Testing and Design of Bolted Connections in Cold Formed Steel Sections

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TESTING AND DESIGN OF BOLTED CONNECTIONS IN COLD FORMED STEEL SECTIONS

By F. Zadanfarrokh[1] and E .R. Bryan[2]

Summary

A detailed analysis of the strength and rigidity of bolted connections in cold formed steel has been carried out. Design expressions are proposed that improve on the existing equations for bearing strength, and for the first time, quantify the rigidity of such connections. The effects on the design of bolted joints are described and it is shown how, by incorporating the proposed expressions, it is now possible to select the correct semi-rigid joint and achieve economy without having to resort to testing.

1 Introduction

New codes of practice for the design of cold formed steel sections allow main frame members, as well as the traditional secondary members, to be designed in cold formed steel. A new impetus has therefore been given to the use of cold formed sections in structures. This has brought a need for modified and reliable design information that improves the economy and versatility of cold formed steel, and maintains its competitive edge over other forms of construction.

Connections are an important aspect of such structures since structural behaviour, and hence economy, is dictated to a large extent by the behaviour of the connections.

There are numerous types of fastenings between cold formed steel components. In cold formed sections however, where holes are punched during forming, bolts are by far the most common type of fastener used in practice. Their ease of application and economy has a particular attraction in all types of steel structures.

An extensive research programme was therefore set up at the University of Salford to study the behaviour of bolted joints in cold formed steel sections, in order to provide adequate information on the real behaviour of bolted joints and enable cold formed steel structures and assemblies to be designed with full economy.

In this paper the findings of this work are described. First the modes of failure of bolted joints are briefly reviewed and standardized parameters for testing of structural bolted connections are proposed, and then the factors influencing the strength and rigidity of such connections are described in detail.

As a result of the tests, a design expression for the bearing strength has been derived with simple coefficients to take account of all relevant factors influencing the strength of bolted connections in cold formed steel sections. The results of the design expression for bearing strength are compared with those presently used in the U.S., European and British codes of practice. Where appropriate, comparisons are made with values obtained by eminent authors in the past.

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Furthermore, as far as is known for the first time, a design expression defining the flexibility of bolted connections is put forward. This is of particular importance to bolted moment connections, since it enables the designer to predict the moment-rotation characteristics of such joints without having to resort to testing. It should then be possible to design optimum structures by specifying joints with appropriate characteristics.

2 Basis of design criteria for bolted connections

Tests initiated by the late George Winter[8] in the 1940's, led to the recognition of four distinct types of failure, which have up to now formed the basis for the provision of design equations in various codes of practice.

The observed modes of failure and their mechanical models are as follows.

i Sheet tearing (Type I) 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)

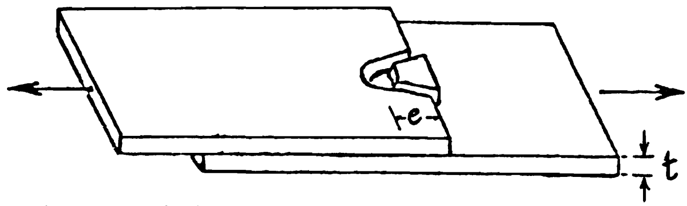

Fig. 1 : Sheet tearing mode of failure.

From first principles the ultimate load is defined as :

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

Substituting 0.6 σ_{ult} in place of τ_{ult} :

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

ii Sheet bearing (Type II) 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. 2)

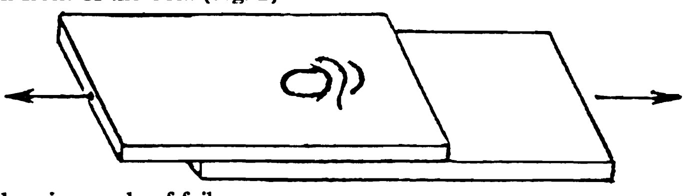

Fig. 2 : Sheet bearing mode of failure.

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.

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

The factor α ($= P_b / dt\sigma_{ult}$ or $= \sigma_b / \sigma_{ult}$) depends on several parameters which will be described later.

The trend is now to combine 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 since, for smaller end distances, the mode of failure will be too sudden.

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:

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

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

The general bolt bearing mode of failure is illustrated in Fig. 3.

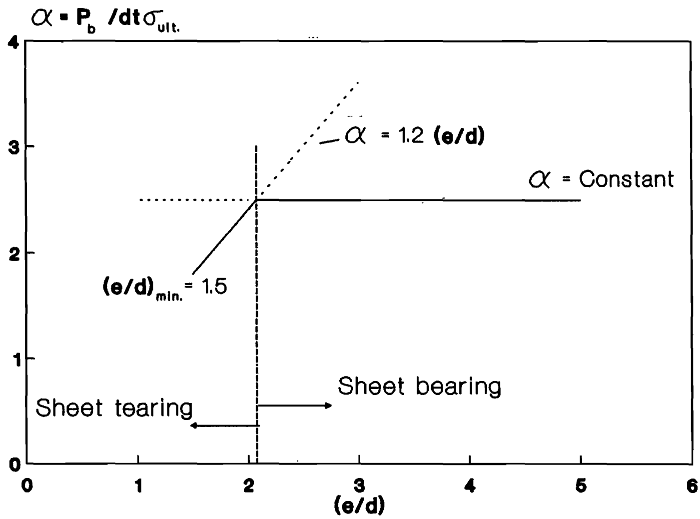

Fig. 3 : Bolt bearing mode of failure

Bolt bearing may be a more logical and succinct approach in defining the bearing modes of failure (Types I & II). It is therefore recommended that design codes should model their design rules on this mode of failure, rather than treating it as two separate cases of sheet tearing and sheet bearing.

The other modes of failure noted by Winter were:

iii Tensile failure in net section (Type III) mode of failure

Where the strength of the fastening is more than the ultimate strength of the net section, it results in tearing failure of the sheet in net section. (Fig. 4.)

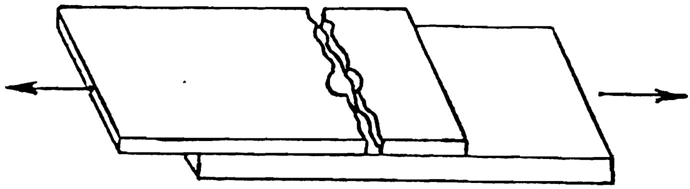

Fig. 4 : Net section mode of failure.

$$P_n = A_n \cdot \sigma_n$$

Consideration of the net section mode of failure is the same as in conventional hot rolled construction.

iv Bolt shearing (Type IV) mode of failure

This mode of failure occurs when the bearing strength of connected parts is greater than the shear strength of the bolt. This is a sudden mode of failure and not common in cold formed steel. (Fig. 5)

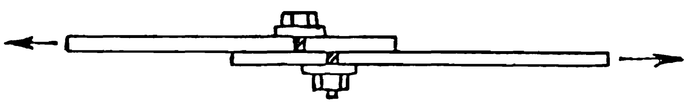

Fig. 5 : Bolt shearing mode of failure.

$$P_s = A_s \cdot \sigma_s$$

The bolt shearing mode of failure, although essentially the same as in hot rolled sections, hardly ever occurs even with mild steel (grade 4.6) bolts in cold formed steel sections up to 3.2 mm thick. This is due to the bolt tilting. This phenomenon reduces the effective shear stress on a bolt by putting it in a state of shear and tension.

Connections are usually arranged so that the bolts are in shear, and because of the thinness of the material, designs tend to be dominated by the bearing capacity of the thinner sheet rather than the ultimate strength of net section or shear strength of the bolt. Bolt bearing (Fig. 3) almost always tends to dominate the design and will therefore be the focal point of in this paper.

3 Testing of structural bolted connections in cold formed steel

As mentioned previously, the objective of the research was to consider practical factors in bolted connections in cold formed steel sections, not previously or adequately accounted for. It was therefore thought essential to devise a standard test procedure to yield the maximum amount of information with a minimum number of tests.

Specifications drawn by the current codes of practice in the testing of bolted lap joints in cold formed steel are briefly described, their short comings are discussed, and further recommendations are made.

3.1 Current directives on testing of fastenings in cold formed steel

AISI[3] does not give specific rules for the testing of connections in cold formed steel, nor does BS 5950 Part 5[4]. Instead the latter code refers the reader to European Recommendations where definite guidelines have been drawn up by ECCS (European Convention for Constructional Steelwork) working group TC7, for testing of bolted connections in cold formed steel.

The procedures drawn by ECCS in the testing of connections are essentially in two categories :- (1) European Recommendation (E.R.) Publication No. 35, reference [7], which covers tests to determine the strength and rigidity of all common types of fasteners in light gauge steel (2) E.R. Publication No. 21, reference [6], comprising of test procedures to determine the properties of the fastenings.

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

3.2 Standard shear test of fastenings in light gauge steel, E.R. Publication No.21[6]

The standard shear test is commonly known as the "single lap joint shear test", in which a two fastener lap joint of standard dimensions is placed in the testing machine and loaded to failure. The minimum standard dimensions (for $d > 6.5$ mm) are shown in Fig. 6.

As can be seen, the test incorporates two fasteners and the results obtained are an average of the two fasteners tested. The E.R.[6] also 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 specimen length may be adjusted accordingly to provide the end distance required for the tests.

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

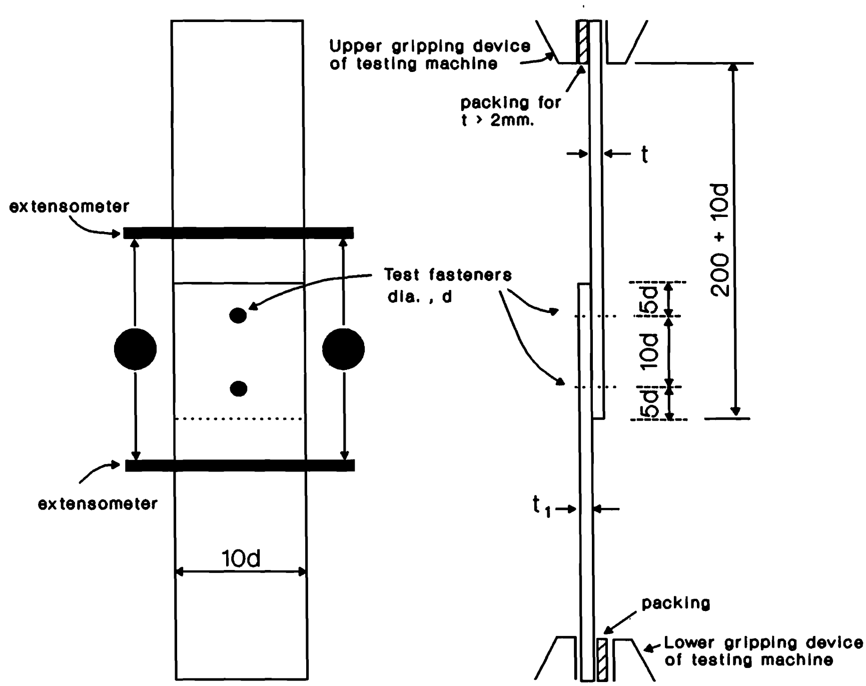

Fig. 6 : Standard shear test specimen specified by the E.R.[6].

A deformation limit of 3mm is also placed on the ultimate load. That is, the ultimate load is defined as the lesser of the load recorded within 3mm deformation or the load at which the first drop in the load-extension curve occurs.

3.3 Application of the "Standard Shear Test" to testing of structural bolted connections

The dimensions specified by the European Recommendations summarized above, seem to be excessive. Reference [17] gives the background to Fig. 6, where it is evident that the specifications in the E.R. 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, with fastener diameters in the range of 6 mm.

It was obviously desirable both in terms of saving material and reducing test time, to test specimens with one bolt only, using smaller specimens provided they were representative of structural bolted connections (i.e. d ≥ 10mm) in cold formed steel sections.

To this purpose a series of preliminary tests was carried out on twenty lap joints, with one and two bolts in the line of stress. Steel sheets at both the "thinner" and "thicker" range of cold formed steel sections were tested, namely 1.7 and 3.0 mm.

Galvanised bolts of 16 mm diameter (grade 4.6) were used in the tests since these are the most common type utilised in practice in the U.K. Grade 8.8 and 10.9 bolts are however the more common practice in Continental Europe and North America. All fasteners (bolts, nuts and washers) are also almost always galvanised. From a structural point of view, galvanising affects the coefficient of friction between the bolt and the parent material. Tests carried out by Winter[9], and later Baehre[15], also show that galvanising yields the lowest coefficient of friction compared to other forms of coating or bare steel.

All bolts were "turned and fitted" into a 16 mm diameter hole, to obtain a perfect fit fastening, and hence eliminate the effect of clearance slip at this stage. Different bolt torque settings, ranging from 60 to 100 Nm, were applied and monitored to ensure that bolt torque did not affect the ultimate load.

In lap joints with two bolts in the line of stress, different bolt spacings of 1e, 1½e and 2e were considered, and their effect on the load carrying capacity of the joint was studied. (e = end distance in the line of stress)

Specimens of the dimensions specified by the E.R.[6], for both two bolts and a single bolt, were tested and the results compared with those of tests on the smaller specimens.

3.3.1 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. Load/extension curves were also recorded by the machine to ensure an accurate register of data up to the failure load. A typical test is shown in Fig. 7.

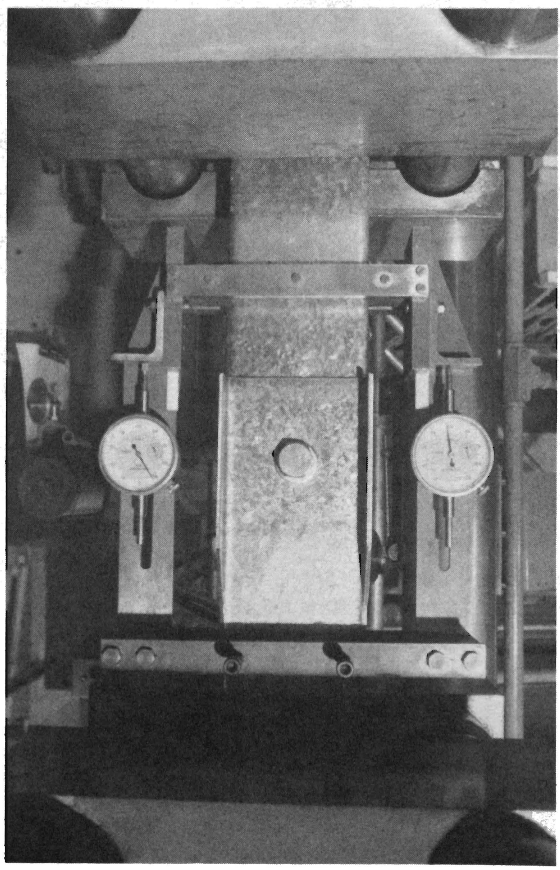

Fig. 7 : A fully assembled specimen, undergoing test.

3.3.2 Test results

Detailed test results are given in Reference [1]: only the main findings are discussed here.

The majority of specimens failed in sheet bearing, with a few in net section.

It was found that in specimens consisting of flat elements, sheets curled out of plane. Increasing the specimen width had an adverse affect on sheet curling. In the case of thinner specimens and those with two bolts in the line of stress, irrespective of the sheet thickness, it was found that sheets curling out of plane badly distorted the results. It was therefore thought necessary to restrain sheet curling by forming lips around the connection area. This device was found to be very effective and resulted in fasteners tilting instead. As a result the deformation capacity of the lap joints increased, making them more representative of the in-situ conditions. The same behaviour, i.e. fastener tilting, was observed in the testing of full scale moment connections. (See § 7, Fig. 24)

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.

Typical load/extension characteristics of a specimen with two bolts in the line of stress, are compared with those of an equivalent specimen with a single bolt, in Fig. 8 (a) and (b) for sheet thicknesses of 1.7 and 3.0mm respectively. All specimens depicted failed in sheet bearing.

As can be seen, the results obtained are very satisfactory and almost the same characteristic per bolt is obtained. It is therefore concluded 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.

It is therefore considered that the dimensions specified in the European Recommendations[6] for the standard shear test, are excessive for load bearing structural bolted connections. The alternative dimensions shown in Fig. 9 are recommended to ensure that the desired mode of failure (i.e. sheet bearing) is obtained.

Where it is deemed that specimens with two bolts in the line of stress are more appropriate, the specimen length should be increased by the bolt spacing S, i.e. $l = (400 \pm 20) + S$. A specimen width of $w = 8.75d$ was found to be adequate in such cases to obtain the correct mode of failure.

Test procedures were kept as close as possible to that specified in the E.R.[6]. However as evident from Fig. 8, the deformation limit of 3 mm set in the E.R. is not realistic for the structural bolted connections being tested here, where movements of the order of 20 to 30 mm are obtained before failure. Moreover, in connections with 2 mm clearance holes, a 3 mm deformation will often correspond to the initial slip load.

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

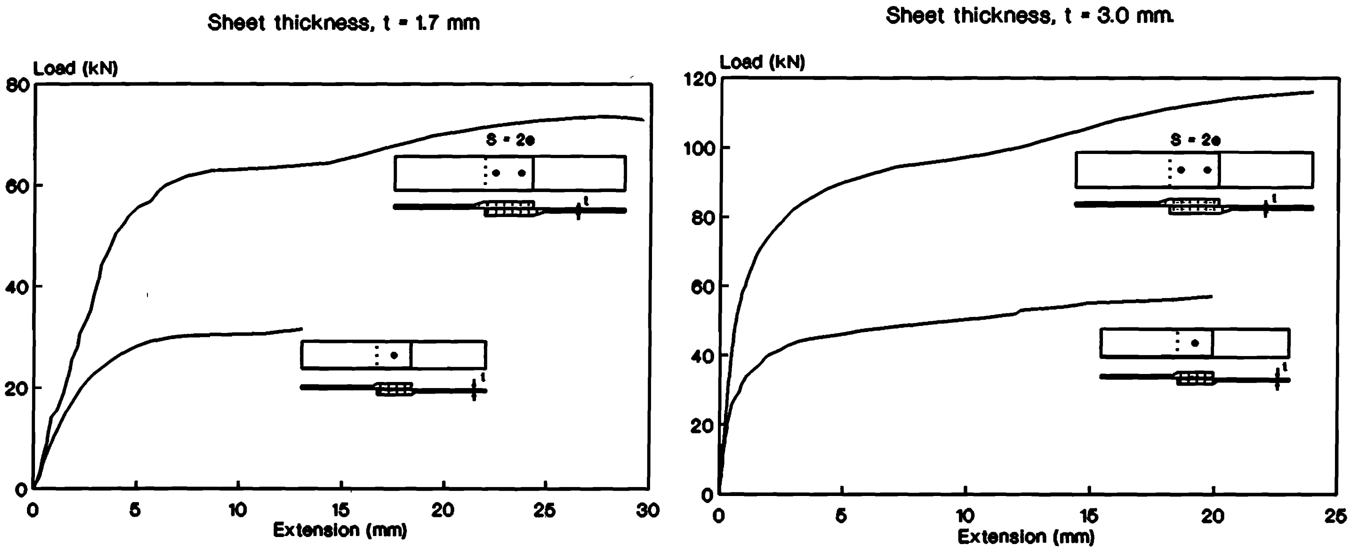

Fig. 8 : Comparison of load-extension characteristics of one and two bolt specimens.

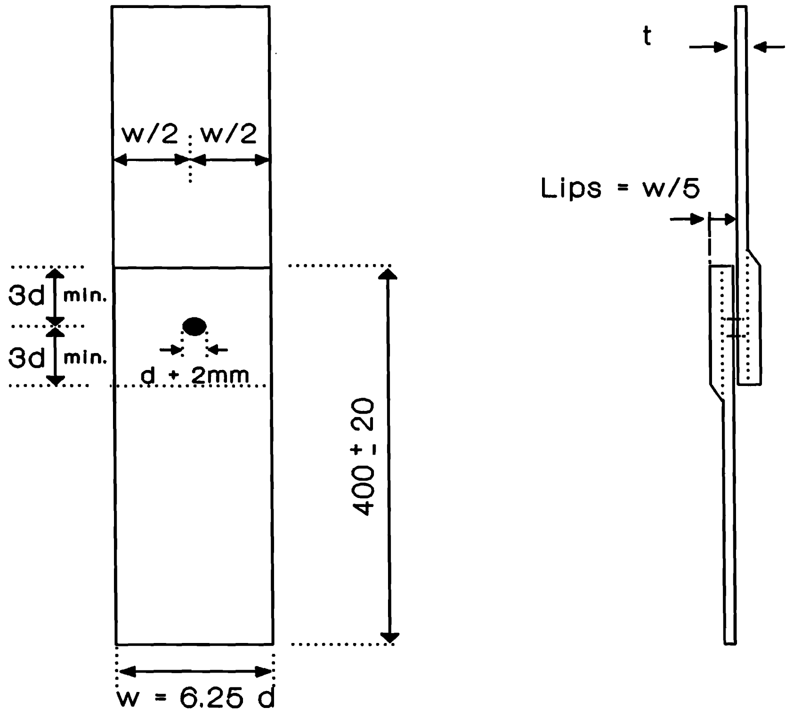

Fig. 9 : Recommended test dimensions for structural bolts in cold formed steel sections. ($d \geq 10$ mm)

3.4 Standard bolt torques

The results obtained confirmed that the ultimate load is independent of the bolt torque. In testing of lap joints however, especially when their flexibility as well as strength is to be investigated, it was thought essential to establish a standard torque setting which corresponded to that used in practice.

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

(i) The induced axial tension and;
(ii) Part of the torque 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 future tests , experiments were carried out to measure the normal torques applied to 16 mm diameter bolts in practice. Special care was taken for the results to be representative of site conditions. Using a torque wrench, typical torque settings of 16 mm diameter bolts in a typical cold formed steel structure were examined. Numerous bolts, with different degrees of accessability were tested at random.

A great scatter of results was obtained. No apparent correlation between the applied torque and bolt location was evident. Based on the results obtained, and taking the relaxation of the bolt forces into account, a standard and conservative torque of 65 Nm is recommended for all 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[8]. For other bolt diameters, values given by Winter are therefore recommended. (Table 1)

Metric ISO Bolts	Torque, N.m
M10	15
M12	50
M16	65
M20	145
M24	335

Table 1 : Standard torque settings.

3.5 Recommendations for "Standard Shear Test" for structural bolted connections

To summarize what has been discussed in § 3, the following procedures for testing of lap joints, resulting in sheet bearing mode of failure (representative of structural bolted connections in cold formed steel sections) are recommended;

- Lap joints with a single bolt may be used to adequately represent bolted connections in cold formed steel. Specimen dimensions should be in accordance with those given in Fig. 9.

- All the steel sections and fasteners (i.e. bolts, nuts and washers) being tested should be galvanised, unless the effect of galvanising is being investigated itself.

- With specimens with more than one bolt in line of stress, a bolt spacing of twice the end distance (S = 2e) is ample to mobilize the strength of the second bolt. Tests indicate however that the required bolt spacing is a function of sheet thickness and may be reduced for the thicker range of steel sections.

- In specimens with more than one bolt perpendicular to the line of stress, the required bolt pitch to develop the full bearing strength of the tension joint may be obtained by ensuring that the tensile capacity of the sheet, over one bolt pitch, is greater than or equal to the bearing capacity.

- The rate of loading should be in accordance with the E.R.[6], as described in § 3.2.

- The ultimate load may be taken as the first drop in the load/extension characteristics.

- Evaluation of test results should be based upon a minimum number of tests specified by either the E.R.[6] or AISI.

- The bolt torque should be in accordance with Table 1.

4 Bearing strength of bolted connections in cold formed steel sections

A total of 228 tests was carried out on bolted lapped joints at Salford. This work coupled with the results of some 700 tests from other British and European research institutions, resulted in a comprehensive study of all factors influencing the strength of bolted connections in cold formed steel sections. The results obtained provide essential information on the real behaviour of bolted joints. A design expression is propounded which enables cold rolled steel structures to be designed with full economy.

The test arrangements and experimental procedures were in accordance with § 3.5, to represent normal structural practice as far as possible.

A minimum of three tests was carried out for any identical set of test parameters. The percentage of variation of ultimate load within any one group tests, 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.

All specimens failed in bolt bearing. Full test details are given in reference [1], only the main findings are summarized here. Some of the specimens tested to failure are shown in Fig. 10.

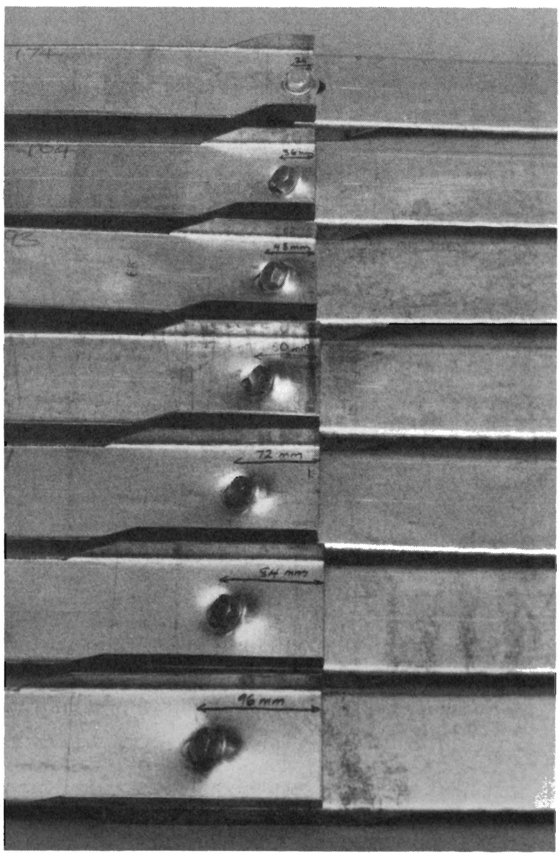

Fig. 10 : Some failed specimens.

4.1 Factors considered in design of bearing strength of bolted connections

4.1.1 Hole tolerance

The majority of tests were carried out on specimens with 2mm clearance holes. Steel sheets were pushed in together prior to the tightening of the bolt to leave the maximum possible initial clearance. Some tests however, were made on specimens with exact diameter holes, i.e. a perfect fit was obtained between the fastener and the fastening.

It was found that, once the joints with clearance holes were pulled into bearing, their behaviour was very similar to that of perfect fit holes thereafter. It is therefore concluded that normal hole tolerance is not detrimental to the joint strength.

4.1.2 Position of the shear plane, on shank or threads of bolt

With bolts, the amount of the plain shank between the bolt head and the last thread is governed by the length of the bolts. British Standard BS4190[27] specifies a thread length of 2d + 6mm for all bolts. Alternative shorter threads of 1½d may be specified, where threads in the shear plane are not desired.

Set screws are differentiated from bolts in that they do not have a plain shank. That is, with set screws the threads run right up to the fasteners head. The most common lengths of 16mm bolts in structural connections are 30 or 35 mm. It follows that strictly speaking they are set screws. Therefore the occurrence of bolt threads in the shear plane of a connection is the norm, in cold formed steel sections.

In calculating the bearing strength of bolted connections however, the nominal bolt diameter is always used. This is taken into account in design expressions, i.e. they are based on the assumption that the shear plane occurs on the threads.

A series of tests was carried out using turned and fitted bolts, with a plain shank in the shear plane, and the same tests were then repeated using turned and fitted set screws with threads occurring in the shear plane.

A study of the failed specimens revealed that in the case of bolts, the sheet steel was squashed in bearing in the area of contact with the bolt, whereas with set screws the threads dug into the bearing area and precipitated tearing. This affected both the strength and flexibility of the tested joints. The effect on flexibility will be considered later.

Since the design expressions are based on the normal case of shear planes on the threads, the ultimate bearing capacity may be increased by $(d/d_{eff.})$ where it can be shown that shear planes occur on the shank.

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

where $d_{eff.}$ is the effective bolt diameter, used in the calculation of A_t.
d is the full shank diameter.
A_t is the tensile area of the bolt.

For most structural bolts, this ratio may be taken as **1.15**.

4.1.3 Bolt diameter

The practical range of bolt sizes, i.e. 10, 12, 16 and 20mm diameter bolts were considered.

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

Taking 16mm diameter bolts as the norm (since they are the most common form of bolts), then the bearing strength of a bolted connection, with bolt diameter d, is

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

4.1.4 Type of bolt

As well as the tests described in § 4, a sample of 94 tests were available from a previous research project at Salford. Sheet thicknesses 1.6, 2.6 and 3.1mm, were tested with 16 and 20 mm diameter ordinary black bolts and High Strength Friction Grip (HSFG) bolts. An analysis of the test results showed that although HSFG bolts produced a higher slip load, they did not improve the ultimate load very significantly. This is due to the low coefficient of friction due to the galvanising of cold formed steel, which often acts as a lubricant.

Results showed that HSFG bolts were most effective when used with the thinnest of sheets, where the ultimate load of the sheets was less than the slip load of the bolt, hence resulting in perfectly rigid joints until failure. In this range (t = 1.6mm) the average increase in ultimate bearing strength as a result of using HSFG bolts, compared to ordinary mild steel bolts, was 51% in the case of 20mm diameter bolts and 38% with 16mm bolts. The corresponding increases in the ultimate strength for 2.6 and 3.1mm sheets were approximately 25% and 20% respectively, almost irrespective of bolt diameter.

Hence, the use of HSFG bolts may not be economical in terms of increase in the ultimate strength, but they may prove advantageous where it is necessary to ensure a perfectly rigid joint at the design loads. These conclusions are in line with those drawn by Winter[9], on tests on high strength, high torqued bolts, in light gauge steel connections.

4.1.5 Sheet thickness

Cold formed steel, as defined by BS5950 Part 5, is primarily concerned with sections up to 8mm thick. AISI considers thicker sections, but in the case of connections gives design guidelines for sheets only up to 4.7mm (3/16in.) thick. Most cold formed sections have thicknesses between 1.2 to 3.2mm. Hot rolled steel sections, on the other hand, usually consist of plate thicknesses of approximately 6mm and over.

It follows that there is an area of overlap between the codes of practice for cold formed and hot rolled steel, 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 main range of sheet thicknesses considered varied from 1.5 to 3.2mm, but further tests were carried out to extend this range from 0.9 to 6.2mm to:

(a) consider the influence of sheet thickness across a wider range of thicknesses covered by cold formed steel codes, and

(b) to verify and achieve a continuity of the ultimate bearing strengths between cold formed and hot rolled steel codes.

It is was found that the bearing strength did not increase linearly with sheet thickness, for the normal range of sections up to 3.0mm thick, but was proportional to (thickness)^2.

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

$$P_{ult} = (1.9 + 0.25\ t)\ t\ d\ \sigma_{ult} \qquad \text{for } t \leq 3.0\text{mm}$$

Above this range, i.e. $3.0 < t \leq 8$mm, the bearing strength equation given in Eurocode No.3[5] Part 1, for design of hot rolled sections, was adequate to predict the ultimate bearing strength, i.e. :

$$P_{ult} = 2.5\ t\ d\ \sigma_{ult} \qquad \text{for } 3.0 < t \leq 8.0\text{mm}$$

However, there is a small discontinuity between the above two equations at t = 3.0mm. This may be removed by using the amended relationships:

$$P_{ult} = (1.9 + 0.2\ t)\ d\ t\ \sigma_{ult} \qquad \text{for } t \leq 3.0\text{mm}$$

$$\text{or} \quad P_{ult} = 2.5\ dt\sigma_{ult} \qquad \text{for } 3.0 < t \leq 8.0\text{mm}$$

The above equations are compared against the test results in Fig. 11. Each point plotted in this figure, particularly for t ≤ 3.0mm, is the result of a lower bound analysis on a great many number of tests.

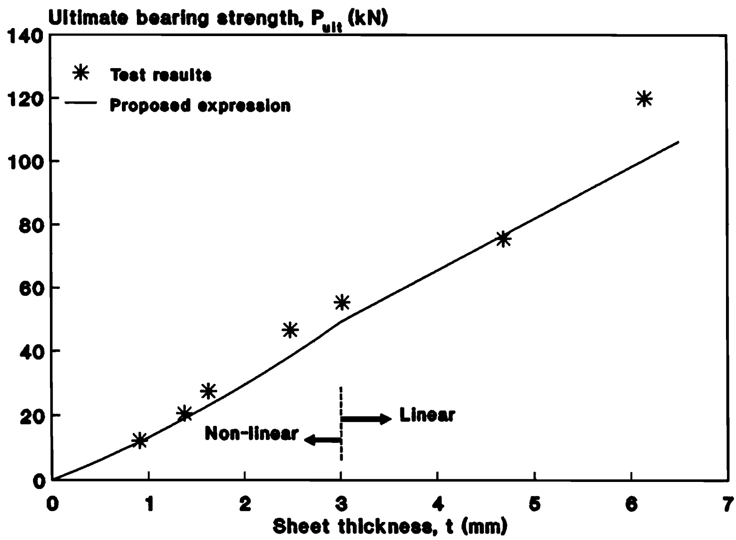

Fig. 11 : Ultimate bearing strength v. Sheet thickness

4.1.6 Mechanical properties of steel sheets

Tests were carried out on different grades of steel, to examine whether an increase in the yield (or ultimate) strength of the connected sheets would lead to an equal increase in the bearing strength of a bolted joint, as is presently assumed in all codes of practice.

At present in the U.K., Z28 (Zinc galvanised, 280 N/mm^2 yield steel) is the most common type of steel sheet used. There is however a growing tendency to use harder steel, Z35 (350 N/mm^2 yield).

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} = 450 N/mm^2) and the results were compared with those of the standard tests, i.e. σ_y = 280 N/mm^2. (nominal ultimate stress, σ_{ult} = 390 N/mm^2 = 1.4 σ_y)

It was found that the bearing strength of bolted joints did not increase linearly with the yield or ultimate strength of the connected sheets - but was proportional to $\sigma_y\,(280/\sigma_y)^{1/2}$ or $\sigma_{ult}\,(390/\sigma_{ult})^{1/2}$. A linear assumption, as used in the present codes, is therefore unsafe.

Moreover the tests showed that the yield stress, rather than the ultimate stress, was a more reliable factor upon which to base the ultimate bearing stress. However it is a requirement of Eurocodes that equations defining rupture, should be expressed in terms of σ_{ult}.

Therefore in terms of design specifications, a design strength factor of $\left(\dfrac{390}{\sigma_{ult.\,design}}\right)^{\frac{1}{2}}$ is recommended,

where $\sigma_{ult.\,design}$ is the design ultimate stress of the sheet material.

4.1.7 End distance of bolt in line of stress

A series of tests was carried out to study the influence of end distance on the mode of failure, and the ultimate bearing strength of bolted connections in cold formed steel.

The test programme adopted was to start with the shortest end distance of 24mm (e/d = 1.5) and increase it 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. 10.

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

The test results are plotted in Fig. 12. In this figure the non-dimensionalized parameters α ($=P_{ult}/dt\sigma_y$) and e/d are plotted for all tested sheet thicknesses. Each point plotted is the average of at least three identical tests.

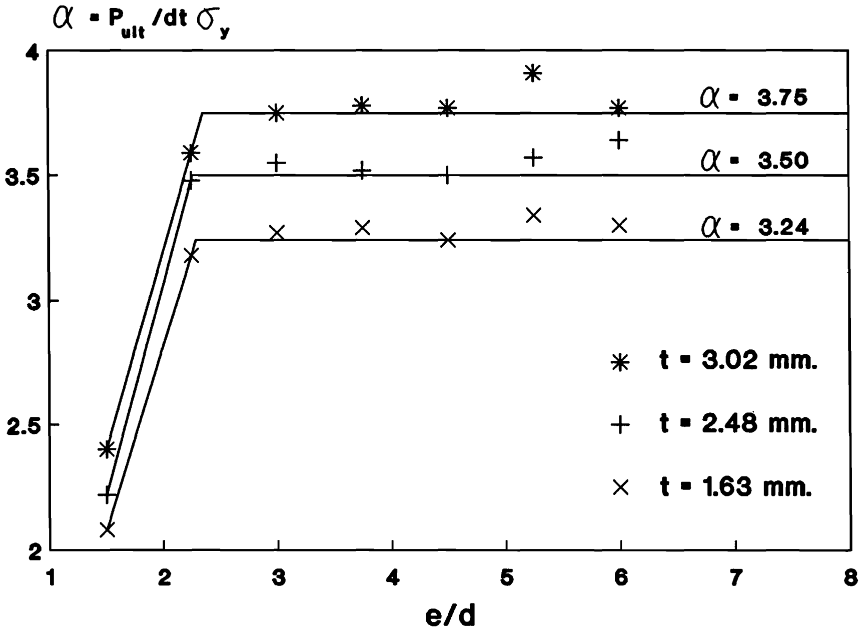

Fig. 12 : Nondimensional factors $\alpha^{\ddagger}$ and (e/d)

It is seen that the ultimate strength P_{ult} increases linearly 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[11].

From Fig. 12, it is evident that ;

for $1.5 \leq e/d < 2.5$ $\quad \alpha \equiv f\,(e/d, t)$ and;

for $2.5 \leq e/d$ $\quad \alpha \equiv f\,(t)$ i.e. α is independent of e/d.

It has been shown[1] that the above criteria are satisfied by incorporating a factor for end distance of bolt in the line of stress as the lesser of e/2.5d or 1.

This is a simple and more accurate solution to what used to be a set of complicated equations in previous drafts of Eurocode 3 (EC3) Annex A[5] (see E.R.[6]). The same comment applies equally to the design expressions in BS 5950 Part 5. In AISI, the permissible end distance in the line of stress, is governed by consideration of the sheet tearing criterion, times a factor of safety of 2. This is coupled with a high factor of safety of 2.22 for bearing, in the sheet bearing criterion, that gives safe bearing capacities for the smallest permissible end distance.

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‡ Note that in Fig. 12, α has been plotted in terms of σ_y ; hence the values shown should be divided by 1.4.

4.1.8 Washer material and diameter

Washers are often punched out of the scrap end of steel strips and usually manufactured by the cheapest available means. Washers, for any one given bolt size, may be specified as normal or large diameter. For 16mm bolts, the outside diameter of these being 30 and 34mm respectively, each with a nominal thickness of 3 mm.

Normal diameter soft commercial washers were used for the bulk of the tests. In addition, tests were made on large diameter soft commercial washers and large diameter bright steel washers - i.e. high quality engineering washers with machined edges and chamfered round the outside edge, known as Turned and Chamfered washers.

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.

Fig. 13 shows a sample of each washer before and after testing. Note the distortion caused by bolt tilting.

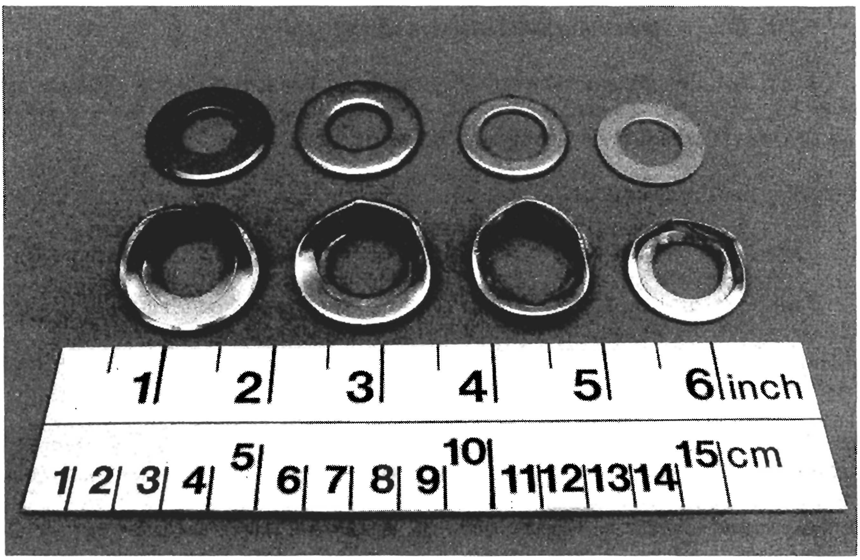

Fig. 13 : Washer distortion due to bolt tilting.

The type of washers shown in Fig. 13, from left to right, are as follows;

(i) large diameter, bright steel washers,
(ii) large diameter, soft commercial washer,
(iii) normal diameter, soft commercial washers, used as standard in all tests and
(iv) normal diameter, soft commercial washers, obtained from a different manufacturer, and of slightly inferior quality compared to (iii) above.

The test results showed that the improvement in the bearing 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 variation is because with thinner sheets, the extra washer diameter provides an added restraint against bolt tilting, so giving 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 giving a reduction in the extra load carrying capacity.

In terms of design specifications, the effect of using larger washers 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.

The effect of washer quality on the ultimate strength was found to be of secondary importance. It is therefore recommended that no modification to the design strength on this account, is necessary.

4.1.9 Number and position of washers

For the main range of sheet thicknesses tested, i.e. $1.5 \leq t \leq 3.2$mm, tests were also carried out on specimens with all possible bolt - washer combinations, 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.

The results were extremely consistent, and showed that 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 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.

It was also found that the slip load, which varied between 3 to 10 kN, was independent of sheet thickness and the use of washers. In terms of joint characteristics the only tangible difference, in use of washers, was in the deformation capacities i.e. the deformation capacity in case (1) was greater than in cases (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 of 25% for cases (2), (3) and (4) alike (almost the same as that in the AISI) is made in BS 5950 Part 5. With AISI however, any discrepancies found in the above recommendations are absorbed in a factor of safety for bearing.

4.1.10 Thin/thick sheets

For the standard range of sheet thicknesses (1.5 to 3.2mm), tests were carried out on single cold formed sheets bolted to a reinforced 9mm thick plate.

So far as the strength of bolted joints is concerned, it suffices to say that the tests vindicated the assumption that the bearing strength of a bolted joint is equal to that of the thinnest connected sheet. 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. The significance of these tests on joint flexibility, will be discussed later.

4.2 Design expression for bearing strength of bolted connections

The design expression for bearing strength, P_{bs}, taking into account all the foregoing factors, described in § 4.1, is defined as :

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

Where α is defined as : $\qquad \alpha = k_1.k_2.k_3.k_3.k_4.k_5.k_6.k_7$

k_1 to k_7 are factors as given below :

Bolt diameter; $k_1 = (16/d)^{½}$	**Sheet thickness;** $k_2 = (1.9 + 0.2\ t)$ for $t \le 3$mm $= 2.5$ for $3 < t \le 8$mm
Mechanical properties; $k_3 = (390/\sigma_{ult.\ design})^{½}$ where $\sigma_{ult.\ design}$ is the design ultimate stress of the sheet material.	**Washer diameter;** For Normal diameter washers, (Form E, BS 4320) $k_4 = 1.0$. For Large diameter washers, (Form F, BS 4320) $k_4 = 1.15$ for $t \le 2$mm $= 1.05$ for $2 < t \le 3$mm $= 1.0$ for $t > 3$mm.
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, with a minimum value of (e/d) = 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.	

5 Comparison of the proposed design expression for the bearing strength with other codes of practice

5.1 EC3, Annex A

Equations originally defining the bearing strength of bolted connections in Annex A (December 1988) were complex, cumbersome and unrepresentative of test results. They also resulted in big step functions existing between the bearing strength equations given by cold formed and hot rolled steel design codes.

In later drafts, the original expressions for bearing strength were withdrawn and replaced by :

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

where α is the lesser of e/3d and 1,
γ_{Mb} is factor of safety for the bolt material = 1.25,
and $(e/d)_{min}$ = 1.5.

The above equation is identical to that in EC3 Part 1, for design of connections in hot rolled sections. Obviously the lower thickness limit of 1.25mm is exclusive to Annex A.

This is a distinct improvement on the earlier version. The previously existing illogical step functions, between the two forms of steel, has also been eradicated. Nevertheless, the revised expression still appears to be markedly inferior to the expression given in § 4.2.

5.2 BS 5950 Part 5

The equations governing the ultimate bearing strength of bolted connections in BS 5950 Part 5, are a conservative simplification of the December 1988 draft of Annex A, (originally published in European Recommendations, Publication No. 21[6]).

The parent expressions upon which BS 5950 Part 5 is based, are not representative of the actual behaviour of bolted connections in cold formed steel connections, and have since been replaced in Annex A. Hence the expressions given in BS 5950 Part 5 define conditions in terms of sheet thicknesses and end distances which are no longer valid. These parameters have been quantified by much simpler and more accurate expressions in this paper, which improve the values given in BS 5950 Part 5 by a factor of about 1.4, and achieve a continuity between the two forms of steel.

Apart from end distance and sheet thickness, it was shown in § 4.1.9 that an incorrect allowance for the use of washers has been made. Other factors considered in this paper have not yet been accounted for in BS 5950 Part 5.

5.3 AISI

AISI is based upon permissible stress design philosophy. The range of sheet thicknesses covered by AISI is from 0.6 (0.024in.) to 4.7mm (3/16in.). For bolted connections with two washers the ultimate bearing strength is defined as 3.00 $dtf_u\ /\Omega_b$. The unfactored ultimate stress 3.00 f_u is too high at first sight, but this has conservatively been accounted for by a high factor of safety for bearing, Ω_b =2.22. Where one or no washers are used, the ultimate strength in AISI is reduced by 26%.

The overall perspective for bearing strength in cold formed and hot rolled steel codes is shown in Fig. 14. In this figure α ($=P_{ult}/dt\sigma_{ult}$) is plotted against sheet thickness, t. Unfactored equations have been plotted although no factor of safety has been described explicitly in BS5950 Part 5, where only the ultimate strength is defined.

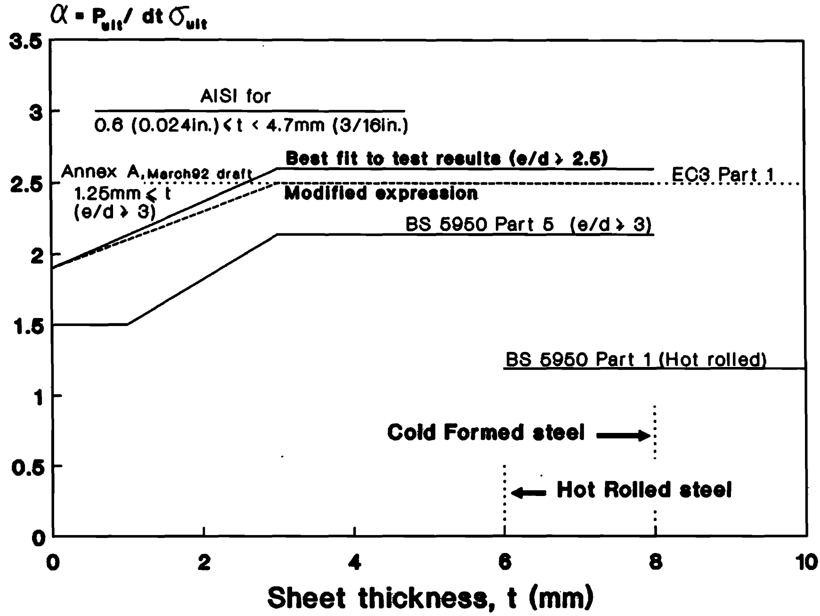

Fig. 14 : Ultimate bearing strength of bolted connections in C.F. and H.R. steel.

It is concluded that the proposed design equation is a significant advance on those given in existing codes of practice. Every appropriate 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 its intended purpose. This is more in line with 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 sheet thickness, except for an upper limit of $t \leq 8.0$mm.

6 Flexibility of bolted connections in cold formed steel sections

So far the focus of all the research carried out on bolted connections in cold formed steel has been on the strength of such joints. However it has long been realised that, as well as strength, connection stiffness is pivotal to a sound design and successful use of steel in any building. 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.

Cold formed steel is no exception to this tradition. At present, many cold-formed sections are designed using 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. 15)

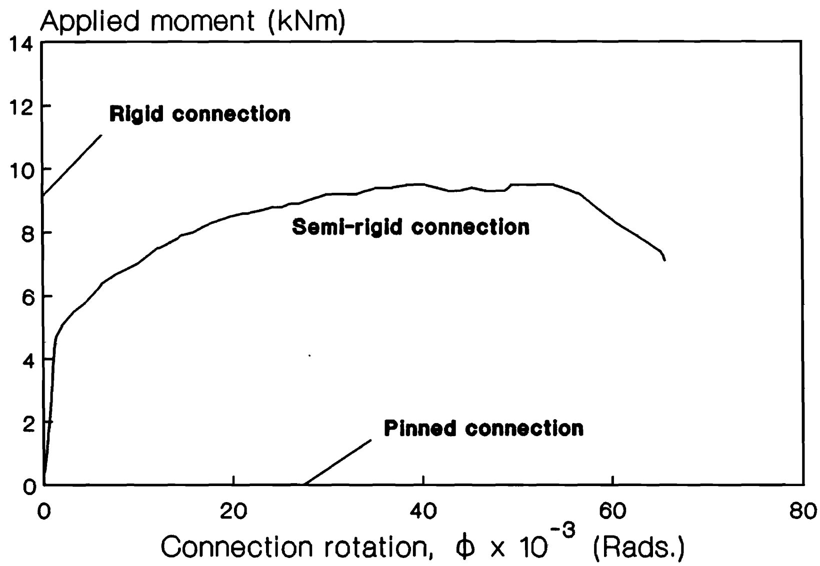

Fig. 15 : Typical moment-rotation characteristics of a bolted connection.

Fig. 16 shows the bending moment distributions for a pinned, rigid and semi-rigid beam. It can be seen that with pinned connections the mid-span moment is critical, whereas with rigid joints the end-moments are critical. If semi-rigid joints are used, these two moments may be more nearly balanced, and the optimum solution is when the mid-span moment and the end-moments are equal. Hence, consideration of the moment/rotation characteristics of bolted connections can be of vital importance to economic design.

Equations defining moment-rotation characteristics can be easily fed into existing 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.

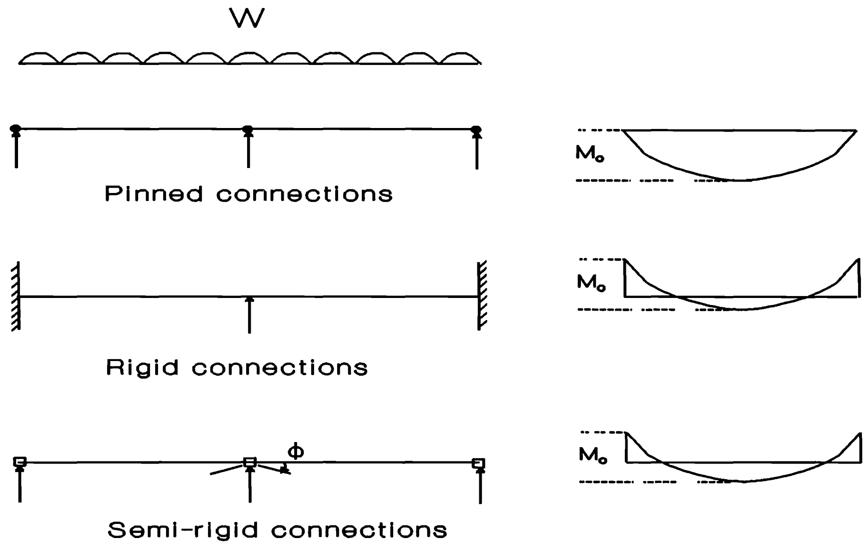

Fig. 16 : Bending moment distribution depending on end conditions.

6.1 Load / extension characteristics of lap joints

In tests on bolted lapped joints the extension of the joints, as well as the applied load, was measured at every stage. 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 the 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 was 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. This is due to two reasons : (1) tolerances 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 the client specifications. So such imperfections are reduced to a minimum. (2) 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, cannot exceed the hole clearance.

The following properties were noted, with regards to the Slip Load :

a) Connection slip cannot be prevented at the working load level in common bolted connections. Only HSFG bolts may prevent this in some circumstances, as described earlier in § 4.1.4.

b) Slip load is independent of the sheet thickness.

c) Slip load is proportional to the bolt torque.

In tests carried out the effect of bolt torque on the slip load was eliminated by applying a standard torque to all lap joints, taking account of the insitu conditions as described previously in § 3.4.

ii) Extension due to deformation, mostly non-linear, of the joined elements.

Deformation extension is usually considerably greater than clearance extension.

6.2 Design models on flexibility of bolted joints

Flexibility of a connection is defined as the deformation sustained per unit load applied.

Therefore flexibility is the inverse of stiffness, i.e. $c = \frac{\delta}{P}$ *(mm/kN)*

In trying to estimate the flexibility of a bolted connection, and hence predict its moment/rotation characteristics, various solutions are possible. Fig. 17 shows two possible design models.

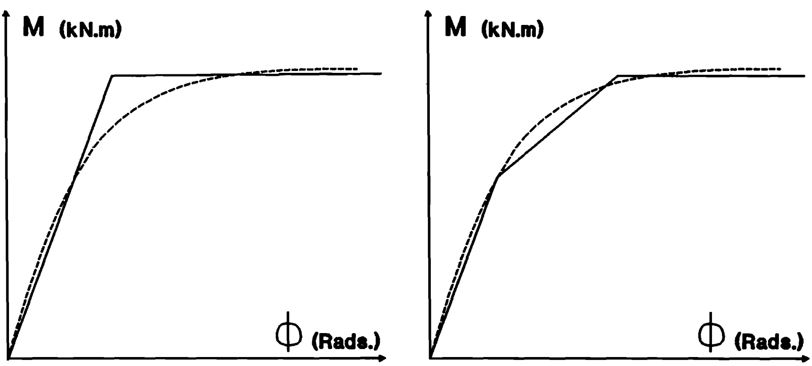

Fig. 17 : Linear and Bi-linear stiffness models.

It is the authors' 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.

Furthermore, 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 underestimate of flexibility will therefore be pursued, coupled with a conservative estimate of the slip load.

6.3 Test results - Joint flexibility

The tests on lap joints, and the variables examined, were described in detail in § 4. Typical load/extension characteristics, for a number of 1.5mm thick sheets, are shown in Fig. 18. The average extension, with the characteristic slip load (i.e. mean - ks), is marked by the dashed curve.

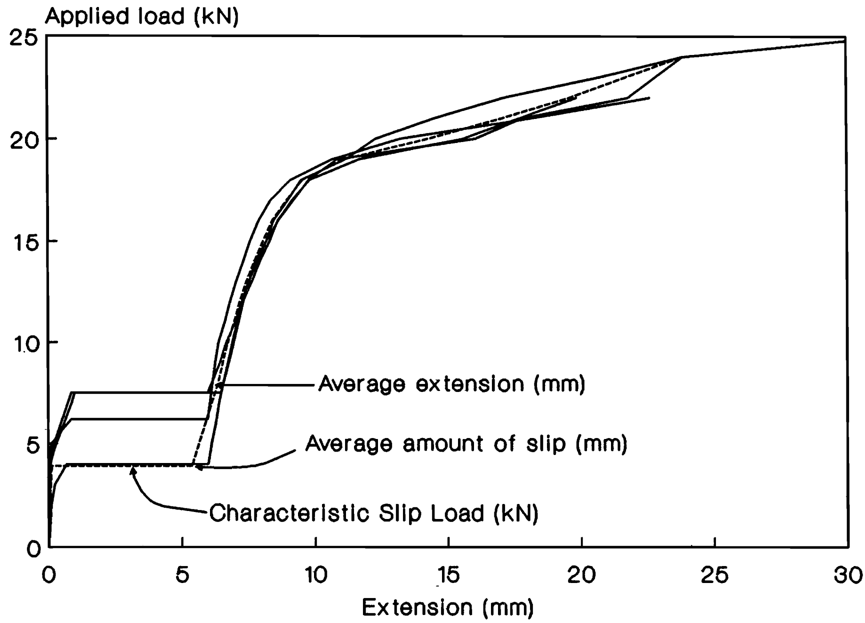

Fig. 18 : Average characteristics plotted against the actual tests.

The same process was repeated for other sheet thicknesses. The results, i.e. average load/extension characteristics for the range of steel sheets considered, is shown in Fig. 19.

From this figure it is seen that the characteristic slip loads are very similar, with the exception of sheets thicker than 3.05mm, where the steel sheets were not galvanised, hence higher slip loads were obtained. It is therefore concluded that a conservative value of 4 kN may be taken as a typical slip load for bolted connections in cold formed steel. This value is independent of sheet thickness, and is recommended for bolt torques representative of site conditions.

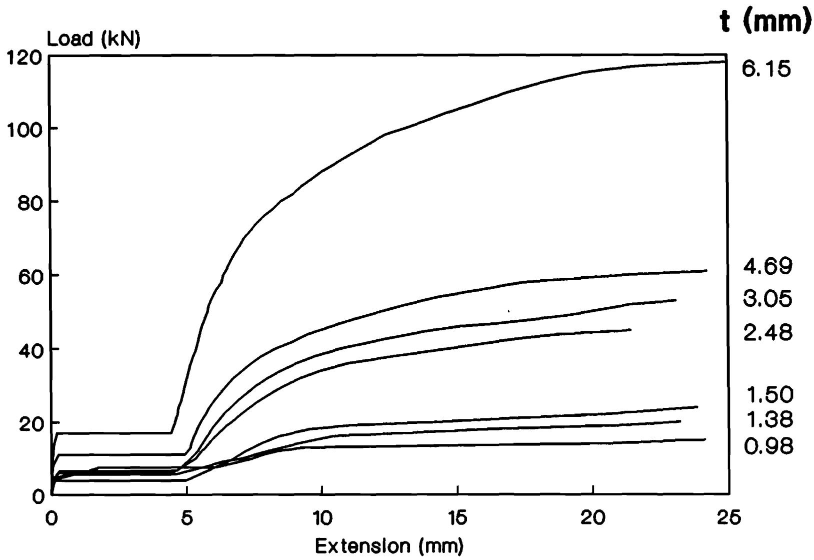

Fig. 19 : Average characteristics of bolted lap joints, for various thicknesses of steel.

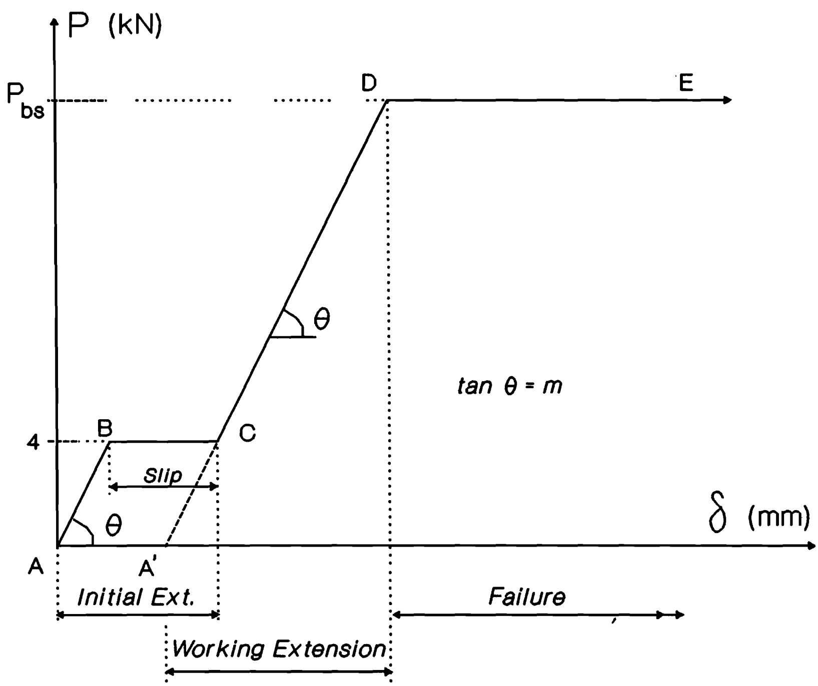

Fig. 20 : General load-extension characteristics for bolted connections, in cold formed steel sections.

The curves shown in Fig. 19, may be idealized to give the general load/extension characteristics of bolted connections in cold formed steel sections (see Fig. 20). The gradients of the two parts of the graph are taken to be equal. The joint flexibility, c, is the reciprocal of the gradient, i.e. $c = 1/\tan\theta = 1/m$.

Of the factors considered in § 4.1, only the following had a significant effect on the flexibility of bolted joints.

6.3.1 Sheet thickness

An underestimate of flexibility, after bolt slip had occurred, was taken for each of the thicknesses considered in Fig. 19.

Fig. 21 shows an example of this, for 1.5mm thick specimens.

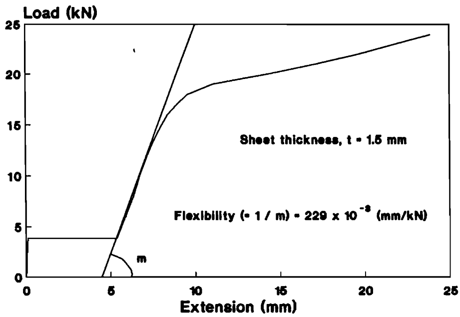

Fig. 21 : Estimated flexibility of 1.5 mm thick specimens.

The above process was repeated for all the load-extension curves shown in Fig. 19. The flexibility values thus obtained, for the whole range of sheet thicknesses tested, are plotted against (1/t) in Fig. 22.

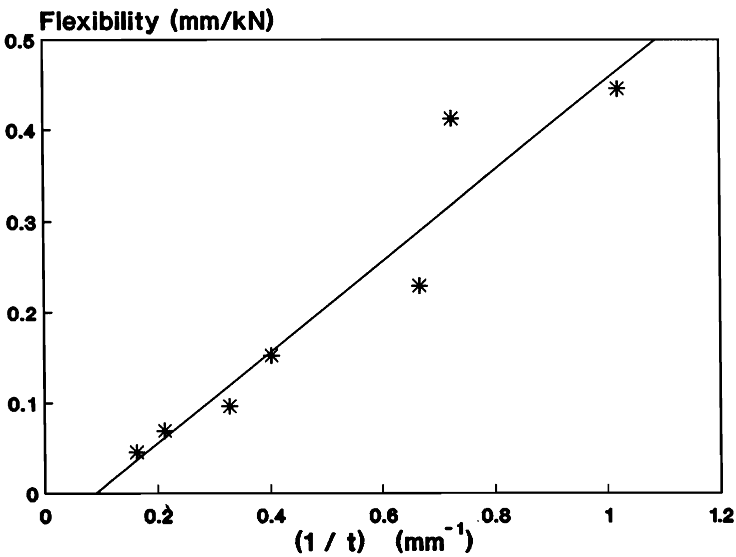

Fig. 22 : Plot of flexibility against (1/t).

It is concluded that the flexibility is inversely proportional to the sheet thickness,

$$i.e. \quad c \propto \frac{1}{t} \cdot$$

A least squares straight line fitted to the above data, yields the following expression :

$$c\ (=\frac{\delta}{P}) = (\frac{10}{t} - 1)\ 0.05 \qquad\qquad for \quad t \leq 8\,mm$$

Note that the above equation is for two sheets of equal thickness.

6.3.2 Thin/thick sheets

The test described previously in § 4.1.10, were primarily intended 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 sum of the separate deformations of the connected sheets. Hence, 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.

Results[1] obtained verified the above principle. That is, the extension ratio of two galvanised sheets of equal thickness, was very nearly 2:1 compared with that of a single galvanised sheet (of identical thickness) and an infinitely stiff plate (9mm thick plate, with the bolt hole reinforced with a hard steel bush).

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. 23)

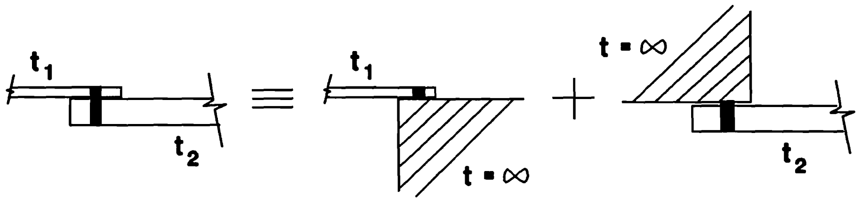

Fig. 23 : Accumulative property of sheet deformation.

Comparisons of load-extension characteristics of thin/thin and thin/thick specimens also showed that, the strength of a bolted connection was governed by the strength of the thinnest connected sheet (i.e. sheet t_1 in Fig. 23).

Therefore, the flexibility of a bolted connection is equal to the sum of the flexibilities of the connected sheets; and the strength of a bolted connection is equal to that of the thinnest connected sheet.

6.3.3 Shear plane on plain shank or threads of bolts

The design assumption has so far been that, the shear plane occurs on the threads of a fastener, rather than its on plain shank. Under load, the threads dig into the area in bearing and increase the joint flexibility. Test results indicate that the flexibility of connections is nearly doubled due to this effect.

Based on the results obtained a design factor of 0.6 is recommended when it can be shown that bolt threads do **not** occur in the shear plane. That is, the flexibility is reduced by 40%.

It is therefore advantageous to design for the correct bolt length whenever possible, so that the shear plane occurs on the plain shank of the bolt. In specifying a bolt length however, 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. That is, the length of the plain shank of the bolt should not exceed the combined thicknesses of the sheets being fastened and that of the washers (if used).

6.3.4 Hole tolerance

In connections with clearance holes even when the joints were pulled into bearing, the effect of hole tolerance was to increase the connection flexibility by nearly a factor 2, compared to those with perfect fit holes and fitted bolts. This result was contrary to expectations, where it might have been originally expected that connection flexibility should remain unaffected once the initial slack has been taken up.

A closer examination of the specimens showed the reason for this. With perfect fit holes the whole bolt circumference is 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 localized stress, since as the load increases, the bolt hole distorts and elongates. The localized stress is therefore dissipated, and the whole bolt circumference is brought into effective bearing by the time the ultimate load is reached.

A similar design factor of 0.6 was found to be applicable in such circumstances. In practice, this has a greater significance than it may first appear. With real load bearing moment connections, often with three or more bolts, as mentioned previously, 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; the whole circumference of the bolts in the connection are in effective bearing from the onset of the service life of the structure. Connection flexibility, for full scale moment connections, is therefore equivalent to that of perfect fit lap joints.

Results of tests on full scale moment connections[1] justify the above premise.

6.4 Design joint flexibility

Based on the conclusions derived in § 6.3, the flexibility of bolted connections in cold formed steel sections is defined as :

$$c \;=\; 5n\;\left(\frac{10}{t_1}+\frac{10}{t_2}-2\right)\times 10^{-3} \qquad (mm\,/\,kN)$$

where c is the joint flexibility (in mm/kN)
t_1 and t_2 are the sheet thicknesses (in mm), where $t_1 \leq$ 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:

$$c \;=\; 5\times 3\left(\frac{10}{1.8}+\frac{10}{2.4}-2\right)\times 10^{-3} \;=\; 116\times 10^{-3} \qquad (mm/kN)$$

Slip load and the amount of slip

Properties of slip load were described in § 6.1. For a bolt torque representative of the insitu conditions, a conservative slip load of 4 kN is recommended.

The initial extension of a connection, depicted in Fig. 20, is considered to occur under the self weight of a structure, and bolt slip has no significance in the service life of structures. It follows that, in calculating the moment-rotation characteristics of bolted connections, the A' DE portion of the general load-extension characteristics is critical for design (the origin is shifted to point A', see Fig. 20).

The general load-extension characteristics of bolted connections in cold formed steel sections, taking account of all relevant factors influencing the behaviour of such connections, has been defined. All the parameters depicted in Fig. 20 are quantified in § 4.2 and § 6.4 above.

7 Moment-rotation behaviour of bolted connections in cold formed steel

Most joints in cold formed steel structures are required to carry moment, with the obvious exception of truss and lattice structures. In the previous sections the load-extension characteristics of bolted connections were defined. This information can then be readily used to calculate the ultimate moment capacity for a group of bolts, and the joint rotation at failure, without having to resort to testing.

Tests carried out at Salford[1], on full scale two, three and four bolt moment connections, under two totally different loading arrangements, show that the proposed design expressions accurately predict the moment-rotation characteristics of such connections. One such test, on a four bolt moment connection is shown in Fig. 24. Note the tilting of fasteners at failure.

Any "nesting" or "interlocking" of sections is of course not taken into account in the proposed equations. These were therefore considered separately, and suitable factors have been introduced[1]. These are tabulated below for completeness.

The effect of various parameters 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

An increase of 20% in the ultimate moment carrying capacity of three or more bolt connections is proposed when sections nest.

The reader is referred to a second paper (reference [2]), based on the same research project where a full account of moment connections, design expressions for moment capacity and moment-rotation relationships for various bolt groups under practical conditions have been given. In this reference by means of design examples, it has been shown how by selecting the correct semi-rigid joint, it is now possible to utilize the design expressions proposed in this paper, to allow a limited redistribution of the applied bending moment, in order to more closely match the resistance moment of the section, and hence achieve an economical design.

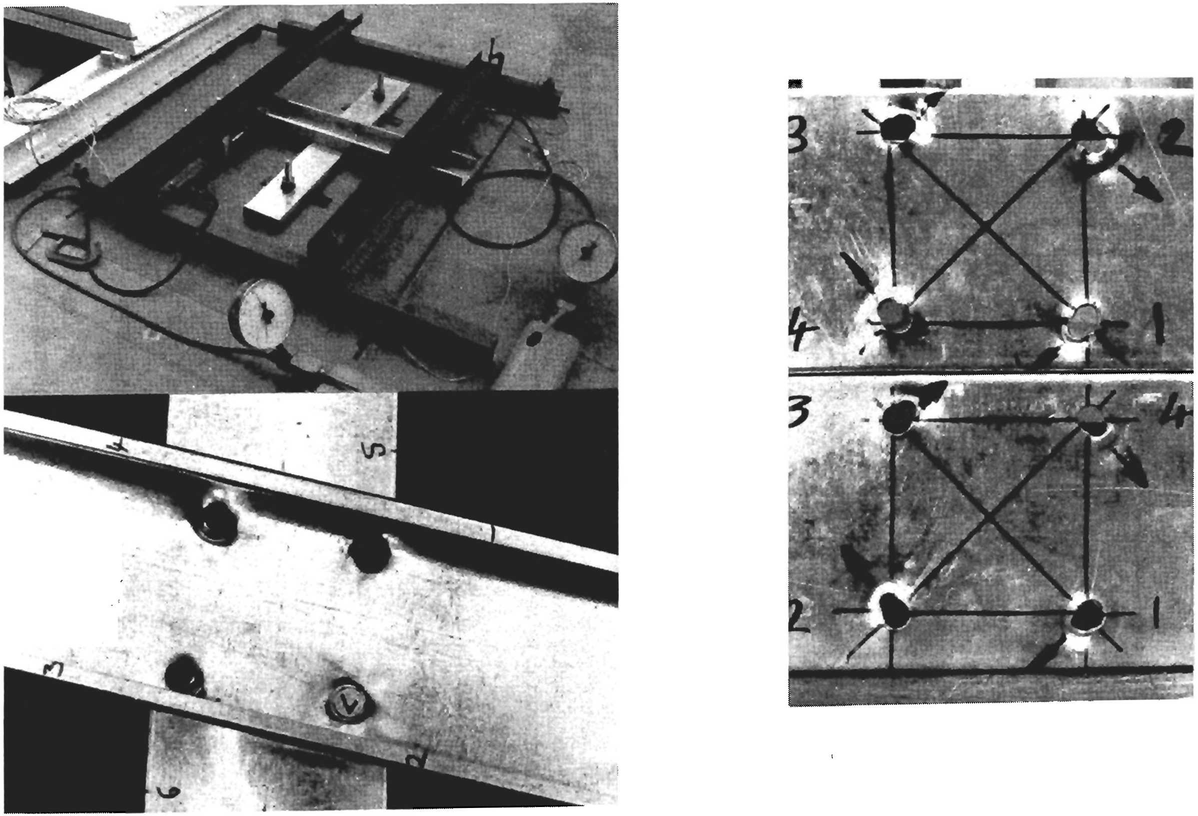

Fig. 24 : Test on a full scale four bolt moment connection.

Conclusions

In cold formed steel sections, because of the thinness of material, the strength of the joints may often dictate the strength of a member or assembly. 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 the 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 paper all relevant factors influencing the strength and rigidity of bolted connections in cold formed steel sections have been investigated individually and quantified. Based on this, design expressions for calculating the bearing strength and estimating the flexibility of such connections have been proposed.

Therefore for the first time, designers are able to estimate the moment capacity and moment-rotation relationship of bolt groups accurately, without resorting to testing.

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Appendix II- Notation

The following symbols are used in this paper:

A_n = net sectional area of connected sheets

A_s, A_t = stress or tensile cross sectional area of a bolt, $\pi d^2_{eff.}/4$

c = flexibility of bolted connections, (inverse of stiffness)

characteristic value = mean -ks

s = standard deviation from the mean of the test results.
k = statistical coefficient depending on the number of test results and type of distribution. Procedures outlined in Appendix E of BS5950 Part 5, have been used where appropriate.

d = nominal bolt diameter

$d_{eff.}$ = effective diameter of a bolt, used in the calculation of A_t $= \left(\dfrac{d_1+d_2}{2}\right)$

where d_1= basic pitch diameter of thread
d_2= minor diameter of thread

e = end distance of bolt in the line of stress

F_b^* = see P_{bs}

f_u = see σ_{ult}

$k_{1\ to\ 7}$ = design coefficients which incorporate the effect of various parameters on bearing capacity

l = lap test specimen length

M = applied moment

n = design coefficient which incorporates the influence of various factors on connection flexibility

P = applied load

P_{bs} = bearing capacity of a bolt

P_n = ultimate strength of the connected sheets in net section

P_s = shear capacity of a bolt

P_t = ultimate strength in sheet tearing

P_{ult} = ultimate strength in sheet bearing

S = bolt spacing in the line of stress

t = sheet thickness

W = total applied uniformly distributed load

w = lap test specimen width

α = non-dimensionalized parameter defined as P_{ult} / $dt\sigma_{ult}$ or = σ_b / σ_{ult}

γ_{Mb} = factor of safety for bolt material

δ	= extension
$\tan\theta$	= slope of load/extension characteristics, i.e. connection stiffness
σ_b	= bearing stress
σ_n	= ultimate stress in net section
σ_s	= ultimate shear stress of a bolt
σ_{ult}	= ultimate tensile stress
τ_{ult}	= ultimate shear stress
ϕ	= connection rotation
Ω_b	= factor of safety for bearing