University of Sheffield
Department of Civil and
Structural Engineering

01480

TELEPHONE 0742-78555

DEPARTMENT OF CIVIL AND
STRUCTURAL ENGINEERING,
THE UNIVERSITY,
MAPPIN STREET,
SHEFFIELD,
S1 3JD.
U.K.

DAN/JP

1084

17 October 1984

Dr J A Edinger
American Institute of Steel Construction
The Wrigley Building
400 North Michigan Avenue
Chicago
Illinois 60611-4185
United States of America

Dear Dr Edinger

Thank you so much for your letter of August 22 enclosing a copy of the AISI publication on type 2 construction, which arrived today. Because of the long delay caused by the mail service I had written to Bob Disque in the interim about this matter and he very kindly sent me a copy of the same document.

The correspondence in the Structural Engineer continues and I suspect that Bob's letter will be included in an issue in the very near future. As a result of the interest in this topic I have been asked to prepare a paper on the whole subject of connection stiffness and steel frame design and am fairly far on with this task. It doesn't include anything new but it does (hopefully) make clear to designers some of the implications of the assumptions inherent in the accepted design approaches - 'simple design', 'wind connection method' etc.

I received a substantial report from Dr Ioannides containing digitised versions of a large proportion of the available experimental moment-rotation data something like two weeks ago. This is proving extremely useful to us. On checking through the coverage, it appears that he and I have something like 50% common ground but that he has unearthed a number of Amercian test series available only in report form whilst I have been able to find several sets of experimental data from sources outside North America including several very recent series. Taken together the two surveys probably cover virtually all the available information.

I note from my letter of 1 August that I had promised you a copy of my draft report but believe that this was not actually despatched. Please accept my apologies for this omission; the document is enclosed. Review by CIRIA is taking a little longer than I had expected so I cannot say when (or indeed how extensive) the amendments will be made.

Yours sincerely

David A Nethercot

D A Nethercot
Senior Lecturer

STEEL BEAM TO COLUMN CONNECTIONS - A REVIEW OF TEST DATA AND THEIR APPLICABILITY TO THE EVALUATION OF THE JOINT BEHAVIOUR OF THE PERFORMANCE OF STEEL FRAMES

BY

D. A. NETHERCOT

Contents

Steel Beam to Column Connections - A Review of Test Data and their Applicability to the Evaluation of Joint Behaviour of the Performance of Steel Frames

by

D. A. Nethercot[1]

Synopsis

The main types of beam to column joints used in structural steelwork are identified and some indication of the more popular forms for U.K. use given. In-plane moment-rotation behaviour is identified as an important characteristic for assessing the influence of the particular form of connection on the behaviour of members (columns and beams) and complete frames, with the joint's M-ϕ curve being the most appropriate measure of this. Data from more than 700 experiments are summarised; for approximately half of these M-ϕ curves are available. The influence of joint parameters e.g. cleat size, number, size and type of bolts, local stiffening arrangements etc., on M-ϕ behaviour of each of the 13 joint types is discussed. Connection restraint in a more general context is mentioned, although only in the case of end torsional restraint for beams has any significant work been conducted. Attention is drawn to those areas in which additional testing is required if a full understanding of the contribution of steelwork connections to the performance of steel frame structures as a whole is to be developed.

----

[1]Department of Civil and Structural Engineering, University of Sheffield, Sheffield, U.K.

1. Introduction

It is now more than fifty years since the Steel Structures Research Committee (1) undertook what remains as the largest concerted attempt to understand the structural behaviour of steel frames. One of the main findings was the crucial role played by the connections, in particular their ability to transmit load and to provide restraint to the beams and columns. Largely as a result of their work the concept of permitting design to incorporate some allowance for semi-rigid action came to be included in British Steelwork codes (2). The fact that very little explicit use has been made of this method is due to a number of factors, two of which are the relative complexity of the necessary design calculations and the lack of comprehensive information on the performance of the full range of modern-day connections.

Interest in trying to remove these hindrances has meant that several attempts have been made by the research community both to achieve the clearer understanding of semi-rigid action which might lead to a simpler design approach and to obtain data on connection response. This report attempts to review activity within the second area which it is felt may provide a useful basis for continued effort in the first. It therefore complements an earlier review (3); specifically it covers work completed since that time, including investigations still in progress, as well as giving rather more attention to the use of the findings in assessing frame behaviour. It does not attempt to deal with the design of different types of joint from the standpoint of sizing the individual component parts e.g. bolts, welds, stiffeners etc; up to date discussions of this topic are available elsewhere (4-7).

2. Types of beam to column connections

Connections in structural steelwork normally involve some combinations of bolting and welding; rivetting, which was at one time virtually the only method of fastening, appears to be hardly used nowadays. Since site welding is generally discouraged in Britain, certainly for routine building frames, it follows that joints will normally be arranged so as to permit the site connection to be made by bolting. Of course, this has certain other advantages in assisting erection, alignment etc. With this limitation in mind Table 1 lists the main forms of beam to column connection assuming the use of I-sections for both types of member. Arrangements not shown include the various forms of moment connection used for example in multistorey rigid-jointed frames in North America (7-9), connections of the type necessary to resist earthquake action as used in Japan and parts of the U.S.A. (10, 11) and all-welded portal frames knee joints (7, 12, 13).

3. **Relative usage of connection types**

Not all of the joint types shown in Table 1 are equally popular. Reasons for particular preferences will vary from one situation to another with economics of fabrication being possibly the most important single factor. Even this aspect of the problem will be viewed differently depending on the exact range of equipment available to the fabricator and the pattern of work within his shops. Thus end plate connections may be preferred by a fabricator using an automatic beam line whilst web cleats may be preferred by another who values the easier adjustment possible during erection provided by this form.

An attempt to establish the current pattern of use and the reasons behind it has recently been made (14). A questionnaire was sent to fabricators and designers listing ten forms of beam to column connection (seven of those shown in Table 1 together with two all welded connections and an extended end plate with a haunch). The responses showed that much the most popular connection type was the flush end plate, largely because of its straightforward fabrication although convenience in containing the joint within the beam depth was also a factor. Next in popularity were web cleats (the survey assumed double sided) and extended end plates followed by the bottom flange cleat with a web cleat and flange cleats. Both combined web and flange cleats and flexible end plates appear to be seldom used in this country.

If it is accepted that there is an element of "fashion" behind the relative popularity or unpopularity of certain forms of joint, then it would be of interest to obtain reactions to the use of single-sided web connections. The web side plate appears to enjoy support in Australia (it is one of the types covered by their standard connections manual), whilst single web angles are often used in North America.

4. Influence of connection behaviour on the performance of members

All of the joints shown in Table 1 are capable of transmitting shear from the beam into the column. The most important difference between each of them in the context of overall structural behaviour lies in their rotational stiffness i.e. their ability to transmit the restraining effects of the beam to the column (or vice versa) and thus also to transmit moments. This facility will vary from about zero for the most flexible connections e.g. web cleats, up to almost 100% of that provided by a rigid connection (one that allows no change in the relative angle between the members) for the stiffest types e.g. tee-stubs. It is most appropriately expressed by means of the connection's moment-rotation (M, ϕ) characteristic.

Fig. 1 shows a schematic version of a simple test designed to provide the M, ϕ relationship for the beam to column connection. The moment M is simply taken as the moment applied to the connection by the beam load W, whilst ϕ is defined as the change in angle between the beam and column centre-lines. Measurement of ϕ in an experiment can be by any suitable arrangement of dial gauges, accelerometers or inclinometers providing these permit the true value to be obtained. This (ϕ_{CON}) is defined as the rotation due to the connection itself together with any local distortions of the beam centre-line and column face due to the connection. Thus ϕ_{CON} can only be obtained from a knowledge of the total rotation of C relative to A in Fig. 1 (ϕ_{TOT}) if the other two components due to flexure of the beam over distance BC (ϕ_{BC}) and of the half-width of the column AB (ϕ_{AB}) are also determined.

Some idea of the range of M, ϕ curves obtainable for the different forms of connection shown in Table 1 is provided by Fig. 2

which shows typical results. Curves which plot close to the horizontal axis imply relatively little rotational stiffness whilst those which approach the vertical axis are performing almost as a rigid joint. Inspection of all the available data listed in Table 2 shows that:

1. All common forms of beam to column connection are more correctly regarded as semi-rigid.
2. All common forms of beam to column connection possess nonlinear M, ϕ curves.

5. Review of test data on connection performance

Starting from the earliest studies of rivetted connections (1, 15, 16), many series of tests on beam to column connections have been conducted in laboratories in different parts of the world. Not all of these have reported on moment-rotation behaviour; in many cases the measurements necessary to determine ϕ were not made. A survey that is complete up to the early 1980's is available (3). For the present study attention has been focussed on those investigations which have provided M, ϕ data suitable for use in future studies of member and frame behaviour, although several of the earliest test series are not reviewed in great detail due to the out of date fastening employed i.e. rivets. For completeness some post-1980 investigations which may not provide adequate M-ϕ data are also included.

Table 2 lists by joint type the relevant test series, whilst a more detailed listing on an individual test basis is given in Tables 3-12. Close examination of these data has enabled several aspects of connection behaviour and its influence on the performance of members and frames to be identified.

5.1 Single web cleat

Three series of tests have been located, all of them conducted in the same Canadian laboratory. Thus test arrangements and procedures were very similar in all cases, comprising a cantilever set-up with a deep beam and a very stiff column. Shear load and beam rotation were produced by two separate jacks acting vertically on the beam. Main variables in the tests were:

i. the number of bolts (from 2 to 12)

ii. the type of fastening to the column (bolted or various weld patterns), see Fig. 3

iii. the type of fastening to the beam (round or slotted bolt holes)

In several cases specimens were subjected to reloading and/or slow cyclic loading. Available M-ϕ curves for the initial loading show considerable variation in initial stiffness with number of bolts; connections with less than 5 bolts sustained very little moment whilst those with more than 10 bolts possessed significant, almost constant, initial stiffness as indicated by Fig. 4. However, the occurrence of slip in the larger connections caused sharp reductions in moment, after which positive rotational stiffness returned. On reloading, rather lower stiffnesses were observed, with behaviour in the third cycle being virtually identical to that of the second cycle. The test series of ref. 15 showed that changes in detailing produced substantial changes in both rotational stiffness and in the moment developed by the connection e.g. a change from round to 1.25:1.00 slotted holes reduced the moment to about one half.

5.2 Double web cleats

Available test data for this type of connection contrasts sharply with the previous case, since it has been provided from a number of different sources. Unfortunately not all investigations have provided M-ϕ curves; the work of refs. 20-22 was addressed principally to the problem of "block shear" failure of the beam web (particularly important if part of the upper flange is removed by end coping) whilst ref. 23 dealt with the assessment of the strength of eccentrically loaded bolt groups. One striking feature of Table 2 is that virtually all the tests used M20 (¾ in.) bolts. In the context of M-ϕ data for subsequent use in assessing the response of subassemblages, probably the most useful studies are those of Munse et al (24), Lewitt et al (25), Bergquist (26), Bose (29) Bjorhovde (30) and Sommer (39). Since 10 of the 12 tests from the first two studies employed some rivets, this means that the number of adequately reported tests in which present day fastening was used is only 20. Of these just 1 is for a column web connection. A fuller description of these 20 tests is provided in Table 3.

Each of the eight test series referred to in Table 3 provides information on different aspects of connection performance. The main individual findings are:

1. Connections with similar geometry, including the fastener arrangement, have broadly similar M-ϕ curves (24), see Fig. 5.
2. Connection flexibility results principally from deformation of the angle cleats in the tension region with smaller contributions being due to extension of the column bolts (if used), column flange deformation and angular movement of the beam web relative to the cleats (24).
3. Connection stiffness reduces significantly at load levels which cause yielding of the cleats; it may increase at higher rotations if the beam toe starts to bear directly against the column (24), see Fig. 6.
4. For modest changes in the moment to shear loading on the connection its stiffness does not alter significantly (24).
5. Some of the specimens used in ref. 24 were similar to that used for an earlier all-rivetted specimen tested by Hechtman and Johnston (31). Despite differences in the techniques used to measure rotations, quite similar M-ϕ curves were obtained, see Fig. 5.
6. For cruciform tests in which nominally identical joints are used for both beams, virtually identical M-ϕ curves are obtained for the two sides (25).
7. The use of hardened steel washers under the bolt-heads and the nuts increases joint stiffness (25), see Fig. 7.

8. Slip of the beam web relative to the cleats is an important source of joint flexibility for connections in which the beam is bolted (using clearance holes) to the cleats. This may be substantially eliminated if two rows of beam web bolts are used (25).

9. Providing the column flange thickness exceeds the thickness of the cleats, deformation of the column flange is not an important source of joint flexibility. Thus double-sided connections to the column web have similar M-ϕ characteristics (25), see Fig. 8.

10. Connection stiffness is approximately proportional to the square of the cleat thickness (25).

11. For this form of connection the presence or absence of paint on the faying surfaces appears to have no effect on connection performance (25).

12. Increasing the distance from the column face to the line of beam web fastening increases joint flexibility (by about 33% initially for connections A and C of ref. 26). This effect is even more important, however, for the position of the column fasteners e.g. connection B of ref. 26 is approximately 4 times more flexible than connections A or C, see Figs. 9 and 10.

13. Connections in which the cleats are bolted to the column flange have sharper unloading characteristics than those (more flexible) connections which employ welded cleats (26).

14. Ref. 26 also contained tests on beam and column subassemblages in which connections of the type shown in Fig. 9 were used but with the important difference that the beams framed into the column web. Some monitoring of connection behaviour during these subassemblage tests was undertaken. This showed, among other

things, that connection to the column web has relatively little effect on initial connection stiffness but column web deformation appreciably reduces the joint stiffness during unloading (26).

15. For connections sustaining high shear the M-ϕ relationship may fall whilst the applied load is still increasing i.e. negative rotational stiffness is present at large rotations (27).

16. For the tests of ref. 27 the change from two to three web bolts increased both the initial rotational stiffness and the moment capacity appreciably.

17. The peak moments sustained by flexible connections of this type are only a few percent (<5%) of the beam fixed end moment (27).

18. Tests in which the cleats are bolted to the beam web exhibit bolt slip at an early stage; this is particularly noticeable for connections with only two beam web bolts. Connection stiffness increases appreciably at higher rotations, however, when the beam bears against the column (30, 39), see Fig. 6.

19. Increasing the number of web bolts increases the initial joint stiffness, although the amount of increase does not appear to follow a consistent pattern (30, 39).

20. The use of a diagonal cope, appears to reduce the joint's ultimate moment capacity significantly (30).

21. Connections in which the cleats are bolted to the column flange appear to undergo gradual softening up to the point at which the beam butts against the column (30).

22. Using welds on the beam and bolts on the column increases the final moment capacity appreciably; for the two pairs of tests of ref. 30 in one case the initial moment capacity was also increased.

5.3 Web side plates

Only one set of five tests has been identified and the results listed in Table 4; this was conducted principally as a check on the design provisions for this form of joint given in the AISC standardised connections manual (32, 33) Thus the determination of moment-rotation behaviour was not one of the direct objectives. The tests were conducted using a propped cantilever arrangement illustrated in Fig. 11 in which the tip support was lowered as loading proceeded in an attempt to simulate the behaviour of a simply supported beam at the connection.

The shear versus beam rotation results provided exhibit, with the exception of W5, almost linear behaviour up to joint failure. Moment versus shear curves provided for tests W2 and W4 also show an approximately linear relationship. However, during the initial phase of each test the authors observed considerable slip between the beam web and the cleat. This suggests that the M-ϕ curves (if available) would contain an almost horizontal region with an increase in stiffness once the bolts had come into bearing. This possibility is supported by the authors' examination of the cleats after failure, which occurred by yielding of the cleat plate for W4, revealing extreme deformation at the inner line of bolt holes. These bolts carried most of the shear with the second line being present principally to provide the connection's moment capacity.

It is understood that BHP will be conducting additional tests on this form of connection in the near future. These will use a deep 760 UB147 with either one or two rows of 9 M20 grade 8.8 bolts.

5.4 Flange cleats

Several of the earliest test series (1, 16, 31) used angle cleats on the beam's top and bottom flanges, usually but not always, with both sets of fasteners being rivets. It appears that interest in this form of connection then lapsed until recent studies (29, 35, 37), each of which has been conducted with the specific aim of utilising semi-rigid joint action. The single test of ref. 29 was actually a pilot test for the programme of ref. 35, whilst the 37 tests performed by Hotz (37) form one of the most detailed studies available for any form of connection. Van Dalen and Godoy (36) tested 3 versions of the same connection; in two of these varying degrees of composite action with a beam slab were present. Full details of the more recent tests referred to in Table 2 are provided in Table 5. Thus some 51 individual $M-\phi$ curves are available, examination of which has produced the following findings.

1. All 12 tests of ref. 35 produced smooth curves; slip of the HSFG bolts on the tension face of the beam being delayed until rotational stiffness had almost disappeared i.e. the $M-\phi$ curve had flattened out, see Fig. 12.
2. Increasing the length of the cleat (from 150mm to 200mm) increased both joint stiffness and joint moment capacity rather more than did maintaining the same dimensions but using higher strength material. However, the most significant of the variables studied in terms of joint characteristics appears to be cleat thickness (ref. 35).
3. For the single uncased test of ref. 36 in which a smaller and much thinner top cleat was used most of the deformation was concentrated in this component. Utilisation of composite action from the beam slab virtually eliminated this source of flexibility leading to joints with much higher initial stiffnesses and significantly greater moment capacities. In the case of the lightly reinforced slab, slip of the beam relative to

the seating angle produced a plateau in the M-ϕ curve at a moment of approximately one quarter of the joint's eventual capacity.

4. For the tests of ref. 37 either 3 or 4 nominally identical connections were tested for each geometrical arrangement; in all cases the resulting set of M-ϕ curves plotted very close to one another, see Fig. 13.

5. Every test of ref. 37 produced a smooth curve, with a progressive reduction in connection stiffness as rotation increased.

6. The 37 tests of ref. 37 may be grouped into three sets -(A, D, E, I), (B, F, J) and (C, G, K) - within which the only change (apart from the extra top cleats in the "special" series E) was in the size of the column (specifically its plate thickness). Whilst there is some effect upon the shape and position of the M-ϕ curves this appears to be only about as great as the scatter between results within any one series.

7. Use of extra internal top cleats in series E of ref. 37 significantly increased both initial joint stiffness and moment capacity.

Ref. 35 also makes several observations based on the results of a preliminary series of tests, results for which (with the exception of the one result given in ref. 29) were not provided. The most important of these are:

1. Shear did not have a significant effect on moment-rotation characteristics.

2. The rotation is greater the first time the moment is applied.

3. Repeatability of results with supposedly identical connections could be obtained.

4. HSFG bolts should always be used to connect the horizontal leg of the angle to the beam flange to prevent rotation due to slip.

5. Stiffness decreases as the distance between the first line of bolts and the root radius of the vertical leg of the tension cleat increases.
6. Increasing either the length of the cleat or using additional bolts in the vertical leg has only a small effect on M-ϕ behaviour.

Both of the main series of tests on this form of connection (35, 37) clearly demonstrate its semi-rigid character, with smooth nonlinear M-ϕ curves which eventually flatten out at moments of the order of 50% of the beam's plastic moment capacity. However, it is worth noting that in all cases four fully torqued HSFG bolts were used to fasten each cleat to the beam flanges and that the cleats were also fastened to the column flange by bolts (8.8 or HSFG). No tests have been located in which any of the fastening was by welding; in view of some of the findings from tests on web cleat connections it seems likely that such an arrangement, whether to the beam or the column, would reduce the connection's performance due to the increased distances between the lines of fastening. Similarly, no results appear to be available in which ordinary grade 4.6 or 8.8 bolts were used to attach the cleats to the beam flanges. This possibility provoked considerable discussion (38) when ref. 35 was presented, leading to one suggestion that any bolt slip on the beam flanges might not actually be harmful since, with a properly controlled clearance between the end of the beam and the face of the column, maximum load in the assembly would now correspond to a more favourable, steeper section of the M-ϕ curve.

5.5 **Bottom flange cleat and web cleat**

Despite that fact that 80 per cent of the responses to the Hatfield questionnaire (14) used this form of connection at least "sometimes", no experimental studies of its performance have been located. (Although one test of ref. 31 appears to have used this form of connection no details of the joint arrangement or the M-ϕ behaviour are provided.) Of course, it is possible that moving the top cleat of type 4 onto the upper part of the web has relatively little effect and that such connections behave substantially as flange cleated connections.

5.6 **Header plate**

The type of connection in which a plate is shop-welded to the end of the beam in preparation for site bolting to the column flange may have several varients. For the purpose of this review these are considered under three headings: a header plate (or flexible end plate) in which the end plate is substantially smaller than the beam depth, an end plate approximately equal to the beam depth with all the bolts contained within this depth which is termed a flush end plate, and an extended end plate for which some of the bolts are located outside the beam flanges. Of these the first acts principally as a shear type connection, whilst the second and third, especially when used with column web stiffeners, possesses considerable moment capacity. Flush and extended end plates are considered in sections 5.7 and 5.8 respectively.

Four series of tests on header plate connections, two conducted in Canada and two in Australia, have been identified and these are referenced in Table 2, with fuller details being provided in Table 6. Ref. 40 states that Sommer (39) conducted 24 tests, 20 of which were on header plate connections. Of the 9 M-ϕ curves given by Kennedy (40), 8 correspond to this form of connection; M-ϕ curves for all 20 tests are available in ref. 40. Ref. 41 provides a full description of the later series of 8 tests. The 2 tests of ref. 27 the 7 of refs.

28 and 34 were all conducted on AISC standard connections (32, 33) with the primary aim of assessing connection strength under high shear loading.

The results presented in refs. 40 and 41 show that the M-ϕ behaviour of this form of connection consists of two distinct phases, the transition between them occurring when the bottom flange of the beam rotates sufficiently that it bears directly against the column flange. Thus each of these M-ϕ curves, an example is given in Fig. 14, exhibits a sudden increase in stiffness (slope), following the initial nonlinear response. The comparatively thin end plate suffers considerable distortion with inelastic action starting at very low loads.

From the discussion of Sommer's tests given in ref. 40 it is possible to note that:

1. Connection stiffness, moment capacity and the point on the M-ϕ curve at which the beam web comes into direct bearing all increase as either plate thickness or plate depth increase or as the horizontal distance between the lines of bolts (gage) decreases, see Fig. 15, 16 and 17.
2. Bearing occurs at moments of between 5 and 15 per cent of the nominal first yield moment of the beam; moments at least twice as large may eventually be developed.
3. Assuming a simply supported span of 24 times the beam depth, first yield would normally be expected to occur in the beam at an end rotation of rather less than that at which web bearing commences i.e. whilst the connection was still on the initial portion of the M-ϕ curve.
4. Kennedy gave a Frye and Morris (42) type of formula, obtained by curve-fitting, that provides a good average representation of the

initial portion of the M-ϕ curves valid for the range of connection geometries covered by ref. 39.

5. Kennedy suggested that changes in the moment-shear ratio have negligible effect on M-ϕ behaviour, although the connections of ref. 39 were all subject to very small shears.

6. The connections of ref. 41 were designed for between 43 per cent and 86 per cent of the beam web capacity in shear. The M-ϕ curves obtained were of exactly the same form as those discussed above; the comments on plate thickness and gage made under 1 also apply to this series of tests.

7. For the tests of ref. 27 the M- curves showed a fall-off as joint failure (by bolt shear in test 1 or beam web cracking in test 2) occurred. No mention was made, however, of the toe of the beam coming into contact with the column face.

8. The initial joint stiffnesses and the maximum moments sustained were both greater than for the equivalent tests on double web cleat connections. The moments were still small, being about 5 per cent of the beam fixed end moment (27).

9. On reloading (the M-ϕ curve having reached a gently falling region) the connection of test 3 exhibited reduced stiffness and carried less moment before collapse (27).

10. Ref. 34 contained two pairs of tests on nominally identical connections; the shear-rotation behaviour was noticeably different for each test of a pair, although part of this is attributed to small variations in testing procedure.

11. Results for two tests of ref. 34 showed that the joint moment either remained almost constant or fell slightly as failure was approached.

For this form of connection, in which the depth of the end plate is between about 40 and 80 per cent of the depth of the beam, moment-rotation behaviour is qualitatively and quantitatively similar to that of double web cleat connections (type 2), see Fig. 18.

5.7 **Flush end plate**

End plates approximately equal in size to the beam depth with all the bolts placed between the beam's flanges have been tested by a number of investigators as listed in Table 2 and detailed in Table 7. Recognising that this form of connection is capable of transmitting significant moments, each of the 8 studies listed have given some attention to the question of moment-rotation behaviour. Thus for the 60 tests covered, M-ϕ curves are available for 54 of them. All but 7 of these, which relate to some of the tests of ref. 43, appear in the published reports. No tests have been identified in which the beam(s) framed into the column web; in several instances columns with very heavy flanges (43) or with a large amount of web stiffening designed to minimise column flange deformation (44, 45, 46) were used. Several of the test series (47, 48, 44, 45, 43, 46) varied only one or two parameters, thereby enabling their influence on joint performance to be directly assessed. The most important of these are:

- Column reinforcement by means of flange backing plates and/or web stiffeners - ref. 47.
- Different forms of column web stiffening - ref. 47.
- Deliberate lack of fit between end plate and column flange - ref. 44, 45.
- End plate thickness - refs. 44, 45, 43, 46.
- Bolt arrangements - ref. 43, see Fig. 19.

Although the M-ϕ curve for this form of connection is again nonlinear, it is noticeable that the great majority of the 54 available results follow the same general pattern, see Fig. 20.

Initially the curve is virtually linear with a slope that appears to depend principally upon end plate thickness. This is clearly illustrated by the 5 tests of ref. 46 shown in Fig. 19 for which initial stiffness increases smoothly each time the plate thickness changes. Conversely altering the column web stiffening in the manner used in ref. 47 or ref. 48, increasing the end plate depth beyond the depth of the beam (47) or introducing deliberate lack of fit (44, 45) produces little if any change, see Figs. 21 and 22. An exception occurs for the shallow set of connections of ref. 43, in which HEA 300 beams were used, where increases in end plate thickness apparently had little effect on joint stiffness. However, for the set in which IPE 400 beams were used, connection stiffness increased with an increase in end plate thickness in the same manner as for the tests of ref. 46. Useful information on the relative efficiency of different bolt arrangements may be obtained from ref. 43, the most important finding being that joint flexibility increases with an increase in either the horizontal spacing of the bolts (gage) or the distance of the top row of bolts below the beam flange, see Fig. 23. For the 300mm deep beams adding either a second row of bolts below the top row or a second line of top bolts outside the first line had almost no effect on connection stiffness. Indeed, using only 2 top bolts close to the upper flange appears to be structurally more efficient than placing 4 top bolts rather lower down. However, connections with a double row of top bolts did perform rather better in the case of the 400mm deep joints.

After this initial phase, connection stiffness reduces due to softening somewhere in the connection. This has variously been attributed to plate separation (44) or yield of the column web (48) and occurs at a moment of about 60 per cent of the connection's capacity. The effect on the $M-\phi$ curve is to produce a rather rounded "knee", the extent of which is influenced by individual joint parameters e.g. end plate thickness, column stiffening etc.

Once the connection has softened its stiffness again becomes approximately constant but at a greatly reduced value, typically about 1/40th of its initial stiffness. Moreover, this reduced stiffness appears to be only slightly affected by changes in joint parameters, including end plate thickness.

The ultimate moment capacity of the connection is influenced chiefly by end plate thickness (46, 44, 45, 49, 43) and column web stiffening (47, 48), not surprisingly the use of a thicker plate and/or more effective stiffening enhancing capacity. One point of interest in the tests of ref. 47 is the apparent lack of improvement produced by conventional compression stiffeners on the column web, equally good joint characteristics being achieved by the simpler method of extending the end plate slightly below the beam flange. Alternatively, more substantial gains in moment capacity were achieved through the use of short backing plates inside the column flange behind the tension bolts, as shown in Fig. 24. The tests of ref. 43 permit some conclusions to be drawn on the effect of bolt arrangement on connection strength. These are broadly similar to those mentioned previously when discussing joint stiffness i.e. the most effective arrangement is to maximise the vertical distance between the top and bottom bolts and to locate the fastener lines as close to the beam web as possible. Whilst the use of a double row of top bolts did improve the strength of the deeper connection it had little effect for the 300mm deep connections.

In several of the tests the full moment capacity of the beam (either M_y or M_p as appropriate) was achieved by the connection. This is of particular importance for the four joints which were tested as part of the frame during the study reported in ref. 50. In each case a moment approximately equal to the full plastic moment capacity of the beams was sustained through a large range of rotation by joints in which backing plates were used.

5.8 **Extended end plate**

Greater stiffness and strength may be introduced into an end plate connection by placing a row of tension bolts outside the beam flange, thereby increasing the effective lever arm of the resultant tensile force. The main disadvantage of this modification is that part of the connection now intrudes into the space between beams. Nonetheless the survey of ref. 14 found this form of joint to be a very popular means of transferring beam moments into columns; not surprisingly it has formed the subject of a significant number of separate studies as listed in Table 2, and Table 8. Although the main aim in conducting the majority of these tests has been to provide information which would assist the designer in proportioning the individual components for adequate strength e.g. end plate thickness, bolt size and number etc., several of the studies have monitored the overall performance of the connection. Thus of the 102 tests referenced, M-ϕ curves have been published for only 53, although it is suspected that rather more may well be available e.g. ref. 57. In certain cases test series have been organised in such a way that the influence of individual features of the joint may be assessed, although this is generally less systematic than was the case for the flush end plate. The most important of these are:

column flange thickness - refs. 56, 58, 59

end plate thickness - ref. 55

column stiffening by means of flange backing plates - ref. 56

column stiffening for haunched connections - refs. 62, 63

haunching - refs. 62, 63

axial column loading - refs. 56, 61

The typical M-ϕ curve for this form of connection is quite similar to that discussed previously for the flush end plate, see Fig. 25. Thus it has an initial, approximately linear portion followed by a large knee, during which considerable softening of the connection occurs, leading into a final phase of approximately constant (possibly almost zero) stiffness. Tests in which connection response after the attainment of peak load was monitored show the unloading behaviour to be generally stable, unless triggered by an event such as a weld failure. In several instances e.g. ref. 56, the connection sustained the full plastic moment of the beam through a large range of rotation, see Fig. 26. Because of the rather different nature of many of these test series it is appropriate to present selected points from each as follows:

1. For the 3 tests of ref. 56 in which specimens were nominally identical except for column size, a reduction in column flange thickness reduced the initial connection stiffness appreciably. It also changed the shape of the M-ϕ curve; for an 11.9mm flange the initial portion up to about 50 per cent of capacity was essentially linear whereas for a 6.5mm flange the curve was nonlinear almost from the start of loading, see Fig. 27. Similar behaviour was observed between tests 4 and 6 of ref. 49, although

these results will also have been affected by a doubling of the end plate thickness (from 12mm to 25mm).

2. Use of too thin a column flange may prevent the attainment of full beam strength (ref. 56), see Fig. 27.

3. The addition of column stiffening in test J4 of ref. 56 enabled the connection to develop the full plastic moment of the beam at large rotations but only increased joint stiffness in the upper regions of the M-ϕ curve.

4. Comparison of the M-ϕ curves for the 2 pairs of tests of ref. 50 in which the effectiveness of flange backing plates was investigated shows increases in both moment capacity and the extent of the initial linear portion of the M-ϕ curve, although the slope of this line does not appear to change significantly, see Fig. 28. Similar behaviour was observed for the paired specimens of ref. 60.

5. After "yielding" of the connection the slope of the M-ϕ curve was between about 4 per cent and 10 per cent of its original value. This figure was rather higher (and more variable) than the 2.5 per cent associated in section 5.7 with flush end plates.

6. For the two deep specimens of ref. 55 in which 8 tension bolts were used identical linear load-deflection behaviour was observed up to 40 per cent and 65 per cent of ultimate for the thinner and thicker end plate respectively. Maximum capacities of both specimens were almost identical.

7. Tests 1 and 2 of ref. 61 show that an axial compressive load which produces a longitudinal stress of 135 N/mm^2 in the column flanges has negligible effect on M-ϕ behaviour, even at extremely large rotations. Nor does it reduce moment capacity. From the other tests in which the column load was applied eccentrically it

would appear that longitudinal stresses up to 170 N/mm^2 have negligible effect. For test J5 of ref. 56 the presence of an axial compressive load in the column of 35 per cent of its working load reduced the initial, linear part of the M-ϕ curve due to earlier column flange yield and flattened the knee leading to a response that was almost bilinear; moment capacity was unaffected.

8. Moderate changes in end plate thickness for end plates with haunches on the compression side have negligible effect on the M-ϕ curve until it becomes noticeably nonlinear at moments in excess of 60 per cent of connection capacity (ref. 62).
9. Initial connection stiffness may be increased by using longer haunches and/or doubler plates which stiffen the column web over the full depth of the end plate. Doubler plates in the compression region of the column only have little effect on M-ϕ behaviour (Ref. 62, 63).
10. Tests 3 and 4 of ref. 49 enable a direct comparison to be made between flush and extended end plates since all other parameters were similar for the two specimens, see Fig. 25. Allaspects of performance appear to be improved for the latter type: initial stiffness was approximately doubled, strength increased by almost 100 per cent and stiffness at large rotations was significantly higher. Similar behaviour may be observed for the tests of ref. 50, although the presence of flange backing plates in some pairs of tests appear to have reduced the differences in response.
11. Limited information is available for the tests of ref. 57 although it is believed that more detailed reports do exist. The four M-ϕ curves provided exhibit nonlinear behaviour throughout, with their initial stiffness being highly dependent upon end plate thickness.

However, these tests are not directly comparable with many of the others in Table 8 as the end plate extended above and below the beam. Axial loads of one third of column yield were present in most of these tests. Ref. 57 also refers to 97 theoretical M-ϕ curves generated from a finite element program, from which an M-ϕ equation has been obtained using regression analysis. This equation suggests that column flange thickness, end plate thickness, gauge and beam depth are the main influences on connection performance.

12. Use of high strength end plates produces a small increase in initial connection stiffness but has negligible effect on moment capacity (ref. 57).

13. Results up to 0.6 M_p for connection C2 of ref. 57 were obtained at different levels of preload in the tension bolts. These showed increases in initial connection stiffness of 50% and 100% for two thirds and full preload with little improvement on changing from hand tight to one third preload.

14. Detailed measurements of connection rotation made in ref. 57 enabled the contributions from each component to be identified for a typical test (C4) as:

shear deformation in column web	50%
end plate flexure	18%
column flange flexure	17%
tensile strain in column web	6%
compressive strain in column web and stiffener	9%

The authors suggested that a similar amount of shear deformation would be expected with fully welded connections.

15. Premature failure due to column web buckling will occur if stiffeners are required but not used (test 1 of ref. 53).

16. Use of heavy end plates may improve the initial stiffness of the connection but will not necessarily enhance its moment capacity (ref. 53).
17. HSFG bolts in the tensile zone may be replaced with "ordinary" high tensile bolts.
18. Providing premature failure due to column web collapse is prevented, the use of progressively heavier column stiffening has little effect on either the initial connection stiffness or on its moment capacity. Similarly, providing end plate thickness exceeds a certain threshold value, further increases do not significantly improve connection performance (ref. 51), see Fig. 29.
19. This form of connection is capable of developing approximately 70% of full rigidity (ref. 51), see Fig. 29.

5.9 Combined web and flange cleats

Only one series of tests appear to have been conducted recently in which combined double web and top and bottom flange cleats were used (50). These formed part of the series of limited frame tests reported at the end of ref. 50 and as such permit useful comparisons to be drawn with the other related tests in which either flush or extended end plates or tee stubs were used. Apart from the early tests indicated in Table 2 no experiments on single joint specimens have been identified. Table 9 provides details of the 4 available tests for each of which an M-ϕ curve is provided in ref. 50.

Each of the four M-ϕ curves is nonlinear up to a moment which is slightly less than the beam's plastic moment in the case of the shallower beams and which is about 60 per cent of this value for the deeper beams. The minor changes between specimens e.g. use of a larger angle, larger bolt size etc. indicated in Table 8 do not appear to affect stiffness to any significant extent.

Comparing results for the 4 different connection types used in these frame tests suggests a hierarchy (in descending order) of: extended end plate, flush end plate with backing plates, combined web and flange cleats, tee stubs. However, the tee stubs are as stiff initially as the flush end plate, becoming more flexible suddenly at about 30-40 per cent of capacity due to what appears to be some form of slip.

5.10 Tee stubs

Tees made from short lengths of I-section split along the centreline of the web provide a potentially very stiff form of flange cleat. Twenty-seven tests in which this connection was used have been identified, although only in the case of the 16 tests of the two most recent series listed in Table 2 are complete results available. Full details of these tests are provided in Table 10.

The M-ϕ curves for the two test series of refs. 64 and 46, see Fig. 29, exhibit one rather noticeable difference; for the latter some form of slip appears to have occurred at a moment of about 30-40 per cent of the joint's capacity. The absence of this from the tests of ref. 64 may be due to a different bolt tightening procedure but neither author provides sufficient information for this to be properly established. Indeed it is noticeable that the connections of ref. 64 behaved in an almost linear fashion over a significant portion of their range. For the tests of ref. 50 the full plastic moment of the beam was effectively achieved for the shallower sections, whilst for the deeper beams, whether or not the bottom tee was replaced with a seat made from a short length of I-section, the connection appeared incapable of providing this level of moment.

5.11 **Top plate and seat angle**

Connections in which a bottom cleat is used in conjunction with a plate butt welded to the column flange and then fillet welded to the top flange of the beam have been tested by a number of authors as indicated in Table 2 with details of the most recent series being given in Table 11. Unfortunately few of these tests are fully reported in terms of M-ϕ behaviour so little in the way of observations on the rotational stiffness of this connection type is possible.

From the M-ϕ curves provided in ref. 36 it is clear that this form of connection acts rather like a stiffer version of the conventional flange cleat arrangement, maintaining its initial stiffness up to about 50 per cent of its maximum moment. Replacing the top plate with a direct weld between the beam flange and the column further increases the range over which this stiffness is available but drastically reduces ductility, producing a very brittle connection. For the composite versions of the top plate connection the presence of the slab approximately trebled moment capacity, with the initial stiffness of the connection, which was slightly greater than that of the equivalent bare steel connection, being available for about two thirds of the range.

A number of varients of this form of connection, embracing both major and minor axis joints, have been tested by Chen and his associates (8, 9). However, since these operate as fully rigid joints, their behaviour is not considered herein, especially as the published reports do not contain M-ϕ curves which could be used as benchmarks against which to compare data for some of the stiffer forms of "semi-rigid" connection.

5.12 **Tee-stubs and web cleats**

Only one recent investigation (50) has been identified in which combined bolted tee-stubs and web cleats have been tested, although four early studies using either rivets or welds are also shown in Table 2. Only one M-ϕ curve is presented in ref. 50; a complete set of thirteen has been obtained privately. These data show that such connections respond in a nonlinear manner over the whole of the loading range, with the M-ϕ curve being rather more rounded than was the case for the comparison tests on flush end plates, see Fig. 30. Moreover, for the particular geometries tested this form of connection appears slightly stiffer initially as well as being capable of sustaining a higher maximum moment. In virtually every case the plastic moment capacity of the beam was attained. The various changes between tests noted in the last columns of Table 12 produced the following effects:

1. Replacing the tension bolts with a smaller size reduced the moment capacity of the joint as well as causing its stiffness to reduce more rapidly with increasing moment.
2. Use of slotted holes for the web cleats appeared to have little effect on performance, although one test result suggests increased deformation at about 60 per cent of capacity due to slip.
3. The presence of compression and tension stiffeners on the column web ensured the attainment of the beams's M_p; in the case of the deeper beam this was reached with little joint rotation.
4. Reducing either the flange width or the flange thickness of the column decreased connection stiffness and appeared to prevent the plastic moment capacity of the beam from being attained.

Despite the points made above, the overall impression is that changes between individual tests had comparatively little effect on connection performance as measured by its M-ϕ curve.

6. Connections using RHS

Numerous investigations into the behaviour of truss-type connections using tubular members have been reported (70); these are not the concern of the present study. Of direct interest though is the work on beam to column connections in which an I-section beam frames into a rectangular hollow section (RHS) column summarised in Table 2.

In the Canadian tests (71, 72) the connection was made by welding suitably coped strap angles to the side walls of the column and to the flanges of beams of equal width and two thirds of the column width respectively, as shown in Fig. 31. For both series two additional tests on conventional "rigid" connections designed according to current CSA practice using I-section columns were included for comparative purposes. Full M-ϕ curves were provided for each of the 38 tests, some examples are given in Fig. 32. These are nonlinear over the full range and demonstrate the feasibility of such connections as moment connections capable of developing the full plastic moment of the beam. For beams of equal width to the columns connection stiffness was just under half that of the "rigid" connections; for the narrower beams, for which some form of additional web cleat was necessary to transfer the shear, almost the full stiffness was achieved.

A rather different form of connection between an I-section beam and a RHS column has been tested by Echeta and Owens (73) in the context of research on composite frames (74), see Fig. 33. Thus the beam was attached to a bottom cleat welded to the column with Gr. 8.8 bolts (HSFG bolts were originally specified in developing the design approach but were replaced in the test). Small stabilising cleats were used near the top of the beam web. The connection was tested under a combination of high shear and high moment with an axial load in the column of approximately 55 per cent of its capacity with the slab in place. Whilst one beam was flush with the column face, a 2mm gap was left on the other side and this was sufficient to introduce a jump in the M-ϕ curve when slip occurred in the bottom cleat bolts. In both cases the M-ϕ curve was approximately linear up to the estimated moment capacity of the beam, see Fig. 34. This was followed by a substantial knee leading to a long plateau. Ref. 73 makes some interesting comments on the interaction between the top cleats and the concrete slab in transferring tensile forces.

Recently tests on a novel method of connecting I-section beams to RHS columns have been conducted in Liege (75). These used threaded shear connectors, stud welded to the column face to effect a site connection via end plates or cleats attached to the beam end in the fabricating shop. A total of 28 tests were conducted on four connection types: flush end plate, extended end plate, double web cleats and flange cleats. Although no M-ϕ curves were reported, the moment versus deflection data provided enables some idea of the restraint characteristics of this type of joint to be ascertained. These suggest that the behaviour of flush end plates and double web cleats will be similar, with extended end plates and flange cleats having comparable initial stiffness, although the extended end plate is capable of maintaining this stiffness up to rather higher loads.

7. **Other forms of end restraint**

All of the foregoing discussion has been limited to the ability of steel beam to column connections to provide restraint against member end rotations associated with planar behaviour. This is quite sufficient for the consideration of frame behaviour on a two-dimensional basis which is the approach most often used. However, a full three-dimensional treatment requires a wider range of joint characteristics. Very little of this type of information appears to be available, especially for the types of joints considered herein. This is preventing development since methods for the analysis of individual members under biaxial bending, which are capable, in principle, of incorporating all forms of end restraint e.g. torsional, warping, major and minor axis bending, have been available for some time (76, 77). For other forms of framing e.g. scaffold assemblies (78) and offshore platforms (79), using different classes of member and forms of joint, some progress has been made.

On a more limited scale Bennetts et al (80) have provided experimental data on the degree of torsional restraint provided by three types of "simple" connection: double web cleat, web side plate and flexible end plate. Twenty-two tests were conducted in which the twist produced by an applied torque was measured. These produced nonlinear relationships with torsional stiffness decreasing as deformations increased. In some cases local distortion of the beam web was a contributing factor. Relating the results back to the requirements for torsional restraint and torsional strength at the end supports for beams so as to ensure adequate lateral stability according to the Australian code (81), casts some doubt on the classification of these particular forms of connection according to the provisions of the AISC manual on standardised connections (32, 33).

The AISC approach is itself of relevance to the present review since it seeks to link joint restraint and member strength in a quantitative way. A performance requirement is first derived based on considerations of the elastic critical stress σ_{cr} for lateral-torsional buckling of beams with finite end torsional restraint (82, 83). Thus if σ_{cr} exceeds 98 per cent of the value for infinite torsional restraint the connection is considered as torsionally fixed and the beam designed for an effective length equal to its span L. Welded moment connections, extended end plates and bearing pads for which the beam's end plate extends over its full depth fall within this category. Incomplete torsional restraint is considered to be provided for connections for which the beam's elastic critical stress lies between 98 per cent and 85 per cent of the full value. This lower limit of σ_{cr} corresponds approximately to the range of beam strengths for which it is appropriate to use an enhanced effective length of 1.2L in conjunction with the beam design rules of AS1250. Connections in this group include angle seats with web top cleats, bearing pads with part depth beam end plates, flexible end plates, web cleats and web side plates. In the evaluation of the torsional flexibility of each type of connection it is of interest to note that coping of the beam (if present) often represents the major contribution. However, it is within this area of evaluation that the later studies of Bennetts et al (80) suggest the need for some modification.

8. Semi-rigid joint action and frame performance

Although it is apparent from the review of test data relating to connection behaviour presented in the previous sections that most beam to column connections behave in a semi-rigid fashion, it has been the normal practice to design steel frames as either "simple", for which connection stiffness is largely ignored, or "rigid", for which full continuity is assumed. However, dating back to the work of the Steel Structures Research Committee (1), several attempts have been made to introduce a more realistic consideration of the actual behaviour of the joints into the process of design. Indeed, many of the references to experimental investigations of different forms of joint listed in Table 2 have, themselves, included material of this nature. Of particular interest in this context are several of the earlier references (25, 31), in which the concept of beam lines was introduced as a means of understanding the behaviour of beams with limited end restraint.

More recent contributions tend to have approached this problem from the rather different standpoint of attempting to assess the extent to which various degrees of semi-rigid action can be accommodated within the spirit of conventional design approaches. Thus for example the work of Kennedy (40, 41) on flexible connections has addressed the problem of ensuring that the beam's moment capacity is attained in the sagging moment region within its span before the connection develops additional stiffness due to direct bearing of the toe of the beam against the column face. At the other end of the spectrum Surtees and Lucking (84) have compared the requirements of the so-called "Joint Committee" (85) method of design for no-sway rigid frames with the experimentally obtained characteristics of extended end plate connections. Their findings suggest that despite increases in column stresses, especially if significant minor axis loading is present, together with larger beam deflections, careful use of the method should still provide acceptable designs. A useful discussion of the incorporation of semi-rigid joint action into an up-to-date frame design method (86) has recently been provided by Roberts (87) who presented examples which demonstrated useful weight savings compared with equivalent "pin-joint" designs. Moreover, these savings approached those obtained with "rigid" design.

Two aspects of this problem have received particular attention of late : detailed studies of the strength of end-restrained columns and the assessment of the effects of joint flexibility on the behaviour of sway frames. A review of the first of these will be published shortly (88). This summarises a number of theoretical studies, the results of which have been used as the basis for a reinterpretation of the effective length concept, which suggests that proper allowance for the beneficial effect of the end restraint provided by several forms of "flexible" connection can lead to significantly enhanced column strength. A more design-oriented presentation of this material is also available (89).

A similar review of the work on sway frames, summarising the results of several numerical investigations, is also due for publication (90). A recent development in this area, which would appear to be of particular significance to designers, has been the demonstration that standard frame analysis programs (90) may be used to perform analyses which include semi-rigid joint action (92, 93).

9. **Recommendations for further testing**

Clearly a prerequisite for an improved understanding of semi-rigid joint action on structural behaviour is the availability of an adequate body of data on connection performance. In this context the more "flexible" forms of connection e.g. types 1-8 of Table 2, are considered to be the most important, particularly as "simple" construction appears to be the more favoured form in the U.K. The Hatfield survey (14), although selective in the types of joints on which information was requested, confirms the predominance of "simple" connections (apart from portal frame connections). Of the connection types considered in Table 2, numbers 1 and 3 are thought not to be used in this country. Ref. 14 suggested the most popular forms of "simple connection" to be: types 7 and 2 with some use of types 5, 4 and 6; significant use is also made of the rather more rigid type 8.

Reference to Tables 2-8 shows the amount of test data relating to these types to comprise:

Type 2	double web cleats	-	75 tests (31)
Type 4	flange cleats	-	83 tests (51)
Type 5	bottom flange cleat + web cleat	-	0 tests (0)
Type 6	header plate	-	37 tests (30)
Type 7	flush end plate	-	60 tests (54)
Type 8	extended end plate	-	102 tests (53)

in which the figure in brackets refers to the number of tests (broadly representative of current practice) for which M-ϕ curves are available. The most striking feature of this summary is the absence of data for type 5, despite 80 per cent of the respondants to the Hatfield survey claiming to use it at least "sometimes". For each of the other five types, a significant amount of data are available, although the number of variables present in any connection problem is, of course, sufficiently large that anything approaching a full coverage would require very many tests indeed. However, inspection of Tables 3, 5-8 shows that in each case enough tests have been performed for a reasonable indication of the interrelationship of the individual variables present within each type e.g. cleat size, bolt arrangement, column flange thickness, column stiffening etc., on joint stiffness to be extracted. This appears to be particularly the case for the three forms of end plate. For web cleat connections behaviour is complicated by the present of slip at some stage as well as by the strong influence of the exact method of fastening the cleats to both the column face and the beam web on the amount of cleat deformation that takes place. The presence or absence of coping of the beam web is another important factor.

Whilst the available connection test data provide much useful information for studies of semi-rigid action, because many of the tests were not conducted specifically with this aim in mind, several important aspects have received little or no attention. It is therefore desirable that information be obtained on:

1. Connections to beams that frame into the column web - no recent tests of this type have been identified, see Table 2. Particularly in the case of unbalanced connections, deformation of the thin column web may be expected to produce some differences from the response of major-axis connections. This arrangement is particularly important in the context of column behaviour since buckling about the minor axis is the usual form of column failure.

2. Variability in the performance of nominally identical connections - only a limited amount of information is available on the spread of M-ϕ curves likely to be obtained for any particular connection

type e.g. from either arm of a cruciform test, the tests of ref. 37 on flange cleats. Any dependance on semi-rigid action needs to establish the sensitivity of the improved performance to the expected variability of the restraint provided.

3. Connection behaviour under repeated and reversed loading - very few tests have investigated the changes in joint stiffness that take place on reloading. For connection types in which a significant amount of slip, bedding-in of bolts etc., takes place on first loading, behaviour in the second and subsequent applications of load may be expected to be rather different. Reliance upon joint restraint requires that the degree of restraint present after an application of the connection's anticipated working load be known. In the case of columns end rotation will meam that the connection to the beam on one side will be unloading and some indication of the stiffness available at this stage is required.

4. Influence of lack of fit on $M-\phi$ behaviour - although the tests of ref. 45 on flush end plates with fully torqued HSFG bolts suggested little effect in that case, tests on other connection types would be useful. This item is one specific component of point 2 above on variability. Because of the importance of bolt slip, the exact stage at which the beam toe comes into bearing against the column face etc., for the web cleat type of connection, it seems likely that factors such as the use of oversize holes will have a significant effect on the stiffness of this form of connection.

5. Tests on bottom flange cleat plus web cleat(s) arrangements should be conducted - no tests on this form of connection have been identified despite its seemingly quite frequent use (14). It is

surprising that no data could be found since it leads inevitably to the question "How sound are present design approaches for basic connection strength if nothing exists against which they may be checked?" It is suggested that the usage of this type of connection be more fully investigated and if the Hatfield findings are confirmed then a representative test series be conducted. This test series would, of course, also provide the missing M-ϕ data for this type of connection.

6. Performance of connections in which both major and minor-axis beams are present - no tests have been identified on three-dimensional arrangements. It is not known to what extent direct bending of the column web by connection to minor-axis beams would degrade the performance of the connection(s) on the column flange(s) and vice versa.

7. Restraint characteristics other than in-plane rotational restraint - apart from the tests of ref. 80 on torsional restraint no work has been identified on the level of restraint provided against types of deformation other than in-plane bending e.g. torsion, warping and out-of-plane bending. Since structures are essentially three-dimensional, even if most analysis and design is based on two-dimensional representation, all types of restraining action at joints should eventually be taken into account. Even the limited study of ref. 80 has revealed the importance of giving proper consideration to the exact capabilities of connections. Already three-dimensional approaches to the analysis of individual members e.g. columns, are available; the appropriate joint data needs to be produced if these are to be properly used to assess the strength of members as parts of structures (rather than continue to work in terms of "ideal" end conditions).

10. Conclusions

A review has been made of over 700 full-scale tests on steel beam to column connections, embracing 13 different classes of joint. It has concentrated on the future application of these data to an improved understanding of the role of connection behaviour on the performance of steel frame structures. Thus the semi-rigid nature of all common forms of connection has been emphasized, with the connection's moment-rotation (M-ϕ) curve being identified as its most important single characteristic. Approximately half the tests provide this type of information. Section 9 has attempted to identify areas in which additional testing is desirable and the particular aspects of connection behaviour that require further consideration.

11. **Acknowledgements**

Much of the information used to compile this review has been supplied by the original investigators in response to requests by the author. To each of those who assisted in this way the author wishes to record his thanks. Without their contributions it would not have been possible to produce the present document.

REFERENCES

1. STEEL STRUCTURES RESEARCH COMMITTEE
First, Second and Third Reports, Department of Scientific and Industrial Research, HMSO, London, 1931, 1934 and 1936.

2. BRITISH STANDARDS INSTITUTION
Recommendations for design, Supplement No. 1 to BS 449. PD3343, 1959.

3. JONES, S.W., KIRBY, P.A. and NETHERCOT, D.A.
The analysis of frames with semi-rigid connections - a state-of-the-art-report.
Journal of Constructional Steelwork Research, Vol. 3, No. 2, 1983, 2 to 13.

4. CHEAL, B.D.
Limit state design of bolted connections
CIRIA Research Project 306, October 1983.

5. HORNE, M.R. and MORRIS, L.J.
Plastic design of low rise frames
Granada, London, 1981.

6. FISHER, J.W. and STRUIK, J.H.A.
Guide to design criteria for rivetted and bolted joints
John Wiley and Sons, New York, 1974.

7. AMERICAN SOCIETY OF CIVIL ENGINEERS
Plastic design in steel - a guide and commentary
Manual No. 41, ASCE, 1971.

8. CHEN, W.F. and LUI, E.
Static web moment connections
Chapter in forthcoming ASCE Monograph, Edited by W F Chen.

9. CHEN, W.F. and LUI, E.
Flange moment connections
Chapter in forthcoming ASCE Monograph, Edited by W F Chen.

10. KATO, B.
Beam to column connection research in Japan
Proc. Am. Soc. Civ. Engrs., J.Struct.Div., Vol. 108, No. ST2, February 1982, 343 to 360.

11. KRAWINKLER, H. and POPOV, E. P.
Seismic beahviour of moment connections and joints
Proc. Am. Soc. Civ. Engrs., J.Struct.Div., Vol. 108, No. ST2, February 1982, 373 to 391.

12. HENDRY, A.W.
An investigation of the strength of welded portal frame connections
Structural Engineer, Vol. 28, No. 10, 1950, 265 to 280.

13. BEEDLE, L.S.
Plastic design of steel frames
John Wiley & Sons, New York, 1958.

14. JENKINS, W.M.
Local failures in steelwork structures
BRE Contract F3/2/256 Interim Report, October 1983.

15. YOUNG, C.R. and JACKSON, K.B.
The relative rigidity of welded and rivetted connections
Canadian Journal of Research, Vol. 11, No. 1, 1934, 62 to 100 and Vol. 11, No. 2, 1934, 101 to 134.

16. RATHBUN, J.C.
Elastic properties of rivetted connections
Trans. Am. Soc. Civ. Engrs., Vol.101, 1936, 524 to 563.

17. LIPSON, S.L.
Single angle and single plate beam framining connections
Canadian Structural Engineering Conference, Toronto, February 1968, 141 to 162.

18. LIPSON, S.L.
Single angle welded bolted connections
Proc. Am. Soc. Civ. Engrs., J.Struct.Div., Vol. 193, No. ST3, March 1977, 559 to 571.

19. LIPSON, S.L. and ANTONIO, M.
Single-angle welded-bolted beam connections
Can. J. Civ.Eng., Vol. 7, 1980, 315 to 324.

20. BIRKEMOE, P.C. and GILMOR, M.I.
Behaviour of bearing critical double angle beam connections
American Institute of Steel Construction Engineering Journal, Fourth quarter, 1978, 109 to 115.

21. YURA, J.A., BIRKEMOE, P.C. and RICLES, J.M.
Beam web shear connections: an experimental study
Proc. Am. Soc. Civ. Engrs., J.Struct.Div., Vol. 108, No. ST2, February 1982, 311 to 325.

22. RICLES, J.M. and YURA, J.A.
Strength of double row bolted web connections
Proc. Am. Soc. Civ. Engrs., J.Struct.Eng., Vol. 109, No. 1, January 1983, 126 to 142.

23. WING, P.C. and HARRIS, P.J.
Eccentrically loaded bolted beam connections
Canadian Journal of Civil Engineering, Vol. 10, 1983, 295 to 305.

24. MUNSE, W.H., BELL, W.G. and CHESSON, E.
Behaviour of rivetted and bolted beam to column connections
Proc. Am. Soc. Civ. Engrs., J.Struct.Div., Vol. 85, No. ST3, March 1959, 25 to 50.

25. LEWITT, C.W., CHESSON, E. and MUNSE, W.H.
Restraint characteristics of flexible and bolted beam to column connections
University of Illinois, Engineering Experiment Station Bulletin No. 500, January 1969.

26. BERGQUIST, D.J.
Tests on columns restrained by beams with simple connections
Report No. 1, American Iron and Steel Institute Project No. 189, Department of Civil Engineering, Univesity of Texas at Austin, USA, January 1977.

27. BENNETTS, I.D., THOMAS, I.R. and GRUNDY, P.
Shear connections for beams to columns
Metal Structures Conference, Institution of Engineers, Australia, Perth, November 1978, 70 to 75.

28. MANSELL, D.S and PHAM, L.
Testing of standardised connections
Metal Structures Conference, Institution of Engineers, Australia, Newcastle, May 1981, 107 to 112.

29. BOSE, B.
Moment rotation characteristics of semi-rigid joints in steel structures
Journal of Institution of Engineers (India), Part CI, Civil Engineering Division, Vol. 62, Part 2, September 1981, 128 to 132.

30. BJORHOVDE, R.
Private communication.

31. HECHTMAN, R.A. and JOHNSTON, B.G.
Rivetted semi rigid beam to column building connections
Progress Report No. 1, Committee of Steel Structures Research, AISC, November 1947.

32. HOGAN, T.J. and FERKINS, A.
Standardised structural connections
Part A: Details and Design Capacities
Australian Institute of Steel Construction, 2nd Edition, October 1981.

33. HOGAN, T.J. and THOMAS, I.R.
Standardised structural connections
Part B: Design of Structural Connections
Australian Institute of Steel Construction, December 1981.

34. PHAM, L. and MANSELL, D.S.
Testing of standardised connections AWRA Contract 76, Australian Welding Research, December, 1982, 15 to 22.

35. MAXWELL, S.M. et al.
Realistic approach to the performance and application of semi rigid joints in steel structures, joints in structural steelwork Edited by J H Howlett, W M Jenkins and R Stainsby, Pentech Press, London, 1981 2.71 to 2.98.

36. VAN DALEN, K. and GODOY, H.
Strength and rotational behaviour of composite beam-column connections
Canadian Journal of Civil Engineering, Vol. 9, 1982, 313 to 322.

37. HOTZ, R.
Traglastversuche Fur Stutzen-Riegel-Verbindungen mit verbesserter Wirtschaftlichkeit
Stahlbau, Vol. 11, November 1983, 329 to 334. (supplemented by private communication).

38. Joints in structural steelwork
Vol. 2, Discussion, Edited by J H Howlett and W M Jenkins, Pentech Press, London, 1983, 87 to 113.

39. SOMMER, W.H.
Behaviour of welded header plate connections
Master of Applied Science Thesis, University of Toronto, January 1969.

40. KENNEDY, D.J.L.
Moment rotation characteristics of shear connections
American Institute of Steel Construction Engineering Journal, October 1969, 105 to 115.

41. KENNEDY, D.J.L. and HAFEZ, M.A.
A study of end plate connections for steel beams
Canadian Journal of Civil Engineering (in press).

42. FRYE, M.J. and MORRIS, G.A.
Analysis of flexibly connected steel frames
Can. J. of Civ. Eng., Vol. 2, No. 3, September 1975, 280 to 291.

43. ZOETEMEIJER, P.
Semi rigid bolted beam to column connections with stiffened column flanges and flush end plates
Joints in Structural Steelwork, Edited by J H Howlett, W M Jenkins and R Stainsby, Pentech Press, 1981, 2.99 to 2.118. (Supplemented by private communication).

44. MANN, A.P. and MORRIS, L.J.
Significance of lack of fit - flush beam-column connections
Joints in Structural Steelwork, Edited by J H Howlett, W M Jenkins and R Stainsby, Pentech Press, Additional Paper, 1081, 622 to 636.

45. MANN, A.P. and MORRIS, L.J.
Lack of fit in steel structures
CIRIA Report 87, 1981.

46. PHILLIPS, J. and PACKER, J.A.
The effect of plate thickness on flush end plate connections
Joints in Structural Steelwork, Edited by J H Howlett, W M Jenkins and R Stainsby, Pentech Press, 1981, Supplementary Paper.

47. ZOETEMEIJER, P. and KOLSTEIN, M.H.
Flush end plate connections
Stevin Laboratory Report 6-75-20, Delft University of Technology, 1975.

48. MORRIS, L.J. and NEWSOME, C.P.
Bolted corner connection subjected to an outer balance moment - the behaviour of the column web panel
Joints in Structural Steelwork, Edited by J H Howlett, W M Jenkins and R Stainsby, Pentech Press, Supplementary Volume, 1981, 6.3 to 6.21.

49. JENKINS, W.M. et al.
Beam-column connections
Current British Practice and Future Developments, Conference on Structural Steelwork, Singapore, 1984.

50. ZOETEMEIJER, P.
A design method for the tension side of statically loaded beam to column connections
Heron, Vol. 20, 1974, No. 1.

51. SHERBORNE, A.N.
Bolted beam to column connections
The Structural Engineer, Vol. 91, No. 6, June 1961, 203 to 209.

52. JOHNSON, L.G., CANNON, J.C. and SPOONER, L.A.
High tensile preloaded bolted joints
British Welding Journal, Vol. 7, No. 9, September 1960, 560 to 569.

53. BAILEY, J.R.
Strength and rigidity of bolted beam to column connections
Conference on Joints in Structures, Paper A4, University of Sheffield, UK, 1970.

54. SURTEES, J.O. and MANN, A.P.
End plate connections in plastically designed structures
Conference on Joints in Structures, University of Sheffield, UK, 1970.

55. GRUNDY, P., THOMAS, I.R. and BENNETTS, I.D.
Beam to column moment connections
Proc. Am. Soc. Civ. Engrs., J.Struct.Div., Vol. 106, No. ST1, January 1980, 313 to 330.

56. PACKER, J.A. and MORRIS, L.J.
A limit state design method for the tension region of bolted beam to column connections
The Structural Engineer, Vol. 55, No. 10, October 1977, 446 to 458.

57. TARPY, T.S. and CARDINAL, J.W.
Behaviour of semi rigid beam to column end plate connections
Joints in Structural Steelwork, Edited by J H Howlett, W M Jenkins and R Stainsby, Pentech Press, 1981, 2.3 to 2.25.

58. BAHIA, C.S., GRAHAM, J. and MARTIN, L.H.
Experiments on rigid beam to column connections subjected to shear and bending forces
Joints in Structural Steelwork, Edited by J H Howlett, W M Jenkins and R Stainsby, Pentech Press, Additonal Papers, 6.37 to 6.56.

59. GRAHAM, J.
Beam to column bolted connections
Ph.D Thesis, University of Aston in Birmingham, 1981.

60. MOORE, D.B. and SIMS. P.A.C.
The influence of backing plates on the behaviour of extended end plate connections
Building Research Establishment Note No. 179/83, December 1983.

61. ZOETEMEIJER, P. and MUNTER, H.
Influence of an axial load in the column on the behaviour of an unstiffened beam to column end plate connection
Stevin Laboratory Report No. 6-84-1, Delft University of Technology, January 1984.

62. ZOETEMEIJER, P.
Bolted beam to column knee connections with haunched beams Tests and computations
Stevin Laboatory Report No. 6-81-23, Delft University of Technology, December 1981.

63. ZOETEMEIJER, P.
Bolted connections with flush end plates and haunched beams Tests and limit states design methods
Stevin Laboratory Report No. 6-81-15, Delft University of Technology, June 1981.

64. DOUTY, R.T.
Strength characteristics of high strength bolted connections with particular application to the plastic design of steel structures
Ph.D Thesis, Cornell University, USA, 1964.

65. BANISTER, A.
Moment-angle change relationships for certain connections incorporating friction grip bolts
Civil Engineering and Public Works Review, Vol. 61, June 1966, 755 to 758, July 1966, 873 to 878.

66. JOHNSTON, B.G. and DEITS, G.R.
Tests of miscellaneous welded building connections
The Welding Journal, Vol. 21, Jan. 1942, 5 to 27.

67. BRANDES, J.L. and MAINS, R.M.
Report of tests of welded top-plate and seat building connections
Welding Journal, March 1944, 146 to 165.

68. PRAY, R.F. and JENSEN, C.
Welded top plate beam-column connections
Welding Journal-Welding Research Supplement, Vol. 35, July 1956, 338 to 347.

69. JOHNSON, L.G.
Tests on welded connections between I-section beams and stanchions
Britsh Welding Journal, Vol. 6, Jan. 1959, 38 to 46.

70. HOWLETT, J.H., JENKINS, W.M. and STAINSBY, R. eds.
Joints in structural steelwork
Pentech Press, 1981, papers 23-27, 4.3 to 4.98.

71. PICARD. A, and GIROUX, Y.M.
Moment connections between wide flanged beams and square tubular columns
Canadian Journal of Civil Engineering, Vol. 3, No. 2, 1976, 174 to 185.

72. GIROUX, Y.M. and PICARD, A
Rigid framining connections for tubular columns
Canadian Journal of Civil Engineering, Vol. 4, No. 2, 1977, 134 to 144.

73. ECHETA, C.B. and OWENS, G.W.
A semi rigid connection for composite frames - initial test results
Joints in Structural Steelwork, Edited by J H Howlett, W M Jenkins and R Stainsby, Pentech Press, 1981, Supplementary Paper.

74. OWENS, G.W. and ECHETA, C.B.
A semi rigid design method for composite frames
Joints in Structural Steelwork, Edited by J H Howlett, W M Jenkins and R Stainsby, Pentech Press, 1981, 3.20 to 3.38.

75. MAQUOI, R., NAVEAU, X. and RONDAL, J.
Beam-column welded steel connections
Journal of Constructional Steel Research, Vol. 4, 1984, 3 to 26.

76. VINNAKOTA, S. and AOSHIMA, Y.
Inelastic behaviour of rotationally restrained columns under biaxial bending
The Structural Engineer, Vol. 52, No. 7, July 1974, 245 to 255.

77. CHEN, W.F. and ATSUTA, T.
Theory of beam columns
Vols. 1 and 2, McGraw Hill, New York, 1976 and 1977.

78. LIGHTFOOT, E. and LE MESSURIER, A.
Instability of space frames having elastically connected and offset members
Second International Conference on Space Strucutres, edited by W J Supple, Sept. 1975, 143 to 149.

79. CIRIA
Node flexibility and its effect on jacket structures - a pilot study on two-dimensional frames
UEG Report UR 22, CIRIA, London, 1984.

80. BENNETTS, I.D., THOMAS, I.R. and GRUNDY, P.
Torsional stiffness of some standard shear connections
Civil Engineering Transactions, The Institution of Engineers, Australia, 1982, 254 to 259.

81. STANDARDS ASSOCIATION OF AUSTRALIA
SAA steel structures code
AS1250, 1981.

82. TRAHAIR, N.S.
Buckling of beams in steel frame structures
University of Sydney, Postgraduate course on the Stability of Steel Frame Structures, School of Civil Engineering, November, 1970, Lecture 10, Vol. 3

83. FLINT, A.R.
The influence of restraints on the stability of beams
The Structural Engineer, Vol. 29, No. 9, Sept. 1951, 235.

84. SURTEES, J.O. and LUCKING, J.C.
Influence of joint characteristics on the performance of braced multi storey frames designed on the basis of full continuity between members
Joints in Structural Steelwork, Edited by J H Howlett, W M Jenkins and R Stainsby, Pentech Press, 1981, 5.3 to 5.17

85. INSTITUTION OF STRUCTURAL ENGINEERS AND INSTITUTE OF WELDING JOINT COMMITTEE
Second Report on Fully Rigid Multistorey Steel Frames, London, 1971.

86. WOOD, R.H.
A new approach to column design
HMSO, London, 1974.

87. ROBERTS, E.H.
Semi-rigid design using the variable stiffness method of column design
Joints in Structural Steelwork, Edited by J H Howlett, M W Jenkins and R Stainsby, Pentech Press, 1981, 5.36 to 5.49.

88. NETHERCOT, D.A. and CHEN, W.F.
Effect of connections on columns
ASCE Monograph on Steel Beam to Column Connections, Edited by W F Chen, in press.

89. BJORHOVDE, R.
Effect of end restraint on column strength - practical applications
AISC National Engineering Conference, Tennessee, 1983.

90. GERSTLE, K.
Effect of connections on frame strength
ASCE Monograph on Steel Beam to Column Connections, Edited by W F Chen, in press.

91. DE SALVO, G.J. and SWANSON, J.A.
ANSYS engineering analysis systems
Users Manual, Vol. 1 and 2, Swanson Analysis Systems Inc., Houston, March 1983.

92. EDGINGER, J.A.
Nonlinear frame analysis using the ANSYS general purpose computer program
AISC Research Report No. 83-1, American Institute of Steel Construction, Chicago, July 1983.

93. EDGINGER, J.A.
Multi storey frame analysis using the ANSYS general purpose computer program
AISC Research Report No. 84-1, American Institute of Steel Construction, Chicago, January 1984.