed with the zed sections, described in Chapter Twelve. With the three bolt connection in the 200 mm purlin on the other hand, the failure moment is well above the predicted M_{nest} value.

ii) The ratios of the actual to predicted $(M/\phi)_{nest}$ at failure are as follows;

Purlin size (mm)	Test / $(M/\phi)_{nest}$
125	0.92
150	1.51
175	1.22
200	1.90

Note that with the 125 mm purlins the connection was not strong enough to make for effective nesting of the sections.

In view of the results it is concluded that, it is amply safe to assume a 20% decrease in flexibility of connections when sections interlock and a further 20% when they nest <u>and</u> interlock.

In the light of the limited nature of the results, it is suggested that no increase in strength (above that already permitted for nesting) should be allowed at this stage.

Chapter Fourteen

Conclusions

14. Conclusions

In cold formed steel sections the strength of the joints may dictate the strength of a member or assembly. This is due to the reduced bearing strength consequent upon the thinness of material. Moreover, the form of typical bolted connections in cold formed steel is such that complete rigidity is difficult to obtain. Therefore consideration of joint flexibility is of fundamental importance and must be considered if structural analysis is to be at all realistic.

Up to date the design of such structures and assemblies has relied upon a combination of testing and rational analysis. Moment-rotation characteristics of various connections are obtained under typical loading conditions. This forms the basis of manufacturers safe load design tables. However, information thus obtained can only be used with reference to few cases where the particulars of a design, such as the span, the number of spans etc. are similar to that of the test conditions.

In this thesis all common factors influencing the strength and rigidity of bolted connections in cold formed steel sections have been investigated individually and quantified. Based on the results obtained a method for calculating the bearing strength of bolts in cold formed steel and estimating the joint flexibility of such connections has been proposed.

The information provided may therefore be used to give economical design of structural assemblies. As for the first time, designers are able to estimate the moment capacity and moment-rotation relationship of bolt groups accurately, without resorting to testing.

A summary of the main points in the thesis is as follows ;

The background to cold formed steel sheeting was given. Particular attention was drawn to the specifications for design of cold formed steel sections. It was shown that the new code of practice BS 5950 : Part 5 has given a new impetus to the use of cold formed sections in structures. The importance of connections in cold formed steel design was emphasized and their modes of failure were derived from the first principles. These principles were then adhered to throughout the thesis.

Reference was made to the past tests carried out on bolted lapped joints at Salford. It was concluded that due to the surface treatment that cold formed sections receive, no significant benefits could be derived from using HSFG bolts in cold formed steel connections.

Current directives on testing of bolted connections in cold formed steel were described. It was concluded that the present specifications in European Recommendations are not economical, in terms of material use and testing time, for structural bolted connections in cold formed steel. Alternative dimensions, equally representative of the insitu conditions, were proposed. Also, a standardised testing method in line with the European Recommendations was put forward.

A comprehensive study of all the factors influencing the strength and rigidity of bolted connections was carried out. These included sheet thickness, bolt diameter, type and use of washers, hole tolerance and end distance of bolts in the line of stress. Design expression for bearing strength and, for the first time, joint flexibility were proposed.

It was shown that the propounded design expression for the bearing strength of sheet material is a significant advance of those existing in the current codes of practice.

The background to the existing codes of practice was given and it was explained how this project has been instrumental in shaping the existing equation for the bearing strength in Annex A to Eurocode No. 3. It is hoped that the conclusions drawn in this thesis are further incorporated in the above mentioned code.

The interaction of the proposed design equation for the bearing strength with other modes of failure of bolted connections were considered and it was proven that propounded equation gave logical boundaries, vindicated by test results obtained at Salford and elsewhere, with other modes of failure. The same however can not be said about the existing equations in the current codes of practice.

It was shown that the strength of a bolted connection is governed by the thinness connected sheet, while the flexibility of such fastenings is equal to the sum of the flexibilities of the connected sheets.

The design expressions derived were used to estimate the moment capacity and moment/rotation relationship of bolt groups, and it was shown how this information may be used to give economical design of structural assemblies.

The location of the centre of rotation of three bolt connections was investigated in detail, and it was concluded that the plastic assumption commonly used with three bolt connections in cold formed steel is not valid. It was suggested that the elastic assumption, as with three bolt connections in hot rolled steel, should be used instead.

The effects of nesting and interlocking of sections were investigated separately and suitable design factors were proposed.

Based on the analyses carried out and results obtained the general load-extension characteristics of bolted connections in cold formed steel sections, taking account of all common factors influencing the behaviour of such connections, were defined as follows ;(Fig. 14-1)

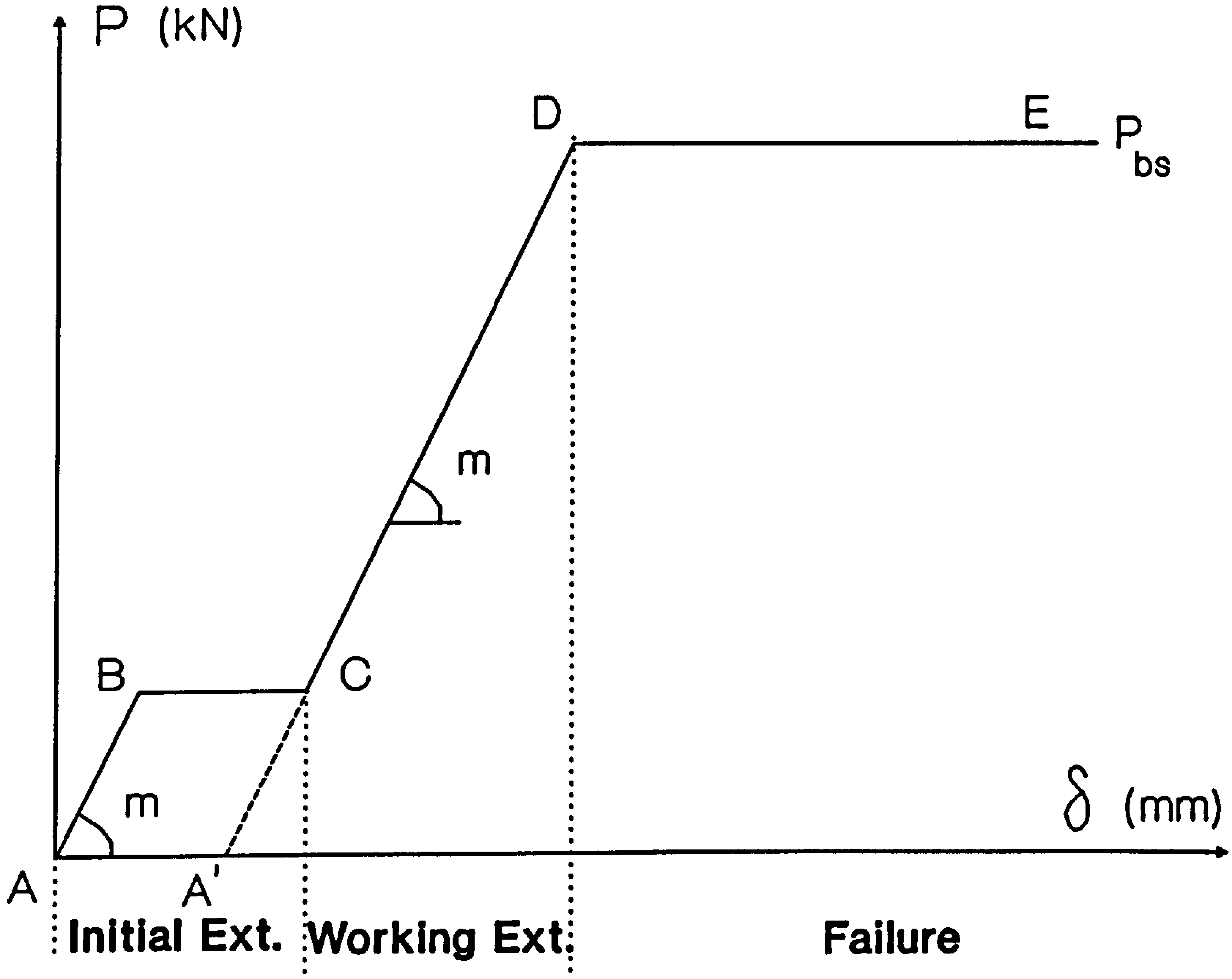


Fig. 14-1 : General Load/Extension characteristics of bolted connections in cold formed steel sections.

The variables listed in Fig. 14-1 above, are as follows:

The Ultimate bearing strength $P_{bs} = \alpha \,.\, d.t.\sigma_{ult}$

Where α is defined as :

$$\alpha = k_1.k_2.k_3.k_3.k_4.k_5.k_6.k_7$$

k_1 to k_7 are factors given for the variables listed below :

Bolt diameter; $k_1 = (16/d)^{1/2}$	**Sheet thickness;** $k_2 = (1.9 + 0.2\,t)$ for $t \leq 3mm$ $= 2.5$ $\quad 3 < t \leq 8mm$
Mechanical properties; $k_3 = (390/\sigma_{ult.\,design})^{1/2}$ where $\sigma_{ult.\,design}$ is the design ultimate stress of the sheet material.	**Washer diameter;** For Normal diameter washers, (Form E, BS 4320) $k_4 = 1.0$. For Large diameter washers, (Form F, BS 4320) $k_4 = 1.15$ for $t \leq 2mm$ $= 1.05$ for $2 < t \leq 3mm$ $= 1.0$ for $t > 3mm$
Number of washers; $k_5 = 1.0$ when two washers are used. $= 0.8$ when only one washer is used. $= 0.7$ when no washers are used.	**End distance in the line of stress;** $k_6 =$ the lesser of $(e/2.5d)$ and 1. where $(e/d) \geq 1.5$.
Shear plane on the plain shank or threads of bolts; $k_7 = 1.15$ where it can be shown be shown that the shear plane occurs over the full shank diameter. $= 1.0$ otherwise	

The **flexibility** of a connection (1/m, in Fig. 14-1) is defined as :

$$\boxed{c = 5n\left(\frac{10}{t_1} + \frac{10}{t_2} - 2\right) \times 10^{-3} \qquad \left(\frac{mm}{kN}\right)}$$

where c = the joint flexibility (in mm/kN)

t_1 and t_2 are the sheet thicknesses (in mm)
note that from definition of cold formed sections $t_1 \leq 8mm$ and $t_2 \leq 8mm$.

The results of all the analyses carried out on flexibility of bolted connections is incorporated in the factor n as follows:

Position of the shear plane on the bolts	For joints in tension	For joints under moment		
		Simple bolted joints	Joints which nest or interlock	Joints which nest and interlock
Full shank diameter	3	1.8	1.4	1.2
Threaded portion	5	3	2.4	2.0

The **slip load** of a bolted connection may be taken as **4 kN.**

The **amount of slip** may be taken as the **bolt clearance.**

If the design assumption is that the shear plane occurs over the plain portions of bolts, the length of the plain shank of the bolts in no circumstances should exceed :

$$l_{ps} \leq (t_1 + t_2) + n_w . t_w$$

Otherwise the bolts can not be tightened.

l_{ps} = length of the plain shank,
t_1, t_2 = thicknesses of the connected sheets,
t_w = washer thickness,
n_w = number of washers.

The general load-extension characteristics may be defined by the following coordinates (Fig. 14-1):

A (0, 0)
B (4c, 4)
C [(4c+ hole clearance), 4]
D [(P_{bs} c + hole clearance), P_{bs}]

It was described that in practice the initial extension of a connection, depicted in Fig. 14-1, is considered to occur under the self weight of a structure - in other words the clearance slip is to be ignored in the design process.
Therefore the A'DE portion of the general load-extension characteristics is critical for the design process (the origin is in turn shifted to point A') i.e. :

A'(0, 0)
D (P_{bs} c, P_{bs})

It was shown that the equation defining the bearing strength could be equally expressed in terms of yield stress as follows :

$$P_{bs} = \alpha \,.\, d.t.\sigma_y$$
$$\alpha = k_1.k_2.k_3.k_4.k_5.k_6.k_7$$

Where $k_2 = (2.6 + 0.3\ t)$ for $t \leq 3$ mm
$= 3.5$ for $3mm \leq t \leq 8$ mm

and $k_3 = (280/\sigma_{y\ design})^{1/2}$

k_1 and k_4 to k_7 all being the same as before.

An increase of 20% in the ultimate moment carrying capacity of connections with three or more bolts was proposed when sections nested. In the light of the limited nature of the test results it was suggested that no increase in strength (above that already permitted for nesting) should be allowed at this stage. The results obtained also suggested that with two bolt connections the sections are not held tightly enough to make for effective nesting.

The above criteria, for strength and rigidity, were successfully applied to typical moment connections and it was concluded that they accurately predict the moment capacity and moment-rotation characteristics of such connections.

As a concluding remark it may suffice to say that one of the grey areas of design in cold formed steel structures, where it has often been necessary to resort to testing, has been successfully clarified.

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Appendix 1

Detailed test programme on lap joints

Hole tolerance	Thickness	Edge	Average σ_y , σ_{ult} (N/mm²)	Test No.	Ultimate load (kN)
Perfect fit	t = 1.50 mm	lips	300.5 , 384.8	1-3	38.0,34.6,34.8
		no lips	300.5 , 384.8	13-15	24.9,23.8,22.0
	t = 1.96* mm	lips	206.2 , 303.9	4-6	35.8,35.7,35.8
		no lips	206.2 , 303.9	16-18	28.5,29.1,29.6
	t = 2.57 mm (2.48)	lips	341.3 , 414.1	(74-76)	61.5,63.0,61.4
		no lips	324.4 , 378.6	7-9,(80)	55.3,47.5,57.0,61.2
	t = 3.17 mm (3.02)	lips	319.0 , 394.8	(77-79)	71.2,70.5,72.5
		no lips	348.2 , 423.6	10-12,(72)	75.8,76.1,75.8,71.1

- * Specimens with sheet thickness of 1.96 mm had a lower yield stress (nominal 210 N/mm²) compared to the rest. (Nominal ultimate stress, 270 N/mm²)
- All specimens with 2 large diameter (M16, BS 4320 Form F) bright steel washers.
- For other test constants see § 4.2.1.

Fig. A1.1: Lap tests detailed programme.

Perfect fit lap joints. Effect of restraining sheet edges

				Average σ_y , σ_{ult} (N/mm^2)	Test No.	Ultimate load (kN)
Perfect fit	occurrence of bolt threads in the shear plane	t = 1.50 mm (with lips)	Large diameter bright steel washers	307.9 , 395.1	36,37	31.8,30.4
			Normal diameter mild steel washers	307.9 , 395.1	35	25.3
		t = 2.57 mm (without lips)	Large diameter bright steel washers	323.0 , 367.0	38-40	54.0,55.5,54.2
		t = 3.17 mm (without lips)	Large diameter bright steel washers (failure by shearing of the bolt)	350.1 , 435.0	41,42	54.0,54.4
			Normal diameter mild steel washers	350.1 , 435.0	43	63.8

- M16 set screws
- Washers as noted
- Failure by sheet bearing, unless otherwise stated .
- Lips as noted; for other test constants see § 4.2.1.

Fig. A1.2 **Lap tests detailed programme**

Perfect fit lap joints **Occurrence of bolt threads in the shear plane**

				Average σ_y , σ_{ult} (N/mm^2)	Test No.	Ultimate load (kN)
Hole tolerance	clearance holes	t = 2.48 mm (2.57)	lips	338.1 , 408.4	70,71,73,81	45.7,47.5,49.0,46.4
			no lips	318.8,366.8	(23-25)	36.0,34.1,44.7
		t = 3.02 mm [3.12] (3.17)	lips	306.7 , 384.4	62,118,[190]	58.7,53.0,59.2
			no lips	348.2 , 423.6	(26-28)	57.5,57.5,59.1

<u>Fig. A1.3</u> <u>Lap tests, detailed programme</u>

<u>Clearance holes</u> **<u>Effect of restraining sheet edges</u>**

	Thickness	Washer	Average σ_y , σ_{ult} (N/mm^2)	Test No.	Ultimate load (kN)
Washer material and diameter	t = 1.63 mm [1.50] (1.57)	(a) Normal dia., standard	306.5 , 392.4	[19-22]	24.8,23.4,24.4,25.1
		(b) Normal dia., different manufacturer	330.3 , 374.9	121,133,(147)	25.9,26.0,26.7
		(c) Large dia., mild steel	329.0 , 374.2	125,132,(155)	33.2,32.9,33.6
		(d) Large dia., machined bright steel	330.7 , 375.1	124,131,(154)	33.2,33.9,33.9
	t = 2.48 mm (2.45)	(a) Normal dia., standard	338.1 , 408.4	70,71,73,81	45.7,47.5,49.0,46.4
		(b) Normal dia., different manufacturer	324.1 , 404.0	122,(140),153	42.8,43.4,42.0
		(c) Large dia., mild steel	324.1 , 404.0	119,(139),152	53.0,52.5,49.5
		(d) Large dia., machined bright steel	323.7 , 405.0	123,(138),151	55.6,52.0,52.8
	t = 3.02 mm (3.12)	(a) Normal dia., standard	306.7 , 384.4	62,118,(190)	58.7,53.0,59.2
		(b) Normal dia., different manufacturer	298.6 , 375.4	127,128,(148)	49.9,49.0,52.0
		(c) Large dia., mild steel	307.3 , 385.1	126,130,(149)	60.2,59.5,59.9
		(d) Large dia., machined bright steel	307.3 , 385.1	120,129,(150)	64.6,59.9,65.0

Fig. A1.4 **Lap tests detailed programme**

Clearance holes **Washer material and diameter**

Number and position of washers	Thickness	Washers	Lips	Average σ_y , σ_{ult} (N/mm^2)	Test No.	Ultimate load (kN)
Number and position of washers	t = 1.63 mm (1.50)	2 washers		306.5 , 392.4	(19-22)	24.8,23.4,24.4,25.1
		1 washer under bolt head		326.6 , 371.3	48-50	21.5,23.3,21.9
		1 washer under nut		326.6 , 371.3	52,55,57	20.7,21.7,21.6
		no washers		326.6 , 371.3	59,61,68	18.7,18.2,17.6
	t = 2.48 mm [2.50]	2 washers		338.1 , 408.4	70,71,73,81	45.7,47.5,49.0,46.4
		1 washer under bolt head		336.5 , 415.6	[51,53,54]	41.0,39.7,39.6
		1 washer under nut		336.5 , 415.6	[56,58,60]	38.6,37.7,38.4
		no washers		338.1 , 408.4	66,67,69	34.7,33.9,33.4
	t = 3.02 mm [3.12] (3.17)	2 washers		306.7 , 384.4	62,118,[190]	58.7,53.0,59.2
		1 washer under bolt head	lips	314.4 , 394.2	63,[188]	47.2,47.2
			no lips	349.2 , 429.3	(29-31)	52.2,53.9,53.9
		1 washer under nut	lips	314.4 , 394.2	64,[189]	45.5,46.8
			no lips	338.5 , 417.1	(32,34),44	48.2,49.8,40.8
		no washers	lips	314.4 , 394.2	65,[178]	40.1,43.4
			no lips	317.2 , 392.8	45-47	38.9,38.4,39.2

Fig. A1.5 Lap tests, detailed programme

Clearance holes Number and position of washers

			Average σ_y , σ_{ult} (N/mm^2)	Test No.	Ultimate load (kN)
Clearance holes	Mechanical properties of steel sheets	t = 1.57 mm	436.4 , 482.0	135,145,146	32.4,32.0,33.0
		t = 2.37 mm	396.0 , 479.7	134,136,141	49.0,45.8,48.5
		t = 3.11 mm	397.2 , 473.5	142-144	63.4,66.7,66.2

Fig. A1.6 Lap tests, detailed programme

Clearance holes Mechanical properties of steel sheets

End distance of bolt in line of stress	e/d	Average σ_y , σ_{ult} (N/mm²)	Test No.	Ultimate load (kN)
t = 1.63 mm (1.50)	1.5	329.0 , 373.6	116,117,169	18.1,17.5,18.0
	2.25	329.0 , 373.6	104,105,173	25.6,27.7,28.6
	3.0	329.0 , 373.6	93,100,161	28.3,28.5,27.4
	3.75	306.5 , 392.4	(19-22)	24.8,23.4,24.4,25.1
	4.5	323.5 , 371.8	89,91,163	27.7,27.2,27.8
	5.25	323.5 , 371.8	108,109,171	27.4,28.8,29.5
	6.0	323.5 , 371.8	113,115,166	28.5,26.9,28.9
t = 2.48 mm (2.45)	1.5	333.2 , 411.2	99,101,(174)	28.5,30.4,28.7
	2.25	333.2 , 411.2	95,96,(178)	45.6,46.8,43.5
	3.0	333.2 , 411.2	92,94,(162)	47.4,45.9,45.3
	3.75	338.1 , 408.4	70,71,73,81	45.7,47.5,49.0,46.4
	4.5	335.7 , 410.9	88,90,(168)	46.5,47.3,45.4
	5.25	334.6 , 410.5	106,107,(172)	47.6,46.9,44.6
	6.0	334.6 , 410.5	112,114,(170)	48.2,48.3,45.5
t = 3.02 mm (3.12)	1.5	304.3 , 381.0	97,98,(179)	36.1,34.4,37.0
	2.25	304.3 , 381.0	86,87,(164)	52.6,53.0,55.2
	3.0	304.3 , 381.0	84,85,(175)	57.7,54.2,56.0
	3.75	306.7 , 384.4	62,118,(190)	58.7,53.0,59.2
	4.5	335.0 , 384.4	82,83,(167)	55.1,54.5,60.6
	5.25	303.7 , 381.1	102,103,(177)	58.5,58.0,58.6
	6.0	303.7 , 381.1	110,111,(165)	54.9,55.0,59.2

Fig. A1.7 Lap tests, detailed programme

Clearance holes End distance of bolt in line of stress

Bolts	Thickness	Bolt grade	Average σ_y , σ_{ult} (N/mm²)	Test No.	Ultimate load (kN)
12 mm ϕ bolts	t = 1.63 mm	Grade 4.6 bolts	331.5 , 375.5	157,159,160	23.3,22.5,23.5
	t = 2.45 mm	Grade 4.6 bolts (failure by shear of bolt)	313.4 , 398.2	156	36.0
		Grade 8.8 bolts	313.4 , 398.2	176,182 196 (e/d = 5)	39.4,38.0 42.0
	t = 3.12 mm	Grade 4.6 bolts (failure by shear of bolt)	309.7 , 394.7	158	35.0
		Grade 8.8 bolts	309.7 , 394.7	180,181	50.0,51.3

- All specimens with 2 mm clearance holes, i.e. 14 mm ϕ holes.
- End distance in line of stress, e = 45mm. i.e. e/d = 3.75, except for test 196 where e = 60 mm, i.e. e/d = 5 .
- 2 M12 normal diameter washers, one under bolt head and one under nut.
- Failure by sheet bearing unless stated otherwise.
- For other test constants see § 4.2.1.

Fig. A1.8 Lap tests, detailed programme,

Clearance holes Bolt diameter

Appendix 2

Load-extension characteristics of lap tests

Contents

Load-extension characteristics of all lap tests listed in Appendix 1 have been plotted in the order listed below.

To further clarify the results produced, each set of graphs has been classified in accordance with Fig. 4.2. The relevant test variables have been high-lighted and underlined, in the same format as the above figure, before each set of graphs.

Fig. A2.14A	Washer material and diameter : Nominal thickness, t = 2.48 mm. (Normal diameter washers)
Fig. A2.14B	Washer material and diameter : Nominal thickness, t = 2.48 mm. (Large diameter mild steel washers)
Fig. A2.14C	Washer material and diameter : Nominal thickness, t = 2.48 mm. (Large diameter bright steel washers)
Fig. A2.14D	Washer material and diameter : Nominal thickness, t = 2.48 mm. (Large diameter washers in B and C above compared against each other)
Fig. A2.15A	Washer material and diameter : Nominal thickness, t = 3.02 mm. (Normal diameter washers)
Fig. A2.15B	Washer material and diameter : Nominal thickness, t = 3.02 mm. (Large diameter mild steel washers)
Fig. A2.15C	Washer material and diameter : Nominal thickness, t = 3.02 mm. (Large diameter bright steel washers)
Fig. A2.15D	Washer material and diameter : Nominal thickness, t = 3.02 mm. (Large diameter washers in B and C above compared against each other)
Fig. A2.16A	Number and position of washers : One washer under bolt head : t = 1.63 mm.
Fig. A2.16B	Number and position of washers : One washer under nut : t = 1.63 mm.
Fig. A2.16C	Number and position of washers : no washers, t = 1.63 mm.
Fig. A2.17A	Number and position of washers : One washer under bolt head : t = 2.48 mm.
Fig. A2.17B	Number and position of washers : One washer under nut : t = 2.48 mm.
Fig. A2.17C	Number and position of washers : no washers, t = 2.48 mm.
Fig. A2.18A	Number and position of washers : One washer under bolt head : t = 3.02 mm.
Fig. A2.18B	Number and position of washers : One washer under nut : t = 3.02 mm.

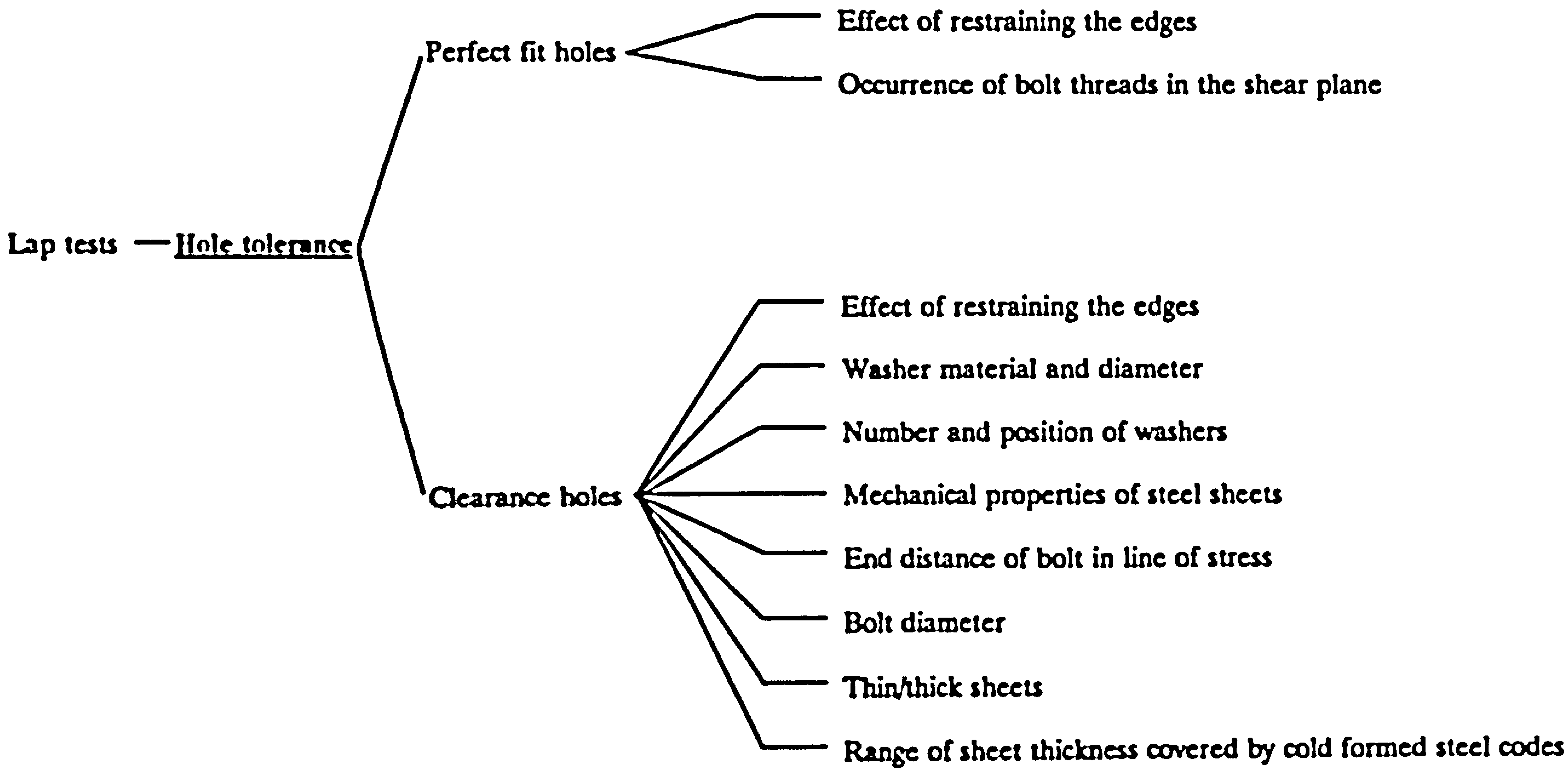

Lap tests
Hole tolerance
Perfect fit holes
Effect of restraining the edges
Occurrence of bolt threads in the shear plane
Clearance holes
Effect of restraining the edges
Washer material and diameter
Number and position of washers
Mechanical properties of steel sheets
End distance of bolt in line of stress
Bolt diameter
Thin/thick sheets
Range of sheet thickness covered by cold formed steel codes

Fig. A2.1 Hole tolerance : Nominal sheet thickness, t = 1.50 mm

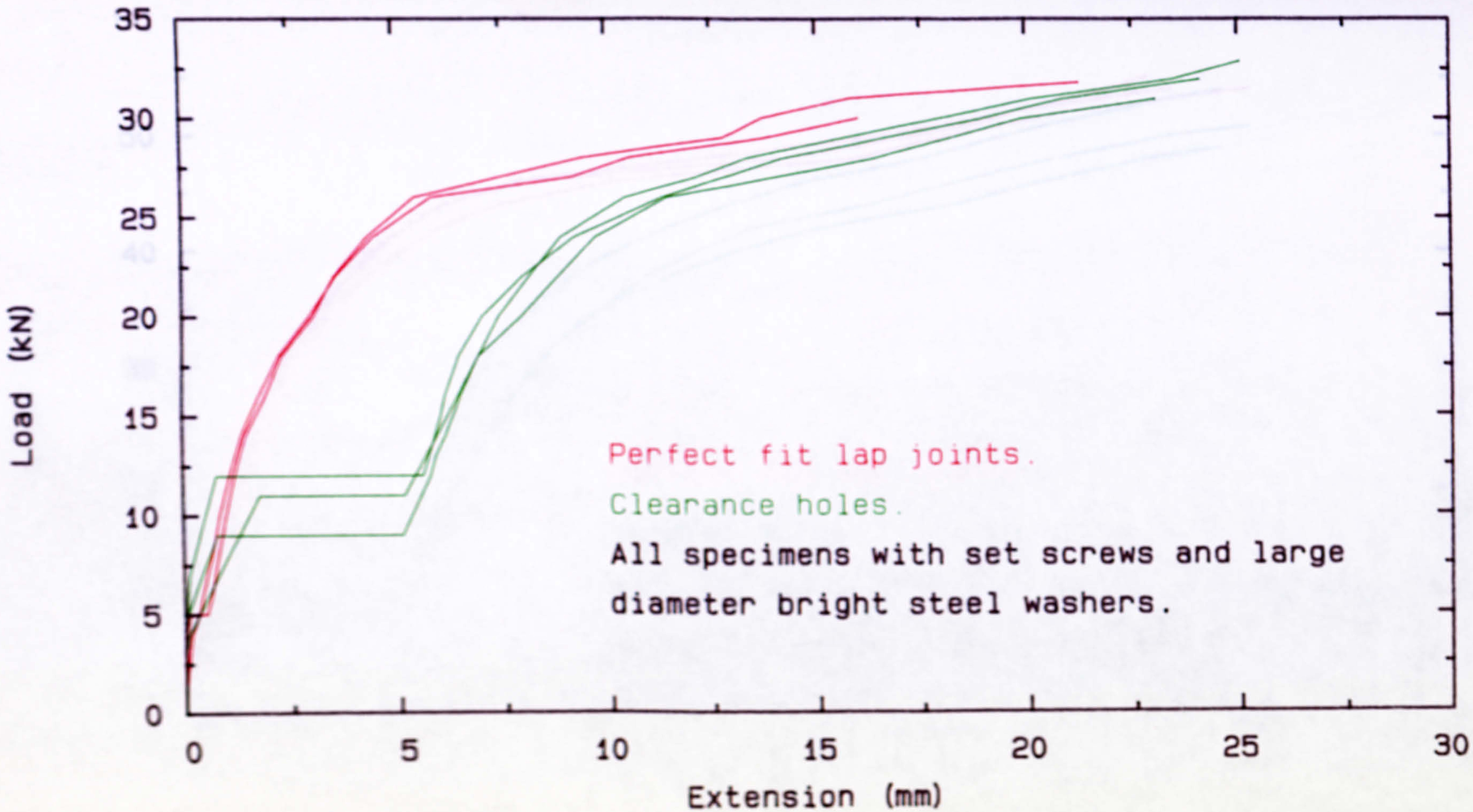


Fig. A2.2 Hole tolerance : Nominal sheet thickness, t = 2.50 mm

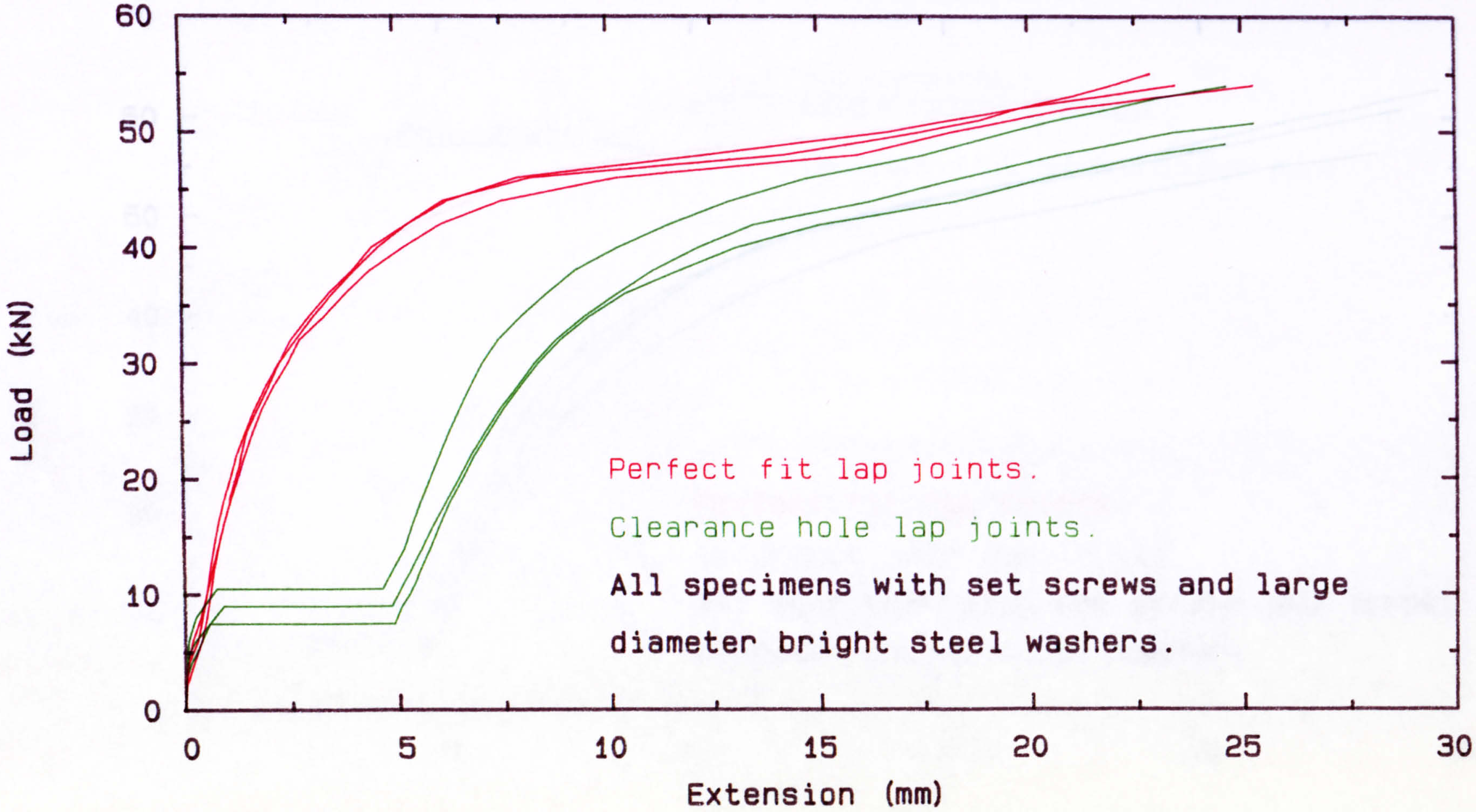
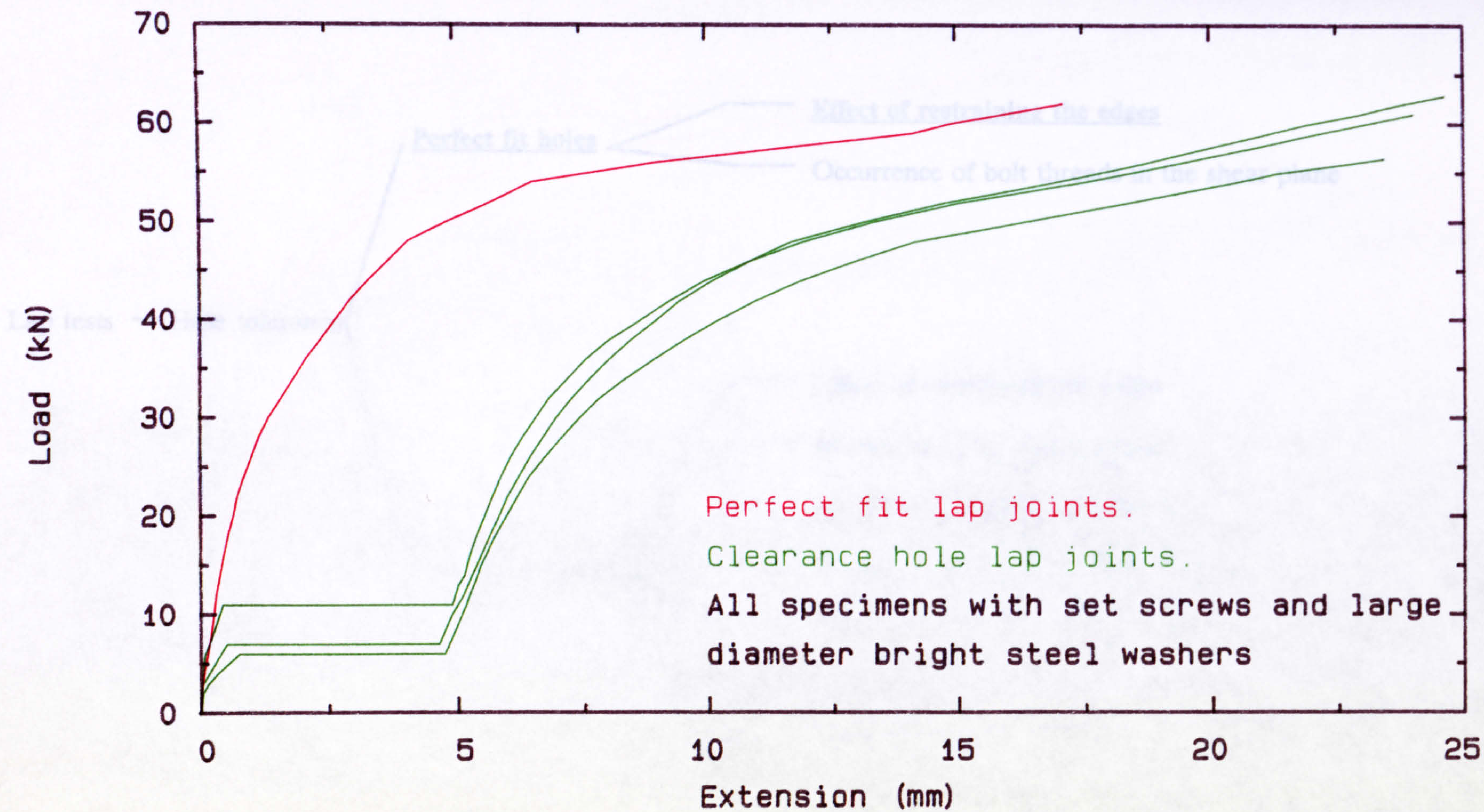


Fig. A2.3 Hole tolerance : Nominal sheet thickness, t = 3.02 mm



- Lap tests
  - Hole tolerance
    - **Perfect fit holes**
      - **Effect of restraining the edges**
      - Occurrence of bolt threads in the shear plane
    - Clearance holes
      - Effect of restraining the edges
      - Washer material and diameter
      - Number and position of washers
      - Mechanical properties of steel sheets
      - End distance of bolt in line of stress
      - Bolt diameter
      - Thin/thick sheets
      - Range of sheet thickness covered by cold formed steel codes

FIG. A2.4 Effect of edge restraint , Perfect Fit joints : t = 1.50 mm

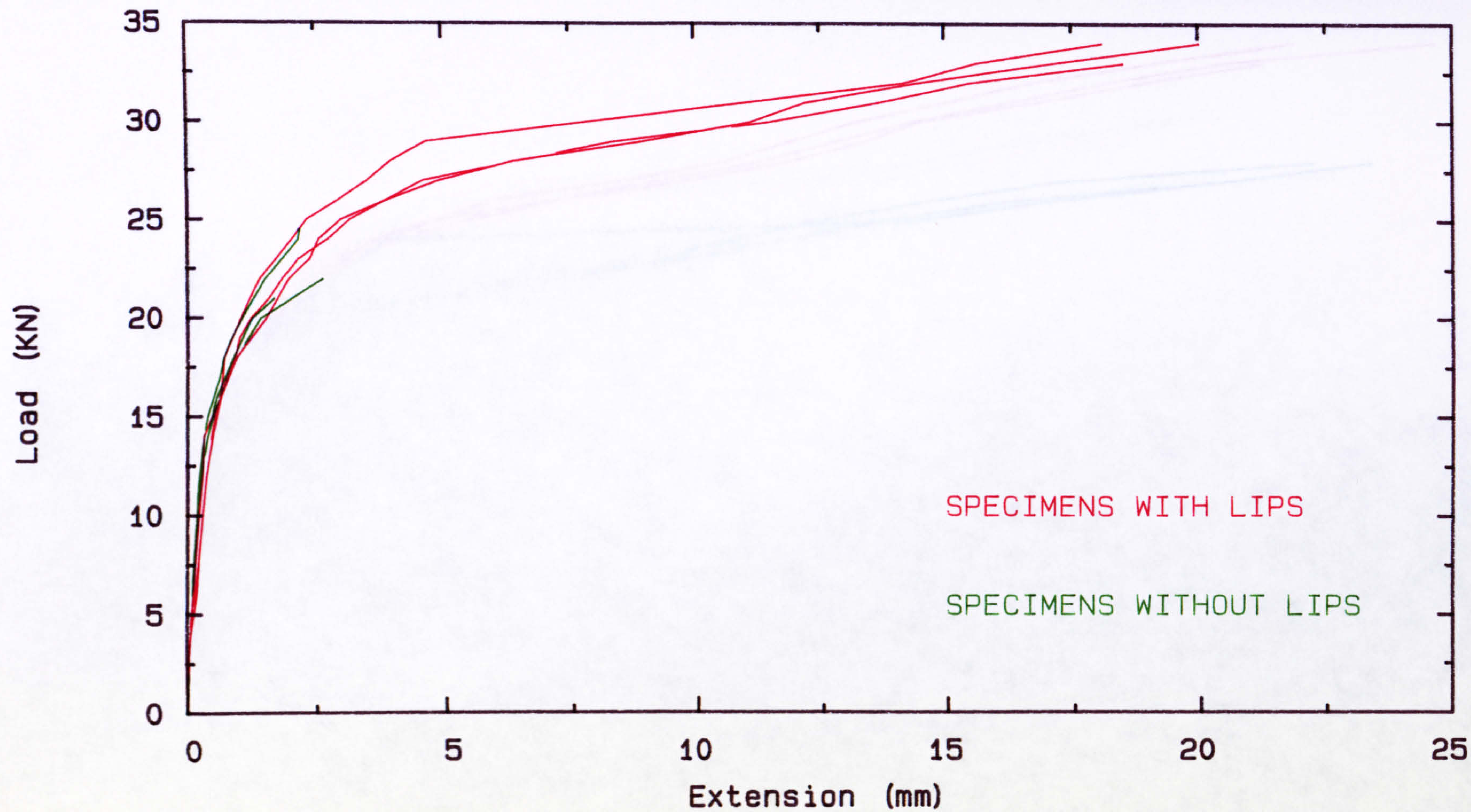


FIG. A2.5 Effect of edge restraint ,Perfect Fit joints :
t = 1.96 mm

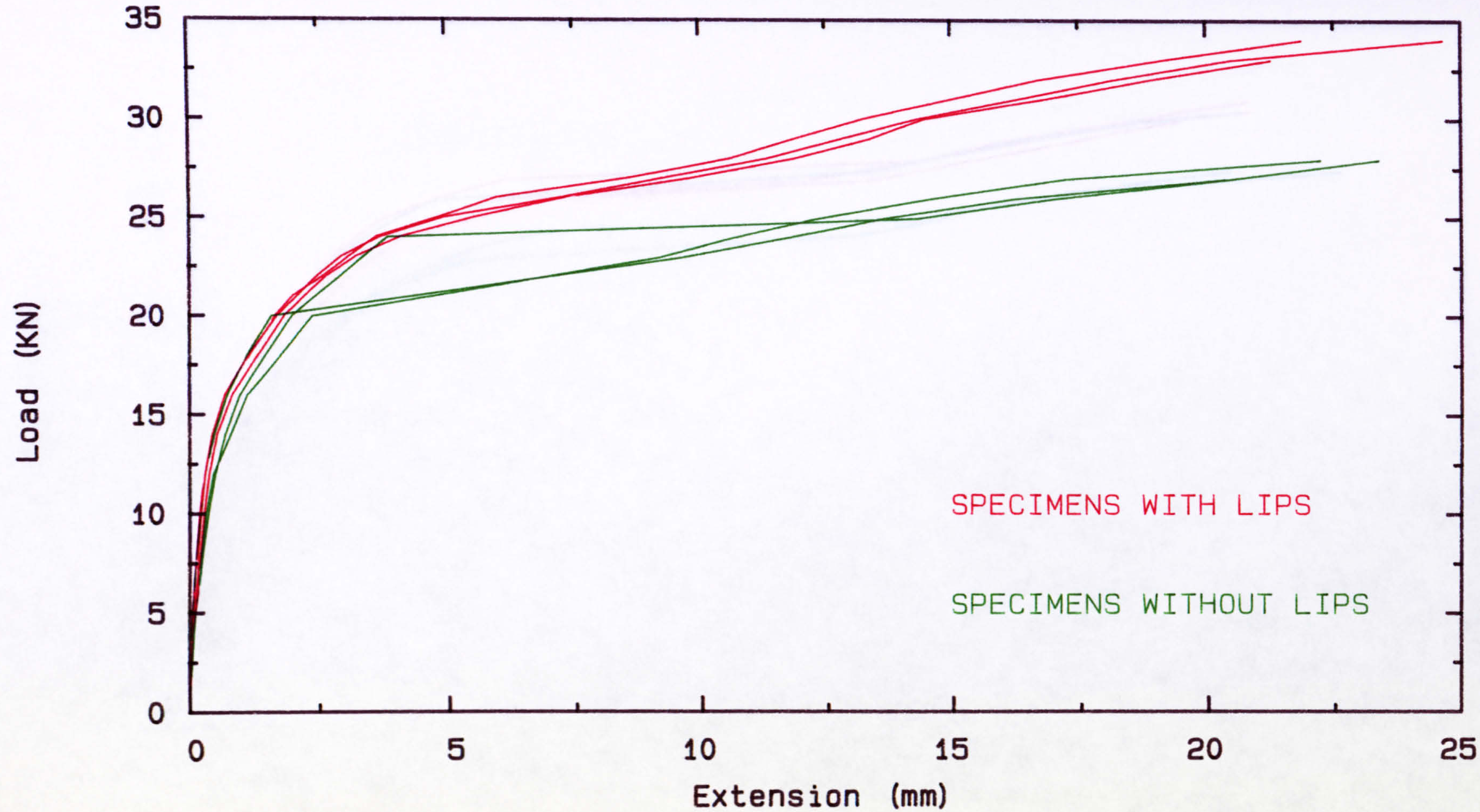


FIG. A2.6 Effect of edge restraint , Perfect Fit joints : t = 2.48 mm

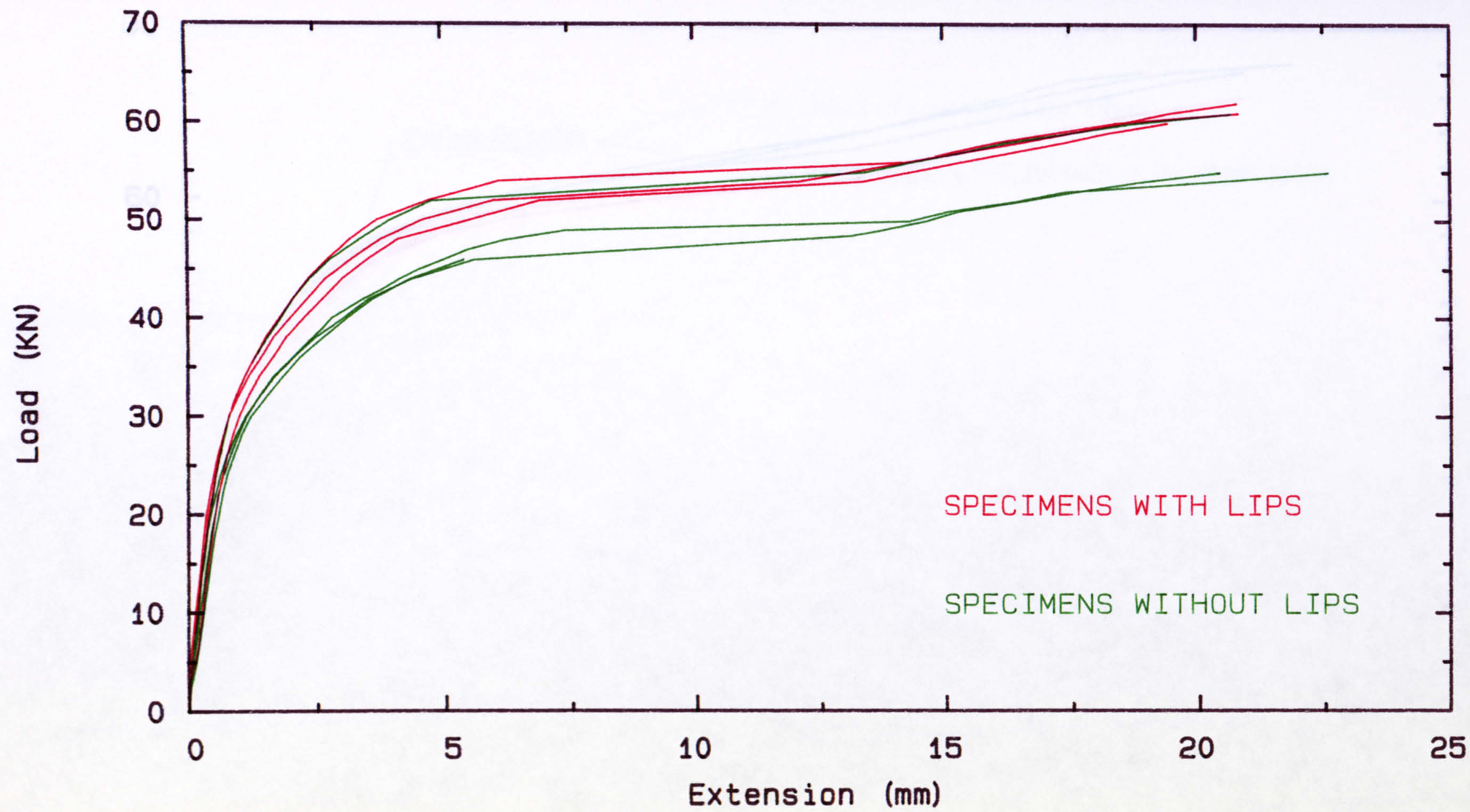
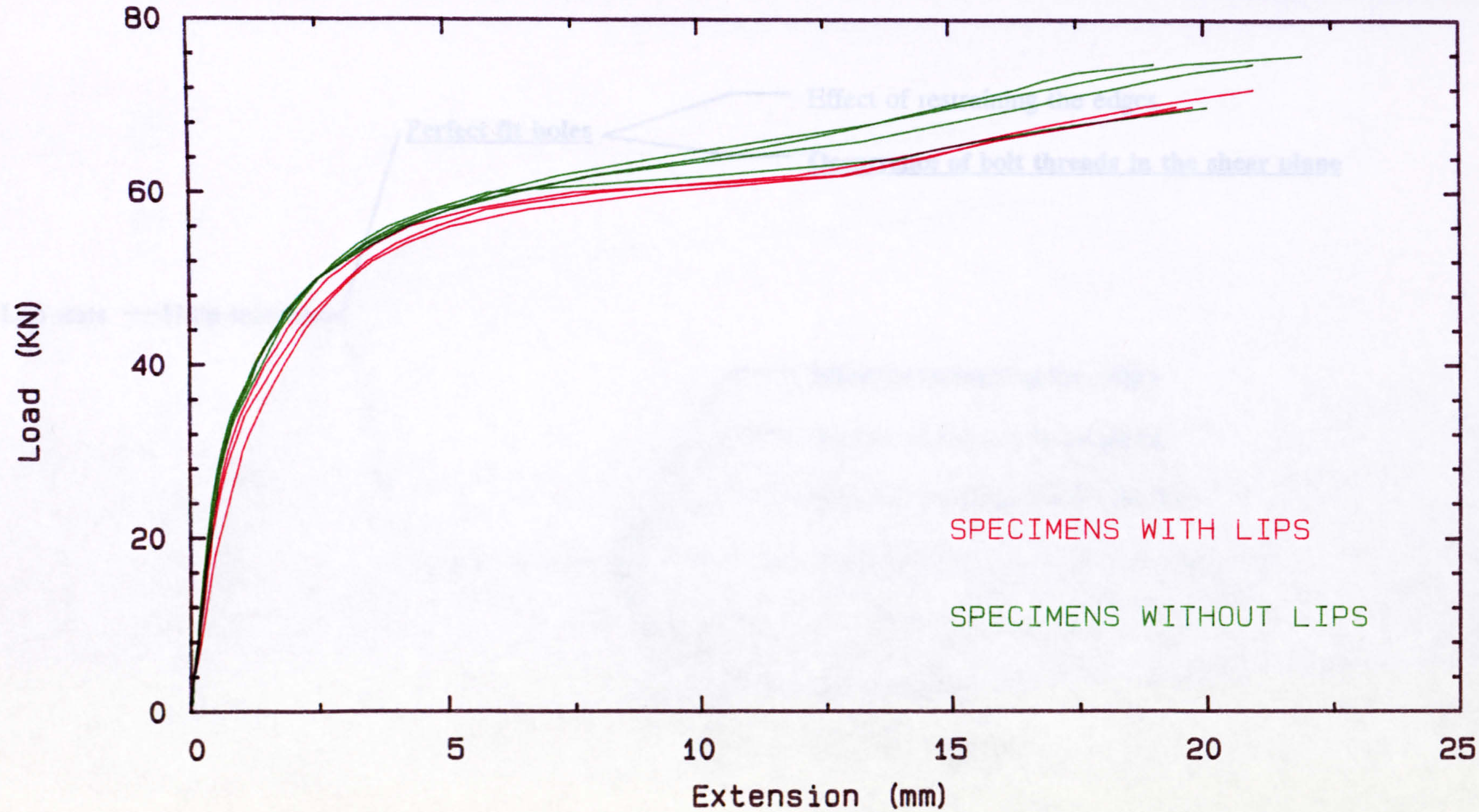


FIG. A2.7 Effect of edge restraint , Perfect Fit joints :
t = 3.02 mm



- Lap tests
  - Hole tolerance
    - **Perfect fit holes**
      - Effect of restraining the edges
      - **Occurrence of bolt threads in the shear plane**
    - Clearance holes
      - Effect of restraining the edges
      - Washer material and diameter
      - Number and position of washers
      - Mechanical properties of steel sheets
      - End distance of bolt in line of stress
      - Bolt diameter
      - Thin/thick sheets
      - Range of sheet thickness covered by cold formed steel codes

FIG. A2.8 Effect of the occurrence of bolt threads in the shear plane : t = 1.50 mm

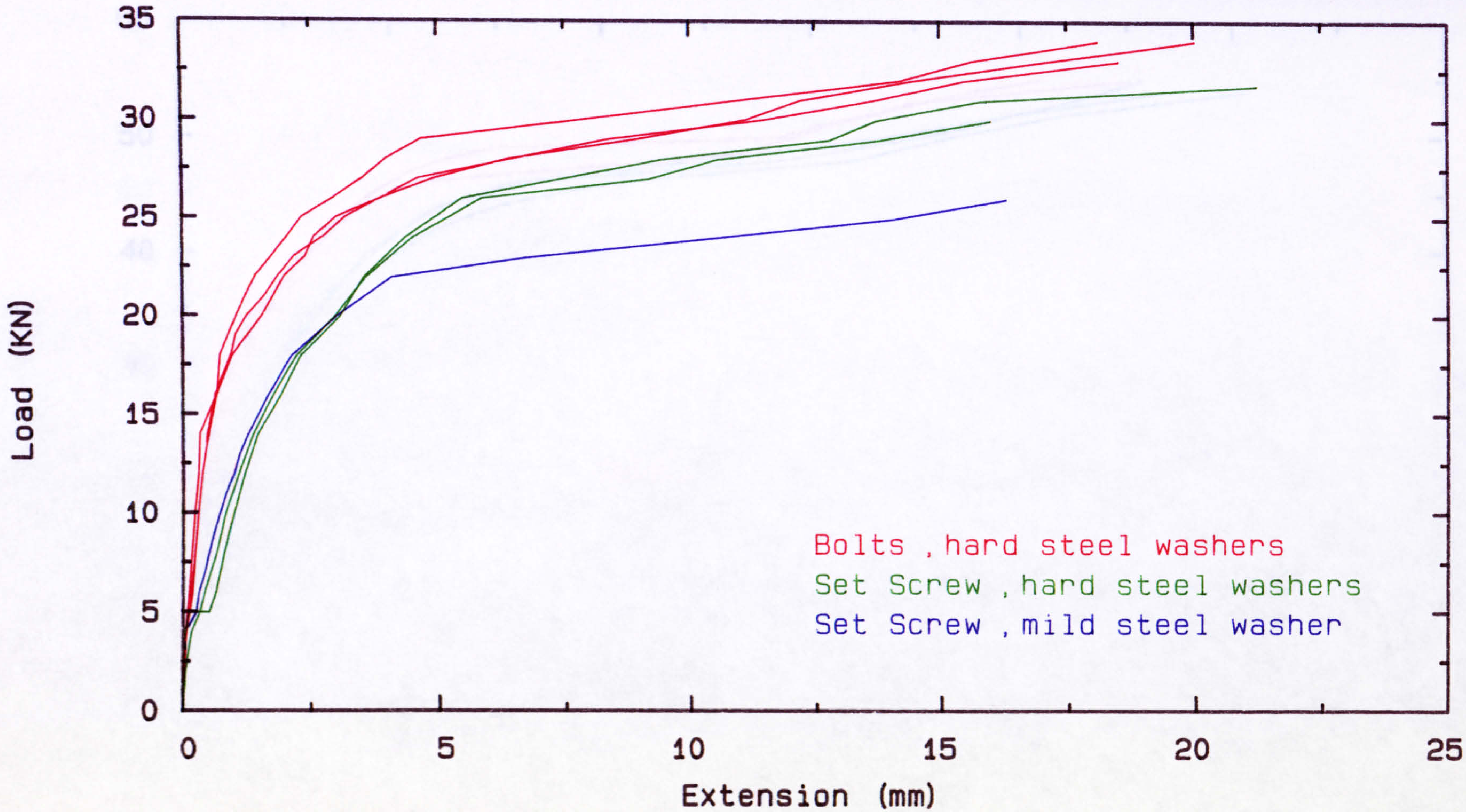


FIG. A2.9 Effect of the occurrence of bolt threads in the shear plane : t = 2.57 mm (No lips)

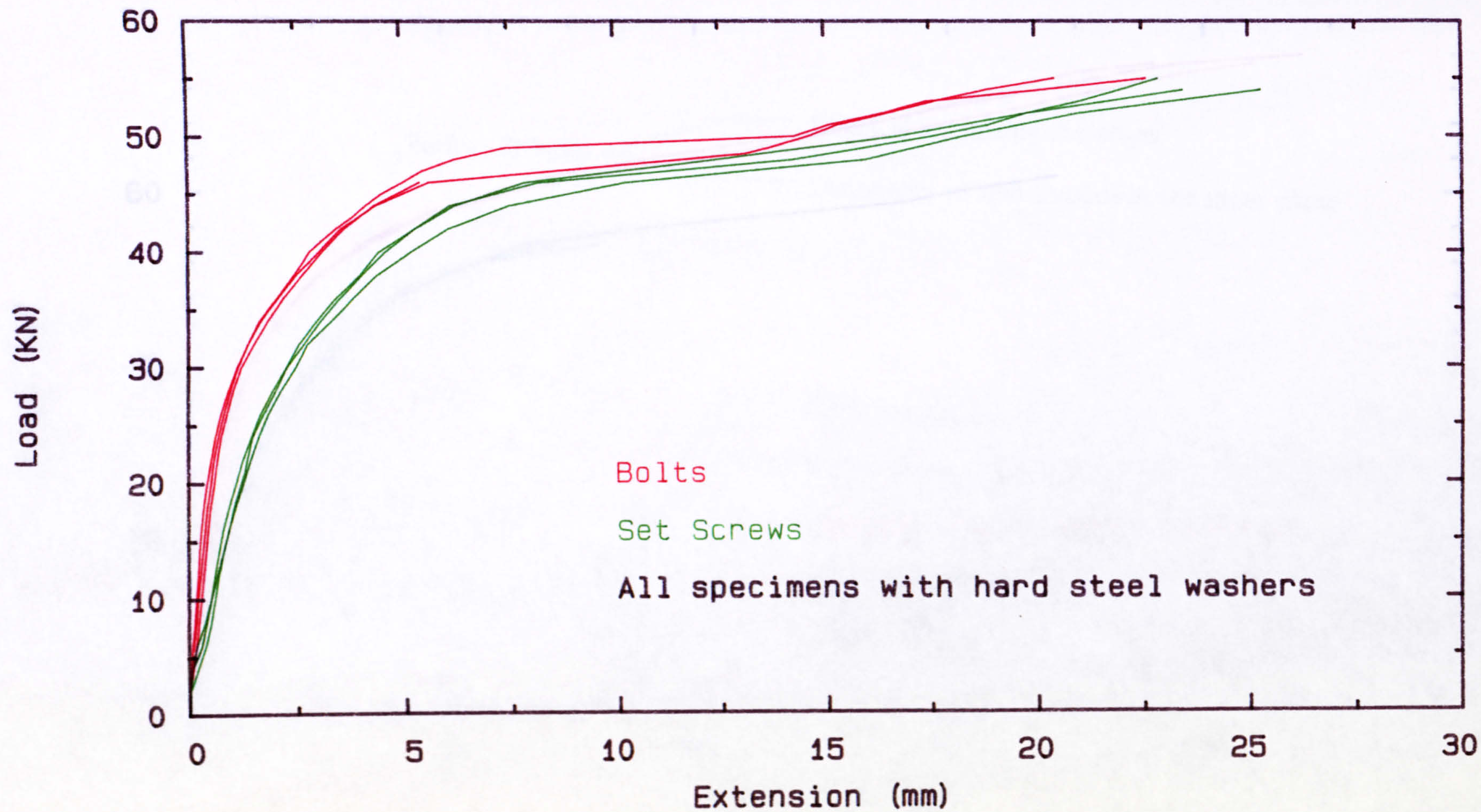
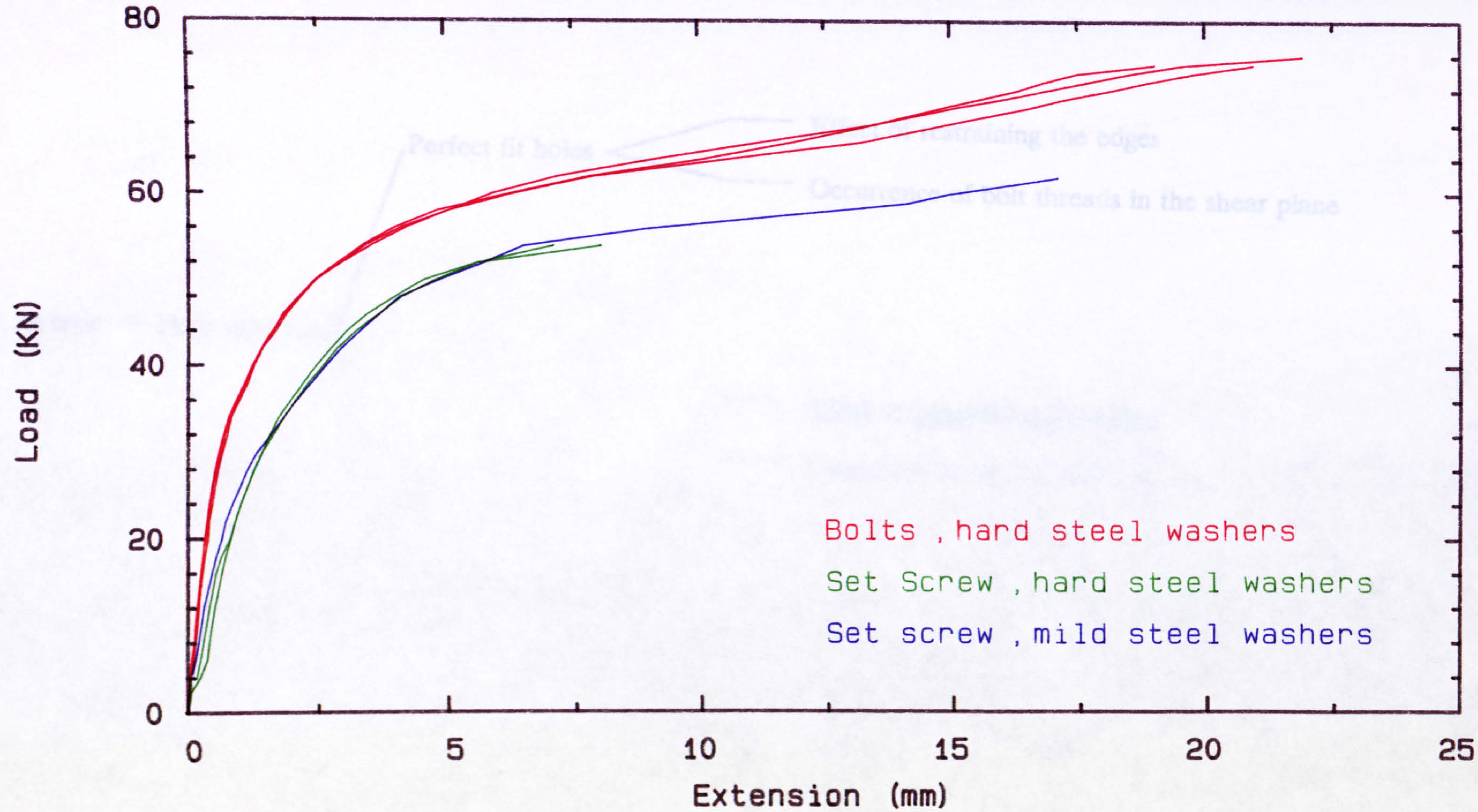


FIG. A2.10 Effect of the occurrence of bolt threads in the shear plane : t = 3.17 mm (No lips)



- Lap tests
  - Hole tolerance
    - Perfect fit holes
      - Effect of restraining the edges
      - Occurrence of bolt threads in the shear plane
    - <u>Clearance holes</u>
      - <u>Effect of restraining the edges</u>
      - Washer material and diameter
      - Number and position of washers
      - Mechanical properties of steel sheets
      - End distance of bolt in line of stress
      - Bolt diameter
      - Thin/thick sheets
      - Range of sheet thickness covered by cold formed steel codes

Fig. A2.11 Effect of edge restraints , clearance holes :
t = 2.48mm

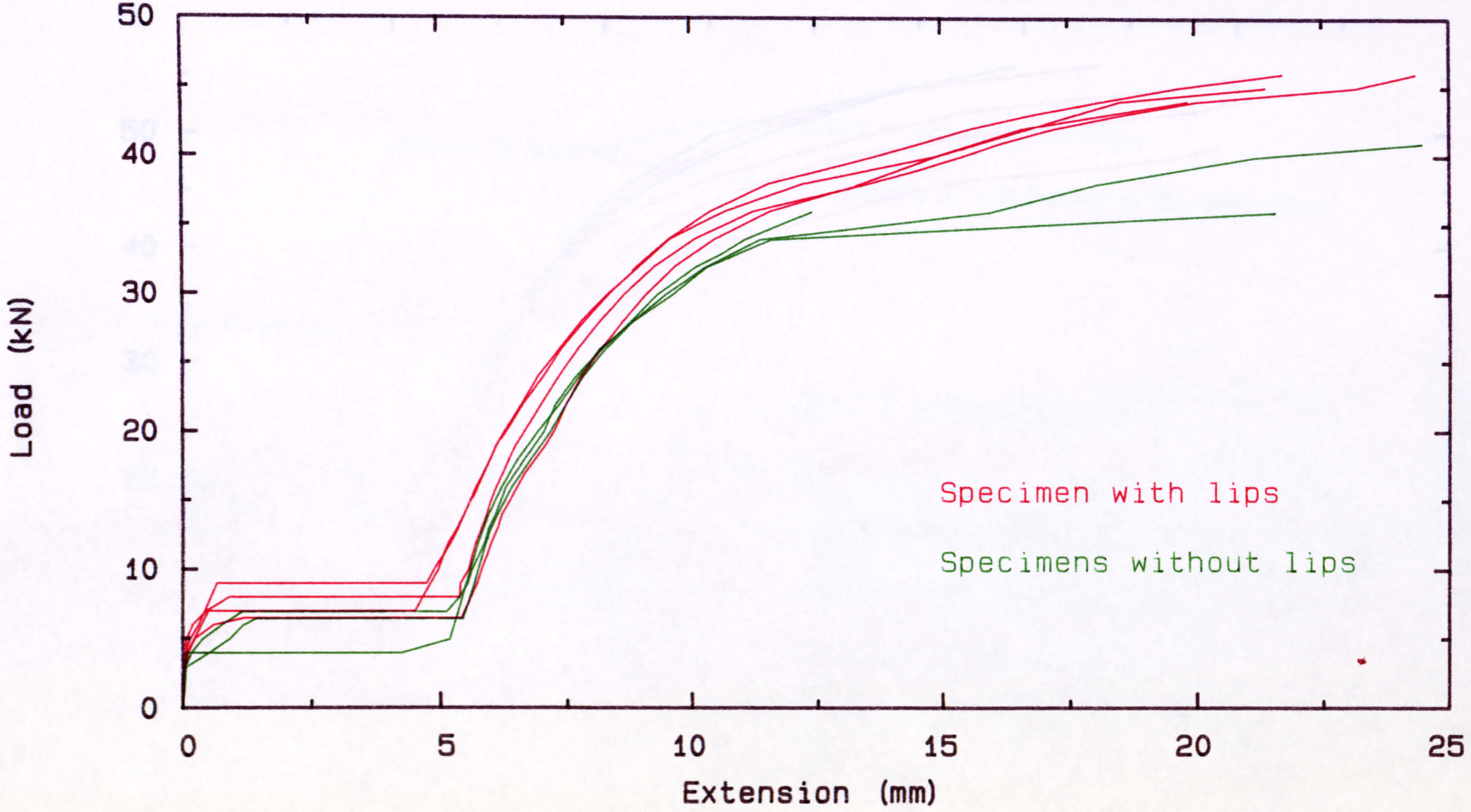
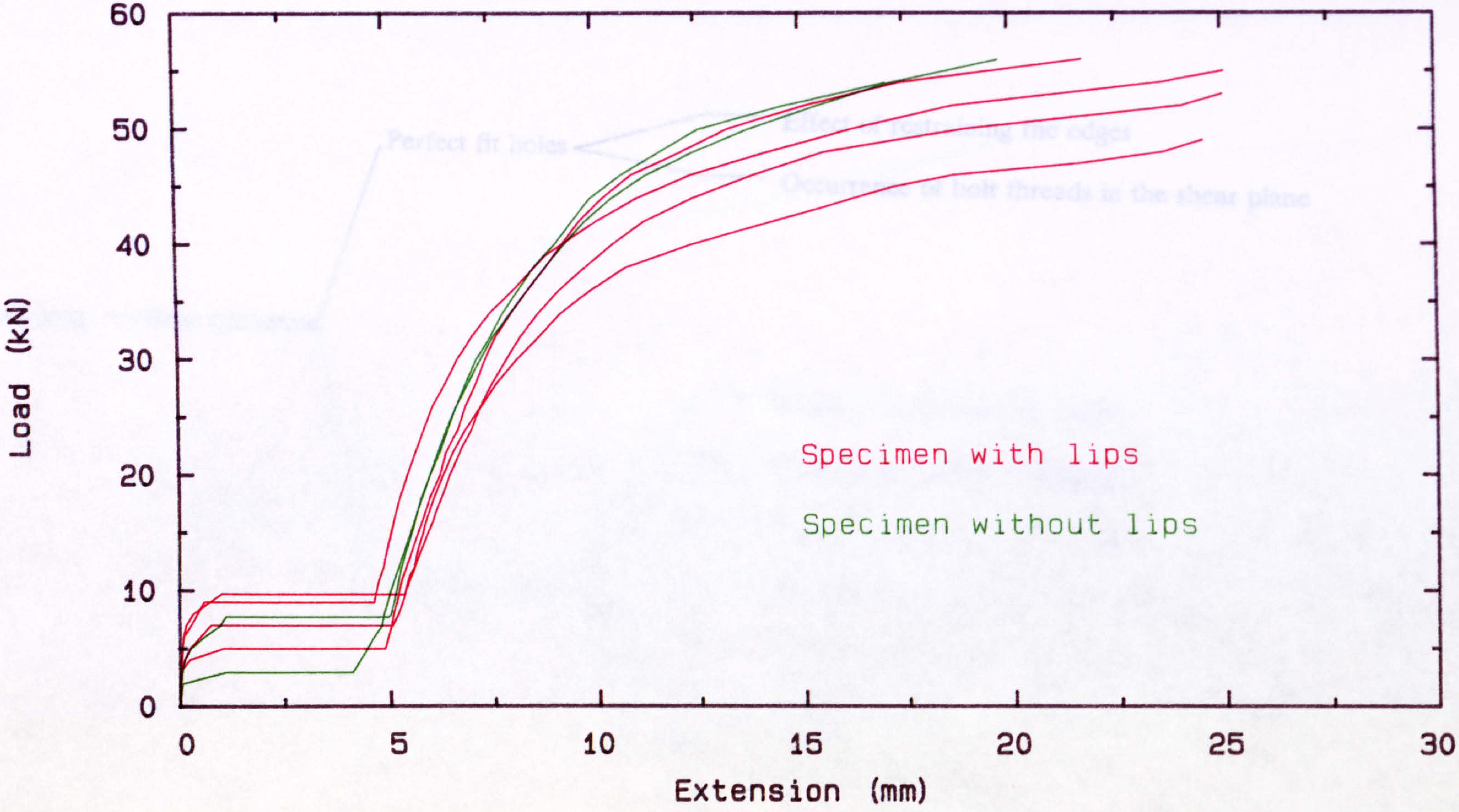


Fig. A2.12 Effect of edge restraints , clearance holes :
t = 3.02 mm



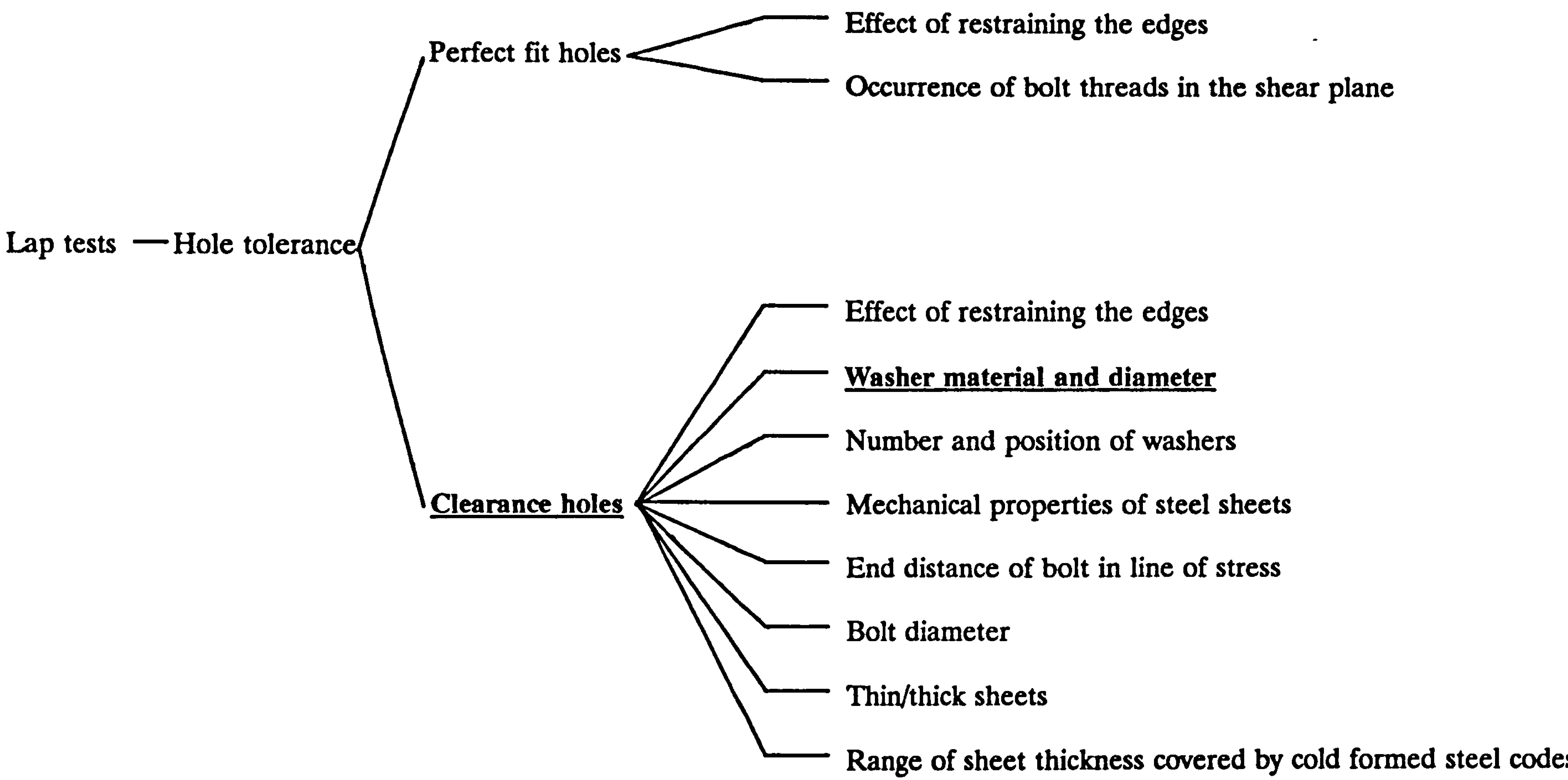

Lap tests
Hole tolerance
Perfect fit holes
Effect of restraining the edges
Occurrence of bolt threads in the shear plane
Clearance holes
Effect of restraining the edges
Washer material and diameter
Number and position of washers
Mechanical properties of steel sheets
End distance of bolt in line of stress
Bolt diameter
Thin/thick sheets
Range of sheet thickness covered by cold formed steel codes

Fig. A2.13A Washer material and diameter : t = 1.63 mm.

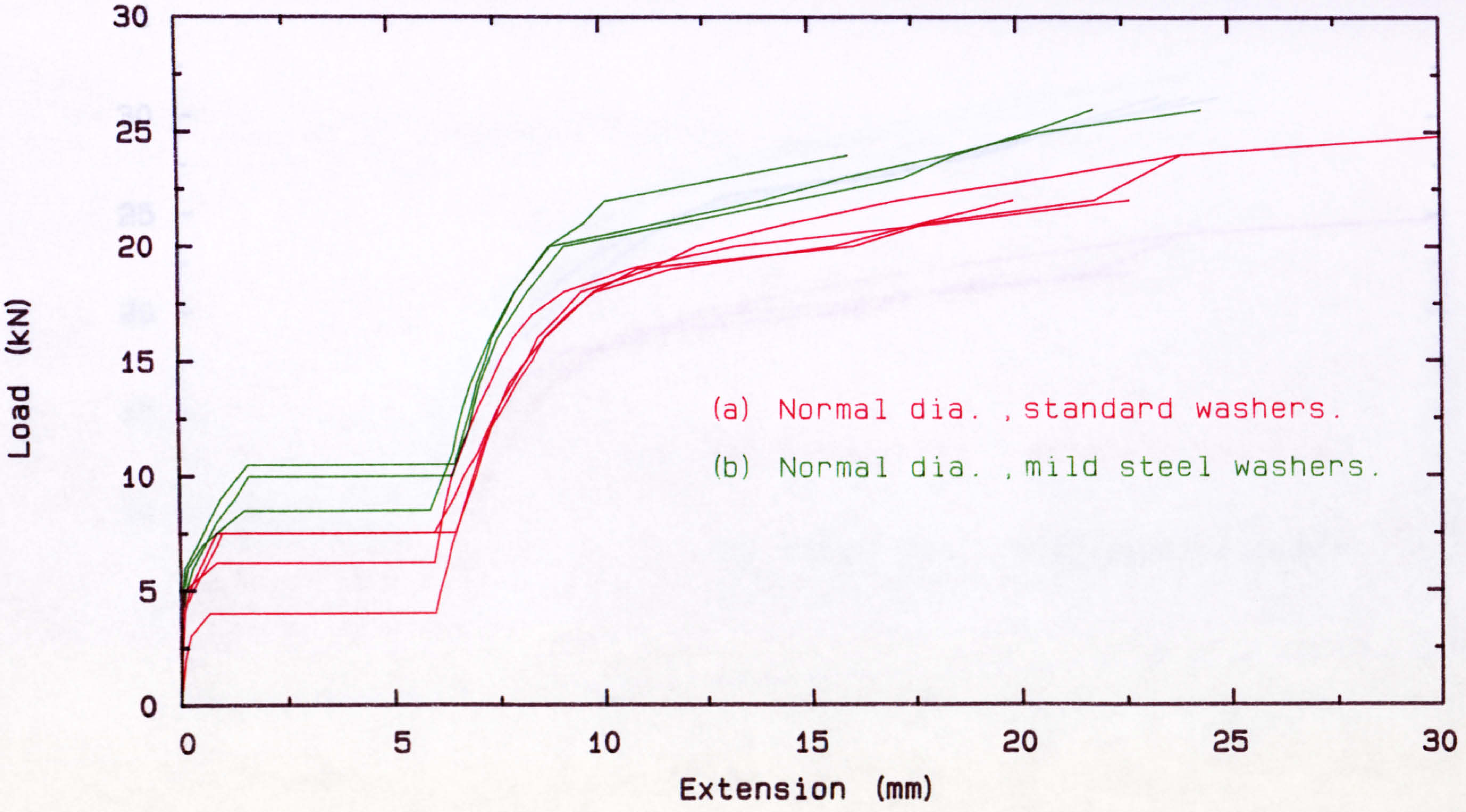


Fig. A2.13B Washer material and diameter : t = 1.63 mm.

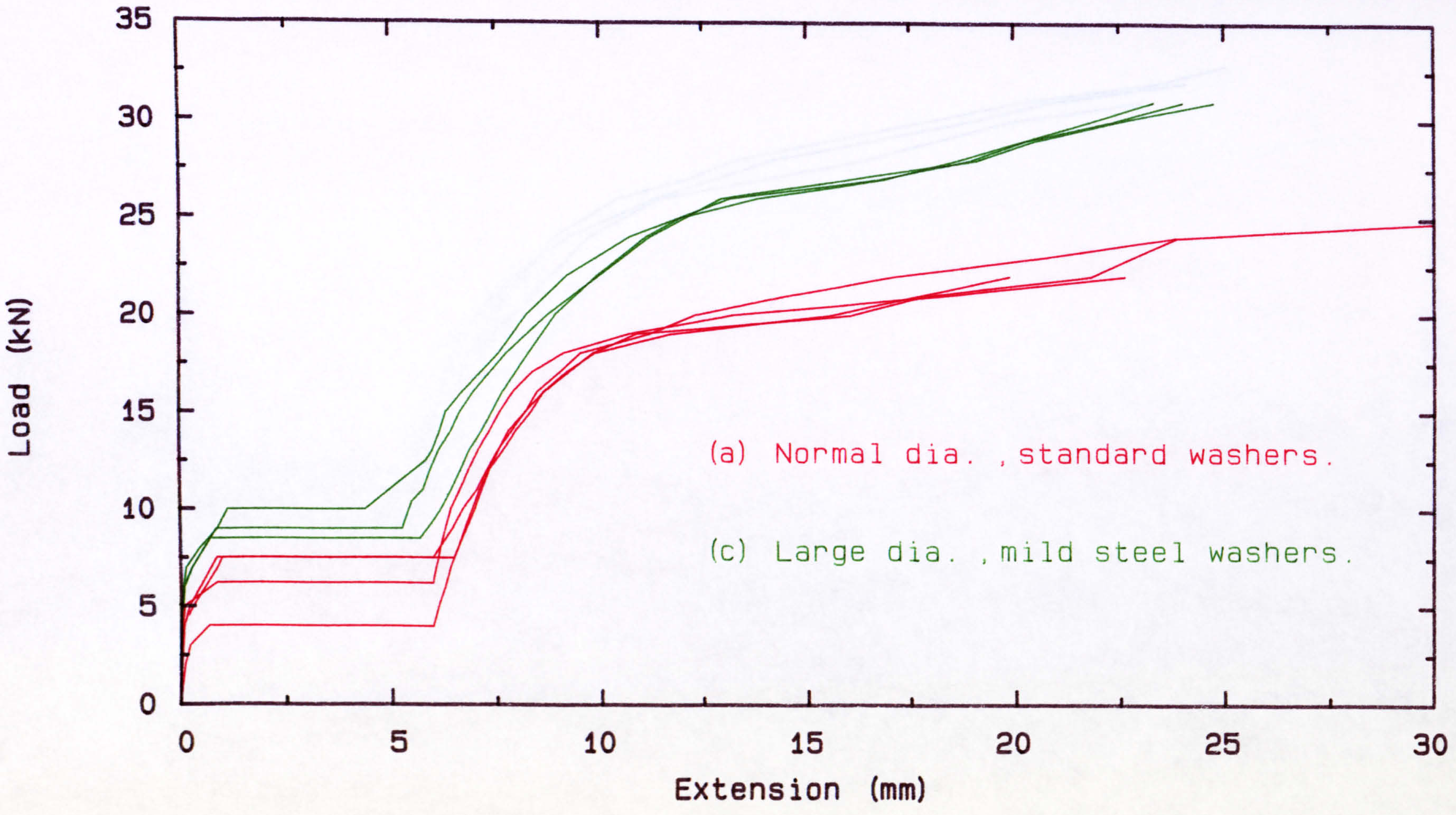


Fig. A2.13C Washer material and diameter : t = 1.63 mm.

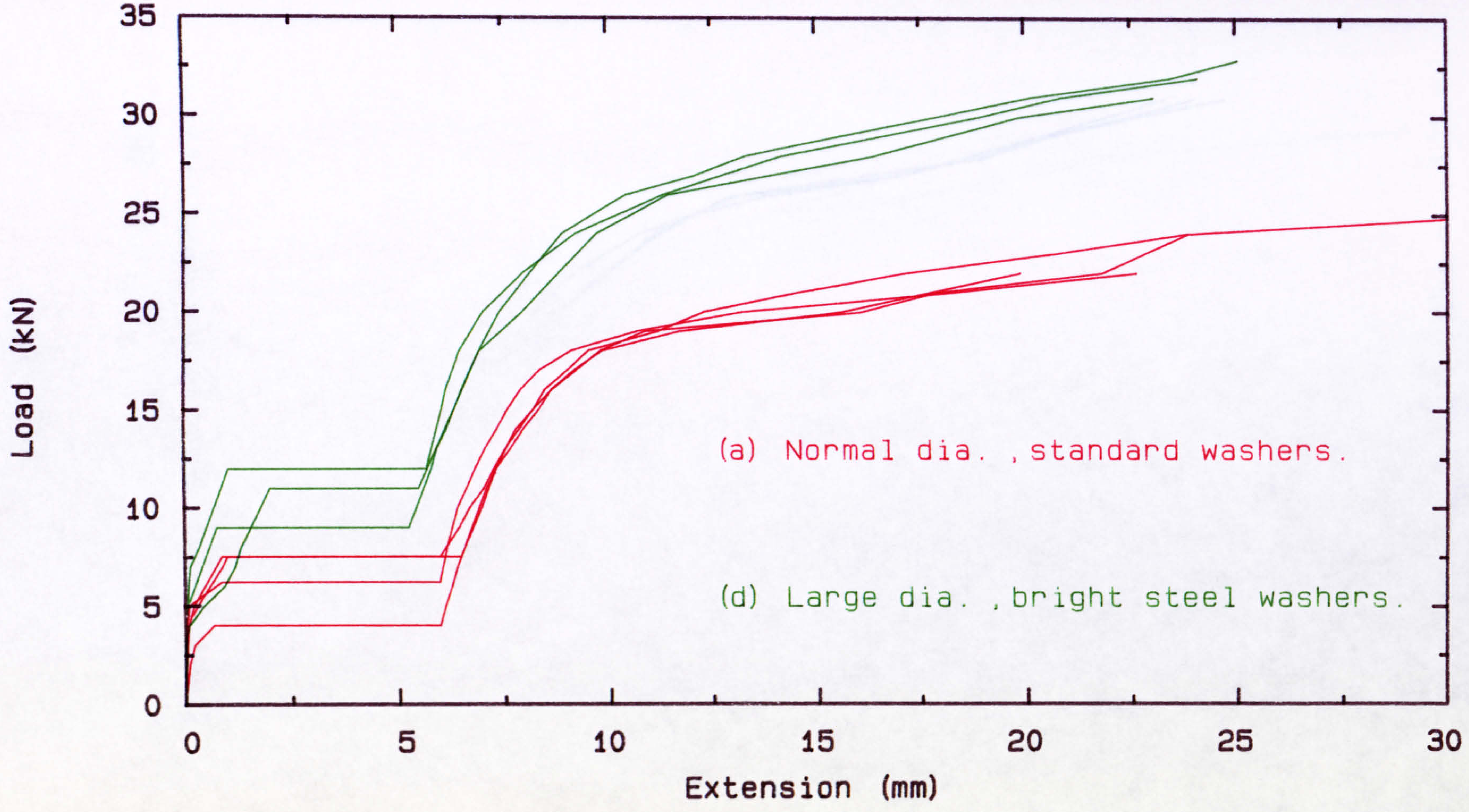


Fig. A2.13D Washer material and diameter : t = 1.63 mm.

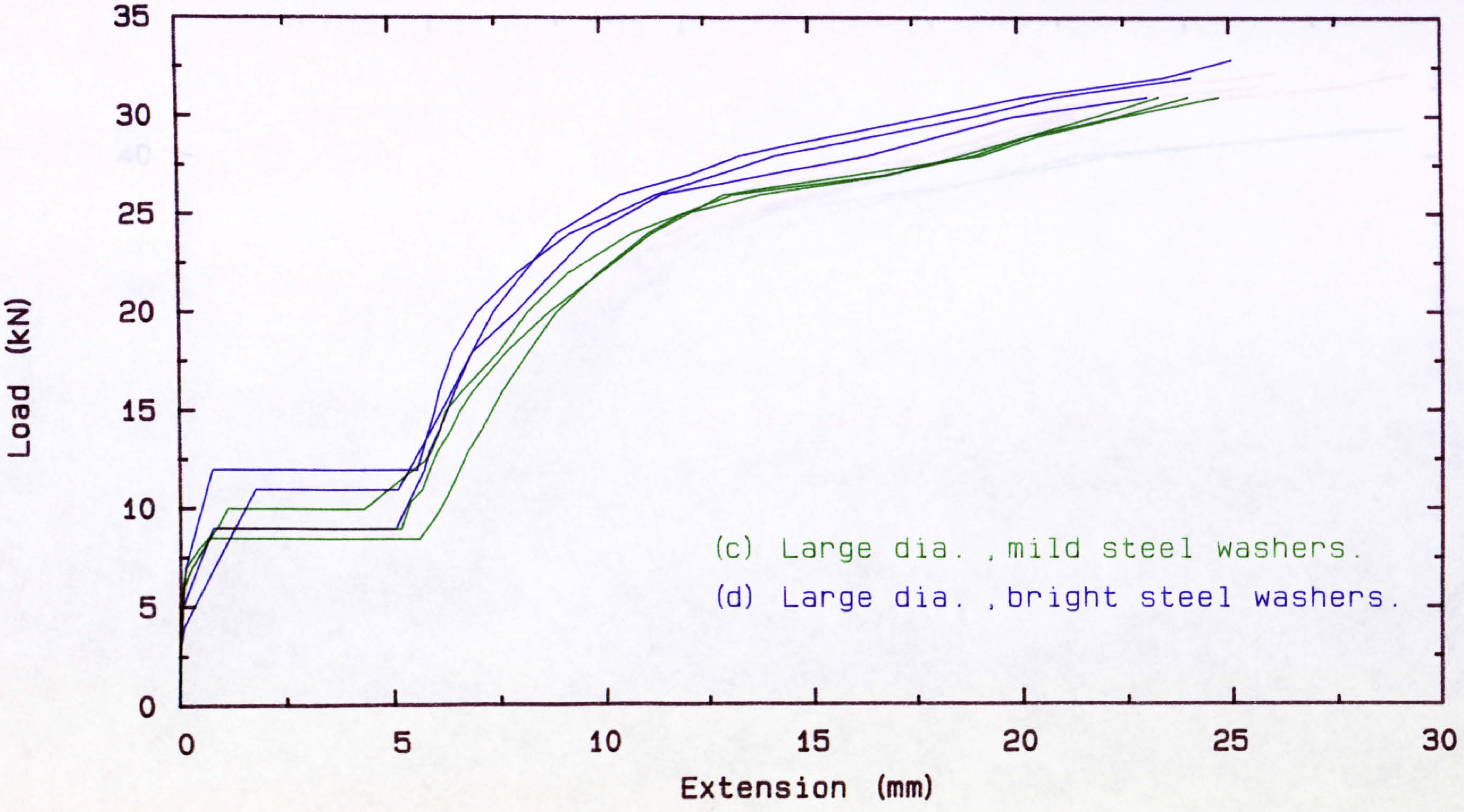


Fig. A2.14A Washer material and diameter : t = 2.48 mm.

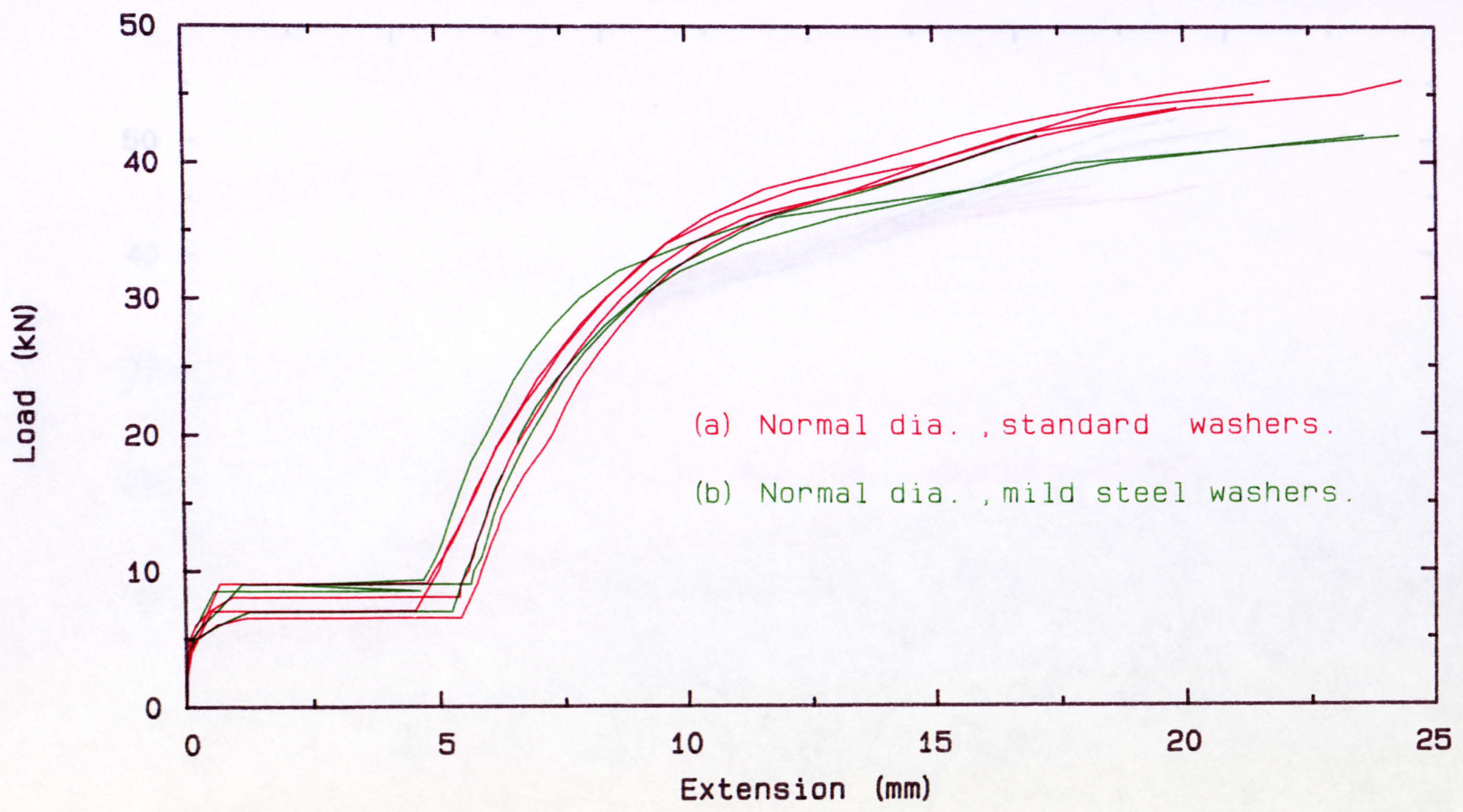


Fig. A2.14B Washer material and diameter : t = 2.48 mm.

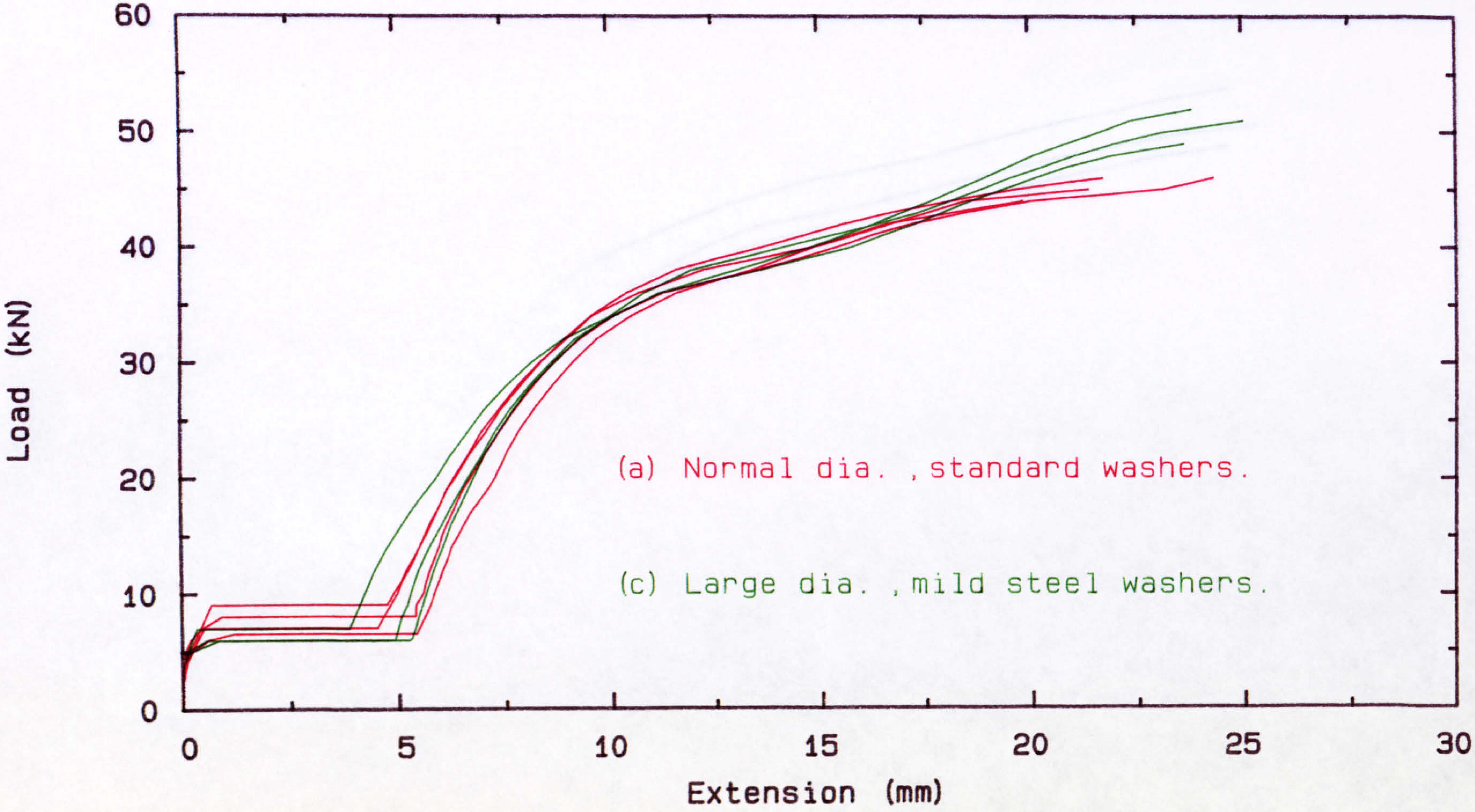


Fig. A2.14C Washer material and diameter : t = 2.48 mm.

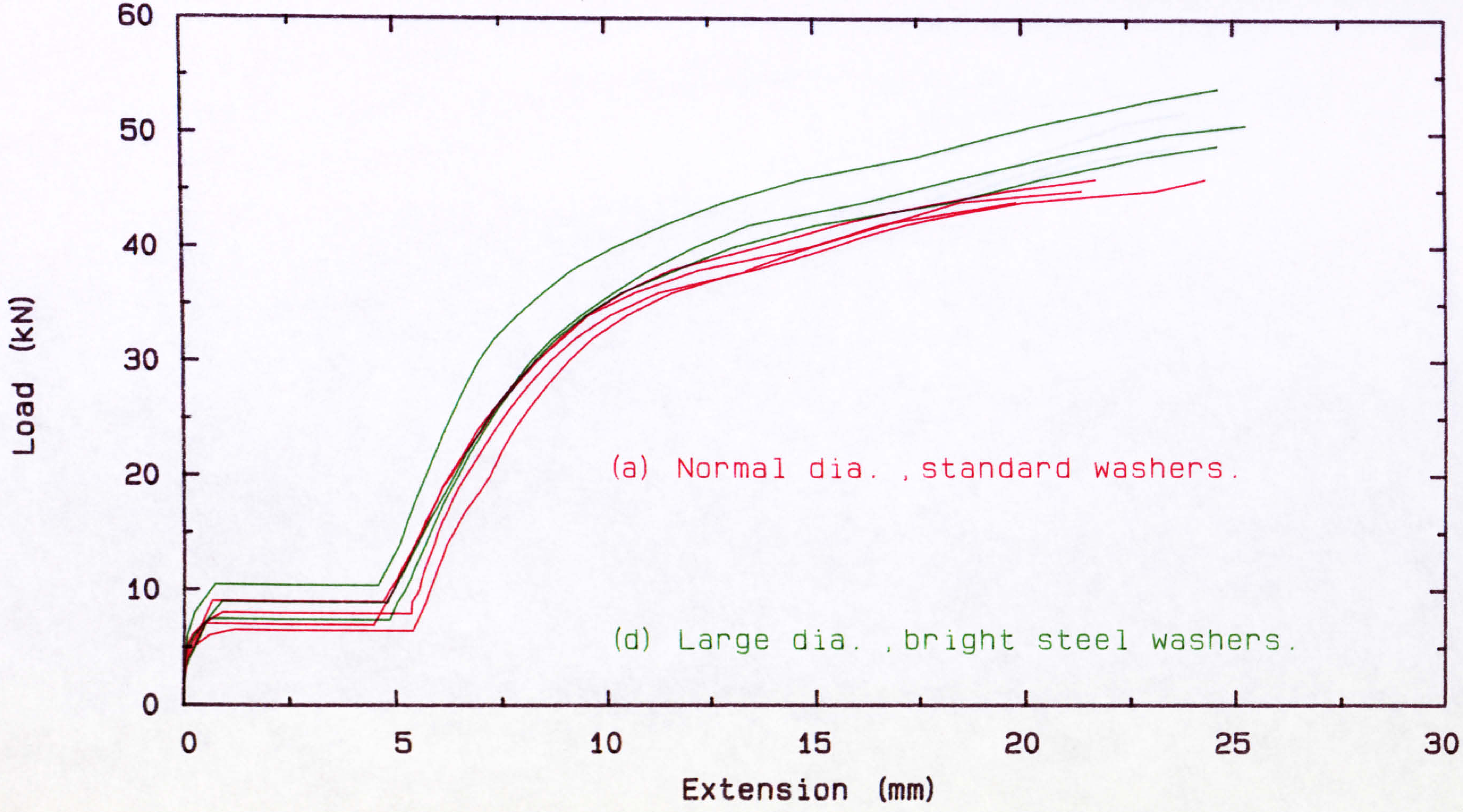


Fig. A2.14D Washer material and diameter : t = 2.48 mm.

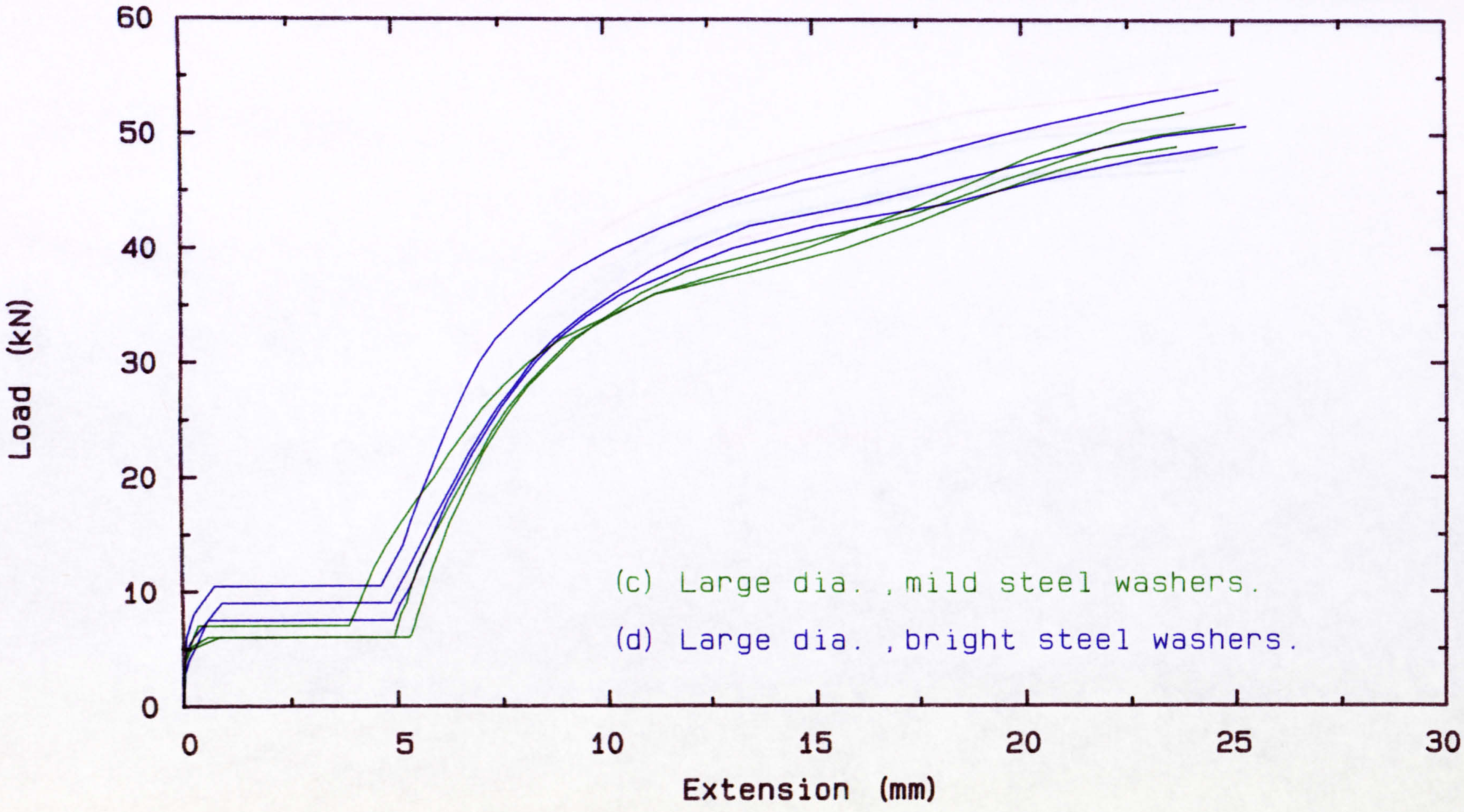


Fig. A2.15A Washer material and diameter : t = 3.02 mm.

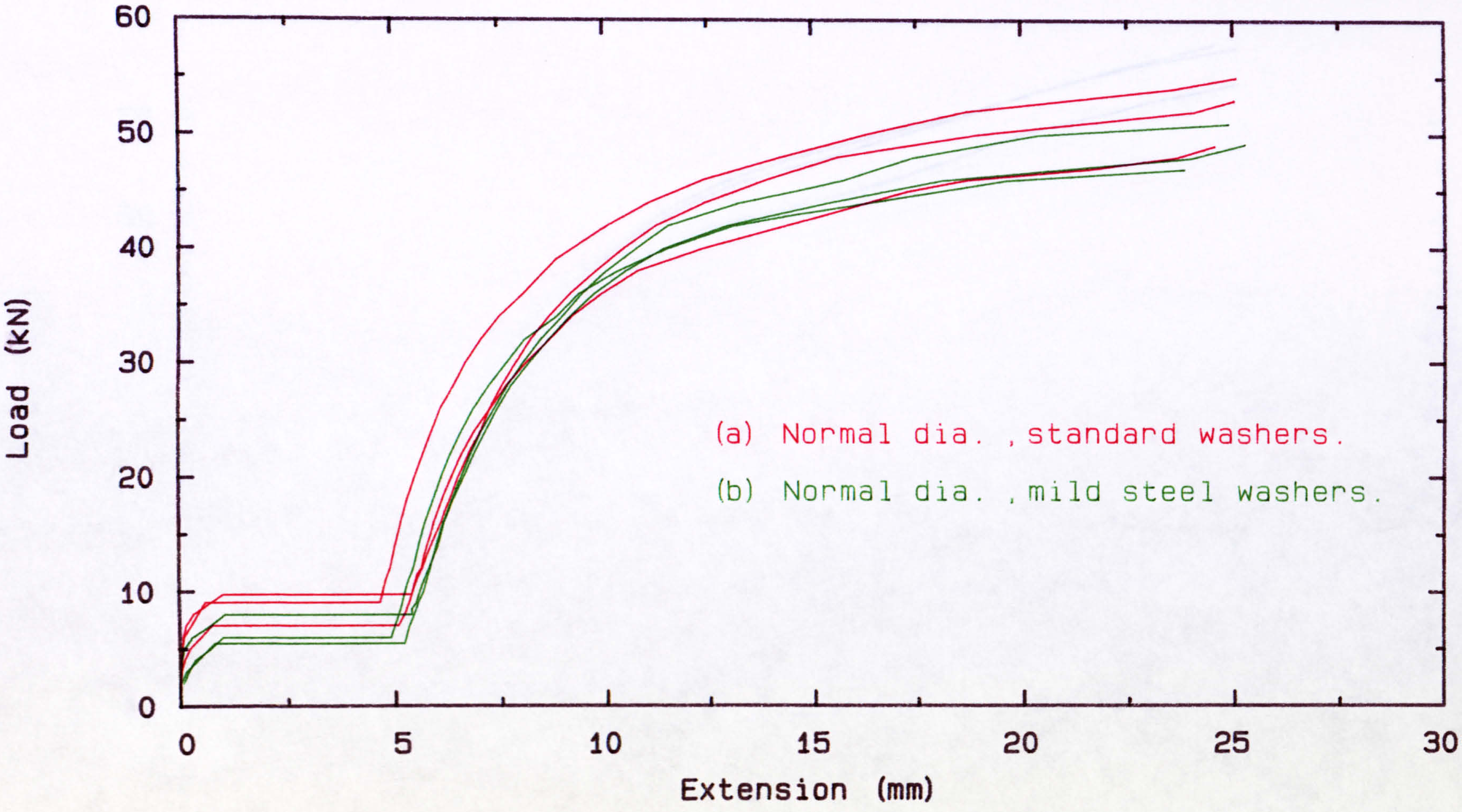


Fig. A2.15B Washer material and diameter : $t = 3.02$ mm.

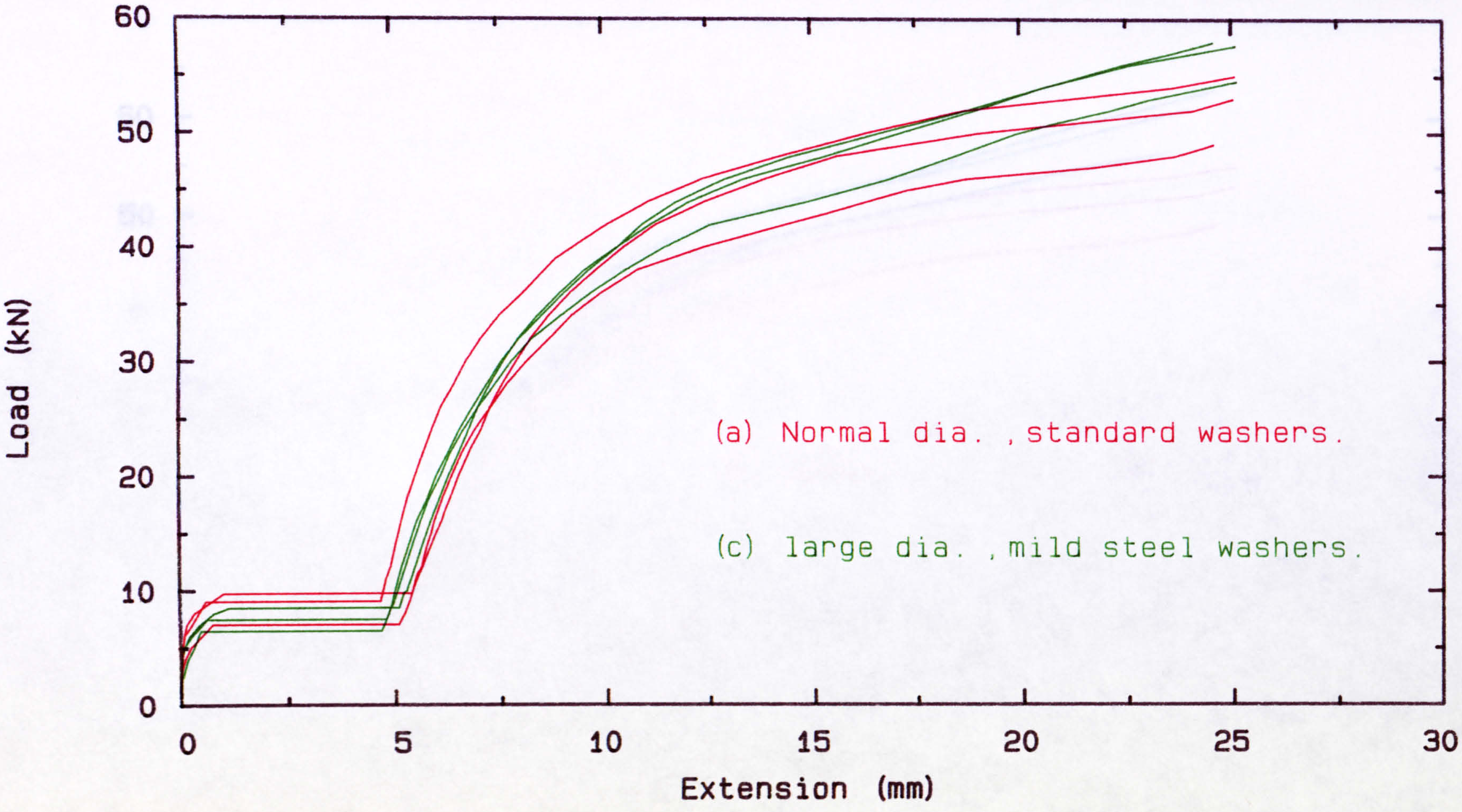


Fig. A2.15C Washer material and diameter : t = 3.02 mm.

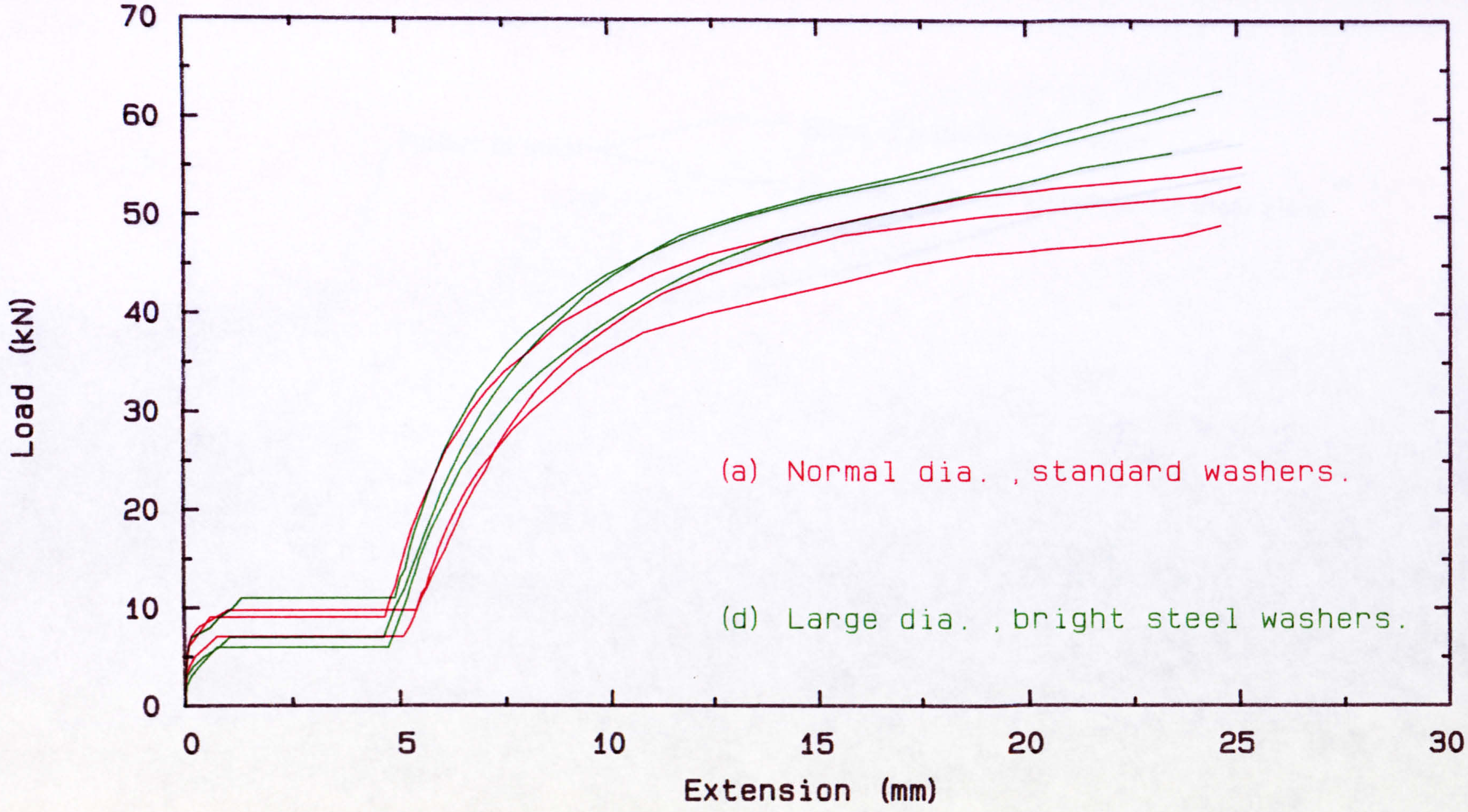
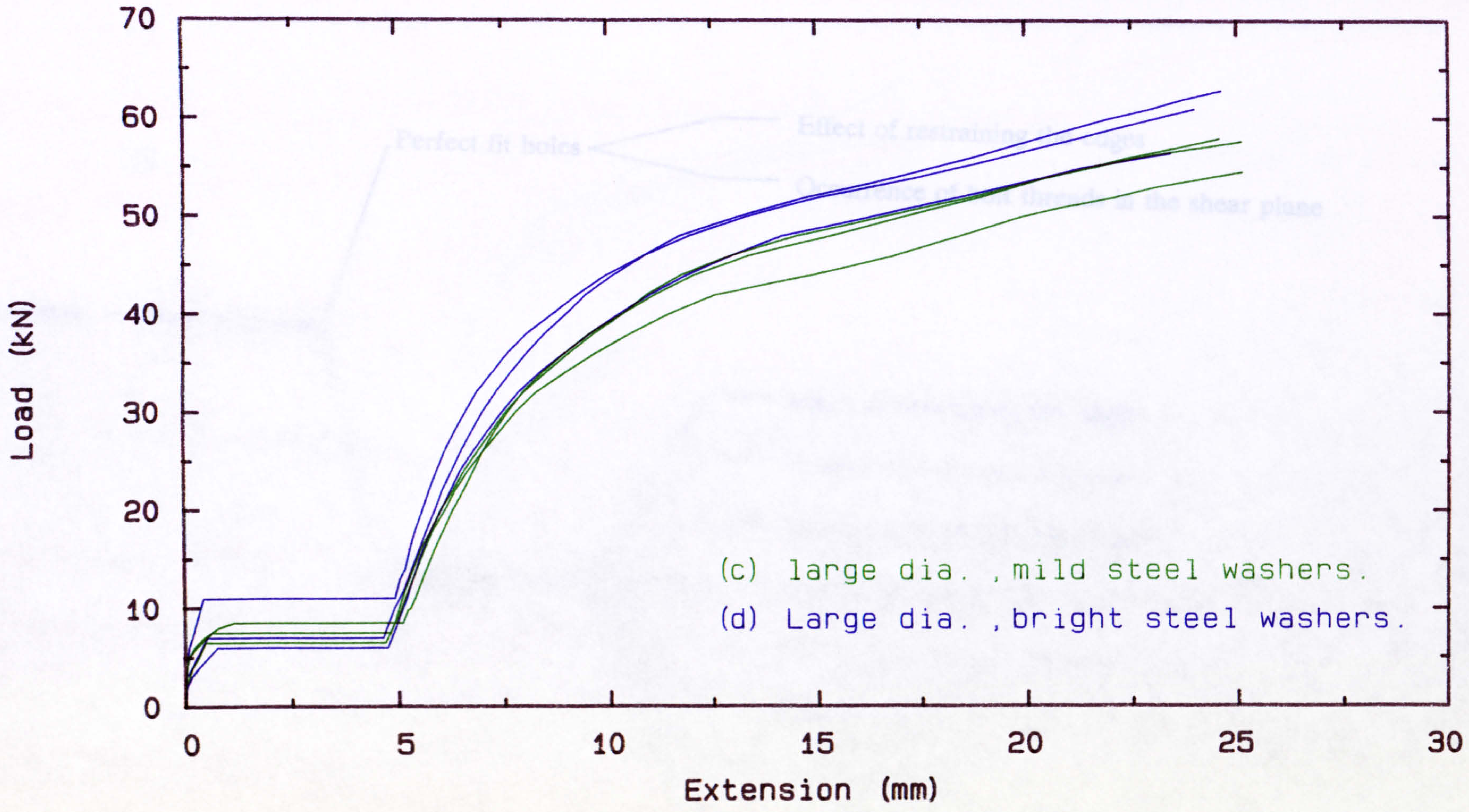


Fig. A2.15D Washer material and diameter : t = 3.02 mm.



- Lap tests
  - Hole tolerance
    - Perfect fit holes
      - Effect of restraining the edges
      - Occurrence of bolt threads in the shear plane
    - <u>Clearance holes</u>
      - Effect of restraining the edges
      - Washer material and diameter
      - <u>Number and position of washers</u>
      - Mechanical properties of steel sheets
      - End distance of bolt in line of stress
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      - Range of sheet thickness covered by cold formed steel codes

Fig. A2.16A Number and position of washers : One washer under bolt head, t = 1.63 mm.

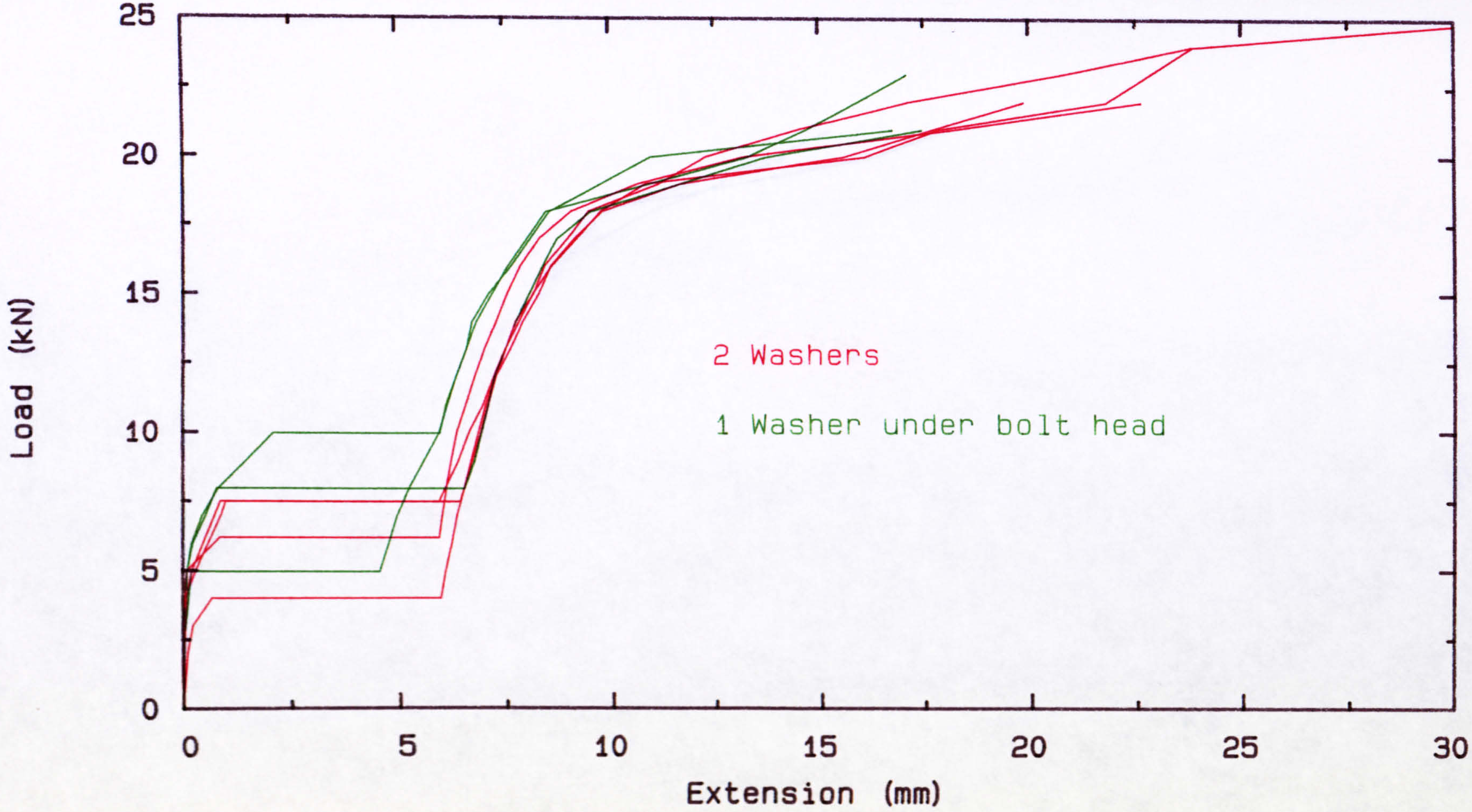


Fig. A2.16B Number and position of washers : One washer under nut , t = 1.63 mm.

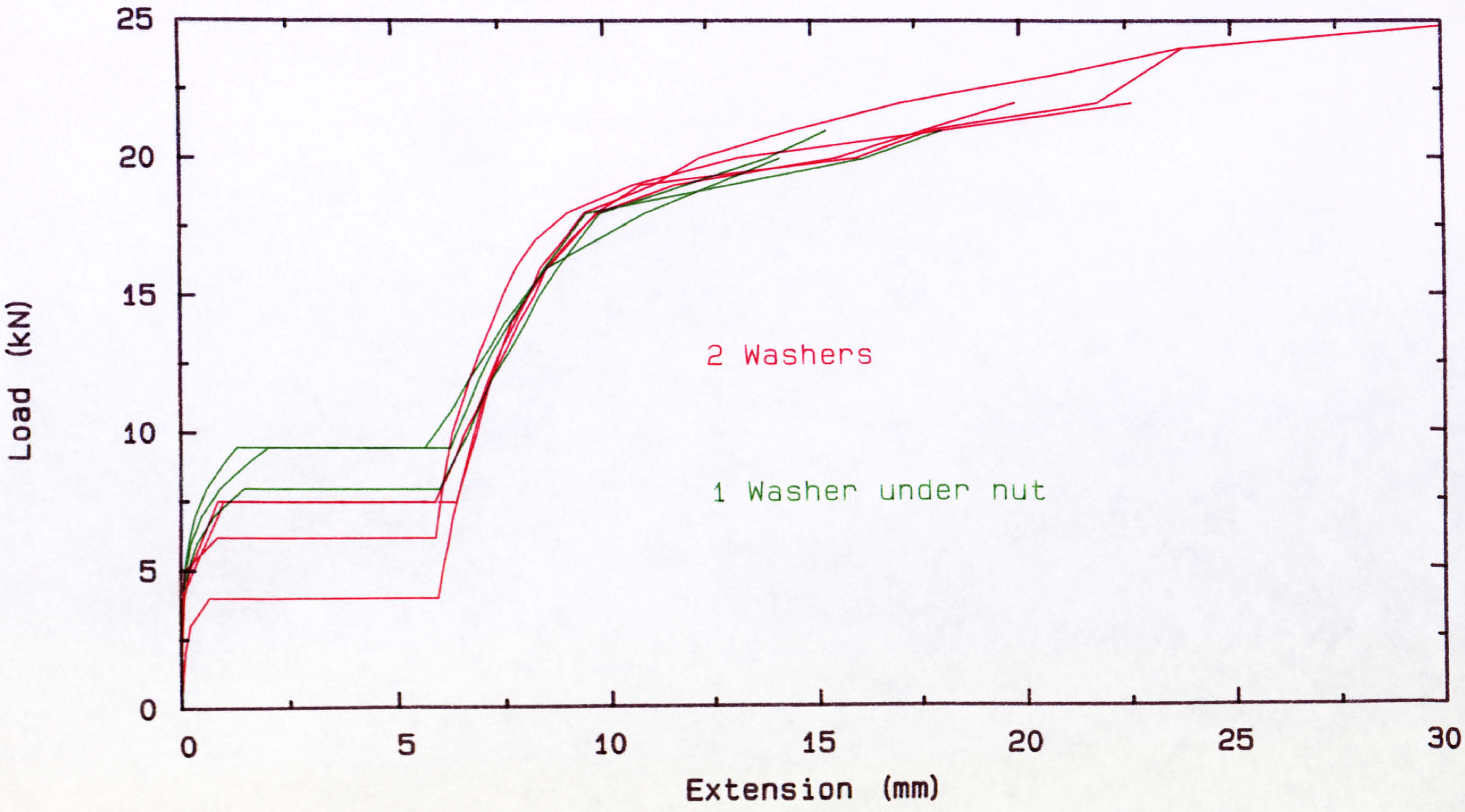


Fig. A2.16C Number and position of washers: No washers, t = 1.63 mm.

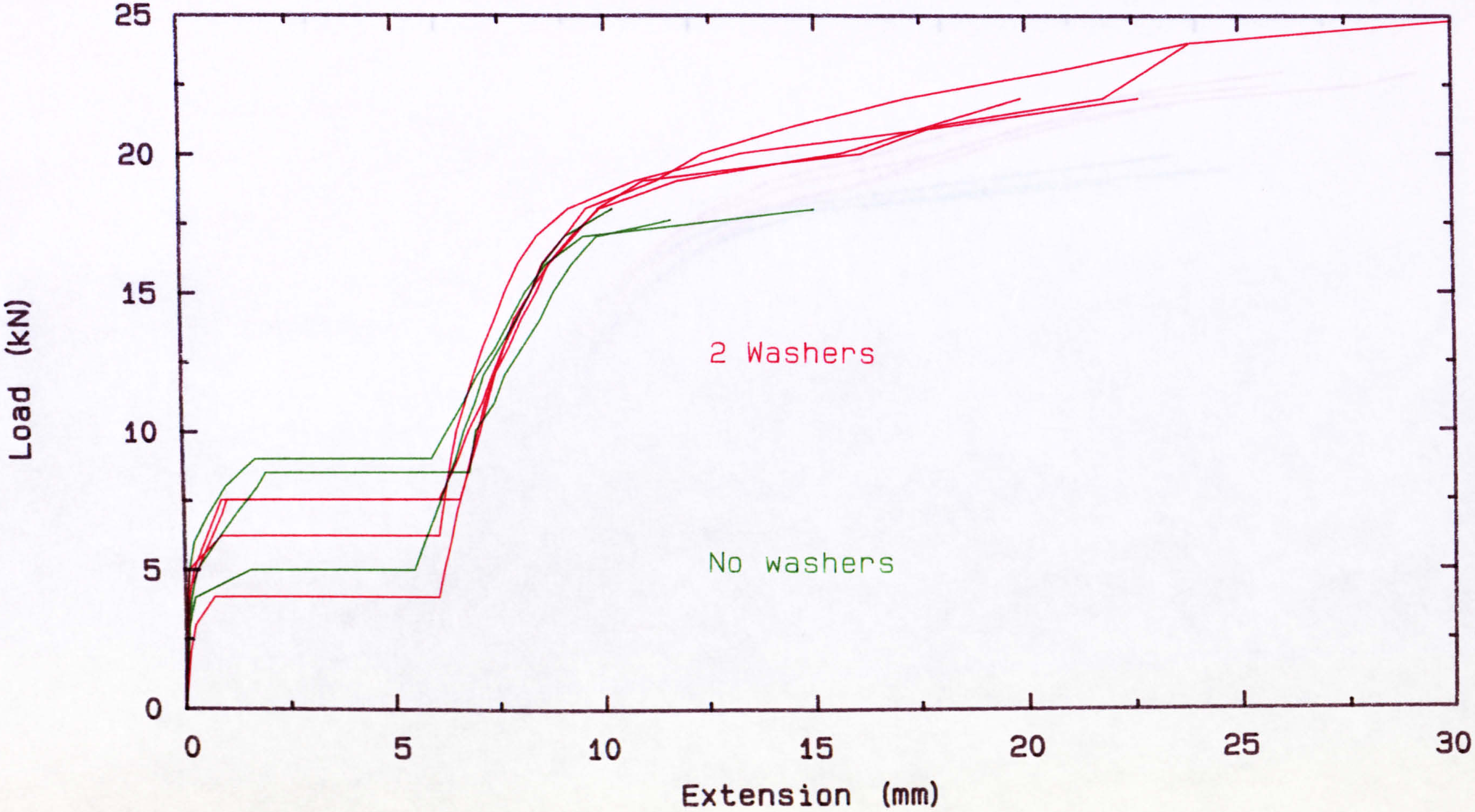


Fig. A2.17A Number and position of washers : One washer under bolt head, t = 2.48 mm.

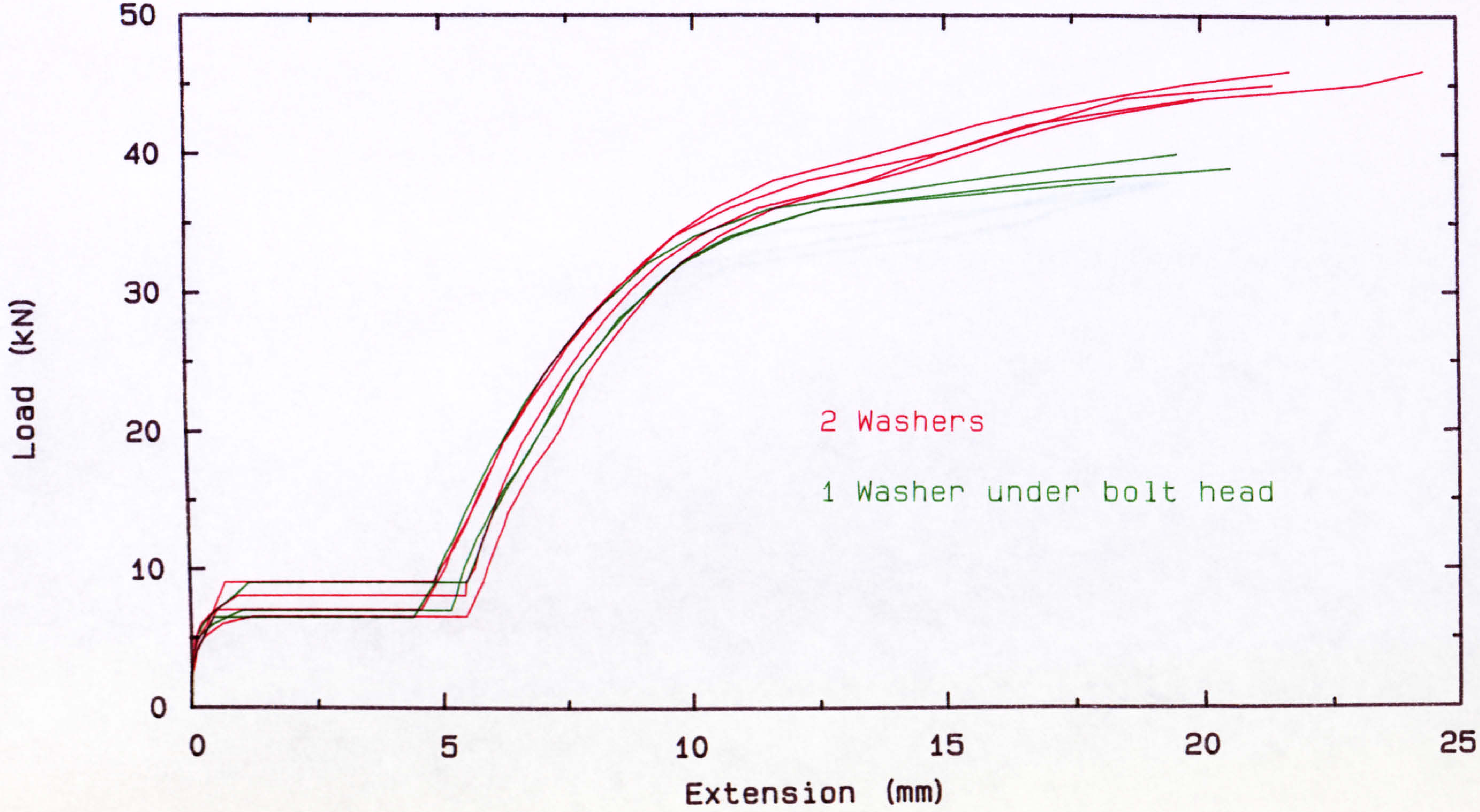


Fig. A2.17B Number and position of washers : One washer under nut, t = 2.48 mm.

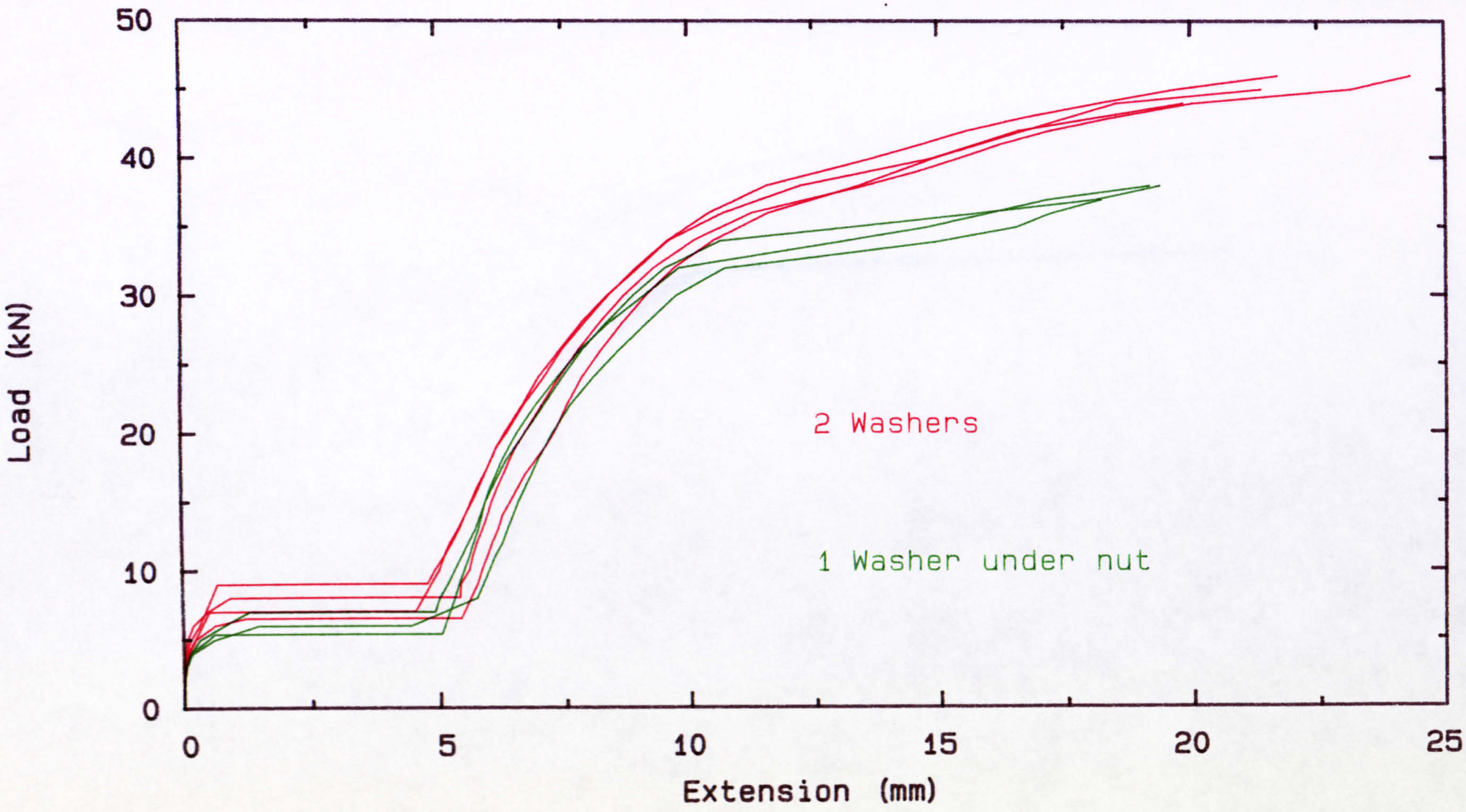


Fig. A2.17C Number and position of washers : No washers,
t = 1.63 mm.

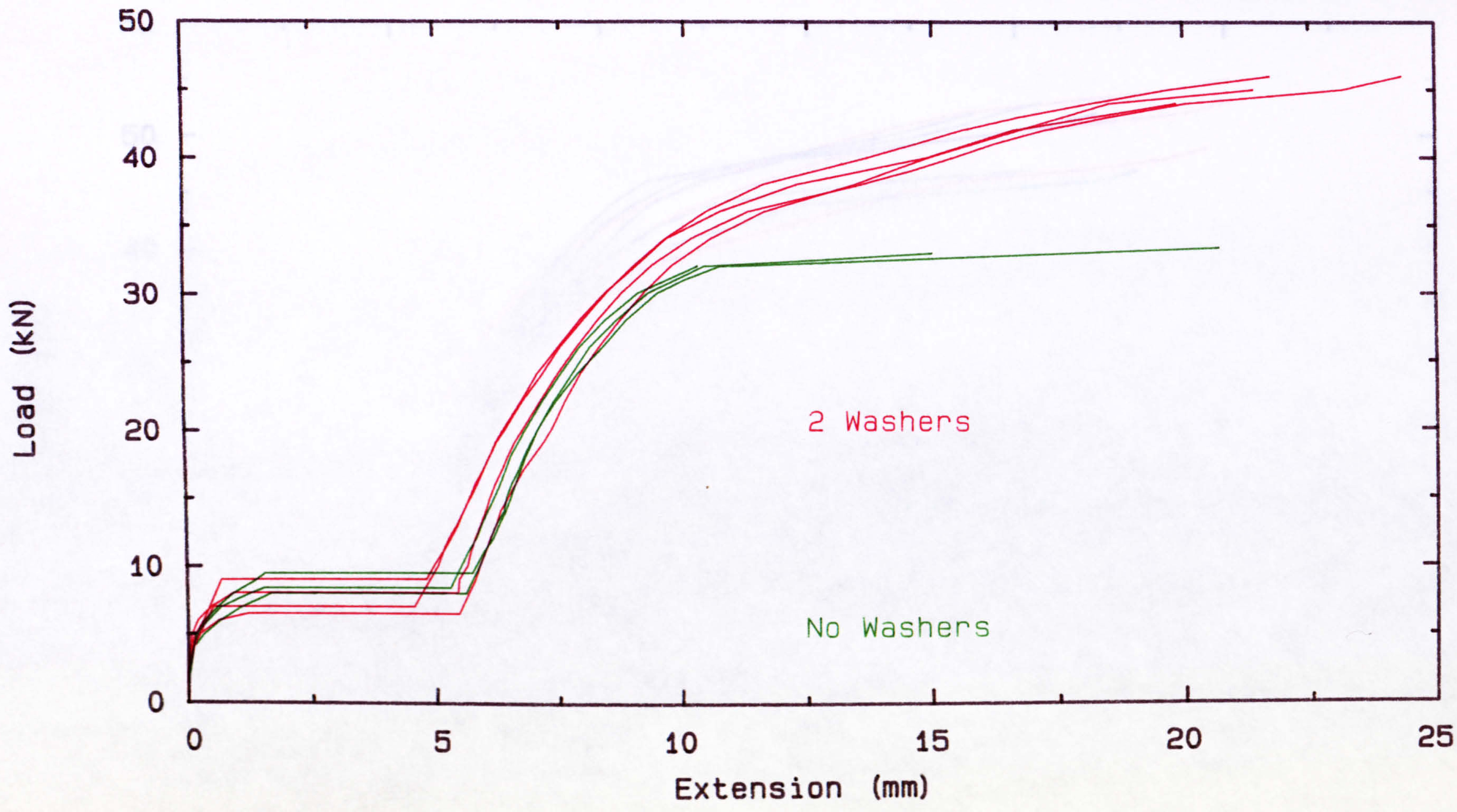


Fig. A2.18A Number and position of washers : One washer under bolt head, t = 3.02 mm.

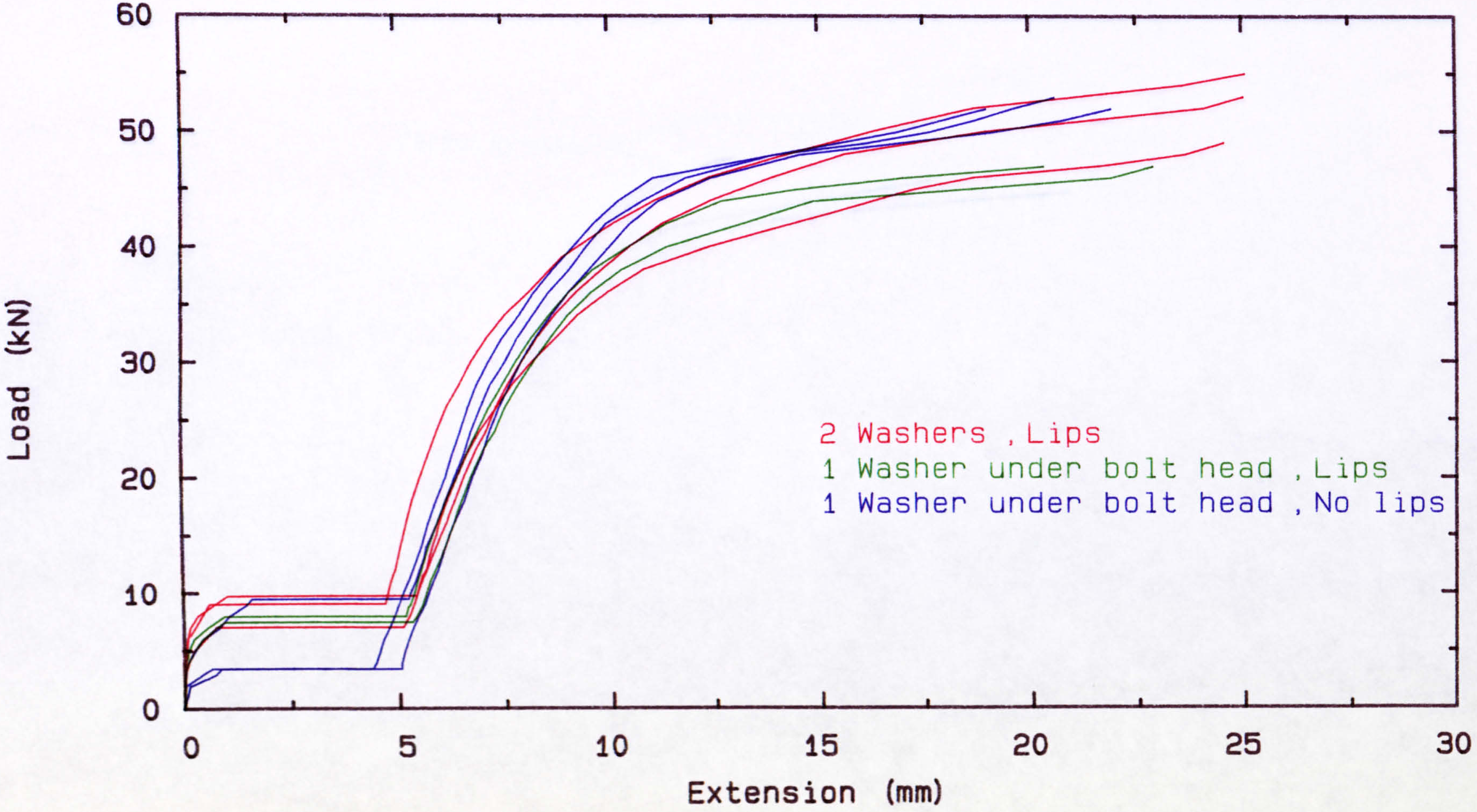


Fig. A2.18B Number and position of washers : One washer under nut, t= 3.02 mm.

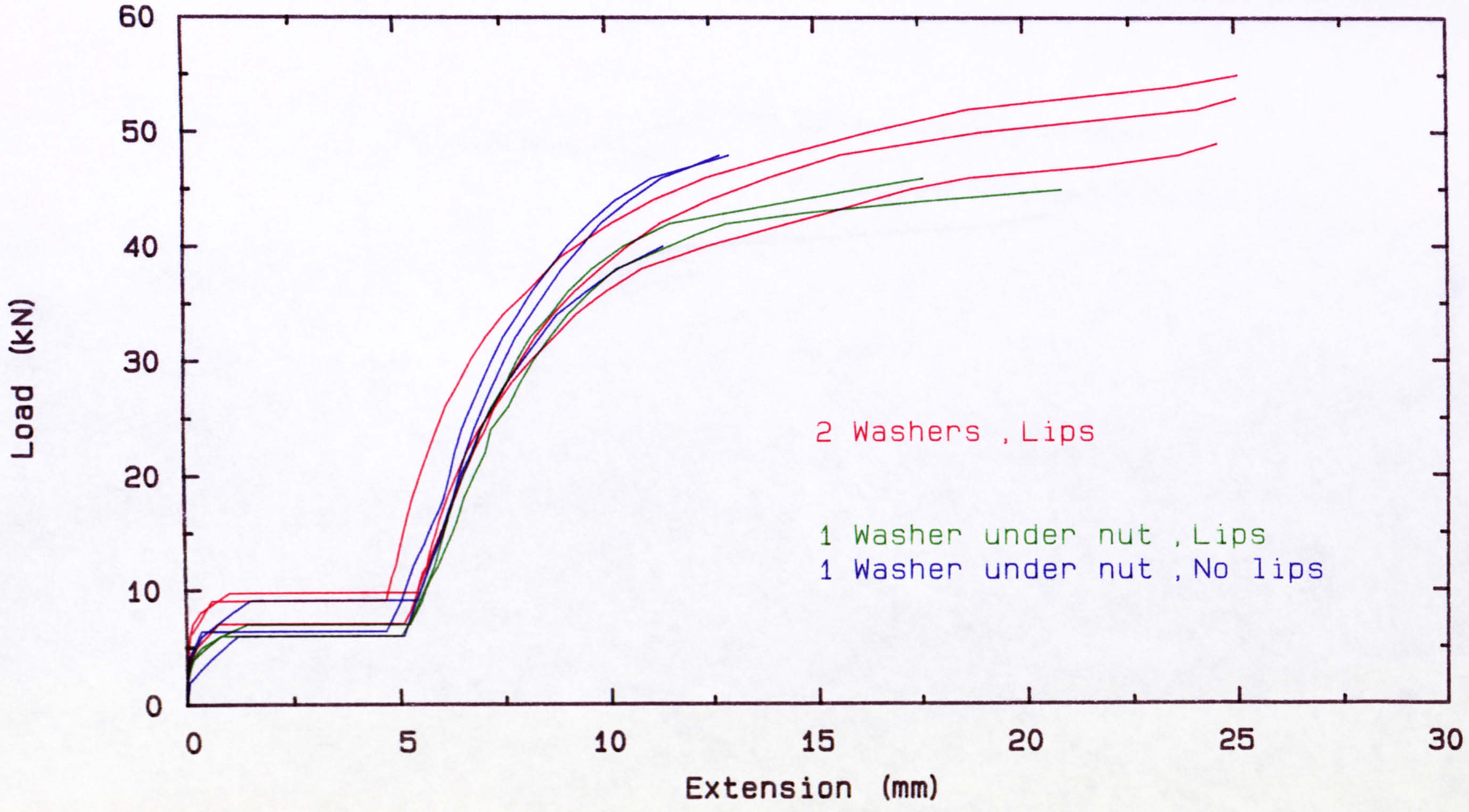
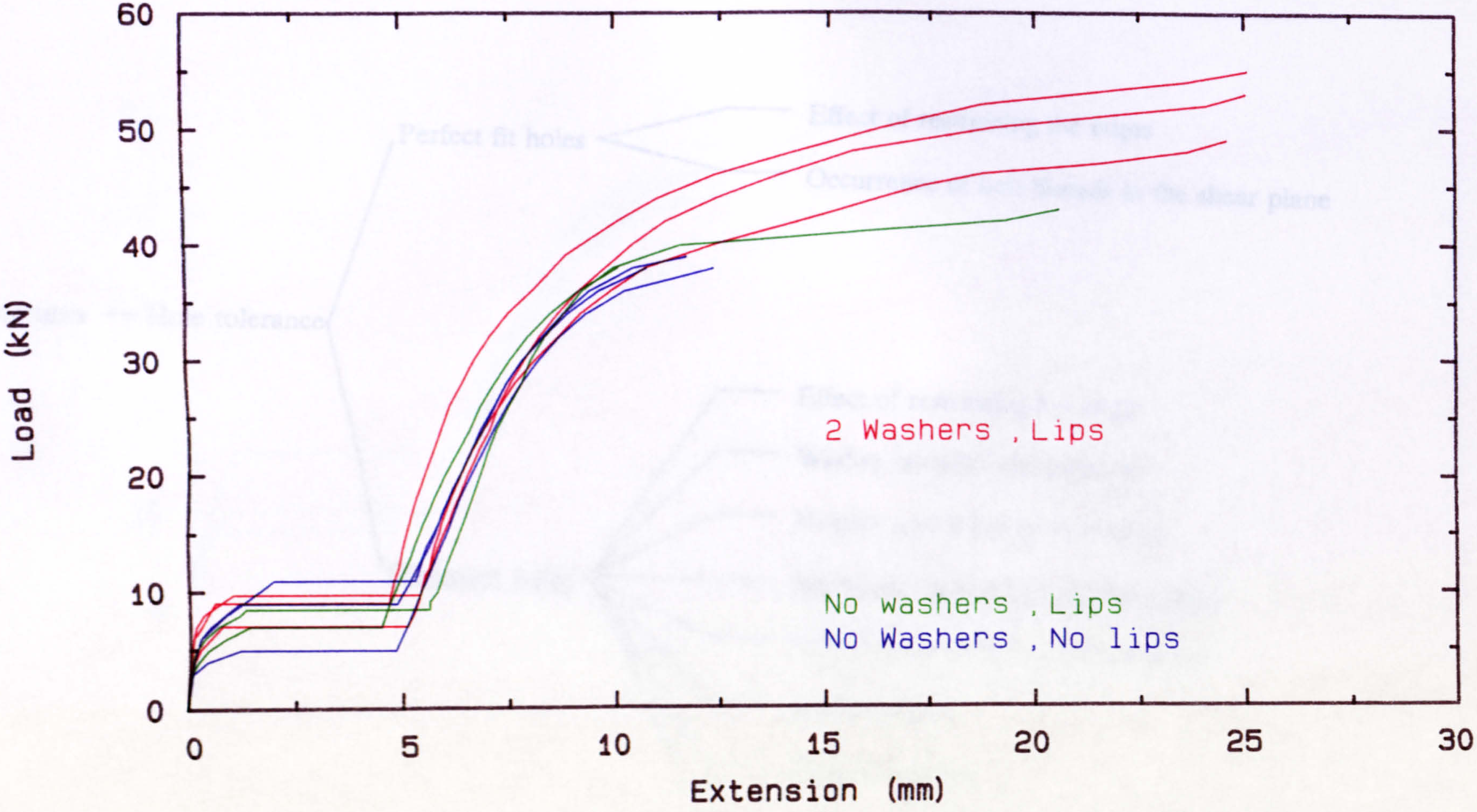


Fig. A2.18C Number and position of washers : No washers, t = 3.02 mm.



- Lap tests
  - Hole tolerance
    - Perfect fit holes
      - Effect of restraining the edges
      - Occurrence of bolt threads in the shear plane
    - <u>Clearance holes</u>
      - Effect of restraining the edges
      - Washer material and diameter
      - Number and position of washers
      - <u>**Mechanical properties of steel sheets**</u>
      - End distance of bolt in line of stress
      - Bolt diameter
      - Thin/thick sheets
      - Range of sheet thickness covered by cold formed steel codes

Fig. A2.19 Mechanical properties of steel sheets :
t = 1.57 mm

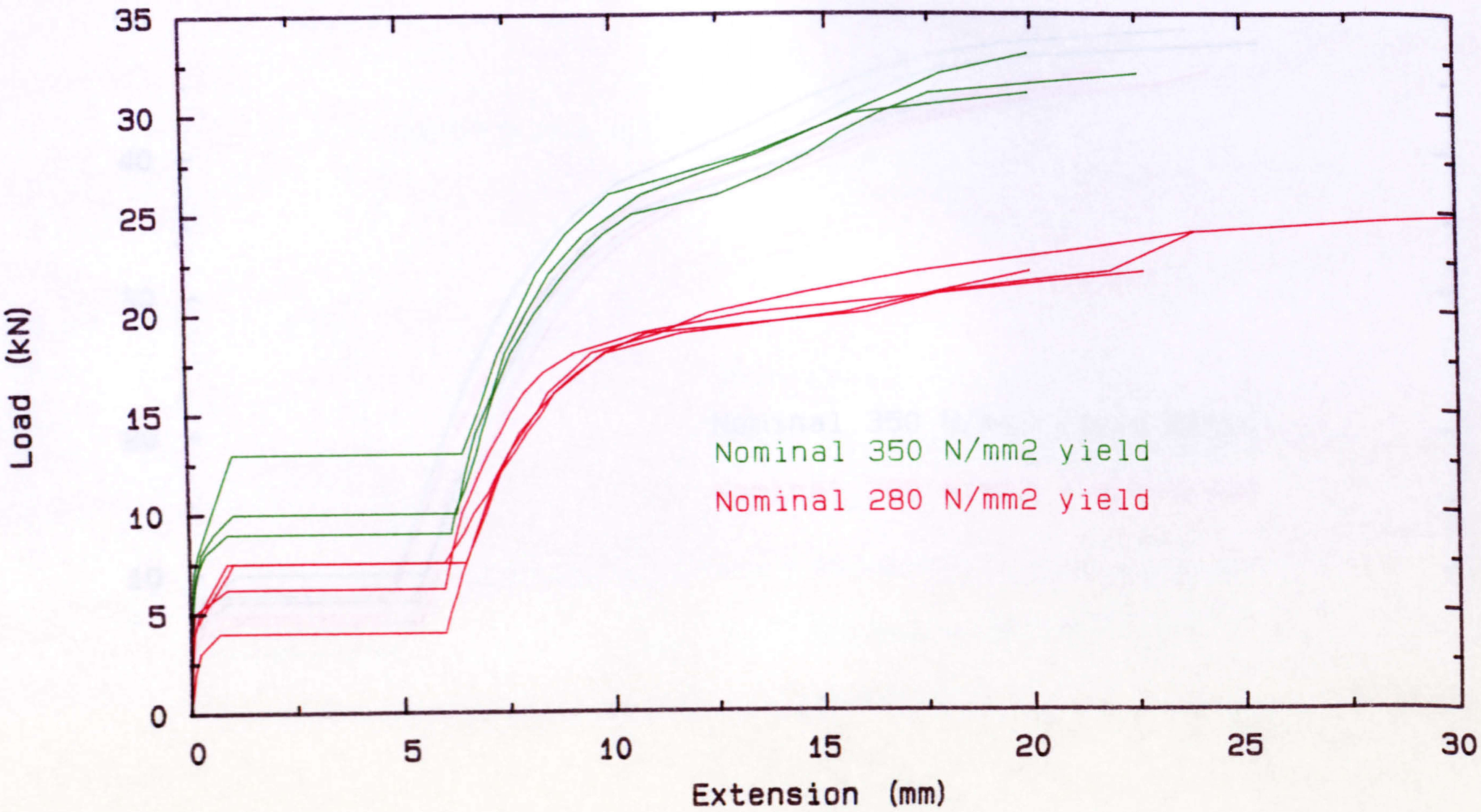


Fig. A2.20 Mechanical properties of steel sheets : t = 2.37 mm

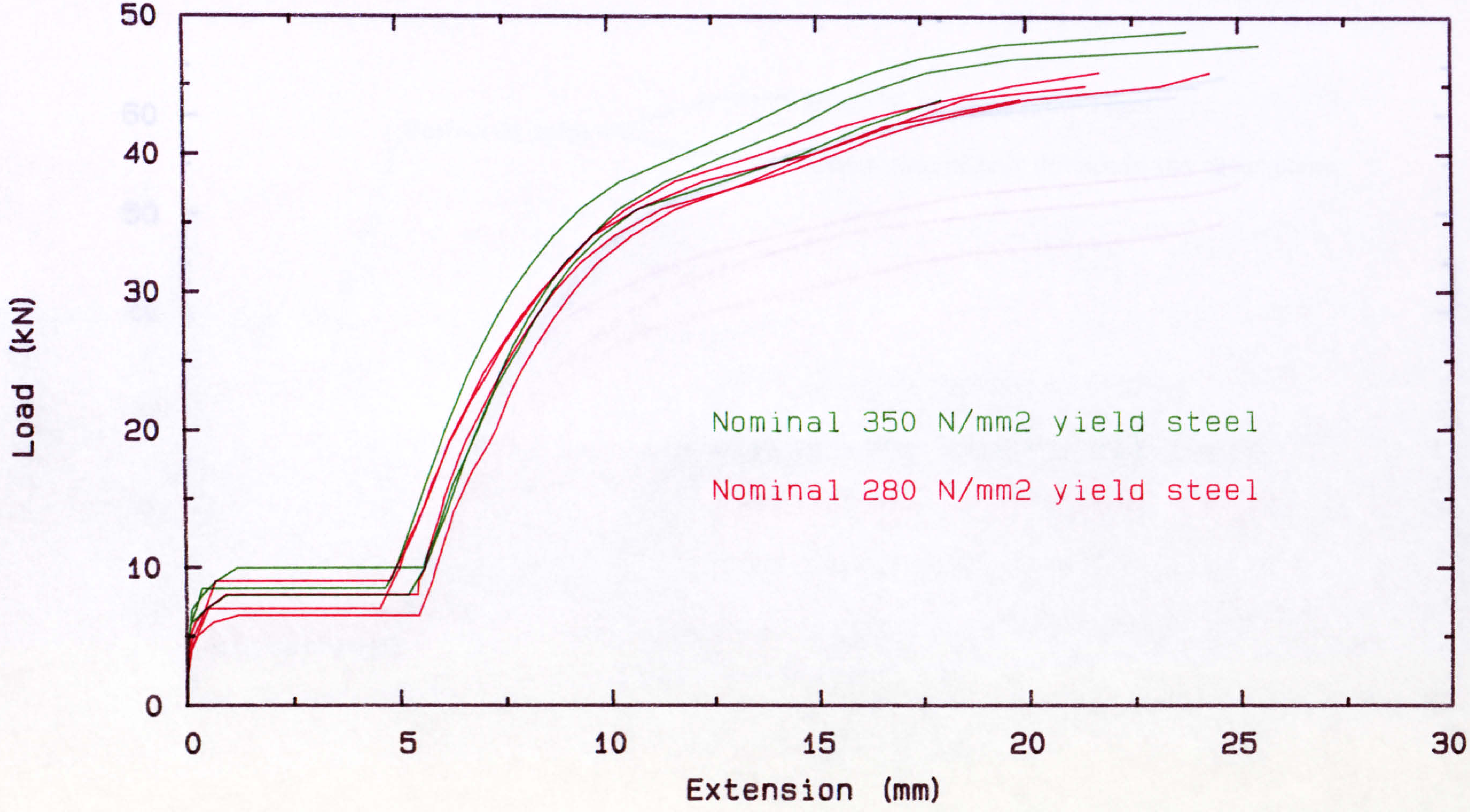
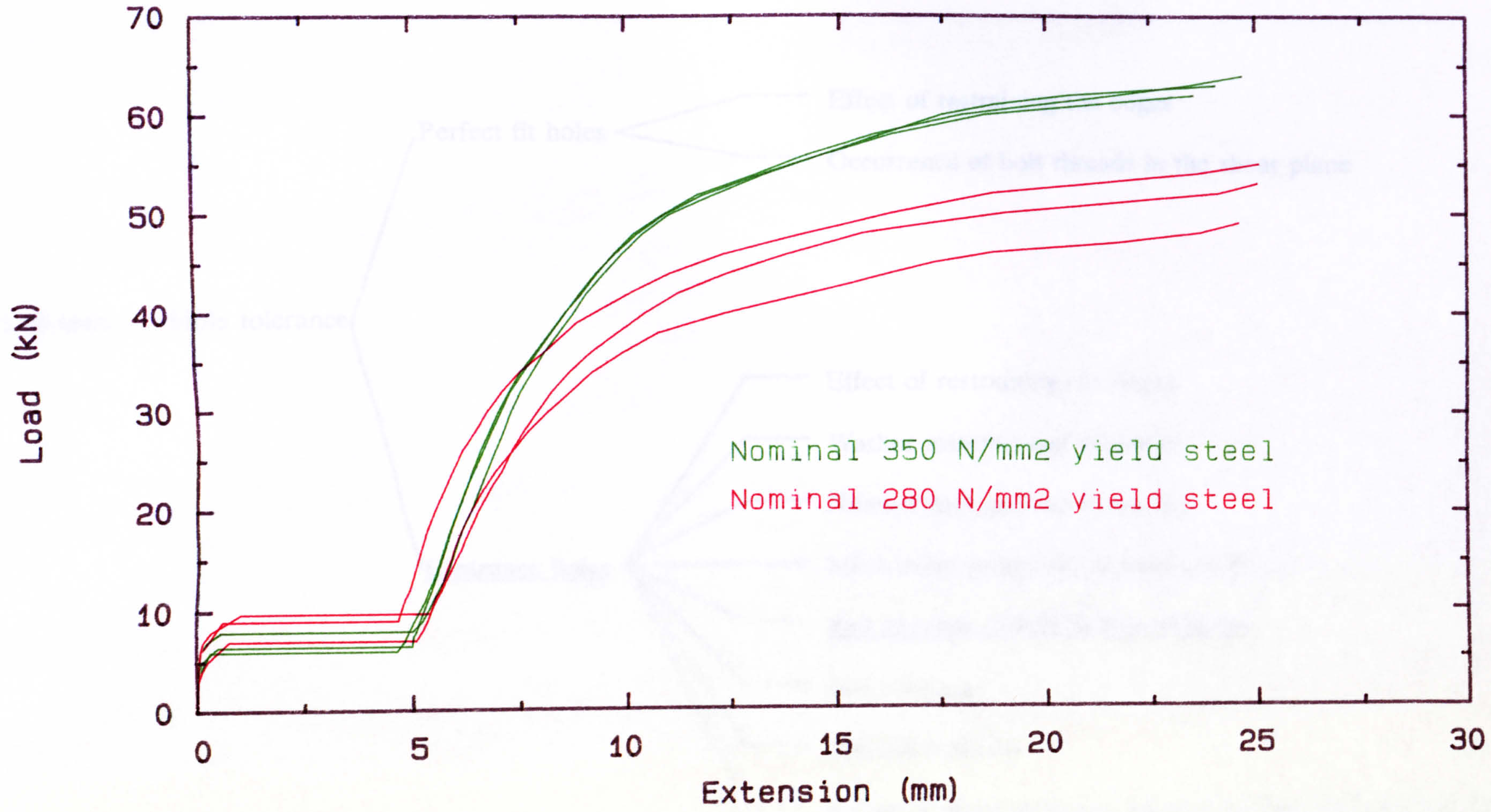


Fig. A2.21 Mechanical properties of steel sheets : t = 3.11 mm



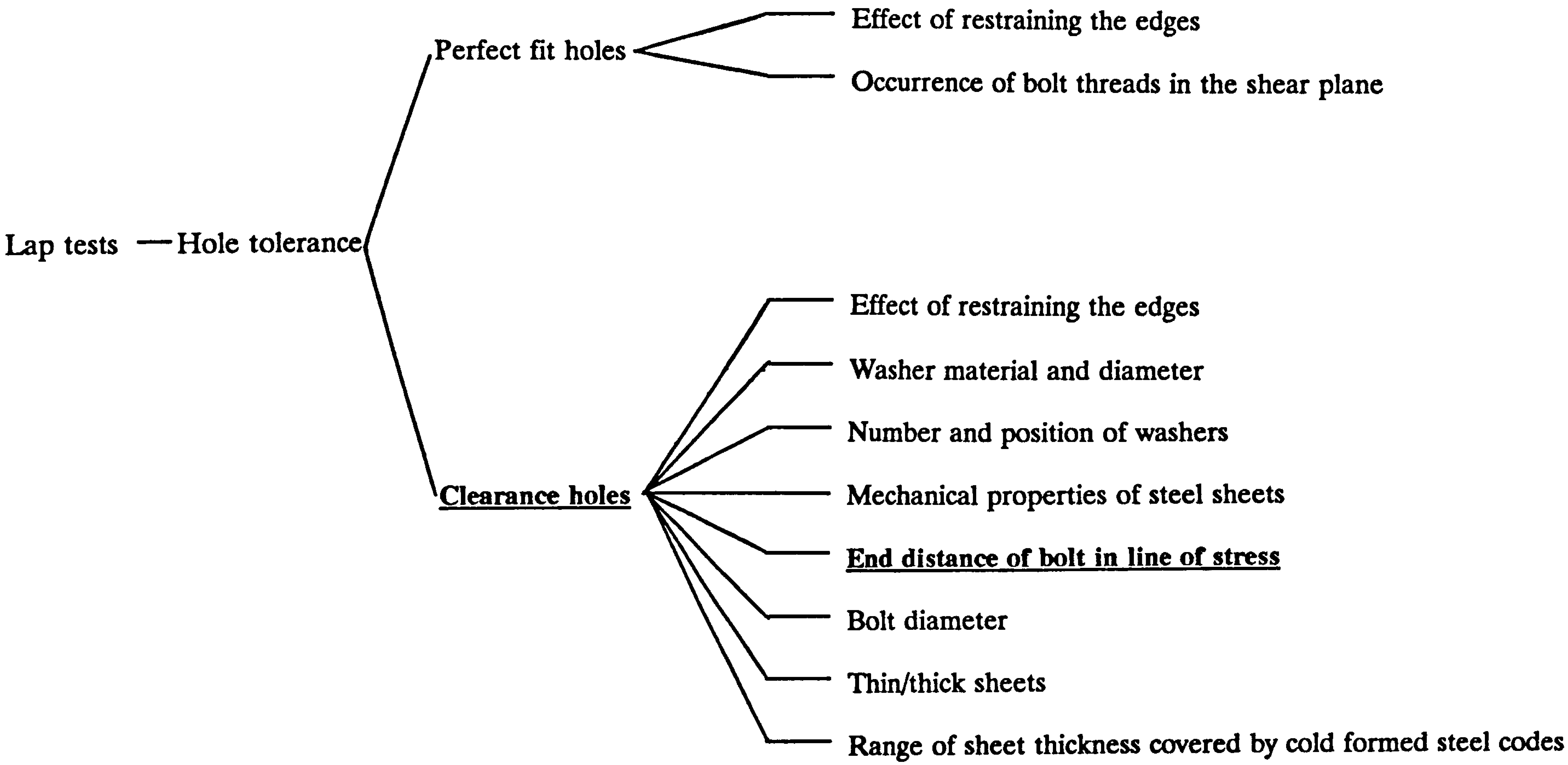

Lap tests
Hole tolerance
Perfect fit holes
Effect of restraining the edges
Occurrence of bolt threads in the shear plane
Clearance holes
Effect of restraining the edges
Washer material and diameter
Number and position of washers
Mechanical properties of steel sheets
End distance of bolt in line of stress
Bolt diameter
Thin/thick sheets
Range of sheet thickness covered by cold formed steel codes

Fig. A2.22a End distance of bolt in line of stress :
t = 1.63 mm

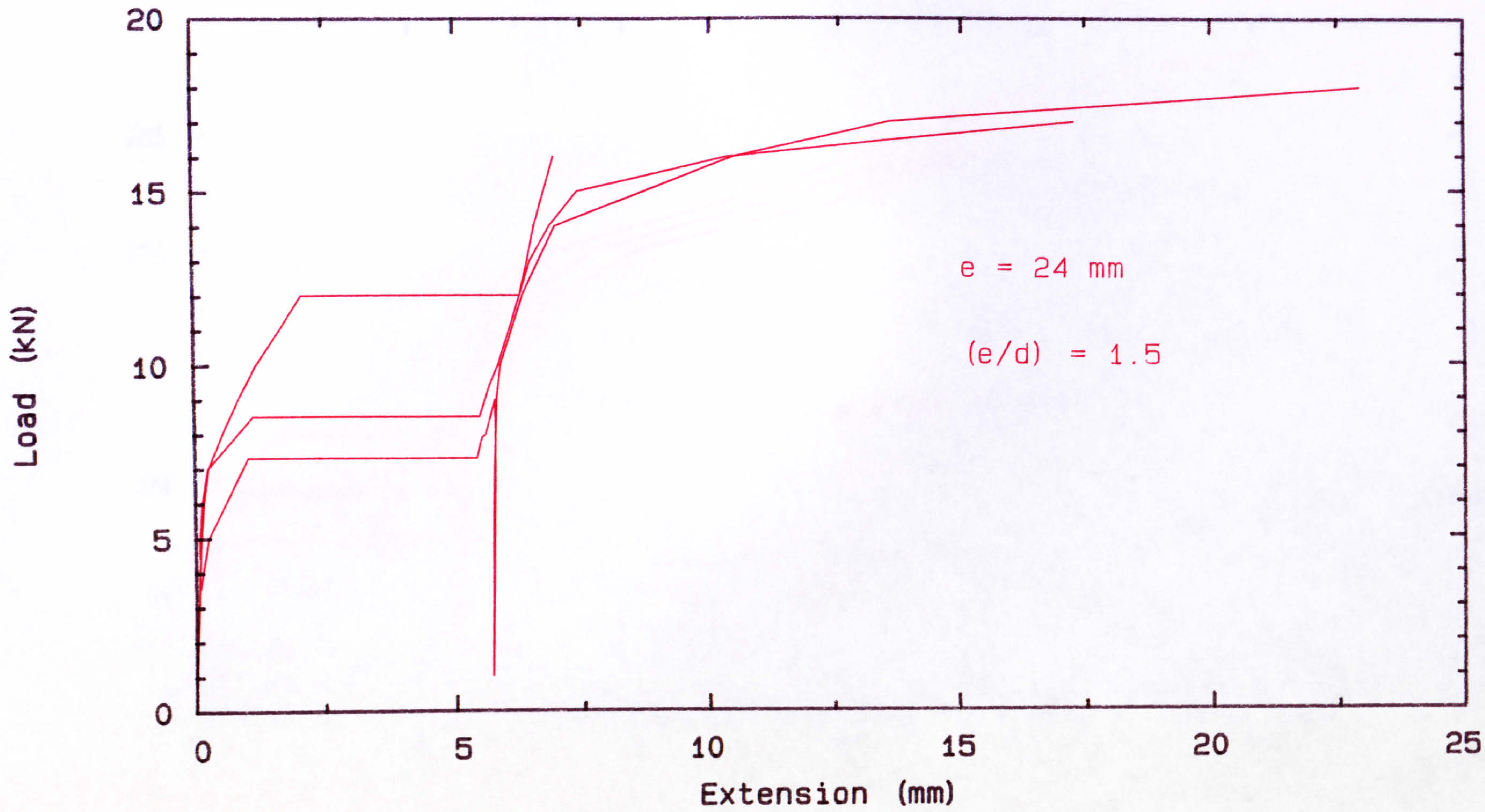


Fig. A2.22b End distance of bolt in line of stress :
t = 1.63 mm

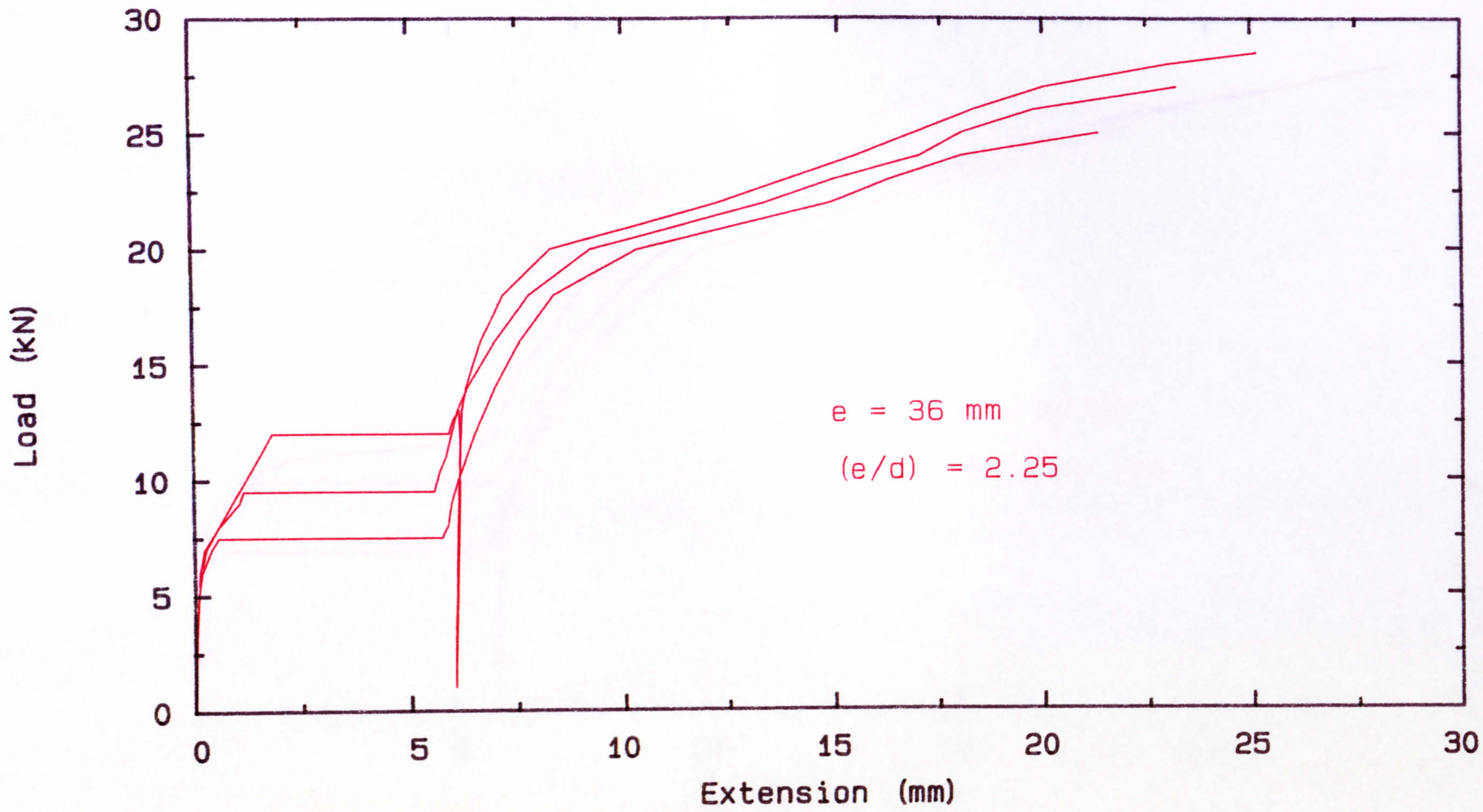


Fig. A2.22c End distance of bolt in line of stress :
t = 1.63 mm

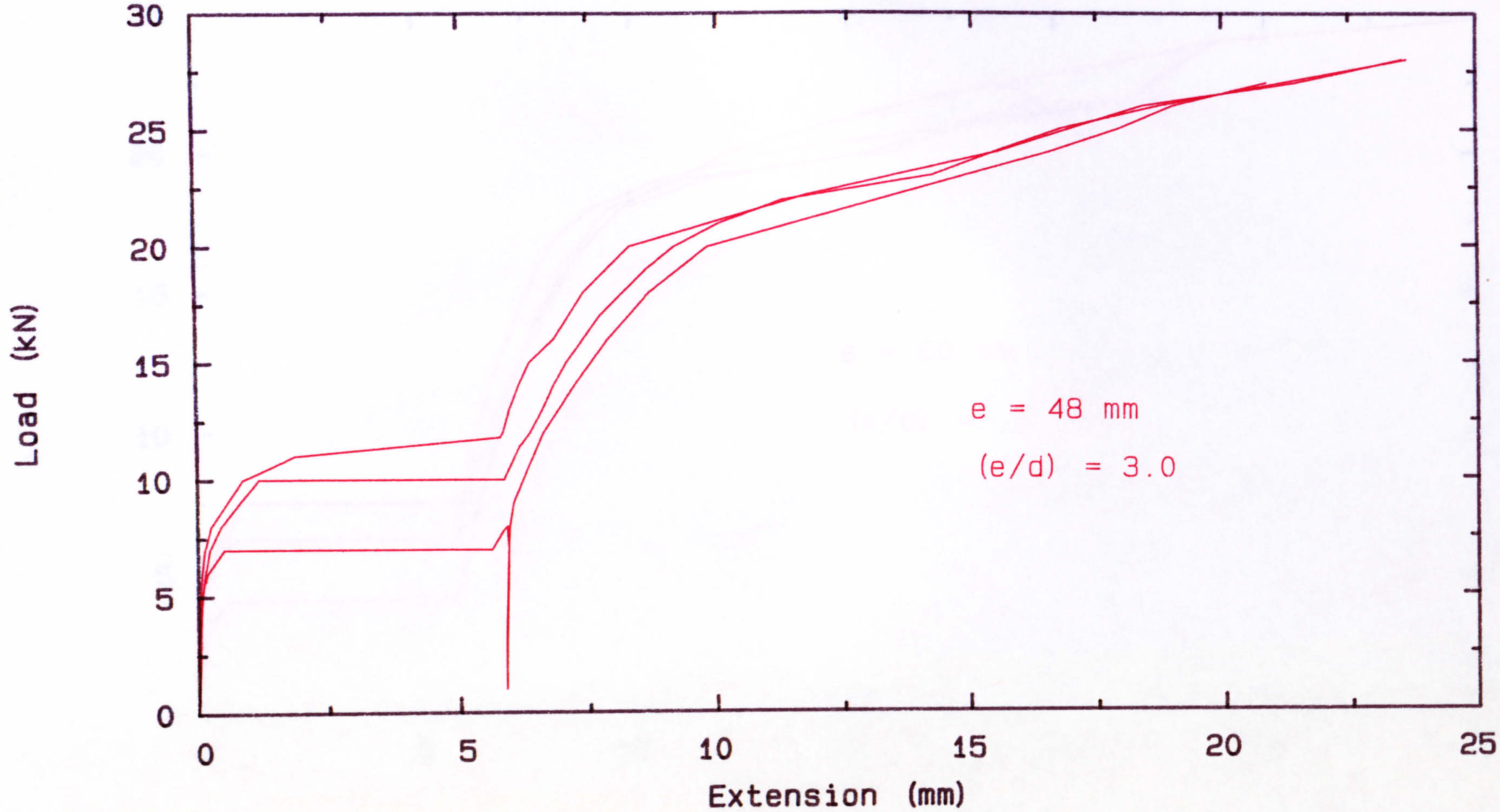


Fig. A2.22d End distance of bolt in line of stress :
t = 1.63 mm

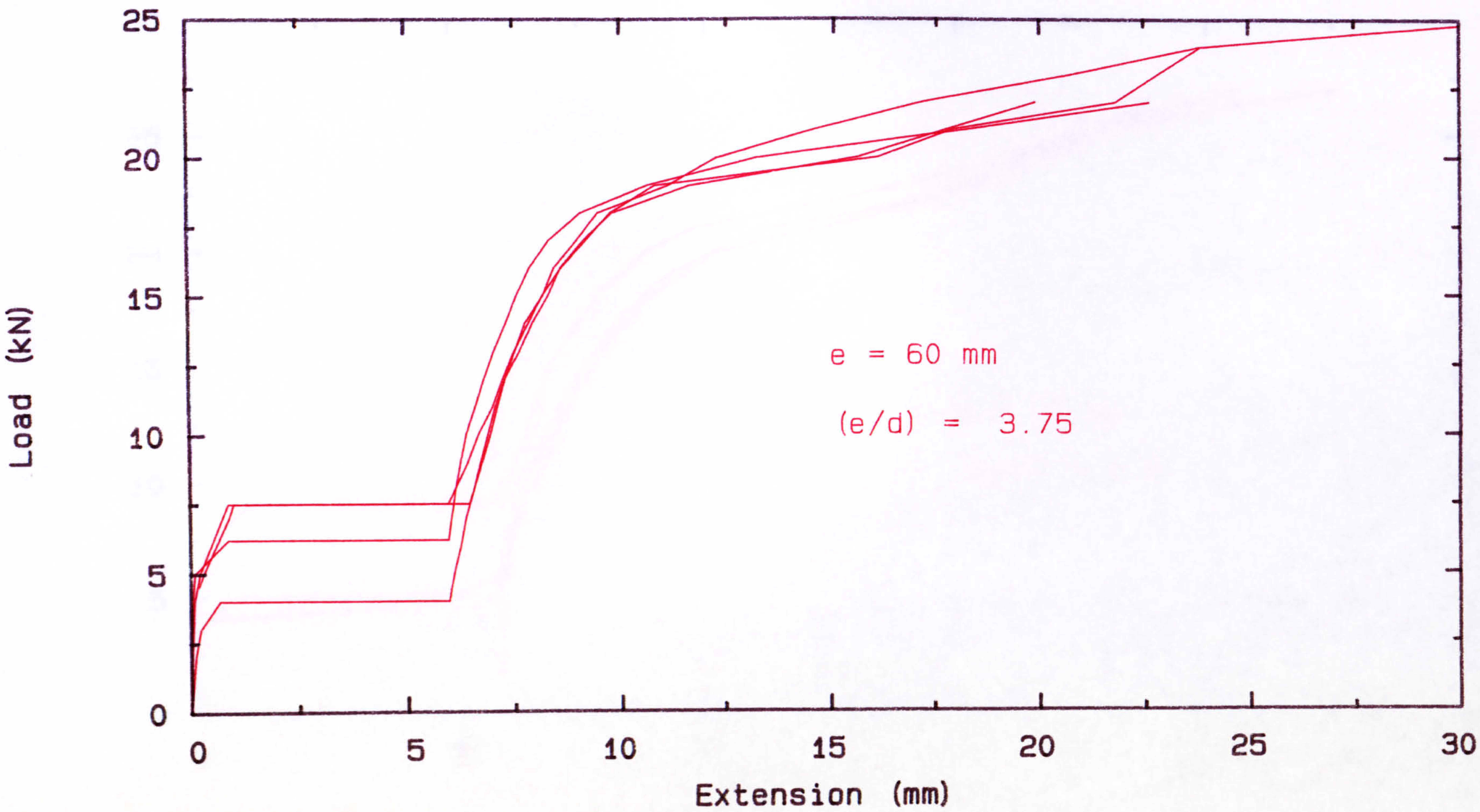


Fig. A2.22e End distance of bolt in line of stress :
$t = 1.63$ mm

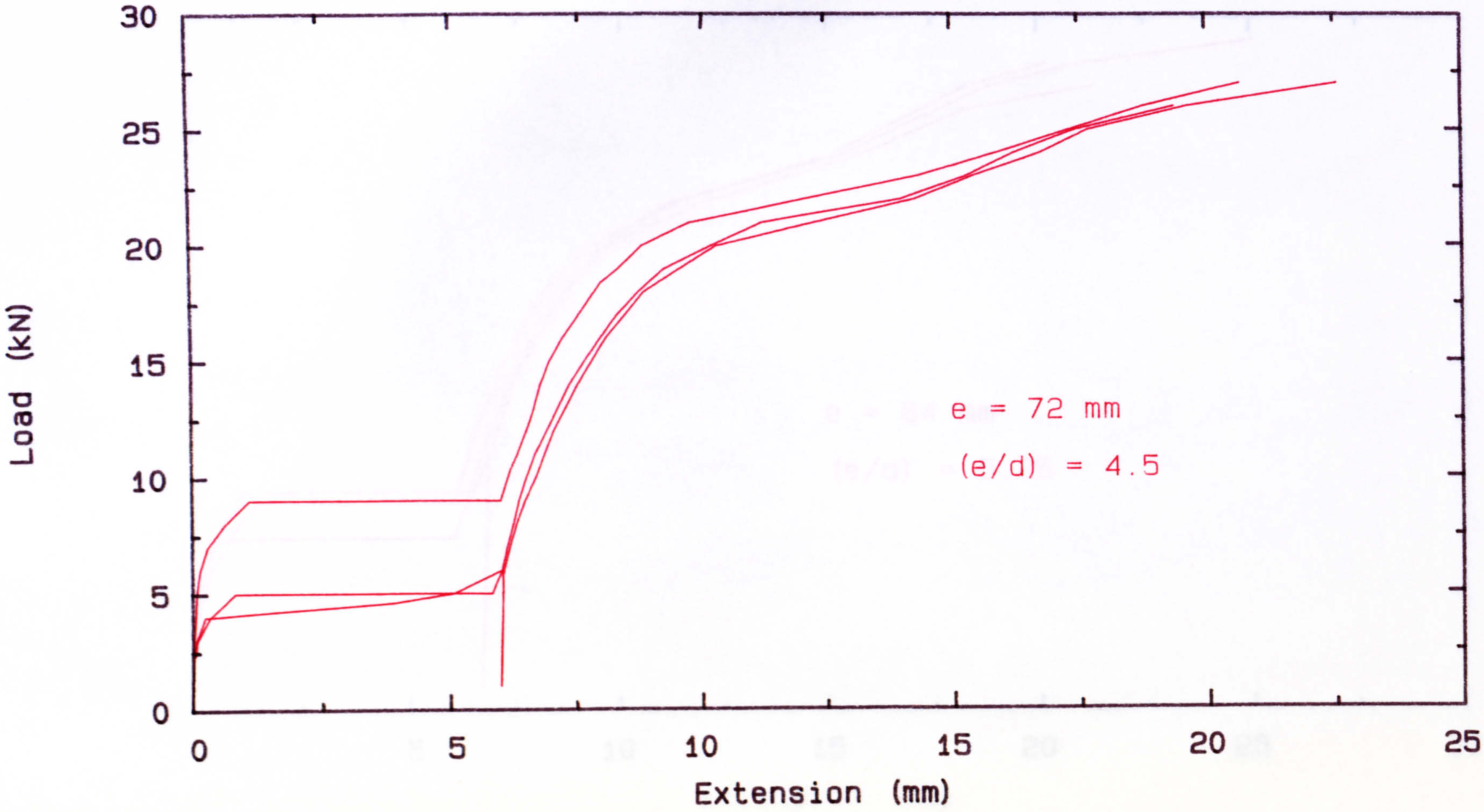


Fig. A2.22f End distance of bolt in line of stress :
t = 1.63 mm

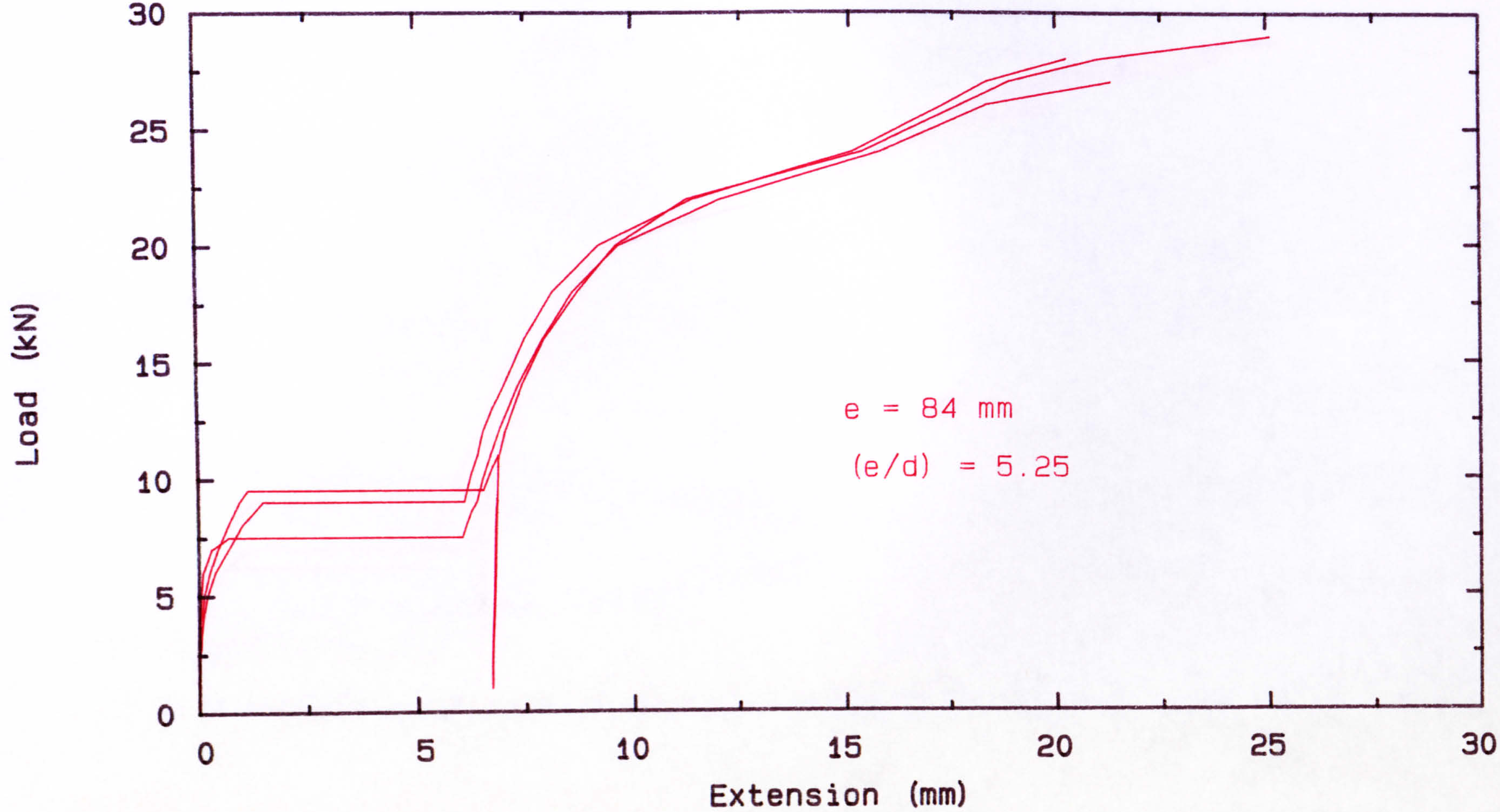


Fig. A2.22g End distance of bolt in line of stress :
t = 1.63 mm

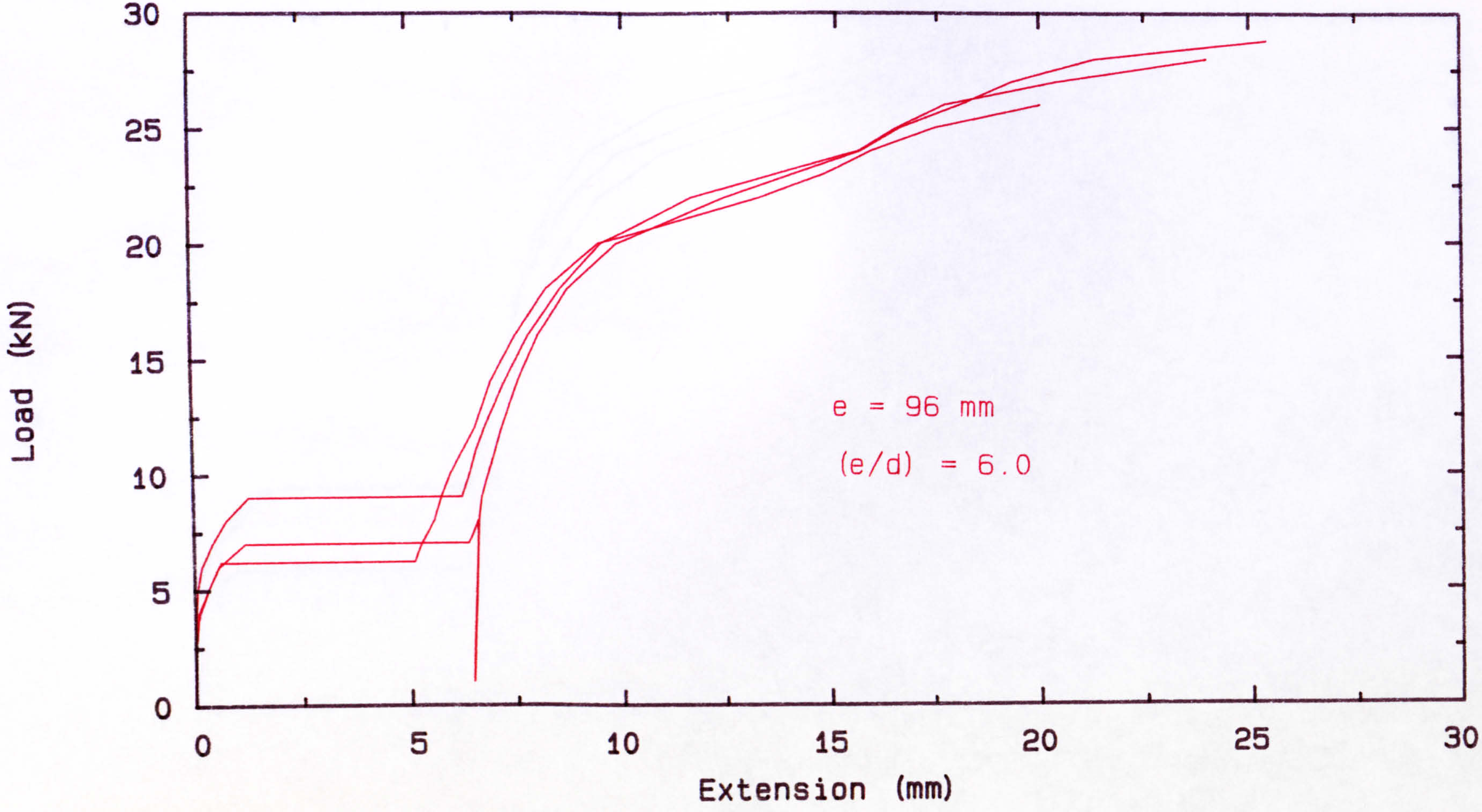


Fig. A2.23a End distance of bolt in line of stress :
t = 2.48 mm

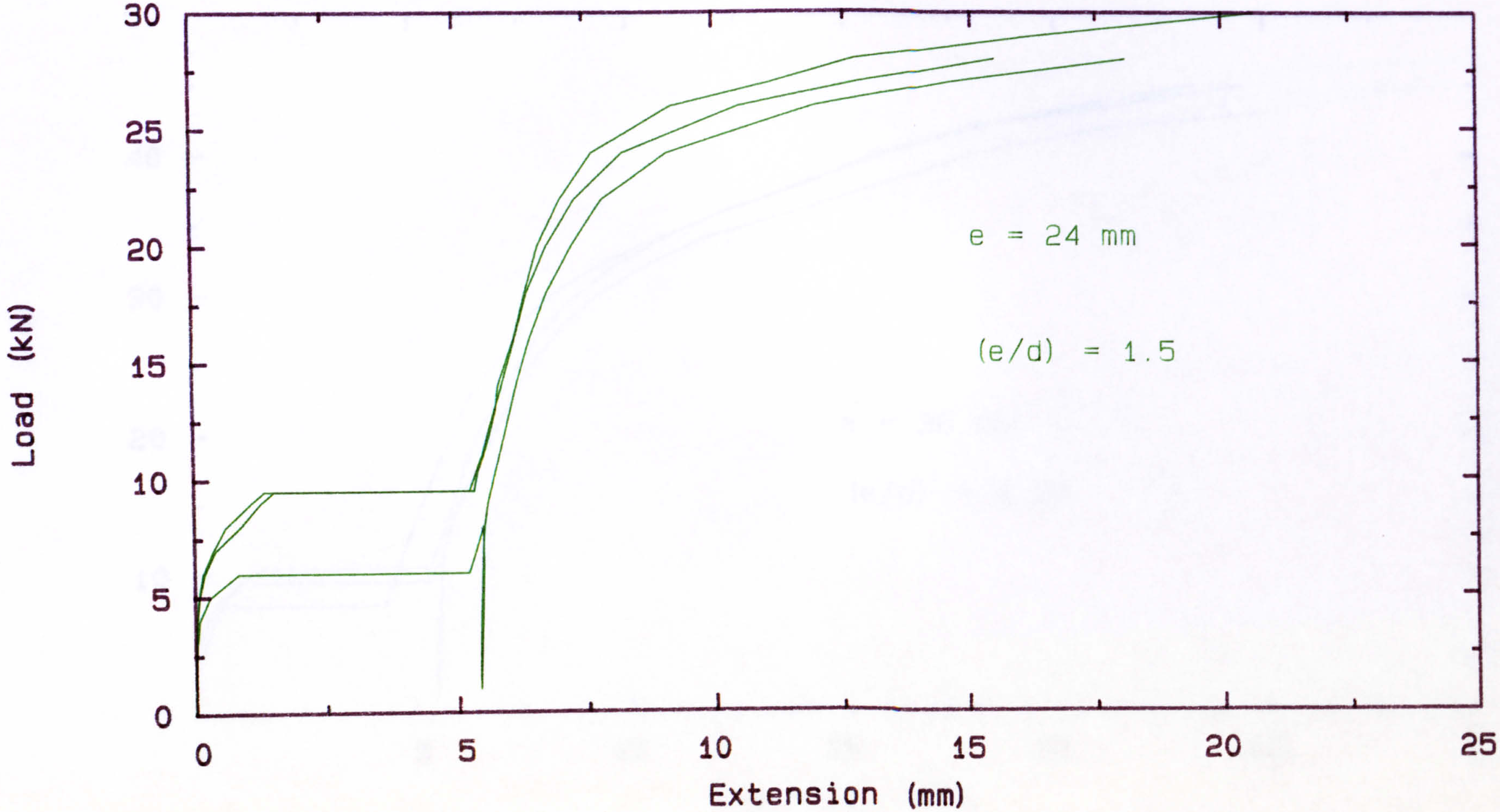


Fig. A2.23b End distance of bolt in line of stress :
t = 2.48 mm

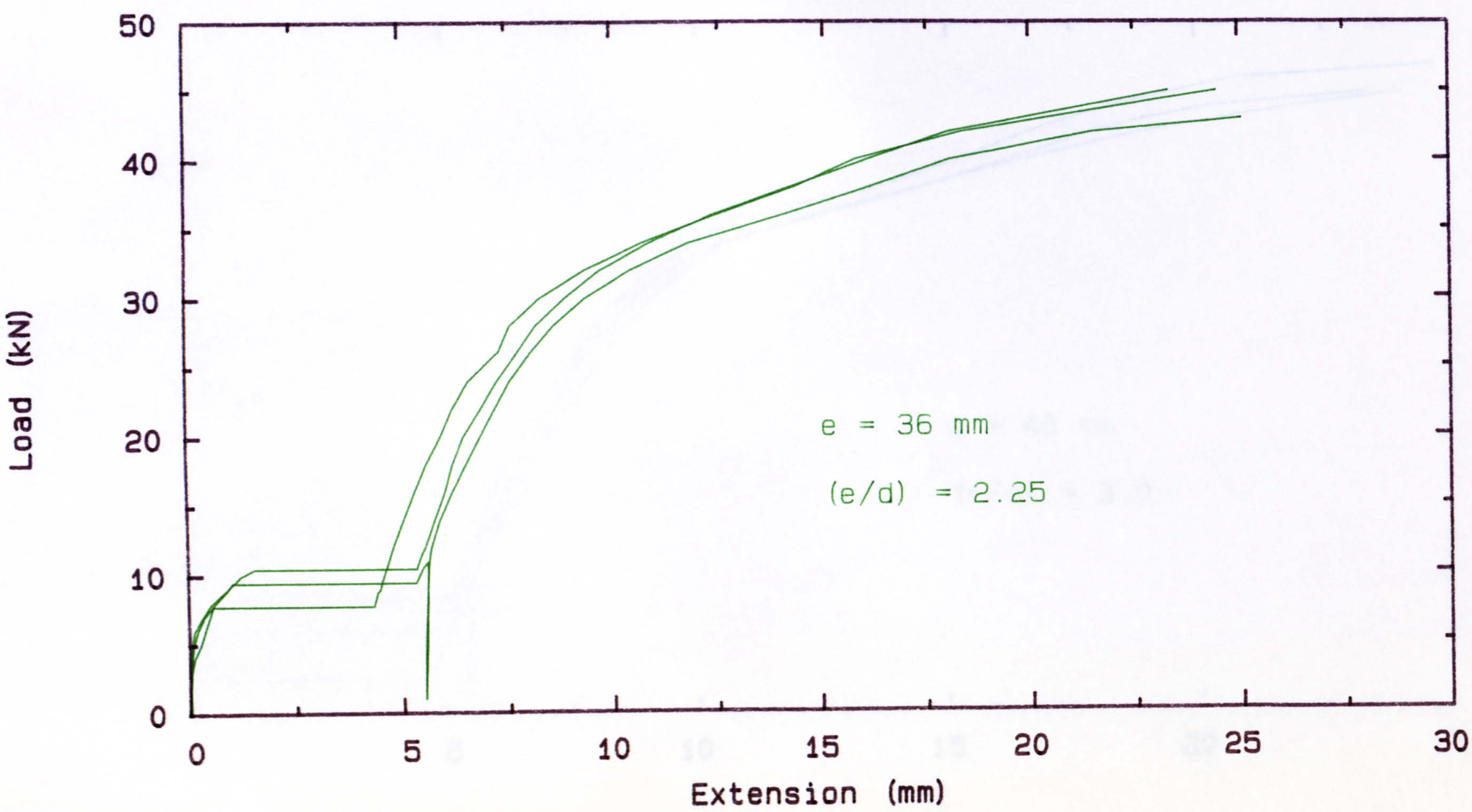


Fig. A2.23c End distance of bolt in line of stress :
t = 2.48 mm

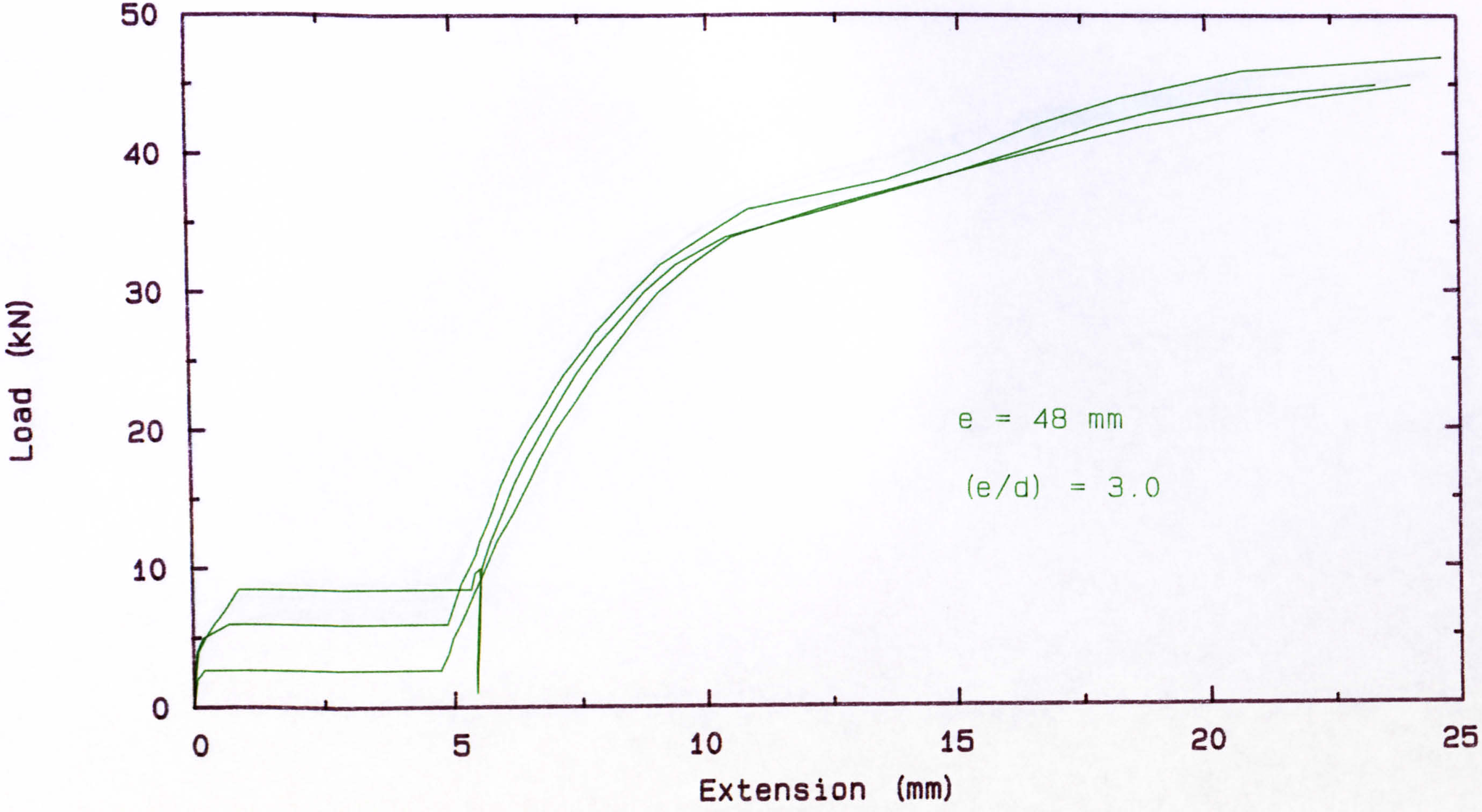


Fig. A2.23d End distance of bolt in line of stress :
t = 2.48 mm

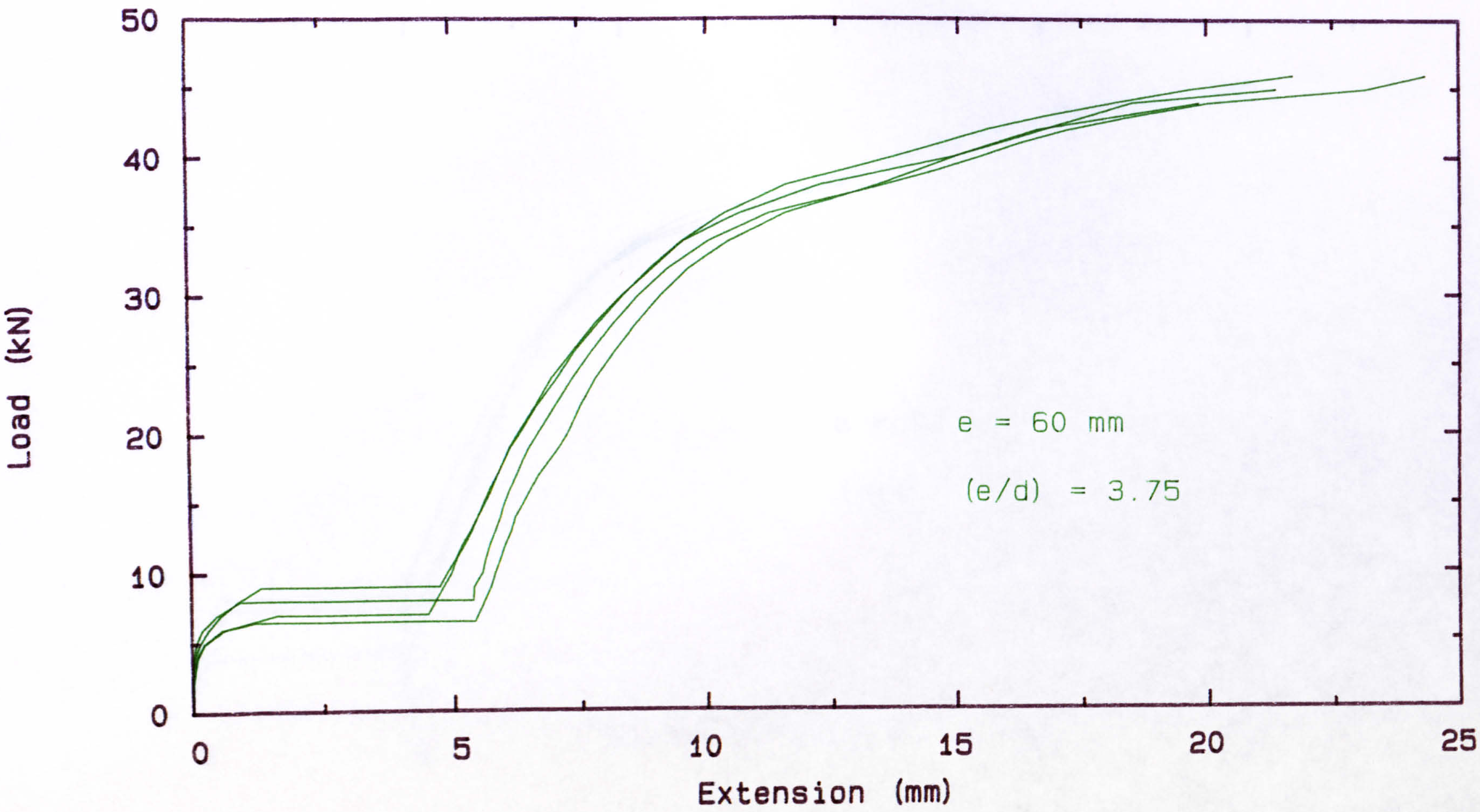


Fig. A2.23e End distance of bolt in line of stress :
t = 2.48 mm

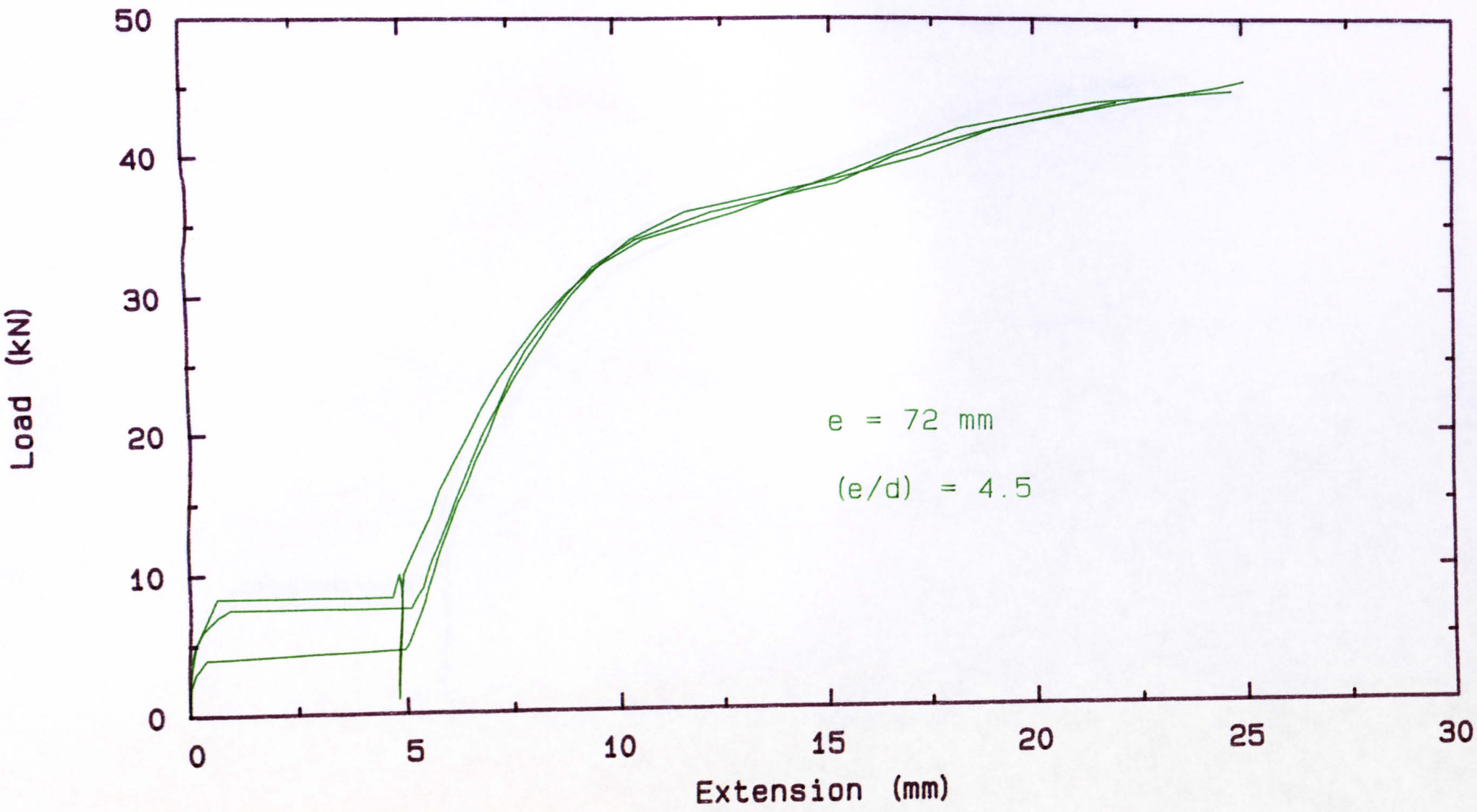


Fig. A2.23f End distance of bolt in line of stress :
t = 2.48 mm

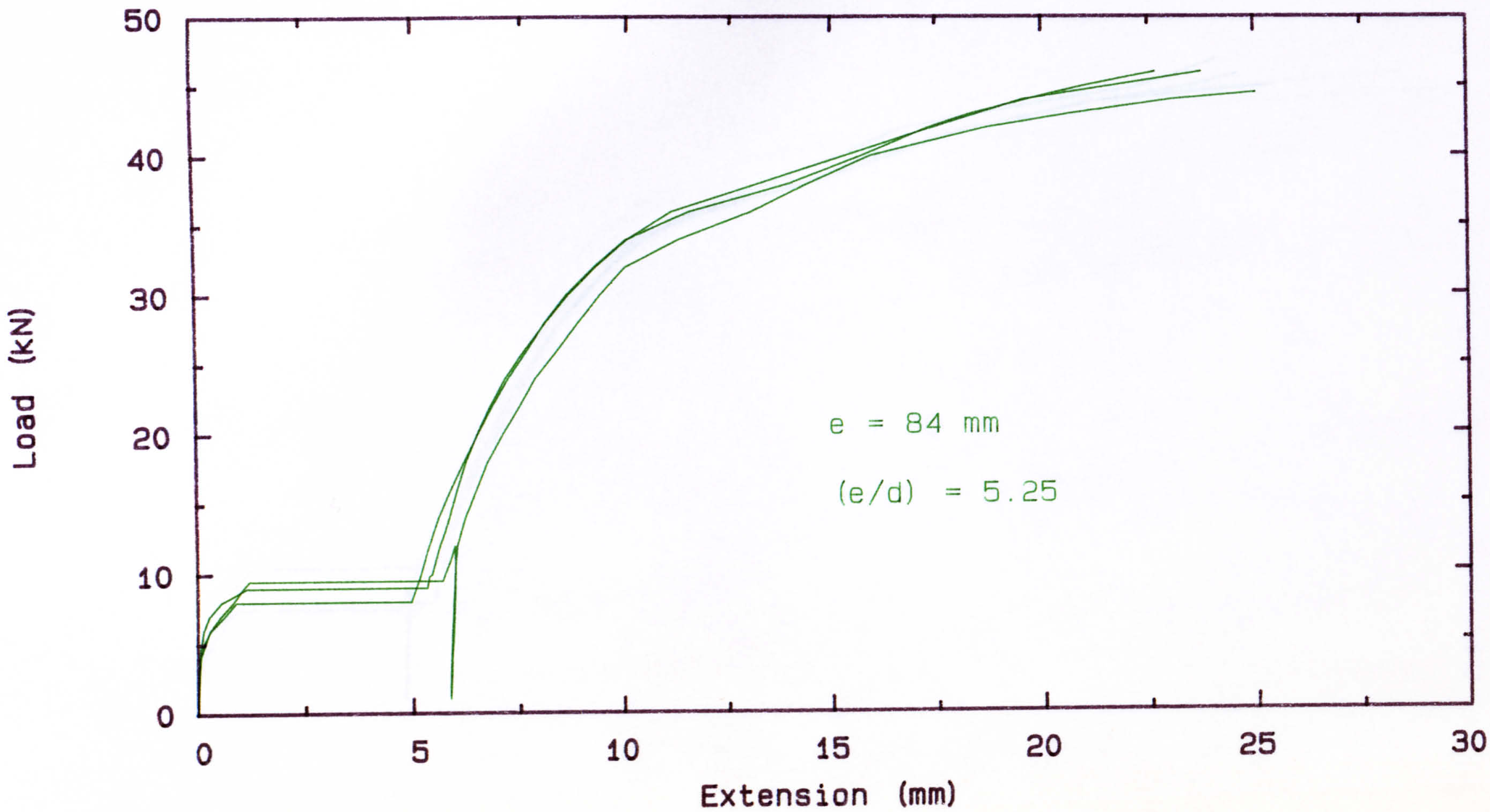


Fig. A2.23g End distance of bolt in line of stress :
t = 2.48 mm

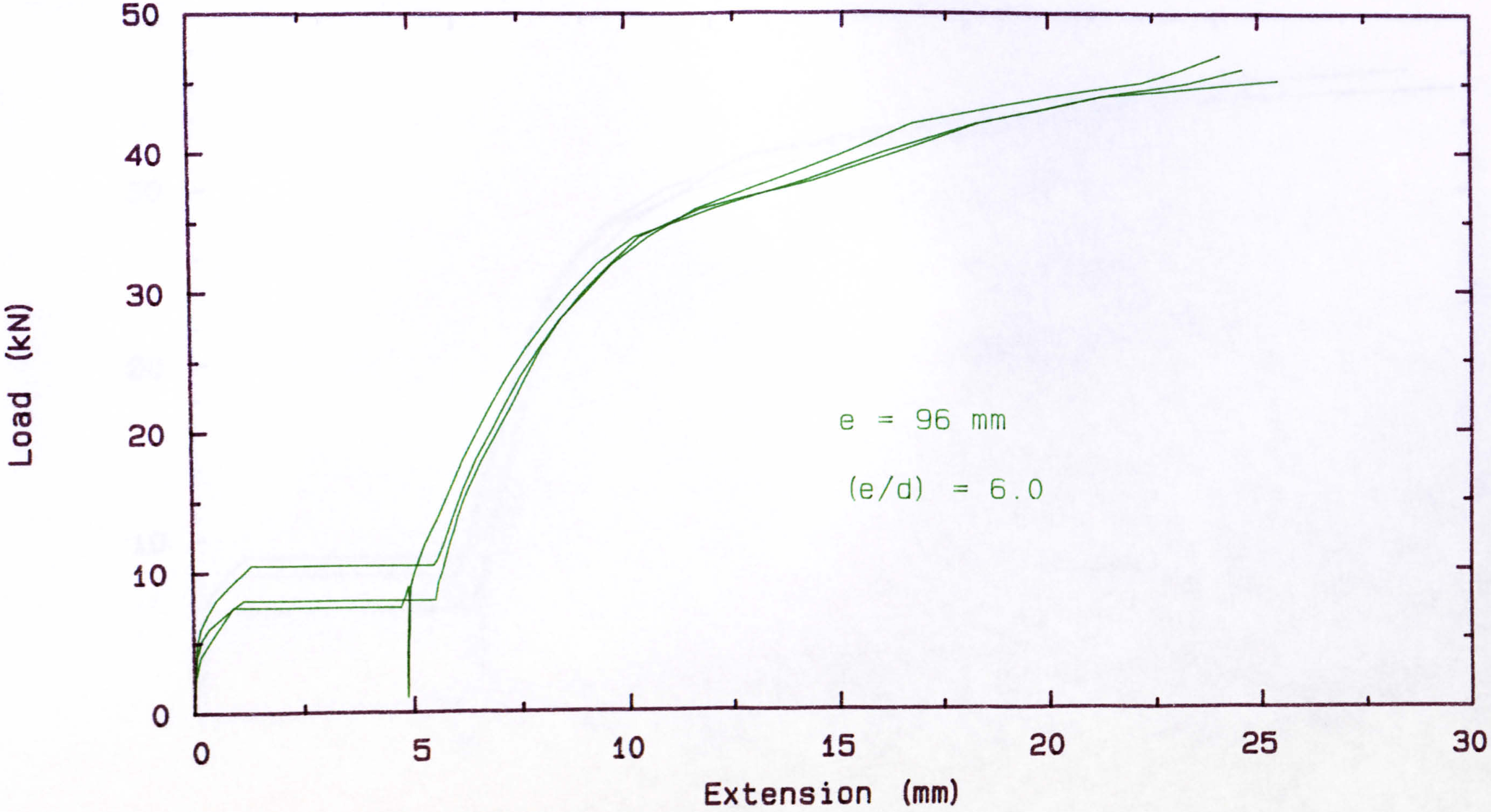


Fig. A2.24a End distance of bolt in line of stress :
t = 3.02 mm

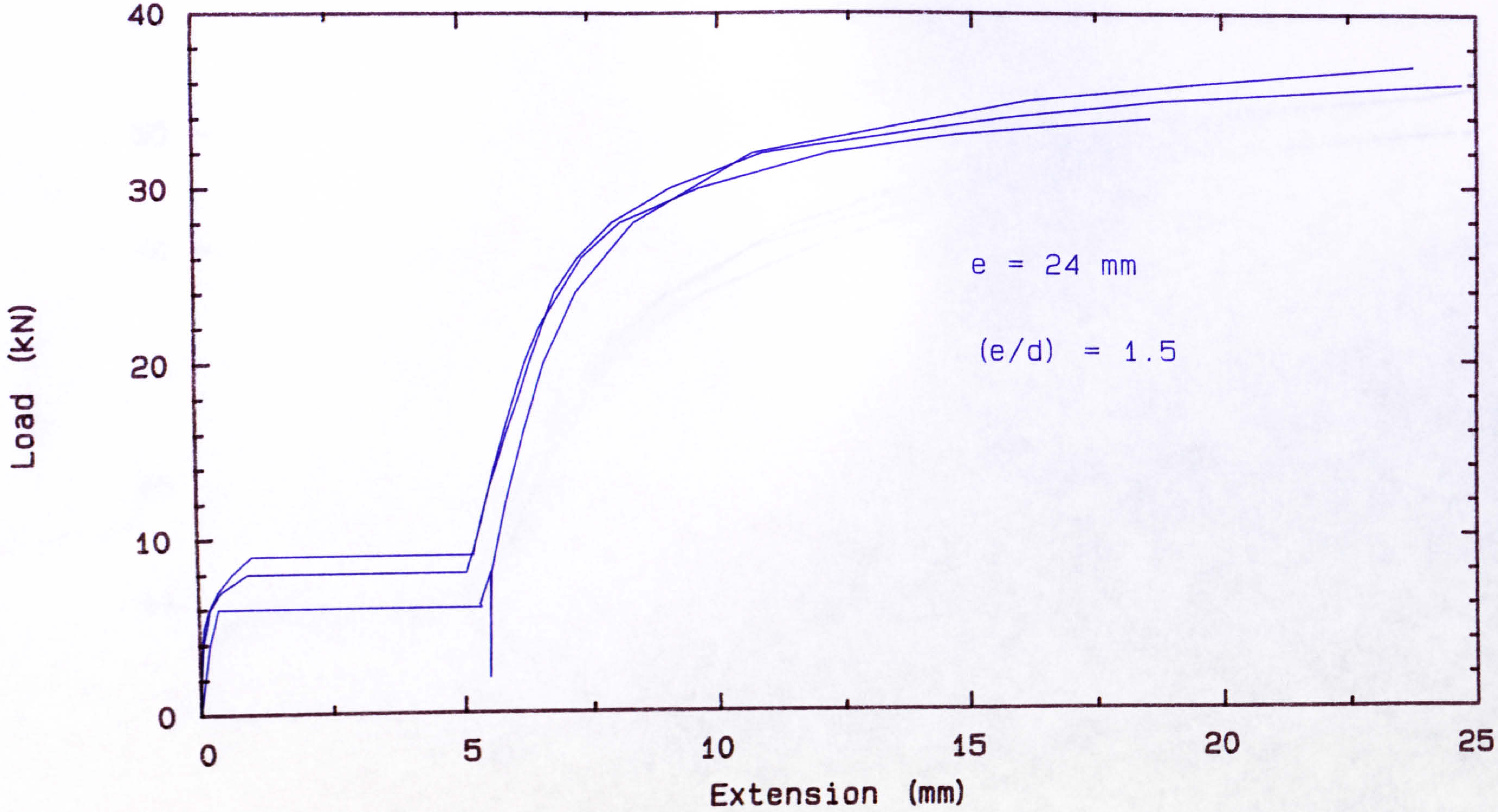


Fig. A2.24b End distance of bolt in line of stress :
t = 3.02 mm

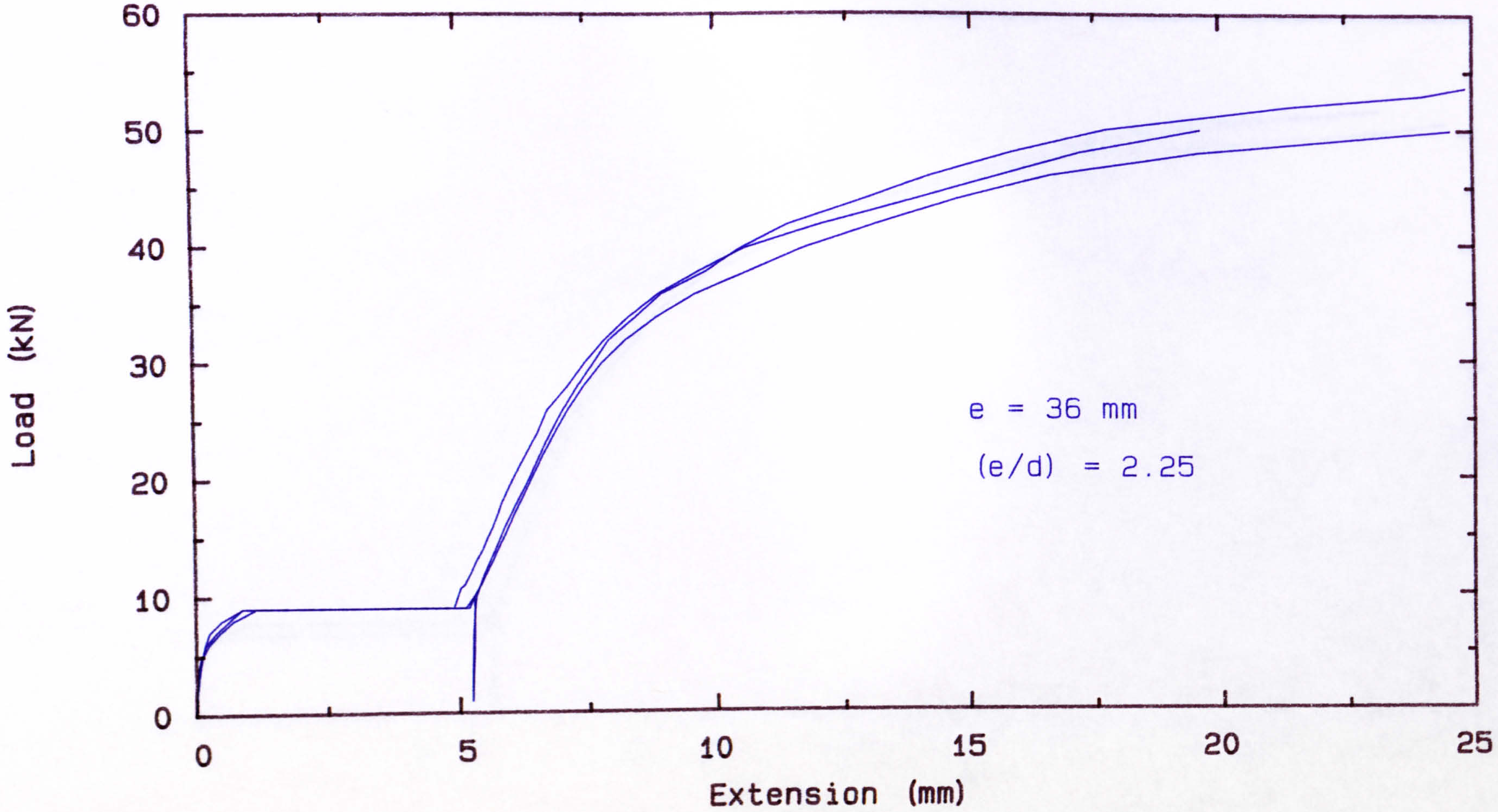


Fig. A2.24c End distance of bolt in line of stress :
t = 3.02 mm

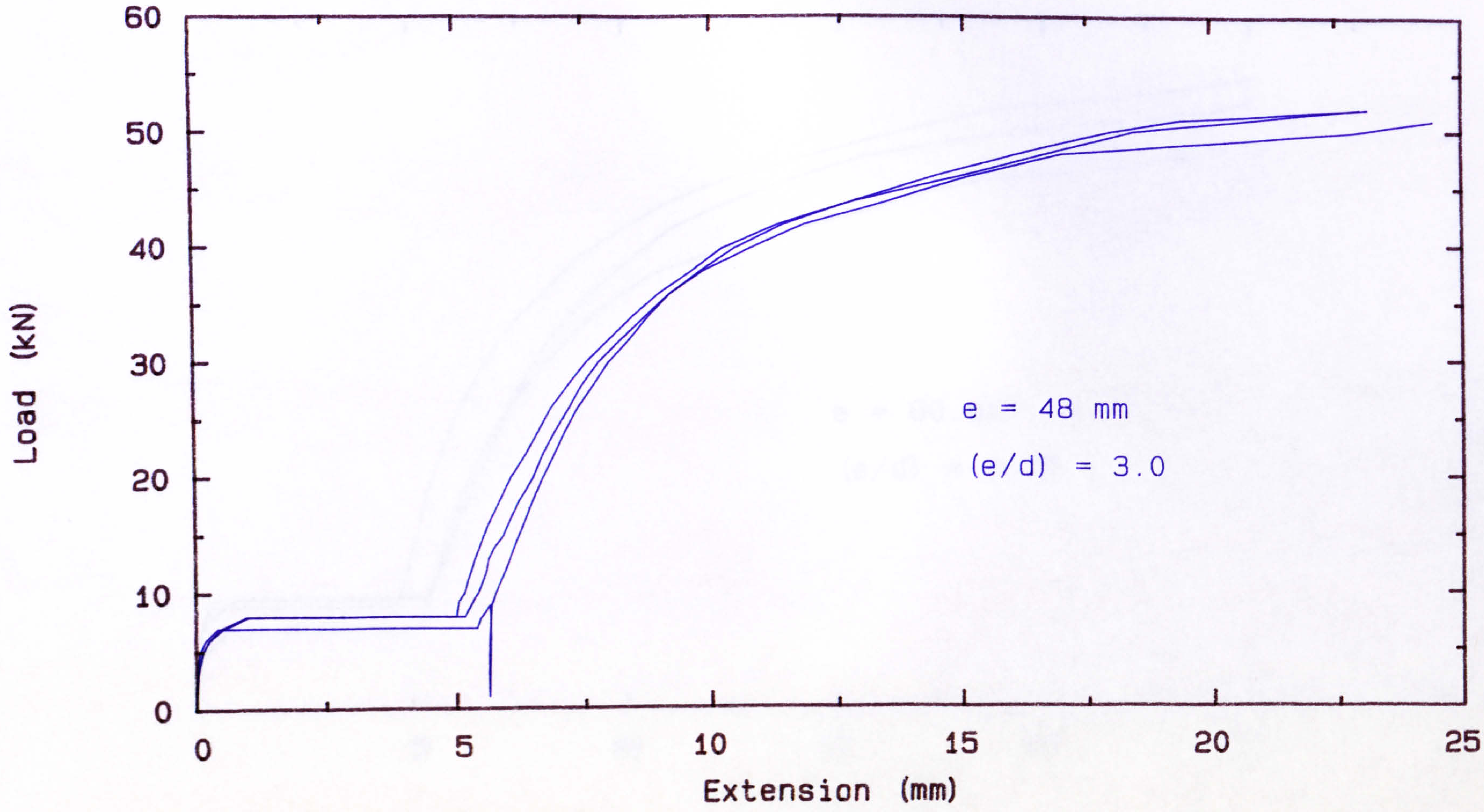


Fig. A2.24d End distance of bolt in line of stress :
t = 3.02 mm

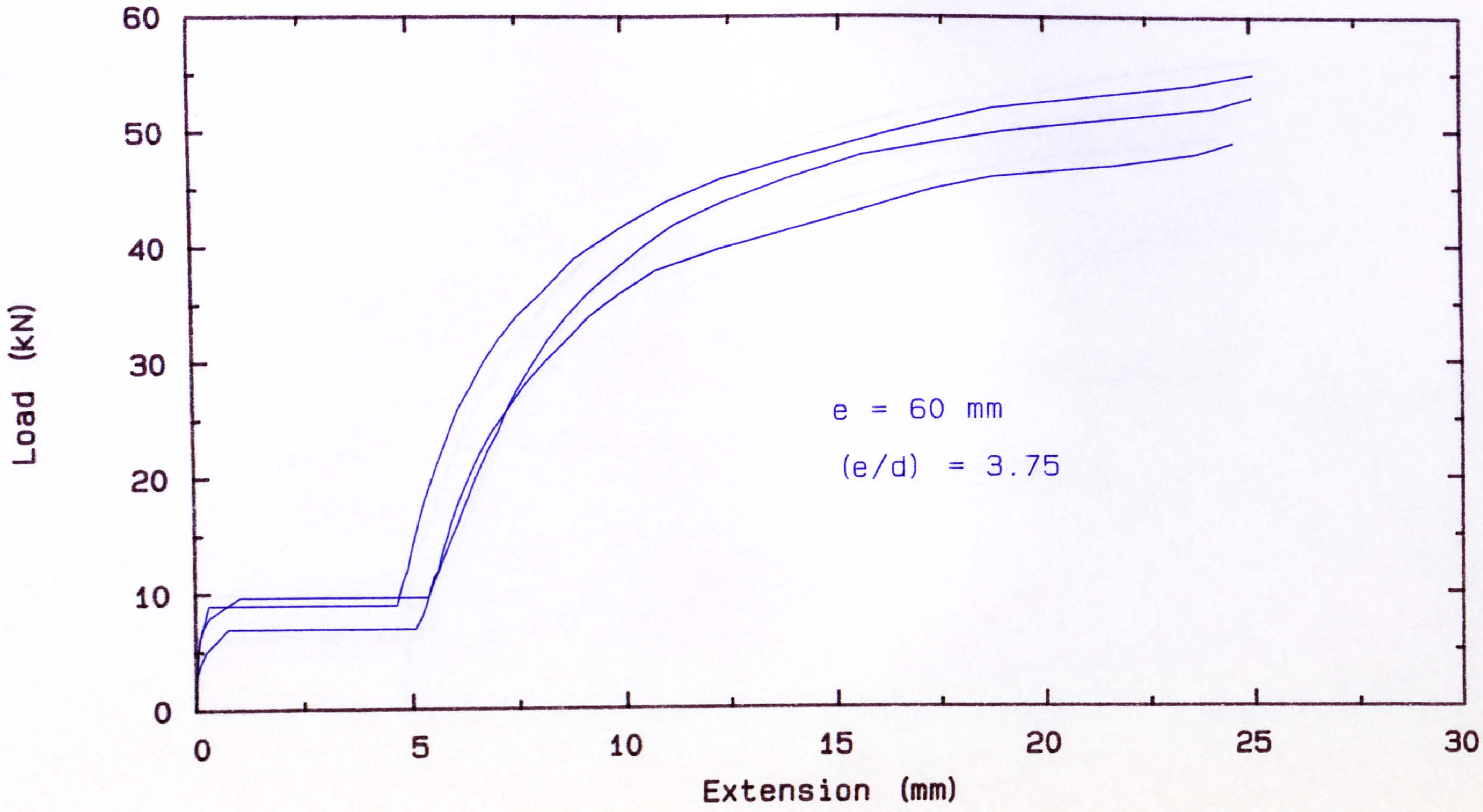


Fig. A2.24e End distance of bolt in line of stress :
t = 3.02 mm

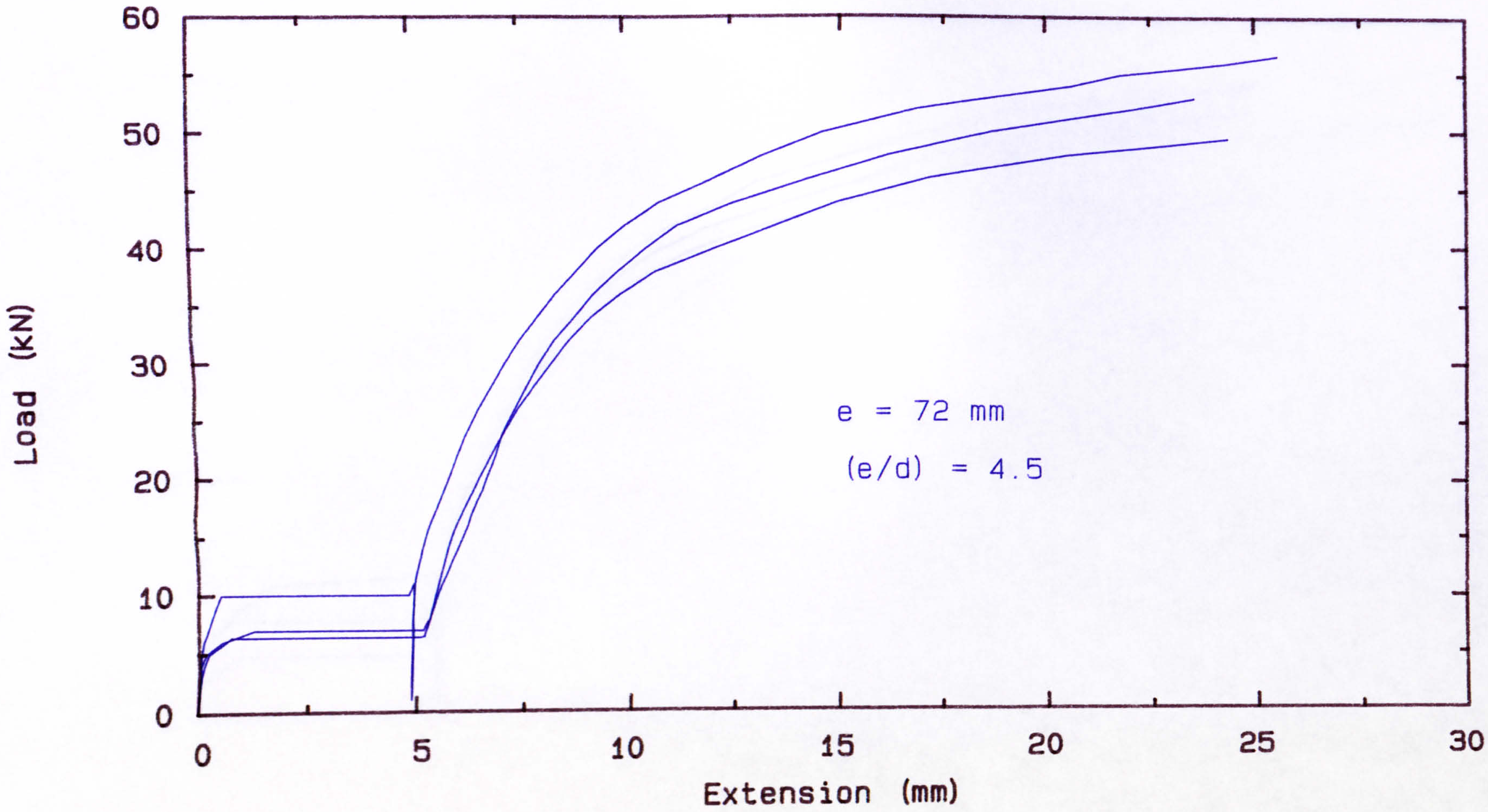


Fig. A2.24f End distance of bolt in line of stress :
t = 3.02 mm

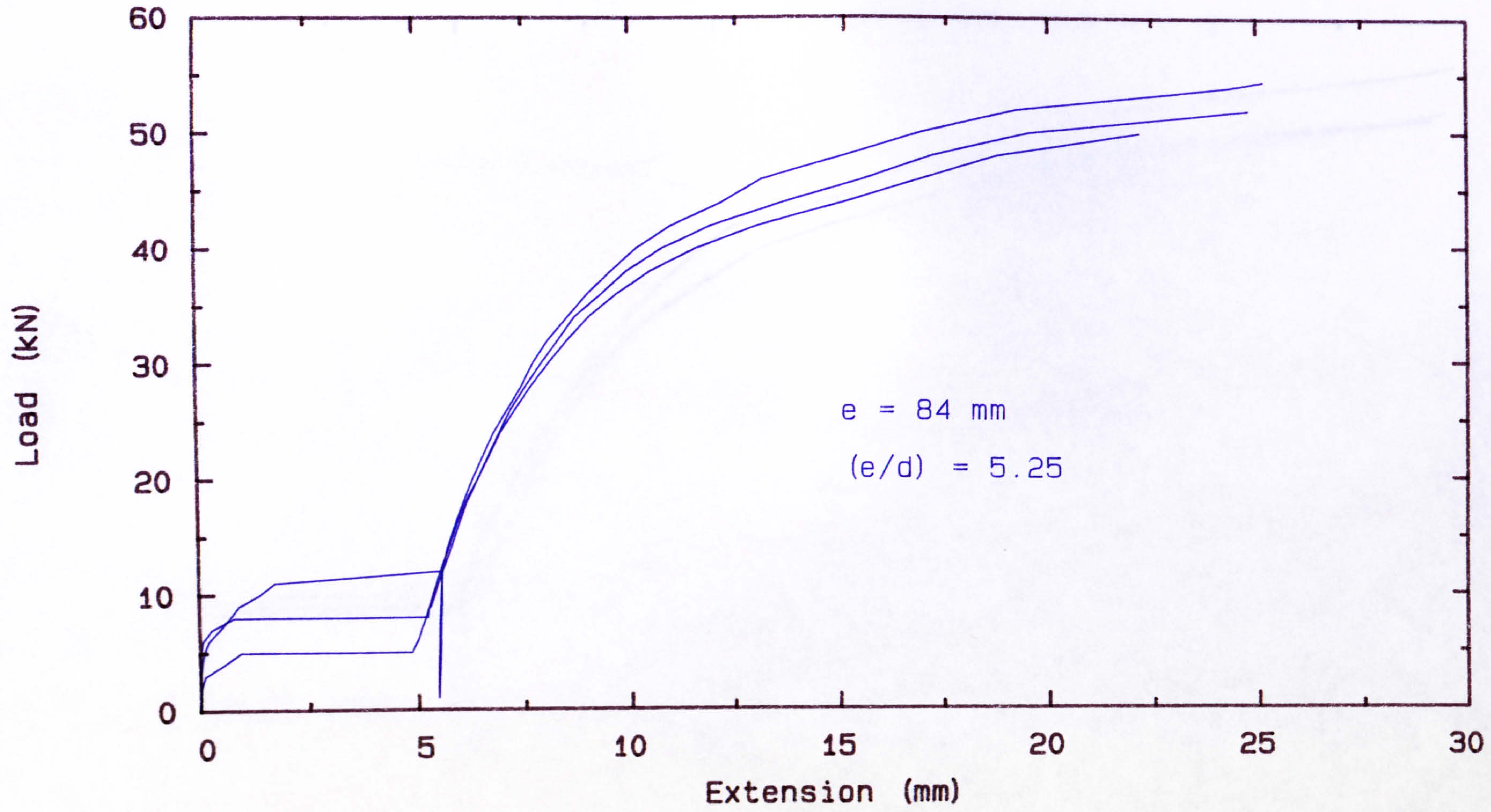
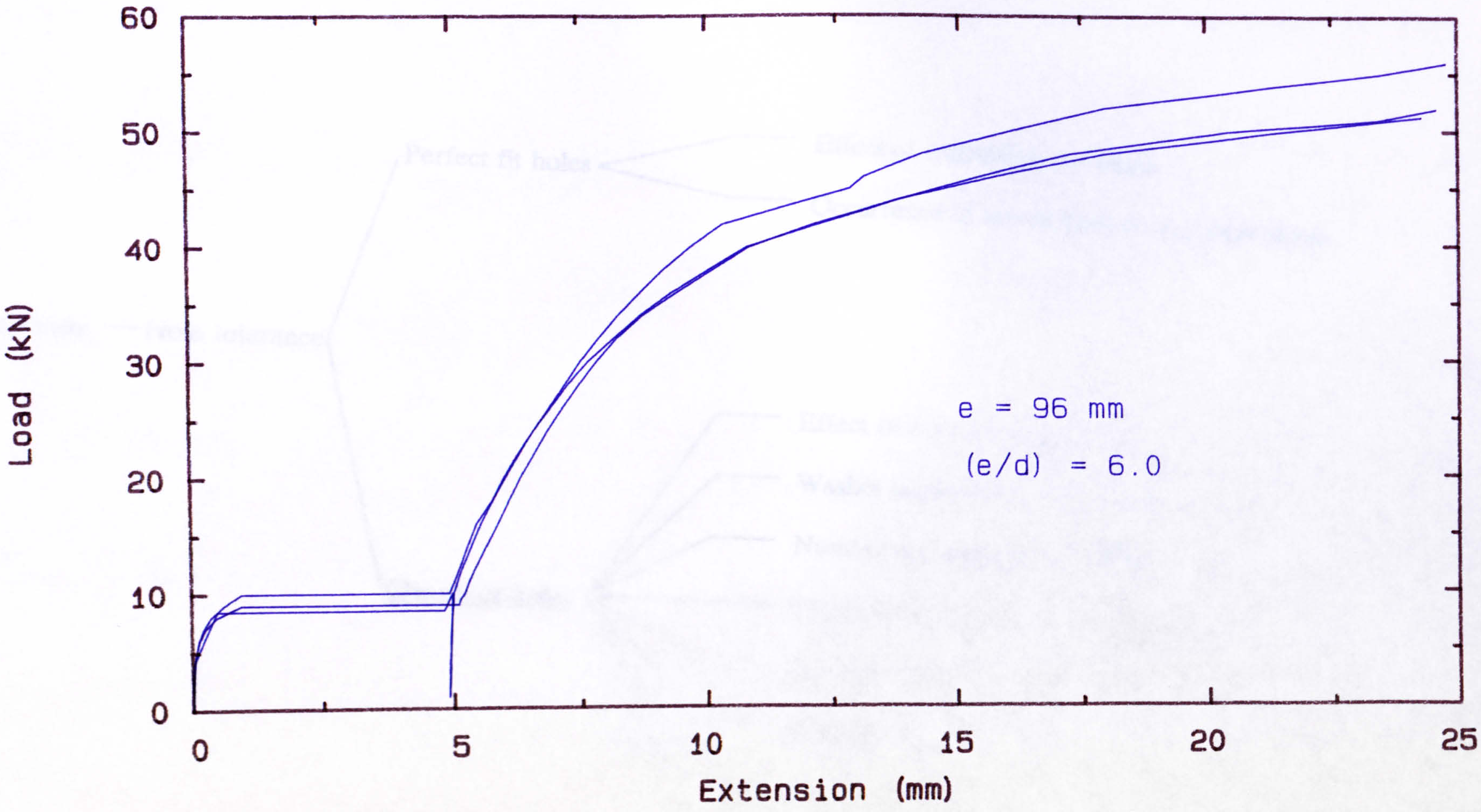


Fig. A2.24g End distance of bolt in line of stress :
t = 3.02 mm



- Lap tests
  - Hole tolerance
    - Perfect fit holes
      - Effect of restraining the edges
      - Occurrence of bolt threads in the shear plane
    - <u>Clearance holes</u>
      - Effect of restraining the edges
      - Washer material and diameter
      - Number and position of washers
      - Mechanical properties of steel sheets
      - End distance of bolt in line of stress
      - <u>Bolt diameter</u>
      - Thin/thick sheets
      - Range of sheet thickness covered by cold formed steel codes

Fig. A2.25 Bolt diameter : t = 1.63 mm

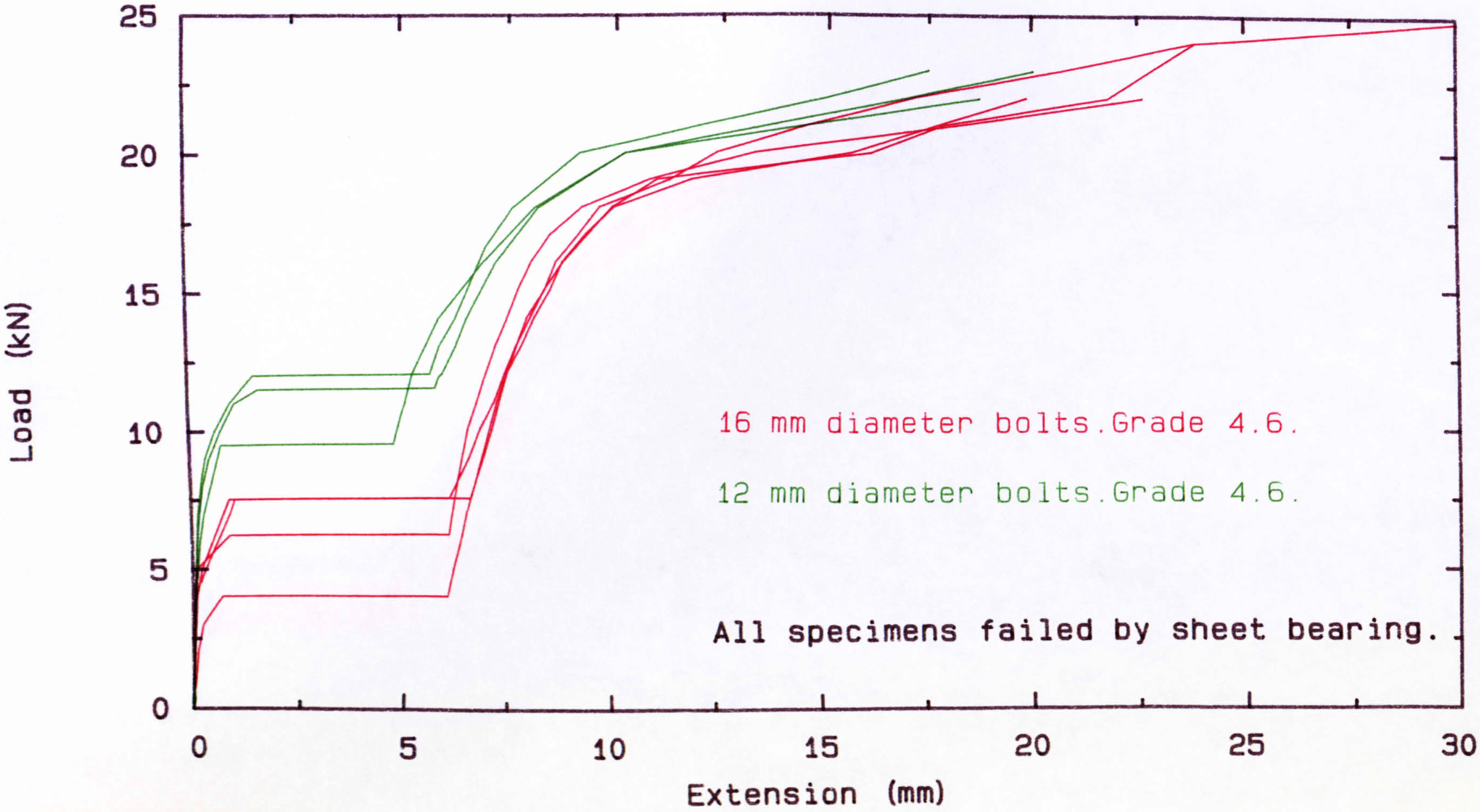


Fig. A2.26 Bolt diameter : t = 2.45 mm

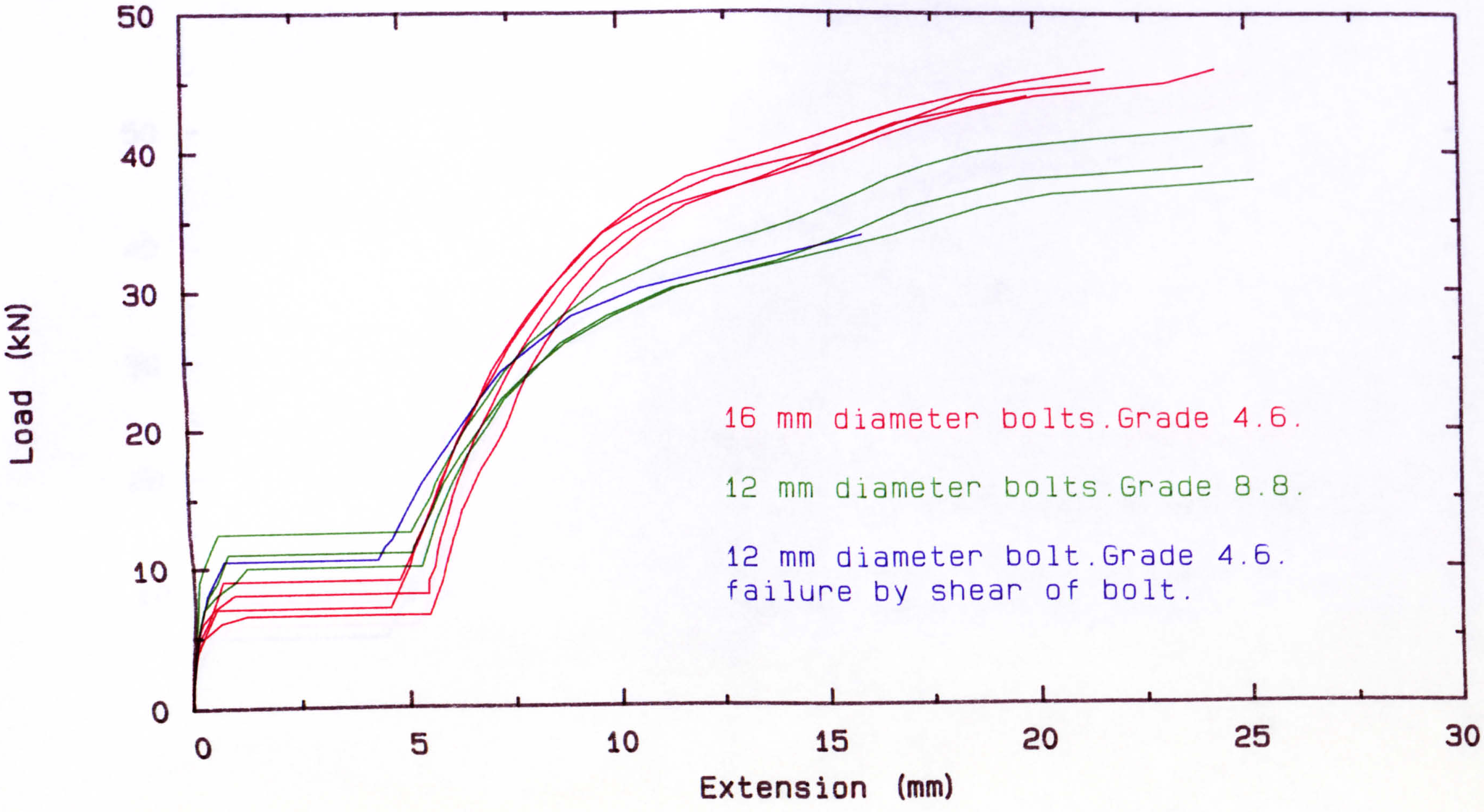
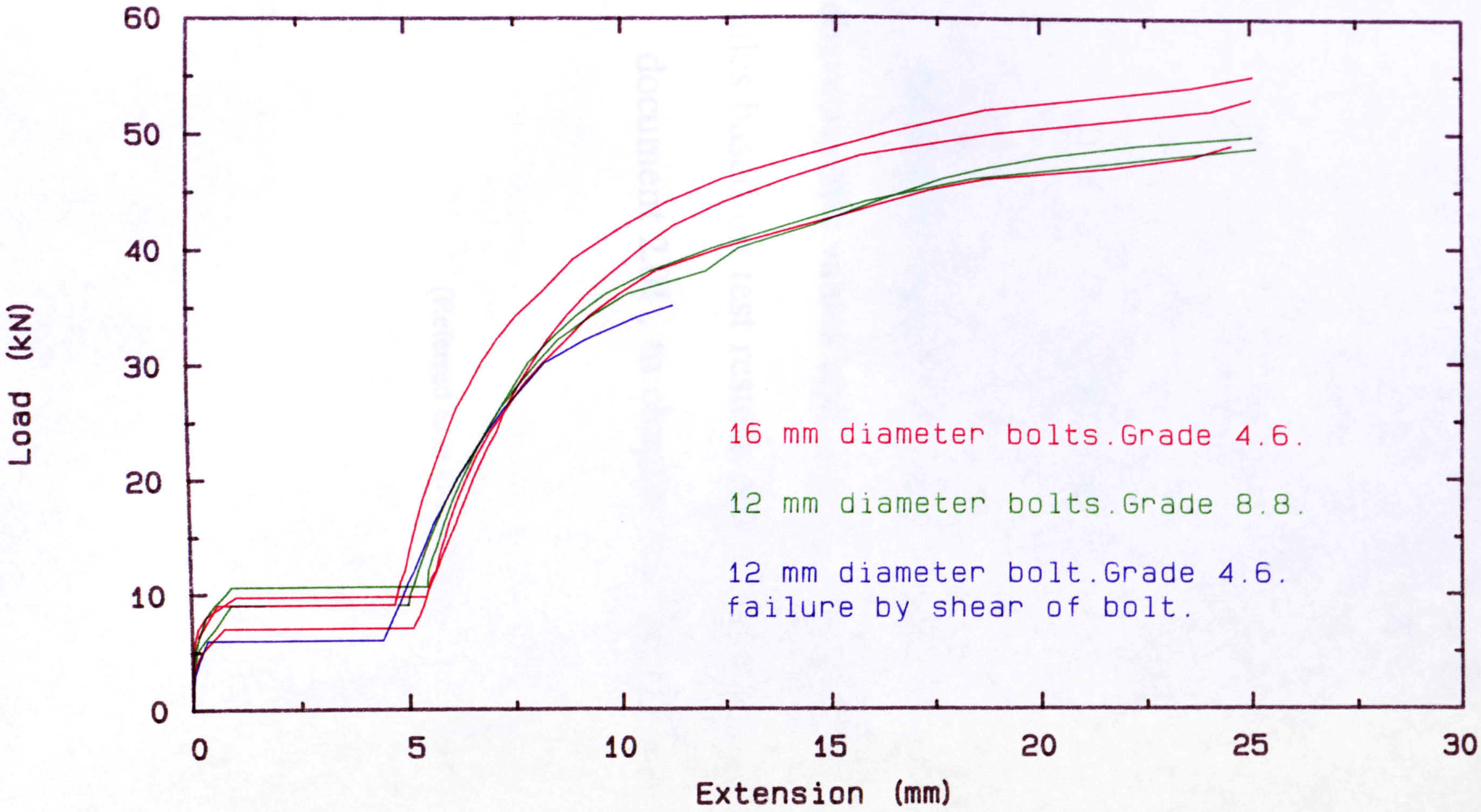


Fig. A2.27 Bolt diameter : t = 3.12 mm



Appendix 3

Reliability analysis in order to determine the characteristic values and factors of safety for design rules based on test results. Specified in background document 2.01 , to chapter 2 Eurocode No. 3.

(Referred to in Chapter Six)

The resistance of structures or part of structures are defined by design models often derived from, or justified, by tests. The procedure for determining partial factors of safety γ_M, for such design models as given in background document 2.01[51], referred to in Chapter Six, is described in this appendix.

Note that the notation used here is exclusive to this appendix, and should not be confused with that found elsewhere in this thesis.

For a given article, eg. bearing strength of a bolted connection, a partial factor of safety is defined as the safety margin between the characteristic strength and design strength of that connection. i.e.

$$\gamma_M = \frac{r_k}{r_d}$$

r_k = characteristic strength
r_d = design strength.

A3.1. Basic defenitions

Most design models are expressed in a multiplicative form, i.e.

$$g_R(\underline{x}) = x_1 \cdot x_2 \cdot x_3 \dots x_j$$

where x_i are the basic variables, with lognormal distributions.

In comparing the results of the strength function $g_R(\underline{x})$, with those obtained from tests, its value is corrected by additional factors $\bar{b}$ and δ.

i.e.

$$\begin{aligned} R_k &= \bar{b} \cdot g_R(\underline{x}) \cdot \delta \\ &= \bar{b} \cdot x_1 \cdot x_2 \cdot x_3 \dots x_j \cdot \delta \end{aligned}$$

$\bar{b}$ is the mean of correction terms, i.e. the ratio of theoretical design value, as given by $g_R(\underline{x})$; and δ is error term due to the scatter of the test results. These will be defined in more detail later.

The factored strength function R, basic variables x_i and δ are considered as stochastic‡ values. Hence when the mean and standard deviations of x_i and δ are known; then the mean value r_m, and the standard deviation σ_R, of the corrected strength function R can be obtained. Therefore the design value r_d, the characteristics value r_k and the partial factor of safety γ_M can be derived.

‡ A stochastic value is one which may be described by a mean and a variance.

The equation defining R_k can be written in logarithmic scale as:

$$\ln R = \ln \bar{b} + \ln x_1 + ... + \ln \delta$$

for simplicity say $$R' = \bar{b}' + x_1' + x_2' + ... + \delta'$$

i.e. $R' \equiv \ln R$, $\bar{b}' \equiv \ln \bar{b}$ and so on.

The standard deviation of R may be determined as:

$$\sigma_{R'}^2 = \sigma_{x'_1}^2 + \sigma_{x'_2}^2 ... + \sigma_{\delta'}^2$$

A3.2. mean correction value $\bar{b}$, and standard deviation of the error terms S_δ

For any given design model ;

$$g_R(\underline{x}) = x_1 \cdot x_2 ...... x_j$$

For any one given set of parameters i, let the theoretical value, i.e. that given by the design expression be :

$$r_{ti} = g_R (x_1 \cdot x_2 ... x_j)$$

and let the experimental value for a specimen with the same set of parameters i, be

$$r_{ei}$$

then the correction term for each test specimen with parameters i, is :

$$b_i = \frac{r_{ei}}{r_{ti}}$$

the mean value of correction terms for n specimens, is;

$$\bar{b} = \frac{1}{n} \sum b_i$$

the error term for each specimen is given as;

$$\delta_i = \frac{b_i}{\bar{b}} \quad \left(= \frac{r_{ei}}{\bar{b} r_{ti}} \right)$$

the mean value of the error terms is;

$$\bar{\delta} = \frac{1}{n_i} \sum_{i=1}^{n} \delta_i \quad (= 1)$$

Standard deviation of error terms is then;

$$s_\delta = \sqrt{\frac{1}{n-1}\sum_{i=1}^{n}(\delta_i - \bar{\delta})^2}$$

A3.3. Standard deviation of R, σ_R'

Standard deviation of predicted values by the design model $g_R(\underline{x})$ is given as :

$$\sigma^2_{R'_t} = \sum\left(\frac{\partial R'}{\partial x'_i}\cdot\sigma_{x'_i}\right)^2$$

where $\sigma_{x'_i}{}^2$ is the standard deviation of each of basic variables x_i, as expected in practice.

The following coefficients of variation for the variables under consideration in document A.01[50], were considered to be typical of that in practice:

bolt diameter	v_d	= 0.005
specimen width	v_w	= 0.005
sheet thickness	v_t	= 0.05
end distance in line of stress	v_e	= 0.005
edge distance perpendicular to line of stress	v_{e2}	= 0.005
sheets ultimate stress	$v_{\sigma_{ult}}$	= 0.07
sheets yield stress	v_{σ_y}	= 0.07

the standard deviation for the error term is:

$$S_{\delta'}{}^2$$

the standard deviation for the total strength is therefore given as:

$$\sigma_{R'}{}^2 = \sigma_{Rt'}{}^2 + S_{\delta'}{}^2$$

with the weighing factors

$$\alpha_{Rt} = \frac{\sigma_{Rt'}}{\sigma_{R'}}$$

$$\alpha_\delta = \frac{S_{\delta'}}{\sigma_{R'}}$$

A3.4. Characteristic, design and partial factor of safety

The characteristic and design values, r_k and r_d can now be determined as below:

$$r_k = \bar{b} \cdot g(R)\,(m_i)\; e^{\left(-1.64\,\alpha_{Rt} \cdot \sigma_{Rt'} - k_s \alpha_\delta \sigma_{\delta'} - 0.5\,\sigma^2_{R'}\right)}$$

$$r_d = \bar{b} \cdot g(R)\,(m_i)\; e^{\left(-3.04\,\alpha_{Rt} \cdot \sigma_{Rt'} - k_d \alpha_\delta \sigma_{\delta'} - 0.5\,\sigma^2_{R'}\right)}$$

values for k_s and k_d in the equations above are tabulated according to the relevant number of test results, in document 2.01.

The partial factor of safety is then given as:

$$\gamma_M = \frac{r_k}{r_d}$$

the factors related to nominal values r_{tk} of the strength, that are calculated with the nominal values x_{nom} of the basic variables x_i :

$$r_{tk} = g_R(x_{nom})$$

Ratio :

$$\Delta_k = \frac{r_{tk}}{r_k}$$

Factor of safety related to r_{tk}

$$\gamma^*_M = \frac{r_{tk}}{r_d} = \Delta_K\, \gamma_M$$

which indicates the partial safety factor to be applied to the nominal value of the resistance $g_R(x_{nom})$.