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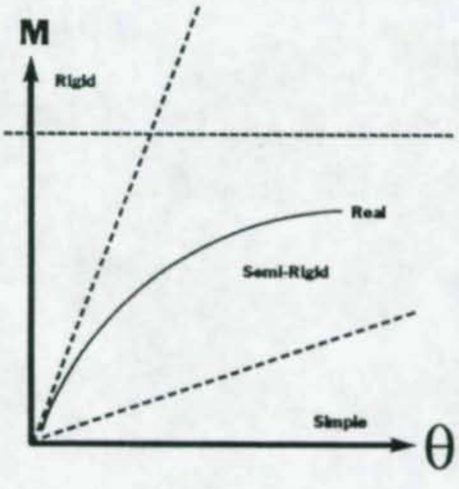
Connections in Steel Structures II:

Behavior, Strength, and Design



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AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.

CONNECTIONS IN STEEL STRUCTURES II:

Behavior, Strength, and Design

Proceedings of the Second International Workshop

held at

Westin William Penn Hotel
Pittsburgh, Pennsylvania, USA
April 10–12, 1991

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FOREWORD

This book is the Proceedings of the ***Second International Workshop on Connections in Steel Structures: Behavior, Strength, and Design***, held at the Westin William Penn Hotel in Pittsburgh, Pennsylvania, USA, during the period April 10 to 12, 1991, under the auspices of the Department of Civil Engineering of the University of Pittsburgh. The First International Workshop was held at École Normale Supérieure de Cachan, in Cachan, France, on May 25 to 27, 1987, and its Proceedings was published by Elsevier Applied Science Publishers in February, 1988.

The workshop organizers wish to express their sincere thanks to the organizations that made the workshop possible through financial support. Thus, the keen interest of the National Science Foundation of the United States, the American Institute of Steel Construction, and the Commission of European Communities, through the European Convention for Constructional Steelwork, is much appreciated.

The workshop was supported through Grant No. 9114319 of the National Science Foundation, under the program on Structures, Geomechanics, and Building Systems of the Directorate of Engineering. Dr. K. P. Chong and Dr. J. B. Scalzi are the Program Directors.

Financial assistance was also provided by a grant from the American Institute of Steel Construction, Inc., through its Engineering Department research and development program. Mr. N. W. Zundel is the President of AISC. AISC arranged a meeting of its committee on Manuals and Textbooks in conjunction with the workshop; the committee is chaired by Dr. W. A. Thornton of the Cives Engineering Corporation.

Financial support was also extended from the Commission of European Communities (CEC) through the European Convention for Constructional Steelwork, in the form of travel and other contributions towards the participation of representatives of the CEC member countries. In addition to providing key input to the workshop in the form of papers and discussions, the members of ECCS Technical Committee 10: Connections, held a meeting in advance of the workshop, making it the first time for a European committee to meet outside of Europe. The Chairman of ECCS TC 10 is Jan W. B. Stark; Mr. J. Van Neste is the General Secretary of ECCS.

The AISC Committee on Manuals and Textbooks and the ECCS Technical Committee 10 convened for a joint meeting and dinner on the day before the opening of the workshop, to provide a forum for the exchange of ideas, experiences, and new knowledge. The meeting was co-chaired by Messrs. G. Haaijer and J. W. B. Stark.

The Department of Civil Engineering of the University of Pittsburgh was the official host of the workshop, through its Chairman, Dr. Reidar Bjorhovde. Extensive logistical and other support was provided by the department, in particular through the work of Mrs. Rae Sutermaster and graduate students Jeffrey Castello and Sherif Morcos.

A number of the workshop participants served as technical session chairmen and reporters. In particular, the efforts of Messrs. R. L. Brockenbrough of the USS Division, USX Corporation; N. Iwankiw, American Institute of Steel Construction; and W. McGuire, Cornell University, who accepted the demanding assignments as Research Reporters for the workshop, are acknowledged.

The support and technical contributions of the participants, without which the workshop would not have been possible, are sincerely acknowledged and appreciated. It is hoped that the kind of international cooperation that was commenced with the First and carried on with the Second International Workshop will continue to enhance research and development efforts in steel structures worldwide.

Pittsburgh, Pennsylvania, USA
December, 1991

Reidar Bjorhovde
André Colson
Geerhard Haaijer
Jan W. B. Stark

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The success of the First International Workshop on Connections in Steel Structures provided the impetus for the Second. The 1987 event was conceived by Messrs. Bjorhovde, Brozzetti and Colson, and following the invitation of 40 internationally recognized research and design experts in the area of steel structures and connections, 37 individuals accepted the challenge to present state-of-the-art reports on this important specialty of the structural engineering profession. Representing 15 countries, these researchers and their colleagues presented a total of 41 papers, co-authored by 76 individual authors. The Proceedings of the 1987 workshop, which was held in Cachan, near Paris, France, documents the technical papers as well as the discussions and the research needs that presented themselves at the time.

Preparing for the second workshop, it was decided to retain the format of attendance by invitation only. The original rationale was that restricted participation would allow for the most advanced topics to be considered, without the need to bring everyone to a common base of knowledge, so to speak. In other words, by gathering experts only, it was felt that it would be possible to move the farthest and fastest in the assessments of ongoing research, developments, and research needs. The first workshop proved this format to be an unqualified success, so much so that since then, a number of restricted attendance workshops have been held, dealing with many and diverse subject areas.

A total of 65 internationally recognized experts were invited to participate in the second workshop, and 60 accepted the challenge, representing 19 countries. 50 papers with 77 authors and co-authors were submitted; these represent the primary contents of this book.

In the initial planning of the workshop program, some of the subject areas addressed in 1987 obviously had such broad interest that it would be important to consider what further progress had been made. Thus, sessions on bolts and welds were selected. Similarly, the topic of semi-rigid connections had assumed increasing prominence, in particular through work on design tools and code criteria. As a result, two sessions on semi-rigid connections were selected, along with one each on actual frame design solutions and the impact on construction economy, and one dealing with computer software. Much work appeared to have been done on the development of predesigned connections; a session was therefore assembled to illustrate some of the practical uses of such solutions. Similarly, much research on composite construction identified the need to deal with this entire subject area, especially to be able to assess or quantify the stiffness and strength characteristics of composite elements.

Although not presented in this book, in the form of written contributions, a special computer program demonstration session was arranged for the evening of April 11. Using the CAD laboratory of the School of Engineering at the University of Pittsburgh, a total of seven connection and frame analysis and design programs were presented. The session was arranged to allow for interaction between the program authors and the attendees, using a variety of practical examples. Special thanks for their contributions are due Messrs. Brozzetti and Colson of France; Messrs. Gross, Deierlein, McGuire and Murray of the United States; Mr. Steenhuis of The Netherlands; and Dr. Zandonini of Italy. It is clear that this is one area where significant progress can be made in transmitting state-of-the-art information to working designers and fabricators.

As was done for the first workshop, Research Reporters monitored all technical sessions for suggestions for needed research and development. The topics that were identified are presented in the last section of this book; they indicate the state-of-the-art as well as the broader outlook on the future of steel construction, considering connections as key elements. The frame design and economy of construction sessions give evidence that significant practical improvements are at hand; it behooves the profession to take advantage of the wide range of ideas and tools that are being developed.

Technical Papers on

BOLTS

DIFFERENCES BETWEEN EUROPEAN AND AMERICAN DESIGN RULES

Colin Taylor[1]

Abstract

Four key differences between AISC LRFD design rules and Eurocode 3 : Part 1 are highlighted. Reasons for differences are examined and suggestions are made for resolving them.

1. INTRODUCTION

The aim in this paper is not to compare in detail, but rather to highlight significant differences between Eurocode 3 and AISC's LRFD specification, as far as connections are concerned. Overall both codes multiply characteristic loads by factors to produce factored loads (AISC) or design loads (EC3). Design resistances in EC3 are obtained ISO style, by dividing by factors greater than unity, whereas AISC uses LRFD format in which resistances are multiplied by factors less than unity, with similar effect. Fundamental differences thus occur only where resistance expressions have a different basic format. This occurs for:

- loss of preload due to external tension
- resistance of long joints
- transverse fillet welds

Another significant feature of EC3 is the inclusion of rules for semi-rigid and partial strength connections.

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[1]The Steel Construction Institute

2. CONNECTION DESIGN

Eurocode 3 differentiates between classification of connections on the basis of:

- rigidity (rigid, semi-rigid, pinned)
- strength (full strength, partial strength, pinned)

For consistency between global analysis and joint design, when the analysis is elastic, it is rigidity of connections that matters. For simple plastic (rigid plastic) design it is strength of connections that counts. For elastic-plastic analyses both need consideration.

A typical moment-rotation curve for a connection is described by three parameters:

- moment resistance (strength)
- rotational stiffness (rigidity)
- rotation capacity (suitability as a plastic hinge)

Rules are given for welded beam-to-column connections, stiffened and unstiffened, including supplementary web plates (reinforced webs). Rules are also given for bolted beam-to-column connections with flush end plates and unstiffened extended end plates, including the effects of backing plates. These include rules for allowing for prying by analogy with equivalent lengths of Tee connections and also for balancing of forces at the interface and redistribution of resistance within connections. Use is made of itemised procedures which facilitate programming as well as hand calculation.

The basis of the method for design of bolted connections has been fully described elsewhere and will not be repeated here.

3. LOSS OF PRELOAD DUE TO EXTERNAL TENSION

In AISC codes the interaction of shear and tension in a slip-resistant connection is treated as linear, on the basis that slip resistance reduces to zero when the external tension equals the initial preload.

However EC3 follows the recommendation of ECCS by allowing for the effects of differential strain, as follows. When the bolt is preloaded, it compresses a zone of the connected plies adjacent to the bolt, so the nut travels down to the shank by a distance equal to the elongation of the grip length of the bolt, plus the contraction of the connected plies. To fully overcome the clamping effect between the plies it is therefore necessary to apply sufficient external tension to counteract the sum of these two strains, by applying a force greater than the initial preload.

The amount of strain in the connected plies depends on the extent of the compressed region. The ECCS recommendation assumes a cylindrical annulus with an effective cross sectional area equal to 4 times that of the bolt, so that when the external tension equals the preload, a residual clamping force of 20% remains to resist slip.

This area appears to be a completely arbitrary value and there is a long-standing difference of opinion about this. The alternative view is that the area of the compressed zone is a function of the grip length, and in practical cases is so much larger that the residual clamping force may be as low as 5% and is best neglected. This seems to be the basis of the AISC rule. It is also the general view of British experts.

No reports can be traced giving quantified support to either view, either from practical tests or from numerical analyses. However a study is available of a very similar condition using finite element analysis and it would appear to be possible to extend the same technique to the present issue.

4. RESISTANCE OF LONG JOINTS

There are two important differences in the treatment of long bolted joints between EC3 and AISC. First, the AISC rule is a step function, whereas EC3 follows the ECCS recommendations and has a plateau followed by a linear reduction. Second the AISC rule is independent of bolt diameter, whereas the EC3 rule is directly proportional to bolt diameter.

The origin of the AISC rule is summarised Fisher and Struik. It is clear that the step function and the linear reduction are two alternative simplifications of a falling curve of reducing slope. The question of bolt diameter is however more fundamental.

A number of reports and papers are referenced by Fisher and Struik. In these the majority of the tests were on one specific size, 7/8 inch. However it is clear that other sizes were also tested and that the effects of bolt diameter were considered in the associated numerical studies. It is concluded that independence of bolt diameter in AISC specifications is intentional. It is not known why the ECCS recommendations were made proportional to bolt size. It is of interest to note that the new Australian code has examined the point (as discussed in the associated commentary) and concluded that the ECCS form of curve should be used, but made independent of bolt size.

5. TRANSVERSE FILLET WELDS

In AISC LRFD no distinction is made between the resistance of fillet welds depending on their orientation relative to the direction of force transfer. This method is also permitted in EC3, but an alternative method is given which leads to a resistance 22% greater for transverse fillets.

This model however also leads to a diference in strength of 73% between fillet welds in pure tension and pure shear, compared to an experimental difference of only 33%, so a modified relationship is needed. The recent Canadian work may prove useful in this.

The EC3 rules are also less adaptable to non-right angled fillets and fillets with unequal legs.

6. INSTALLATION OF HIGH STRENGTH BOLTS

The differences in installation do not warrant a separate paper. Essentially EC3 permits all the same techniques as in American codes. It also features the Combined method in which a bolt is first torqued to a predetermined value less than the specified preload, then a fixed part turn is also applied. The reliability of this is better than either torque control or simple part-turn.

The real differences are that all techniques are intended to give the same preload and the proposed technique should be checked to ensure it can reach the required preload without eroding the margin before the torque-preload curve peaks. This also gives a margin of safety against breaking the bolt.

The significance of the relation to bolt size can be seen by considering a case where a joint is "long" and it is desired to avoid this. According to the AISC rule, a change to a higher strength grade of bolt, giving a similar number of bolt rows but a smaller bolt diameter, would be a valid solution because the bolt rows could be closer together, thereby reducing the joint length. However the ECCS rule would not allow this because the maximum length for a "short" joint would also be reduced.

The research shows the number of rows to be of little relevance (so long as there are more than two rows) so for a simple examination consider three rows. It is assumed in allocating forces that each row of bolts transfers the same proportion of the total. It is when this assumption is not possible (without initiating a "progressive" failure starting from the most heavily loaded rows) that the joint becomes classified as "long", but when there are only two rows both are similarly loaded anyway.

Relating the bolt forces to those in the lapping plies, it can be seen there is a differential strain problem between the plates, because the force in one ply is double that in the other. This differential strain is proportional to the length of the joint. It is also proportional to the maximum stress in the plies and therefore indirectly to the strength grade of the material.

For a joint to function as assumed for a short joint, the differential strains must be taken up in localised deformations of the bolts and the bolt holes due to bearing pressures.

The relationship of this deformation to bolt size can be deduced by considering dimensional similarity. Imagine a comparison of two bolt grades, one four times the strength of the other. The required bolt size for the stronger would be half that for the other with all dimensions proportionate, including thickness of material, for similar shear stress in the bolts the deformation for the smaller size would logically be half that for the larger size. With similar thickness material in each case, the ratio of deformation would drop to a quarter, but then allowing for the higher bolt stress the deformations would be the same in each case, so deformation is effectively independent of bolt size.

This simple review thus indicates that the "long joint" rule should be independent of bolt diameter. However the strength of the material should be included and possibly the strength of the bolts. It is likely that other geometrical parameters are also relevant. A full numerical study is being carried out at RWTH Aachen.

VERIFICATION OF QUALITY ASSURANCE ON EUROPEAN 4.6- AND 8.8-BOLTS

Herbert Schmidt[1]

Abstract

The factual level of quality assurance on the European market for ordinary hexagon head bolts with medium thread lengths or threads up to the head has been verified by collecting an extraordinarily large random sample of bolts with strength grades 4.6 and 8.8 and determining their properties in view of the statistically based design rules for structural bolted connections under static loading as specified in the new European codes.

1. INTRODUCTION

Eurocode 3 for the design of steel structures, as well as the new German design code DIN 18800 (11/1990), allow that bolts in shear/bearing type connections may extend with their threaded portion into the shear plane. That has not been allowed under the recent versions of DIN 18800. Therefore, the experience of German steel fabricators concerning the quality of structural bolts referred exclusively to the approved special "constructional steelwork bolts" with short thread length according to DIN 7990 (property classes 4.6 and 5.6, normal width across flats) and according to DIN 6914 (property class 10.9, large width across flats). The new more liberal regulation means that in future ordinary hexagon head bolts with medium thread length according to DIN 601/ISO 4016 (property class 4.6) and DIN 931/ISO 4014 (property classes 5.6, 8.8, 10.9) and with threads up to the head according to DIN 558/ISO 4018 (property class 4.6) and DIN 933/ISO 4017 (property classes 5.6, 8.8, 10.9) will be used for structural bolting in Germany in increasing numbers.

The quality assurance of bolts, being a typical industrial mass product, lies in the own responsibility of the manufacturing industry. The number and (possibly) the qualitative variety of manufacturers of ordinary hexagon head bolts is, because of the much larger market, considerably larger than that of the mentioned special German "constructional steelwork bolts". On the other hand, the two new design codes lead to a generally

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higher utilization of bolts under both, tensile and shear/bearing loading. This overall situation made it desirable to check the factual level of the quality assurance situation on the European bolt market with regard to the properties which are relevant for the ultimate limit state design of structural bolted connections under static loading.

The investigation has been carried out in 1989, its results have been documented in a research report (Knobloch and Schmidt, 1990).

2. RANDOM SAMPLE OF BOLTS

As typical bolt types of expected common use in the above mentioned sense, the following two types were chosen:
- M16-4.6 DIN 558 or DIN 601 respectively,
- M16-8.8 DIN 931 or DIN 933 respectively.

Especially the 8.8-bolts, until now not being used for structural bolting in Germany, are supposed to spread rapidly.

A sample unit existed of four bolts stemming from one production batch. The sample units were bought anonymously in bolt trading houses (wholesale or retail) in 10 selected industrial areas of West Germany and the neighbouring European countries. The bolts were defined to be acceptable for structural use if they had, according to ISO 898 part 1, markings of the property class and the manufacturer on their head. It was made sure as good as possible, that no production batch was included twice. Thus, the whole sample may be looked at as a random sample of statistically independent bolt units.

192 units M16-8.8 and 75 units M16-4.6 were included in the investigation. They showed 45 different manufacturer markings, of which nine could not be identified by means of the available lists (N.N., 1983/1988/1988). Nevertheless the 22 units showing these not identifyable markings were also included in order to make the investigation as realistic as possible. The identified markings represent nine European coutries. About 50% of the bolts were black or galvanized respectively. The units included bolt lengths from 35 mm up to 160 mm, with 40% of the 4.6-bolts and 55% of the 8.8-bolts having threads up to the head.

3. THREAD GEOMETRY

The outside diameters d and root diameters d_3 of all bolts were measured, and average stress areas for each sample unit were determined from

$$A_S = \pi/4 \cdot (0{,}75\ d_3 + 0{,}25\ d)^2. \qquad (1)$$

This equation is geometrically identical with the one given in ISO 898/1 where the thread-pitch diameter d_2 is used as second defining diameter (instead of d). Figure 1 shows the histograms of the A_S-values, together with their average values MV and their 5% and 95% characteristic values; the latter have been determined by simple counting. Also shown in figure 1 are the tolerance fields according to DIN 13/ISO 695 of which 6g is specified for product grades A and B (relevant for the 8.8-bolts of this investigation) and 8g is specified for product grade C (relevant for the 4.6-bolts).

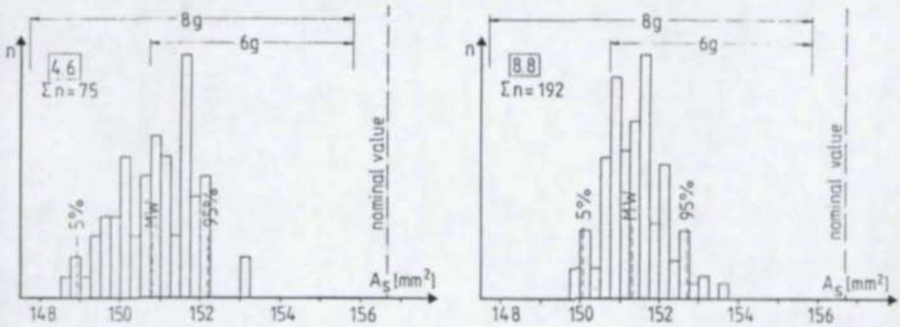

Figure 1. Histograms of stress areas A_S

It can be seen from figure 1,
- that all bolts are well inside the tolerance field 8g;
- that the **8.8-bolts** are significantly more carefully manufactured than the **4.6-bolts**, although even the 5%-fractile of the 8.8-bolts is a little below the lower limit of tolerance field 6g;
- that no bolt at all is in the upper half of tolerance field 6g;
- that the average stress area is only 96% of the nominal value $A_S = 156{,}7\ mm^2 \cong 157\ mm^2$.

It is an interesting aspect of fastener quality assurance that obviously all bolt manufacturers try very hard, to realize the male threads at the lower limit of the tolerance field, supposedly in order to gain a high degree of acceptance for their products in terms of easy screwing. Of course, the situation would be the other way round with the female threads of nuts, so that the thread engagement depths of bolt/nut assemblies will preferably be at the very lower limit of their tolerance fields.

4. TENSILE STRENGTH

One bolt of each sample unit was tested in a direct full size tension test according to ISO 898/1. The test speed was 15 mm/min, which is well below the specified limit of 25 mm/min. Figure 2 shows the histograms of the ultimate tensile forces F_{tu}, including their mean and fractile values, together with the nominal values $f_{ub} \cdot A_S$=62,8 kN and 125,6 kN respectively.

It can be seen from figure 2,
- that for the **8.8-bolts** the 5%-fractile of the ultimate tensile forces is 3,5% above the nominal value, with the scatter being reasonable (variation coefficient V=4,2%);
- that for the **4.6-bolts** the 5%-fractile is slightly below the nominal value with a much wider scattering (V=14%!).

Especially for the 8.8-bolts, it may be stated from the foregoing that - from a statistical point of view - the 4% deficit in the stress areas is conservatively compensated by the real tensile strength values. It should be mentioned that, besides the "technical" ultimate tensile forces, "quasi-static" ultimate tensile forces (analogously to the "static yield stress") have been determined, too; they lie about 5% lower.

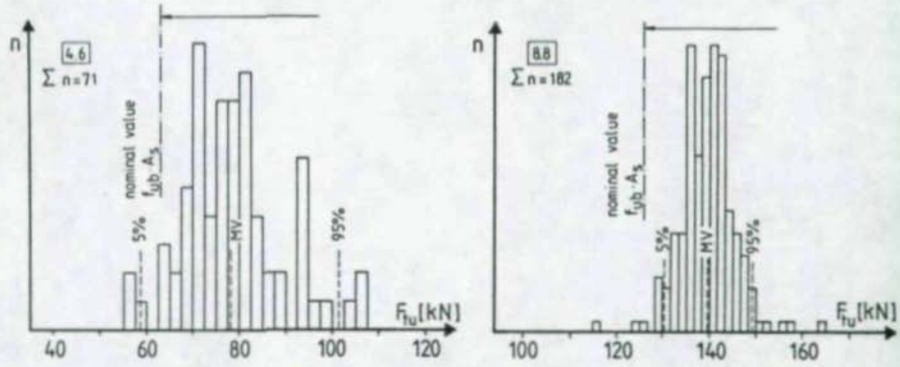

Figure 2. Histograms of ultimate tensile forces F_{tu}

5. SHEAR STRENGTH

Two bolts of each sample unit were tested as a pair in a two plane shear test in their threaded portions. The shear device was of the "tension jig" type (Fisher and Struik, 1987). Figure 3 shows the histograms of the ultimate shear forces F_{vu} per shear plane in the same presentation as the tensile forces. The nominal values are $0.6 \cdot f_{ub} \cdot A_S$=37,7 kN and 75,4 kN respectively, 0.6 being the specified shear strength coefficient for 4.6-, 5.6- and 8.8-bolts in both mentioned new design codes.

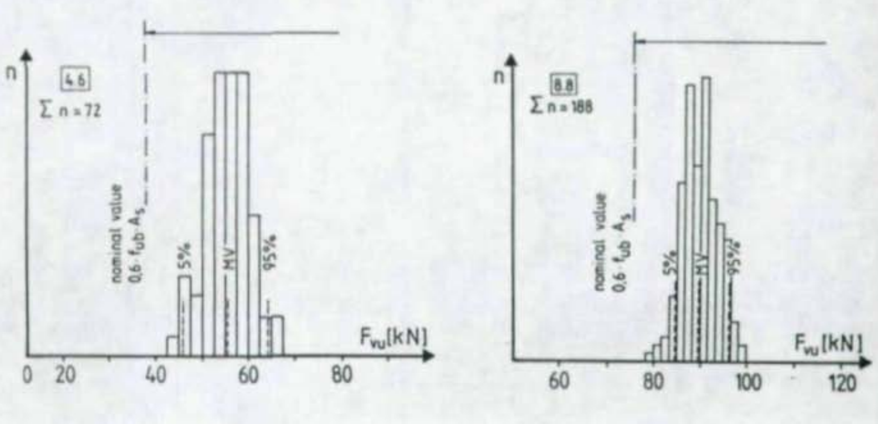

Figure 3. Histograms of ultimate shear forces F_{vu}

It can be seen from figure 3,

- that for the **8.8-bolts** the 5%-fractile of the ultimate shear forces is 12% above the nominal value, with the scatter being reasonable (V=4,5%);
- that for the **4.6-bolts** the 5%-fractile is even 23% above the nominal value but again with a wider scattering (V=9%).

The reason for the shear forces of the 4.6-bolts seeming more conservative than their tensile forces is the well-known fact, that the shear strength coefficient is slightly variable with tensile strength (Knobloch and Schmidt, 1987; Knobloch, 1990); it could be set to at least 0.65 for strength grade 4.6.

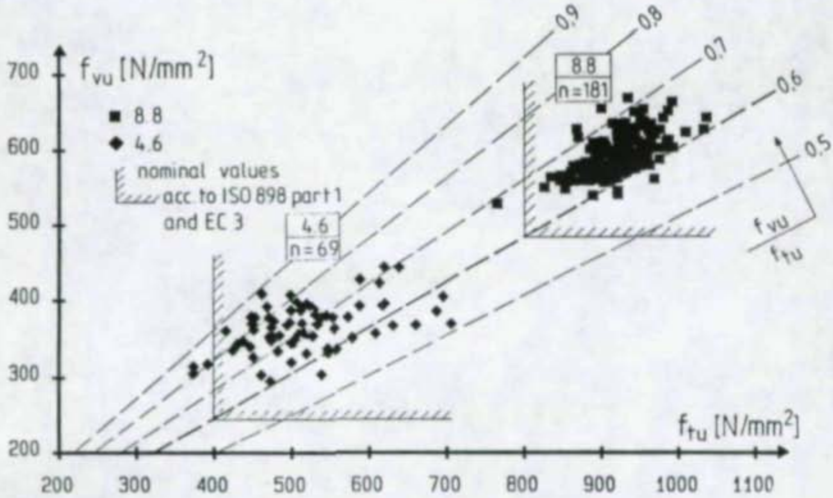

Figure 4. Shear strengths versus tensile strengths

The correlation between the shear strength $f_{vu} = F_{vu}/A_s$ and the tensile strength $f_{tu} = F_{tu}/A_s$ of all sample units may be seen from figure 4. Physically seen, it is not correct to interpret the plotted test points as shear strength coefficients, because they do not stem from **one** bolt, but from two or three bolts of one production batch, which unavoidably incorporates an own statistical scatter (Knobloch, 1990). But approximately the f_{vu}/f_{tu}-values may be looked at as shear/tensile strength ratios. It is interesting that a couple of them are below the specified code value 0.6, although the latter value, as just seen (fig.3), yields conservative characteristic shear forces. The reason is that the equation for the nominal design shear resistance

$$F_{v,Rk} = 0.6 \cdot f_{ub} \cdot A_s \qquad (2)$$

is not a deterministic one, but has a statistic background: Not all of the three factors in (2) must necessarily be 5%-fractiles to make their product a 5%-fractile. This has recently been demonstrated for the present problem by numerical simulation (Knobloch, 1990). This simple fact should be kept in mind when dealing with member strength problems in view of the modern partial safety factor concepts.

6. MATERIAL DUCTILITY

From any of the 4.6-units and from those 15 of the 8.8-units that had yielded the highest or lowest tensile or shear strength values respectively, one bolt was machined into a circular coupon according to ISO 898/1 and checked in a strain-controlled tension test. Besides tensile strength R_m and upper and static yield stresses R_{eH} and R_{eS}, the percentage elongation A_5 after fracture - as one the critical ductility measures in structural bolts - has been determined. Figure 5 and figure 6 show the results versus tensile strengths and shear/tensile strength ratios respectively.

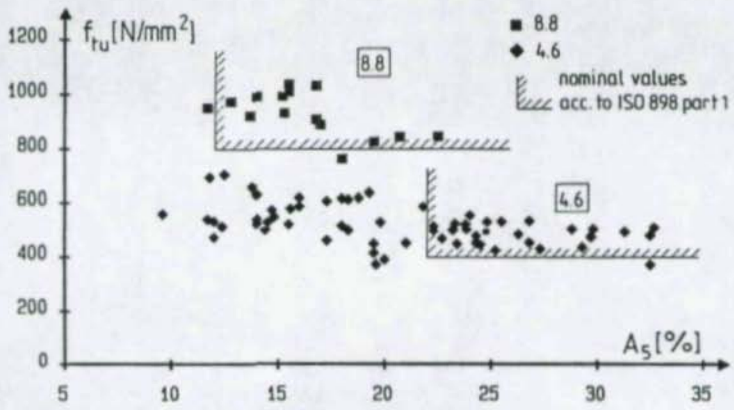

Figure 5. Tensile strengths versus percentage elongations after fracture

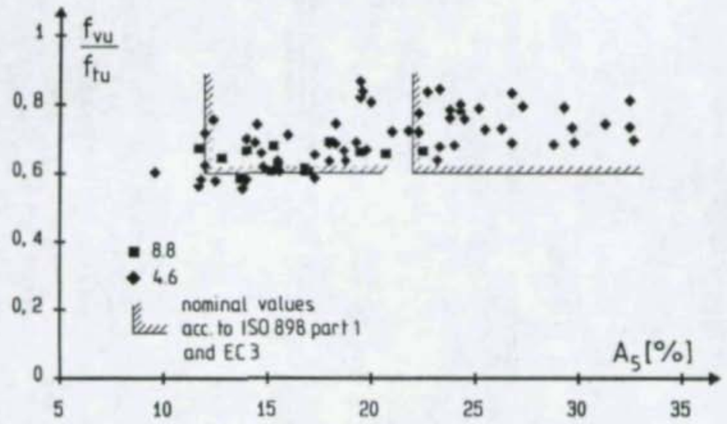

Figure 6. Shear/tensile strength ratios versus percentage elongations after fracture

It can be seen from figures 5 and 6,

- that for the **8.8-bolts** the specified minimum value A_5= 12% (ISO 898/1) is adhered satisfactorily;

- that for the **4.6-bolts** more than 50% of the units show percentage elongations below the specified minimum value A_5=22% (ISO 898/1) with the lowest values even falling below the 8.8-minimum value (!);
- that the shear/tensile strength ratios, as a whole, correlate in the well-known positive way with the material ductility (Knobloch, 1990);
- that nearly all shear/tensile strength ratios <0.6 belong to sample units with far to small percentage alongation values.

The significant ductility deficit of the 4.6-sample, together with its wide scattering of tensile strength values and its less satisfactory thread tolerances, shows - in the author's opinion - very clearly that bolt manufacturers distinguish carefully in their quality assurance efforts, depending on the property classes.

7. CONCLUSIONS

a) The **8.8-bolts** according to DIN 931/ISO 4014 and DIN 933/ ISO 4017 exhibit a good quality assurance level. They comply fully with all geometric and material requirements. No hesitations should exist at their application to carrying connections in steel structures.

b) The **4.6-bolts** according to DIN 558/ISO 4018 and DIN 601/ISO 4016 exhibit a slightly unsatisfactory quality assurance level. Many of them do not comply with the ductility requirements. Precautionary measures should be taken if the plastic deformation capacity of such bolts shall be made use of in the design of carrying connections in steel structures.

c) The **design rules** of DIN 18800 and Eurocode 3 for bolts subjected to shear in their threaded portion are conservative.

8. ACKNOWLEDGMENTS

The author thanks Dr.-Ing. M. Knobloch, who collected the random sample and carried out and evaluated the shear tests, and cand. ing. U. Simanzik, who carried out and evaluated the tension tests. The financial support by DASt and AIF is greatly acknowledged.

REFERENCES

Fisher, J. W./Struik, J. A.: Guide to Design Criteria for Bolted and Riveted Joints, 2nd. edn.
New York, London, Sydney, Toronto: J. Wiley & Sons 1987.

Knobloch, M./Schmidt, H.: Tragfähigkeit und Tragverhalten stahlbauüblicher Schrauben unter reiner Scherbeanspruchung und unter kombinierter Scher- Zugbeanspruchung.
Forschungsberichte d. FB Bauwesen der Univ. GH Essen, No.41, 1987.

Knobloch, M.: Zum Tragsicherheitsnachweis scherbeanspruchter Schrauben im Stahlbau unter vorwiegend ruhender Belastung.
Dr.-Dissertation Uni Essen 1990.

Knobloch, M./Schmidt, H.: Statistische Tragfähigkeitsdaten industriell gefertigter Schrauben unter vorwiegend ruhender Zug- und Abscherbeanspruchung im Gewinde.
Forschungsberichte d. FB Bauwesen der Univ. GH Essen, No.52, 1990.

N. N.: Fastener Manufacturer Identification Symbols.
Köln: European Industrial Fasteners Institute 1983.

N. N.: Herkunfts- (Hersteller-) Zeichen für Schrauben, Muttern und andere mechanische Verbindungselemente nach DIN-Normen.
Berlin: Deutsche Gesellschaft für Warenkennzeichnung GmbH 1988.

N. N.: A Guide to the Markings of Fastener Manufacturers.
New York: ASME 1988.

Sedlacek, G./Stark, J./Hemmert-Halswick, A.: Auswertung von Versuchen mit Schrauben und Schraubenverbindungen für den Eurocode 3.
In: Festschrift Joachim Scheer (ed.: Maier, W. et al.)
p. 239-252.
Braunschweig: Inst. f. Stahlbau der TU, 1986.

Sedlacek, G./Stark, J./Hemmert-Halswick, A.: Evaluation of Test Results and Determination of Partial Safety Coefficients for the Resistance of Connectors. Documentation (preliminary) for Eurocode 3 (ed.: Commission of the European Communities).
Eindhoven, Aachen Dec. 1986.

Schmidt, H./Knobloch, M.: Schrauben unter reiner Scherbeanspruchung und kombinierter Scher- Zugbeanspruchung.
Stahlbau 57 (1988), p. 169 - 174.

Stark, J./Bijlaard, F./Sedlacek, G.: Bolted and Welded Connections in Steel Structures.
In: Bjorhovde, R. et. al. (ed.): Connections in Steel Structures - Behaviour, Strength and Design, p. 8 - 17.
London, New York: Elsevier 1988.

BOLT PRELOADS IN LABORATORY AND IN FIELD CONDITIONS OF ACCEPTANCE

Eugène PIRAPREZ[1]

Abstract

A lot of researches about the preloading of HSFG bolts were carried out in laboratories and the behaviour of the bolt, in these conditions, is well-known. But the bolts are used on site, and it was necessary to control that the tightening gives the same effects in this case. The results of the comparison were unsatisfactory; the dispersion of all parameters and mainly of the preload is larger on site and the security is sometimes too small. It is not sufficient that the components comply with the stipulations for the mechanical and dimensional characteristics mentioned in the respective standards. A new acceptance inspection is proposed.

INTRODUCTION

We have at our disposal a lot of results of tightening tests carried out in laboratories on high strength bolts. But a few years ago, we had no comparison between the preload estimated by tests and the actual preload obtained on site. And there are many reasons to have a difference; it is not easy to evaluate the influence of some factors such as :
- the number of bolts in the connection,
- the lack of fit between the plates,
- the variation of the properties of the lubrication,
- the sequence of the tightening of the bolts,
- the variation of the calibration of the tool,
- the position of the tool,
- the type of the tool (continuous or step by step tightening),
- ...

A large series of tests were carried out in field on different types of connections and the results are compared with the predicted values obtained in laboratory on bolts of the same lot.

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A statistical evaluation is done on basis of about 1500 results of acceptance inspection coming from a branch of the Belgian Ministry of Public Works which does not recognize the quality insurance of the manufacturers of bolts.

1. DESCRIPTION OF THE TESTS IN SITE

Several kinds of connections are considered; the number of bolts, their disposition and their diameter are different, some examples are given on figure 1. Bolts with axial strain gages are used, their position is indicated on the figure. Two curves are plotted during the tightening :

$F_v = f(M_a)$ and $F_v = f(\alpha)$

where :
F_v = the preload in the bolt (calculated from the indications given by the strain gages)
M_a = the applied torque
α = the rotation of the nut.

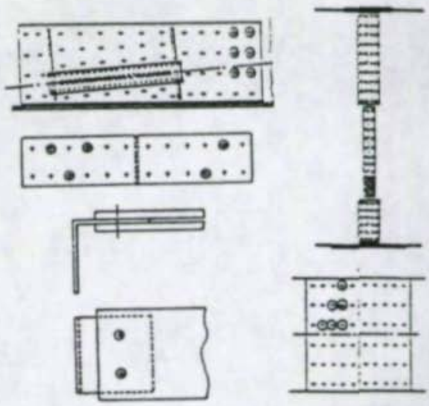

Figure 1 : Examples of connections

Another type of connection was tested to evaluate directly the influence of a gape between the plates; it is shown on figure 2. Three different ratios of the plates thickness were considered. During the tightening, the gap between the plates was plotted on several places.

On the whole, 204 were tested. The tests were stopped (when it was possible) after reaching the maximum of the curve $F_v = f(\alpha)$.

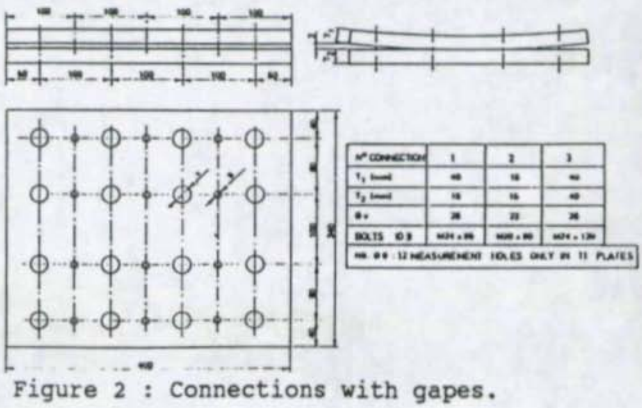

Figure 2 : Connections with gapes.

2. RESULTS OF THE TESTS CARRIED OUT ON SITE

2.1. Maximum axial stress

For a large part of the bolts, the maximum of the axial stress in the stress area (σ_{tAm}) does not reach the yield stress. A summary of the results is given in table 1.

Table 1 : Maximum stress in regards to the yield point (percentage of bolts)

Maximum of σ_{tAm}	Percentage of bolts
$\geq 0.90\ f_y$	90
$\geq 0.92\ f_y$	86
$\geq 0.95\ f_y$	79
$\geq\ f_y$	54

In laboratory the maximum stress in all bolts reach at least the value 0.9 f_y.

2.2. Stress after tightening by the torque method

Based on calibration tests in laboratory, the stress after tightening by the torque method would be 630 MPa (0.7 f_y). The mean of the plotted values on the diagrams ($\bar{X}$), their coefficient of variation ($V = \bar{X}/s$) and the relative range ($W/\bar{X}$) are given on table 2. The dispersion is clearly shown on figure 3. It is also possible to compare on table 2, the value of k determined in laboratory and the value deduced from the curve $F_v = f(M_a)$.

Table 2 : Axial stresses after tightening by the torque method

Connection n°	$\bar{X}$ (MPa)	V	$W/\bar{X}$	k_{lab}	k_{site}
1	644	0.070	0.17	0.061	0.067
2	601	0.069	0.19	0.055	0.064
3	659	0.112	0.33	-	0.112
4	455	0.182	0.48	0.073	0.194
5	625	0.335	0.91	0.082	0.260
6	684	0.073	0.31	0.087	0.071
7	702	0.091	0.35	0.131	0.092
8	634	0.058	0.20	0.099	0.057
9	551	0.089	0.23	0.064	0.092
10	604	0.104	0.27	0.085	0.105
11	586	-	0.12	0.082	-
12	545	0.075	0.19	0.873	0.079

50 % of the values are not acceptable; they are lower than the theoretical stress.

The differences between the mean values are mainly influenced by the diameter of the bolts : if the diameter is larger, the stress is lower. They also depend on the ratio d/Σt where d is the bolt diameter and Σt the total thickness of the assembled plates. The stress is directly proportional to this ratio.

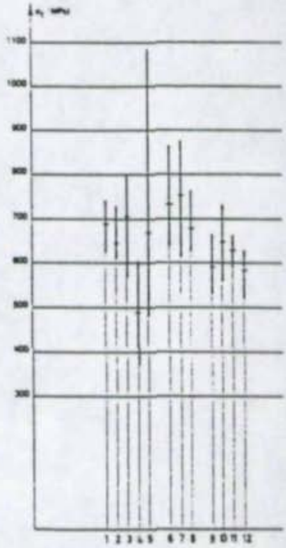

Figure 3 : Dispersion of the axial stress after tightening by the torque method.

2.3. Stress after tightening by the combined method

The combined method considered is :

$$0.75\ M_a + \alpha_c$$

where M_a is the theoretical torque : $M_a = k\ d\ 0.8\ f_y\ A_s$ and α_c is given in table 3 in regards with the total thickness of the plates (Σt).

Table 3 : Values of α_c (degrees)

Final stress	Σt ≤ 2d	2d < Σt < 6d	6d ≤ Σt ≤ 8d
$0.7\ f_y$	50°	60°	90°
$0.8\ f_y$	60°	90°	120°

With this method, it is usual to prestress the bolt until reaching at least the value $0.8\ f_y$ in the stress area, for several reasons :

- economical considerations;
- no risk of loosing;
- a very smaller loosing dispersion of the preload.

Theoretically, the minimum is thus 720 MPa; it can be compared with the values obtained in site and given in table 4.

This table gives also (when it is possible) the value of α_r which is a "security angle", because it is not allowed, during the tightening, to reach the maximum of the curve $\sigma = f(\alpha)$ and usually, a "security" of 30 degrees is required.

The dispersions of the stresses in each connection are illustrated on figure 4.

Table 4 : Axial stress after tightening by the combined method

Con. 'n'	$\bar{X}$ (MPa)	V	$W/\bar{X}$	α_r (°)
1	787	0.038	0.09	-
2	852	0.052	0.15	-
3	748	0.114	0.31	132
6	958	0.048	0.20	180
7	913	0.022	0.08	144
8	870	0.055	0.21	164
9	780	0.080	0.19	133
10	934	0.126	0.28	-
11	876	-	0.01	216
12	900	0.023	0.05	-

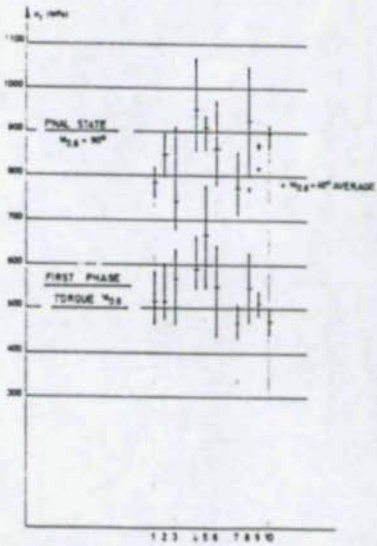

Figure 4 : Dispersion of the axial stress after tightening by the combined method.

3. CONCLUSIONS ABOUT TESTS ON SITE

3.1. Torque Method

It is remarkable that the values of k are very different on site than those determined in laboratory. The main reason is probably the alteration of the lubricant due to the time and the conditions of stocking. But the value of this coefficient also depends upon the gap between the plates, on the rapidity of the tightening and as it is calculated by means of the torque M_a, it is also influenced by the position of the tool.

These considerations are proved by the values given in table 2 : the dispersion of the stresses corresponding to a connection is not much larger than the dispersion of the values determined in laboratory. On the other hand, the mean values are very different from one connection to another and from laboratory to site.

In all cases this method is dangerous because 50 % of the bolts have not the minimum preload.

3.2. Combined Method

This method gives much better results : if we consider the mean values, the required preload is always obtained and the reserve of rotation is sufficient. But this conclusion is only valid if the value of k is known for each lot of bolts; a uniform value of k is not acceptable. And even in this case, all the individual results are not correct : 3 % of the bolts have not the required preload and 5 % of them do not present a sufficient security of rotation.

If the required preload was limited to $0.7\ f_y\ A_s$, the method $0.75\ M_a+60°$ would be satisfying for all bolts, but in these conditions, the dispersion of the preload is much larger, and using this preload is not economical.

3.3. General Remarks

For both methods, to know the accurate value of k is important. Using the torque method, it can considerably reduce the dispersion of the preload (without eliminating the influence of the site conditions), and with the combined method, it is necessary to be sure to reach the required preload. It is also necessary to control the value of the reserve of rotation (α_r) to avoid important damages and consequently the relaxation and the loose of preload.

4. CONDITIONS OF ACCEPTANCE

4.1. General Considerations

As mentioned above, acceptance controls must be executed on each lot of bolts. Even the geometrical characteristics and the mechanical properties must be checked if they are not guaranteed by a quality control system from the manufacturer.

But all the rules to control these factors are based on statistical considerations and they need a large number of values; they are only valid for a quality control system. For the acceptance of a lot with a small population, it is necessary to define other rules [2], [3] which can be resumed as follow.

4.2. Definition of a lot

A normal lot of bolts contains only bolts of one diameter, of one property class and from one manufacturer, but it may contain material from several cast numbers (with an upper limit of 6). If such a lot is presented for inspection, bolts from each cast number must be clearly indicated.

A restricted lot of bolts contains bolts from one production unit only.

A lot of nuts or washers contains items from one production unit only.

Depending on the tightening method, two different definitions of lots of fasteners are used :

a) for fasteners tightened by direct tension or by turn-of-nut method, a lot of fasteners comprises bolts from one normal lot only, but may include nuts and washers from several lots.
b) for fasteners tightened by the controlled torque or by the combined method, a lot of fasteners comprises only one normal lot of bolts, one lot of nuts and one lot of washers.

4.3. Dimensional and mechanical characteristics

The required values for these factors are given in ISO 892/2 standard [4][5]. The test programmes with the corresponding AQL values are proposed in detail in references [2] and [3].

4.4. Additional controls for fasteners to be tightened by the torque or combined method

The feasibility of tightening the fasteners by one of these methods has to be verified.
The suitability of the fastener for a particular tightening method is determined by a tightening test, measuring the variables : M_m, σ_{tAm} and α.

For both tightening methods the following acceptance criteria have to be fulfilled :
1) the ratio σ_{tAm}/M_m has to be constant between $\sigma_{tAm} = 0.4\ f_y$ and $0.8\ f_y$ for each individual fastener. The graph $\sigma_{tAm} = f(M_m)$ has to be linear (± 5 %) between these limits.
2) the mean $\bar{k}$ has to be ≤ 0.14

3) the coefficient of variation V_k has to be ≤ 0.08.

For the combined tightening method, the following additional criteria have to be fulfilled :

4) the σ_{tAm} value obtained after actual tightening has to be $\geq 0.8\ f_y$.
5) the reserve angle α_r has to be $\geq 30°$.

In some cases, the results may be improved by elimination of one (and only one) outlying result. This procedure is permitted because tests carried out on many supplies have revealed that the distribution of the k values is almost always normal : the elimination of the one outlying value restores the normal distribution.

The sampling plans proposed are given in reference [3].

5. GENERAL CONCLUSIONS

The combined method of tightening is very satisfying; it gives a good security and economical connections. To be absolutely sure and to give a fully guarantee of the preload for a long time, the characteristics of the bolts must be checked by the quality control service of the manufacturer or by acceptance tests. In any case, tightening tests must be carried out.

6. REFERENCES

1. P. BESEM, E. DEHAN and E. PIRAPREZ
   Le serrage des boulons à haute résistance dans les assemblages réels. Publication CRIF, MT 171, juillet 1987.
2. E. DEHAN and E. PIRAPREZ
   Considerations about the acceptance inspection of H.S. Fasteners intended for controlled tightening. Publication CRIF, MT 175, March 1989.
3. Acceptance inspection of grade 10.9 structural fasteners intended for controlled tightening.
   ECCS-TC10/WG4 Publication, to be published.
4. ISO 898/1 : Mechanical properties of fasteners, part 1, bolts.
5. ISO 898/2 : Mechanical properties of fasteners, part 2, nuts - style 1.

USE OF SNUG-TIGHTENED BOLTS IN END-PLATE CONNECTIONS

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Kamal B. Rojiani[3]

Abstract

The results of an experimental investigation on the behavior of snug-tight bolts in moment end-plate connections are presented. Eleven specimens representing six different end-plate configurations were tested. Cyclic loading represented expected wind loads in the range of 33% to 100% of the connection allowable stress design moment. Ten specimens were tested using a loading sequence considered to be representative of a site subjected to severe wind loading. One specimen was tested with a loading commensurate to a worst case wind loading scenario. After cyclic loading was completed, each specimen was statically loaded to failure. All specimens survived the cyclic loading without bolt loosening or rupture. Eight of the eleven specimens provided capacities which were consistent with strength predictions.

1. INTRODUCTION

Moment end-plate connections are frequently used in gable frame metal buildings. Metal building manufacturers prefer this type of connection for use in beam-to-beam or in beam-to-column connections. Bolted end-plate connections are preferred for their excellent rotational restraint characteristics and because they require no field welding.

In current practice, bolts in end-plate moment connections are required to be pretensioned to approximately 70 percent of the bolt tensile strength. The American Institute of Steel Construction (AISC) and the Research Council on Structural Connections (RCSC) have approved three methods for tightening bolts: turn of nut, calibrated wrench, and tightening by use of a direct tension indicator (AISC, 1989; RCSC, 1985). Each of these methods requires special tools, considerable effort on the

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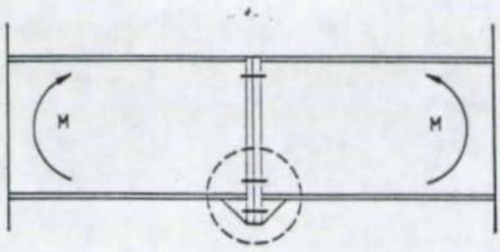


Figure 1. Tension Zone in a Moment End-Plate Connection

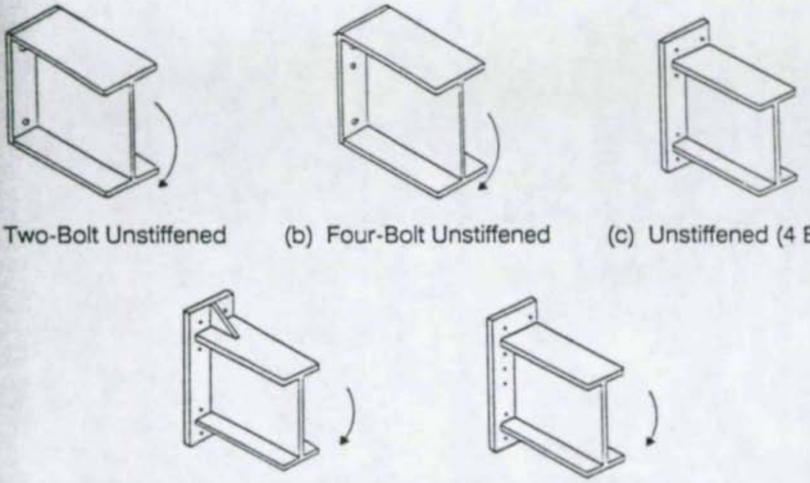
part of the iron worker, and careful inspection. Considerable savings could be effected during erection if the requirement for bolt tensioning were relaxed for some applications.

A snug tight condition is defined by RCSC as "the tightness that exists when all plies in a joint are in firm contact. This may be attained by a few impacts of an impact wrench or the full effort of a man using an ordinary spud wrench". There are primarily two concerns with the use of snug-tightened bolts: (1) the effect of repeated environmental loadings on the performance of the tension bolts, and (2) the possible loss of connection stiffness due to the lack of full tightening. The emphasis of this paper is on the former topic.

Justification for bolt tensioning is well established. When a moment end-plate connection is loaded, the bolts near the tension flange of the beam are in axial tension as indicated in Figure 1. If the bolts are preloaded, then ideally the bolt stress remains constant through at least a variation in applied moment below the design moment for the connection. With snug-tightened bolts, the force in the tension bolts changes as the applied moment varies. The increase in bolt tension below the design moment is the primary concern with snug-tightened bolts in moment end-plate connections. Snug-tightened bolts must endure larger stress variations than pretensioned bolts, which could lead to premature strength degradation.

The static behavior of moment end-plate connections has been studied thoroughly (Murray, 1988). Chasten et al (1989) have studied the behavior of snug-tightened bolts in large capacity moment end-plate connections. Research on cyclic loads applied to moment end-plate connections has been very limited. Several studies have been conducted to evaluate the behavior of bolted connections in severe seismic environments, in which the connection is strained well beyond yield (Dicorso, et al, 1989; Tsai and Popov, 1989).

A study conducted by the authors is the only known work available on snug-tightened bolts subjected to cyclic loading (Kline, Rojiani and Murray, 1989). In this investigation, wind loads were considered to be the dominant contributor to life-time loading on a building. A test loading sequence was established based on statistics of wind speeds in the United States. Since it is known that the wind loading distribution on low-rise buildings is site dependent, the test loading was intended to be representative of the more severe wind loading locations. The experimental part of the study included tests of eleven full scale end-plate connections representing six different configurations. The six end-plate configurations are shown in Figure 2. ASTM A325 bolts were used exclusively. The following sections summarize the results of the authors' study.



(a) Two-Bolt Unstiffened (b) Four-Bolt Unstiffened (c) Unstiffened (4 Bolts)

(d) Stiffened (4 Bolts) (e) Multiple Row Extended

Figure 2. End-Plate Configurations Used in Study

2. WIND LOADING ANALYSIS

The predominate cause of non-seismic cyclic loading of moment end-plate connections in low-rise buildings is wind. Obviously, snow loads can be large; however, the total number of snow loading cycles is small compared to the total number of wind loading cycles. Temperature loads are insignificant.

A procedure to determine the mean wind speed magnitudes and the corresponding frequencies over the 50 year life of a low-rise building, and to establish a test loading sequence for moment end-plate connections in low-rise buildings is:

1. Find extreme value distributions of maximum annual wind speed at a set of sites.
2. Determine wind speeds, V_n, corresponding to return periods, n, ranging from 2 years to 50 years at each site.
3. For the selected sites, compute the ratio of V_n to V_{50} and the mean and standard deviation of this ratio. From this data, establish storm magnitudes for selected return periods.
4. Determine the expected number of wind gusts in a storm and the distribution of these gusts relative to the mean wind speed. The number of cycles for selected wind speed and wind loading increments versus return periods is then determined.
5. Evaluate the effects of wind direction at an average site, realizing that only wind loading components in the plane of the resisting rigid frame are of interest.
6. Establish a test loading sequence in terms of applied moment as a function of design moment capacity of the connection.

Table 1
Total Wind Loading Distribution

M/M_{design}	M/M_{design}	Test Cycles
0.92 - 1.0	1.0	18
0.85 - 0.92		
0.78 - 0.85	0.85	294
0.71 - 0.78		
0.65 - 0.71		
0.59 - 0.65	0.65	1492
0.53 - 0.59		
0.48 - 0.53		
0.43 - 0.48	0.48	6306
0.38 - 0.43		
0.33 - 0.38		
	Total Cycles	8110

Table 1 is the result of the above for 100 selected sites in the United States. In Table 1, M_{design} is equal to 1.33 times the connection working moment to account for the allowable stress increase in the AISC Specification (AISC, 1989a). The four loading ratios given in the second column of Table 1 were selected to simplify the physical testing. The minimum loading ratio cut-off was arbitrarily selected to be 33 percent of the design loading. One-eight of the total cycles at a site were used to account for the effect of wind direction.

3. EVALUATION USING CYCLIC LOADING

To evaluate the performance of snug-tightened bolts in moment end-plate connections, eleven tests, two for each of the configurations shown in Figure 2, except the multiple row extended configuration, were conducted. One test was conducted for this configuration.

The test setup consisted of two equal length beam sections and two point loading, resulting in pure moment at the connection plates. A closed-loop, servo-controlled hydraulic system was used to apply the cyclic loading sequence of Table 1. The multiple row extended configuration was subjected to ten times the number of cycles shown. Eight of the specimens were tested in a high-to-low load sequence and the remaining three in a low-to-high load sequence. A sinusoidal wave form was used to generate the cyclic loading. After all cyclic loading was complete, the specimens were statically loaded to failure.

Instrumented bolts, one per bolt row at the tension flange, were used to monitor bolt strains resulting from the applied load. The bolts were instrumented by installing a strain gage into a small hole which had previously been drilled through the bolt head and into the unthreaded portion of the bolt shank. (The material removed was less than the difference between the gross and tensile areas of the bolt shank.) After installation of the strain gage, the bolts were calibrated using a universal testing machine to obtain a calibration factor for calculating bolt force.

Table 2
Initial Bolt Tension

Test Specimen	Pretension Inside Bolt[1] (kips)	Pretension Outside Bolt[2] (kips
2-Bolt Flush	18.0	--
2-Bolt Flush	14.7	--
4-Bolt Flush	18.0	14.7
4-Bolt Flush	10.1	11.0
4-Bolt Extended	16.1	11.0
4-Bolt Extended	5.9	11.9
4-Bolt Extended	13.1	13.7
4-Bolt Extended	11.9	7.6
4-Bolt Extended, Stiffened	14.0	7.5
4-Bolt Extended, Stiffened	15.1	12.4
Multiple Row Extended	6.1	8.0
	4.9	7.7

[1]Inside Bolt is nearest to beam neutral axis. [2]Outside Bolt is farthest from beam neutral axis.

For each specimen, the bolts were tightened prior to testing using an ordinary spud wrench. During this procedure, the instrumented bolts were connected to a data acquisition system so that the exact bolt pretension could be recorded. Bolt pretension levels were not predetermined. The bolts were tightened by the full effort of a man with a spud wrench; no extraordinary effort was expended. Table 2 shows the pretension forces for all instrumented bolts.

A large variability in bolt pretension levels is evident in Table 2. This variability is likely due to the loosening of some bolts as others are tightened. The bolt pretension as a percentage of the AISC specified pretension level is directly related to bolt diameter. On average the 5/8 in. diameter bolts were pretensioned to 89 percent, the 5/8 in. diameter bolts were pretension to 89 percent, the 3/4 in. diameter bolts to 37 percent; and, the 1 in. diameter bolts to 25 percent of the full pretension values listed in the appropriate specifications (AISC, 1978; RCSC, 1985).

Typically, the residual bolt forces decreased as the number of loading cycles increased. Figure 3 is a plot of residual bolt force versus number of load cycles for a typical test. The data points are plotted on a semi-log scale to emphasize the settling effect of the residual bolt forces. The residual bolt forces decreased rapidly during the first few loading cycles and asymptotically approached a lower bound. Even though there was a general decrease of bolt clamping force as the cyclic loading progressed, no bolt in any of the eleven test specimens ruptured or became loose, including those in the test in which 80,000 cycles were applied.

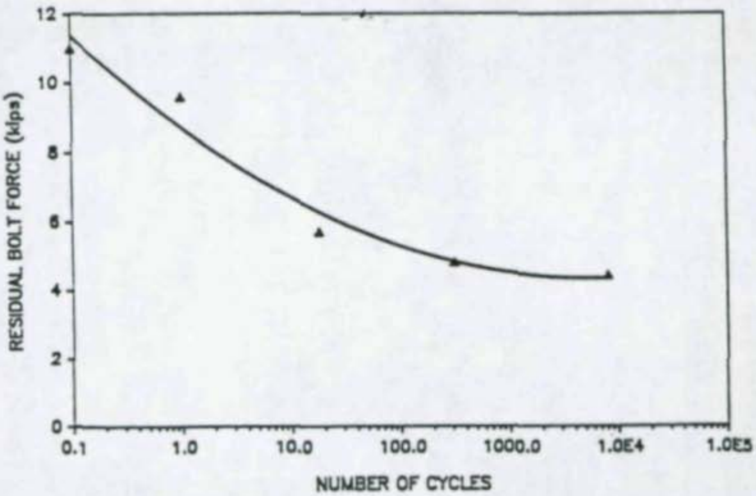


Figure 3. Residual Bolt Force versus Number of Load Cycles, Typical Results

4. STRENGTH EVALUATION

Ultimate moment capacities for the end-plates were calculated using the methods for statical loading summarized in Murray (1988). The so-called "modified Kennedy method" (Schouji, Kukreti and Murray, 1983) was used to predict bolt forces, including prying forces, for all of the end-plate configurations except the four-bolt, extended, unstiffened configuration. Prying force effects for this configuration were not considered.

Bolt force limit state results are summarized in Table 3. "Predicted moment at bolt proof", listed in the second column, is the predicted moment at which the bolt stresses are at their tensile strength levels. The actual measured bolt tensile strength levels were used to determine these values. The "maximum applied moment" shown in Table 3 is the maximum moment prior to bolt rupture, or in four cases, the maximum applied moment prior to test beam failure that was applied in the final static test to failure.

The ratio M_{ub}/M_u in Table 3 gives a comparison of the predicted and experimental moment capacities based on the bolt force limit state. Also shown in Table 3 is the ratio of the experimentally determined end-plate yield moment, M_y, to the maximum applied moment, M_u, for the nine tests in which end-plate yielding was observed. In all nine tests, this ratio is less than the ratio of the predicted moment at bolt proof load-to-maximum applied moment, M_{ub}/M_u, which indicates that bolt strength was not the controlling limit state for these tests. Yielding of the end-plate material at a moment below the predicted moment at the bolt proof load results in increased bolt force. Therefore, it is not unexpected that the maximum applied moment for these tests did not exceed the predicted moment required to cause a bolt force equal to the bolt proof load. Consequently, the bolt strength performance observed in these tests is judged to be satisfactory.

Table 3
Bolt Limit State Results

Test Specimen	Measured Bolt Tensile Stress, F_{yb} (ksi)	Predicted Moment at Bolt Proof, M_{ub} (k-ft)	Maximum Applied Moment, M_u (k-ft)	M_{ub}/M_u	$M_y{}^1/M_u$
2-Bolt Flush	107.6	68.4	75.6	0.90	0.75
2-Bolt Flush	115.5	104.2	88.4	1.18	0.68
4-Bolt Flush	107.6	93.0	113.6	0.81	0.80
4-Bolt Flush	115.5	139.0	112.8[2]	1.23	0.62
4-Bolt Extended	111.4	287.2	227.2	1.26	0.97
4-Bolt Extended	111.4	287.2	230.7	1.24	----
4-Bolt Extended	102.5	263.2	237.7	1.11	0.82
4-Bolt Extended	116.2	298.3	265.0	1.13	----
4-Bolt Extended Stiffened	112.0	404.0	468.0[2]	0.86	0.66
4-Bolt Extended Stiffened	118.1	476.0	482.7[2]	0.99	0.89
Multiple Row Extended	115.5	405.0	467.6[2]	0.87	0.71

[1] M_y is the experimental end-plate capacity. [2] Test beam failed by flange local buckling.

In the two remaining tests, end-plate yielding was not observed prior to bolt rupture. The corresponding M_{ub}/M_u ratios are 1.24 and 1.13, meaning that the maximum applied moments for the two tests were 80.3% and 88.8% of the predicted bolt proof load moments. These four bolt extended, unstiffened end-plate connections were designed using the procedure in the 9th edition AISC Manual of Steel Construction (AISC, 1989). This procedure does not account for possible prying action forces, as does the *procedure used for the other configurations.*

5. USE OF SNUG-TIGHTENED BOLTS

From the study reported, snug-tightened bolts in moment end-plate connections are not susceptable to premature failure when subjected to cyclic wind loading for a 50 year building life. Further, the ultimate strength of moment end-plate connections with snug-tightened bolts can be predicted using methods developed for similar fully tightened connections if prying forces are considered. Thus, snug-tightened bolts can be used in moment end-plate connections if a proper design procedure is used. However, moment end-plate connections with snug-tightened bolts, pending further study, are not recommended for other than low-rise buildings, buildings subject to crane loads or buildings in areas where seismic loads control the design.

ACKNOWLEDGEMENTS

The study reported in this paper was co-sponsored by the American Institute of Steel Construction, the Metal Building Manufacturers Association and the Research Council on Structural Connections.

REFERENCES

AISC (1989) Manual of Steel Construction. American Institute of Steel Construction, Chicago, Ill.

AISC (1989a) Specification for the Design, Fabrication and Erection of Structural Steel for Buildings. American Institute of Steel Construction, Chicago, Ill.

Chasten, C. P., Fleischman, R. B., Driscoll, G. C. and Lu, L. (1989). "Top-and-Seat-Angle Connections and End-Plate Connections: Behavior and Strength Under Monatonic and Cyclic Loading." Proceedings of the 1989 National Steel Construction Conference, American Institute of Steel Construction, Chicago, IL, June 21-24, pages 6-1 to 6-32.

Dicorso, P., Harper, W., Reinhorn, A., and Dickerson, J. (1989). "Structures with Semi-Rigid Bolted Connections." Proceedings of the Sessions Related to Steel Structures, ASCE Structures Congress '89, San Francisco, May 2-5, pages 529-538.

Kline, D. P., Rojiani, K. B. and Murray, T. M. (1989). "Performance of Snug Tight Bolts in Moment End-Plate Connections." Research Report CE/VPI-ST-89/04, submitted to the Metal Building Manufacturers Association, the Charles E. Via Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA, February.

Murray, T. M. (1988). "Recent Developments for the Design of Moment End-Plate Connections." Journal of Constructional Steel Research, 10, pages 133-162.

RCSC (1985), Specification for Structural Joints Using ASTM A325 or A490 Bolts, Research Council on Structural Connections, Chicago, Ill.

Srouji, R., Kukreti, A., and Murray, T. (1983). "Yield-line Analysis of End-Plate Connections with Bolt Force Predictions." Report No. FSEL/MBMA 83-05, University of Oklahoma, Norman, Oklahoma.

Tsai, K., and Popov, E. (1989). "End Plate Moment Connections for Cyclic Loads." Proceedings of the Sessions Related to Steel Structures, ASCE Structures Congress '89, San Francisco, May 2-5, pages 569-578.

Technical Papers on

WELDS AND LOCAL STRENGTH CONSIDERATIONS

STRENGTH AND DEFORMATION CAPACITY OF FILLET WELDS IN Fe E 460

A.M. Gresnigt[1]

Abstract

In order to extend the scope of Eurocode No. 3 to include also Fe E 460, a European research programme was carried out. In the framework of this, tests on lap joints and T-joints were executed at TNO. In this paper, the test set up and the test results are summarized. Also the main results of the statistical evaluation are given. Finally some preliminary results of finite element calculation are presented.

1. INTRODUCTION

As a result of the improvement of the mechanical properties and weldability of modern high strength steels and furthermore the availability of improved welding consumables and welding processes, there is a growing number of applications where high strength steels (HSS, with a yield stress above 355 N/mm^2) may lead to economical advantages above "normal" steels. Examples are large and tall buildings, bridges, offshore structures, cranes, vehicles.

For the application of HSS guidelines and codes for the design, calculation and fabrication are necessary. In many countries the absence of these guidelines and codes is a drawback for the application of HSS.

In Eurocode No. 3 (April 1990) the applicability of the rules is limited to steel grades up to and including Fe E 355. In order to extend the scope to Fe E 460, a research programme was established to collect the available knowledge and to identify additional research needs. Tests on bolted and welded connections appeared necessary. The tests on bolted connections were carried out at the Technical University in Braunschweig (Germany) and at Imperial College in London (UK).

The tests on welded connections were carried out at INSA in Rennes (France) and at TNO in Delft (The Netherlands). The research was coordinated by the RWTH in Aachen (Germany).

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The evaluation of the test results was also carried out by RWTH (Background documentation 1990).
The research has resulted in a preliminary draft of Annex D of Eurocode No 3: The use of steel grade Fe E 460 (January 1991).
In this paper a summary of the tests on fillet welded joints at TNO is presented, together with some preliminary results of finite element calculations on lap joints, carried out at the Delft University of Technology.

2. TEST SPECIMENS

2.1 Parent material

The specimens were fabricated from Fe E 460 TM plates provided by Arbed Luxembourg. These plates were taken out of the flanges and webs of rolled H- and I-sections. Table 1 gives an average of the chemical composition. Table 2 gives results of tensile coupon tests.

C	0.15	Si	0.28	Cr	0.05
Mn	0.54	Mi	0.06	Mo	--
P	0.023	Al	0.04	V	0.07
S	0.011	Cu	0.07	Nb	0.04

Table 1. Average chemical composition of the parent material

Plate	f_y (N/mm^2)	f_u (N/mm^2)	ε_u (%)
HE 200 A web	482 (33)	592 (22)	24 (2)
HE 200 A flange	506 (34)	616 (25)	26 (2)
HE 400 M web	490 (20)	713 (17)	25 (2)
HE 400 M flange	491 (19)	685 (23)	31 (4)
HE 500 B web	541 (9)	730 (10)	29 (3)
HE 500 B flange	493 (23)	741 (23)	31 (7)
IPE 360 flange	453 (5)	574 (2)	41 (1)

Table 2. Mean values and standard deviations of tensile coupon test results. The standard deviations are given between ( ).

2.2 Welding processes and welding consumables.

Most test specimens were welded using the Smitweld Conarc 60 G electrode.
Some of the specimens were welded using the Smitweld Kardo electrode or the Philips PZ 6030 wire (15% CO_2 and 85% Argon). The specified and measured mechanical properties are given in table 3.

Welding consumable	f_y (N/mm²) spec	pure	par	f_u (N/mm²) spec	pure	par	ε_u (%) spec	pure	par
Conarc 60 G	550	549	687	620	656	753	22	26	24
Kardo	420	396	444	470	478	511	26	30	32
PZ 6030	430	411	534	520	540	633	22	31	29

Table 3. Specified and measured mechanical properties of the welding consumables. The coupons were taken from pure weld metal according to ISO 2560-1973 ("pure") and from dp 5 specimens out of a fillet weld in Fe E 460 parent material ("par").

The Conarc 60 G and Kardo electrodes were vacuum packed. The hydrogen content was specified to be below 3 ml/100 g. Considering the steel and the low hydrogen content, it was decided not to preheat the specimens, nor a post weld heat treatment was applied.
As can be seen from the tables 2 and 3, the Kardo electrode was undermatched. In fact, this electrode is a so called buttering electrode.
The idea to choose this electrode for a limited number of specimens is that an undermatching electrode may result in a better toughness, while the lower strength may be compensated for by a thicker (fillet) weld (Treiberg 1991).

2.3 Test specimens

The shapes of the test specimens are showed in figure 1. See also the photographs in figure 6.

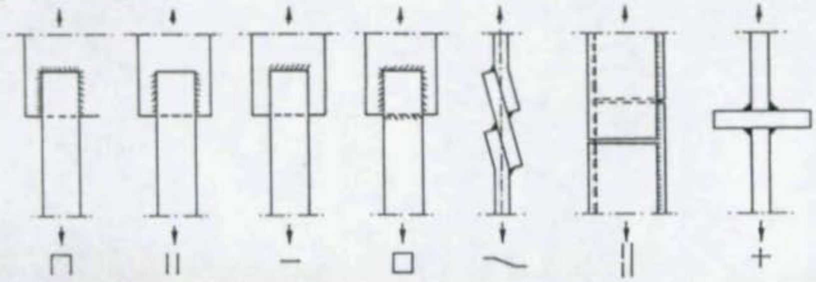

Figure 1. Shapes of the test specimens.

The main plate thicknesses were 10 mm, 28 mm and 40 mm. The total number of tests was 96. In most test specimens the weld thickness was designed such that rupture of the weld occurred before yielding of the plates. Eight lap joints were designed such that the plates would yield before rupture of the welds. In the lap joints, the following combinations were tested:
- length side weld / length transverse weld : 45/80; 55/55; 80/40 (mm)
- throat thickness side weld / throat thickness transverse weld : 0,5 ; 1,0 ; 2,0.

3. TEST RESULTS

3.1 Test rig and measurements

The smaller test specimens were tested in a 1000 kN Amsler tensile machine, while for the bigger specimens a 10.000 kN test rig was necessary. During testing, the loads and the displacements in the welded connections were measured, see figure 2.

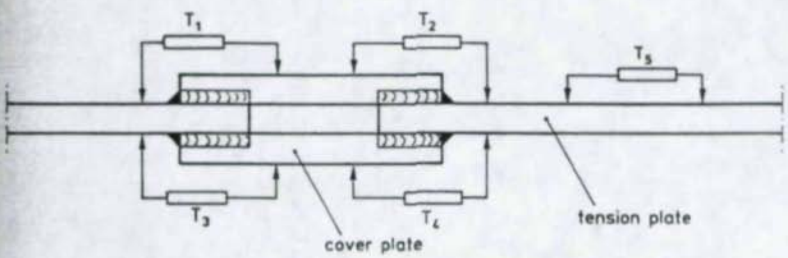

Figure 2 Electric transducers T_1, T_2, T_3, T_4, T_5. The measuring length of most specimens was 110 mm. The cross section of each cover plate was 0.5 times the cross section of the tension plate.

3.2 Test results

From every test specimen the followig data were recorded:
- Dimensions of plates and welds before the test;
- Dimensions and orientation of the rupture surface;
- Load-displacement diagram.

In figures 3, 4 and 5 the rupture surfaces and load displacement diagrams are given for a number of specimens. The dimensions and data about the welding consumables are given in table 4.
Some photographs of the test specimens are given in figure 6.

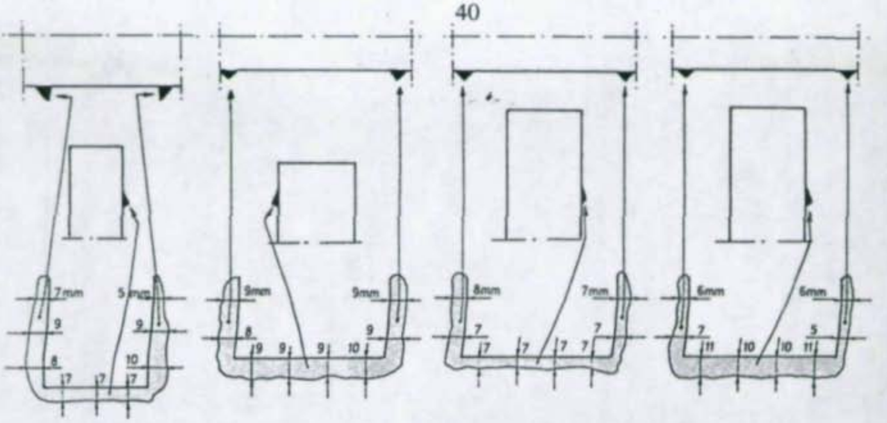

Figure 3 Measured rupture surfaces for test specimens 1.17, 1.32, 1.35 and 1.36. Note that the rupture angle of the end fillets is about 25° and for the side fillets about 45°.

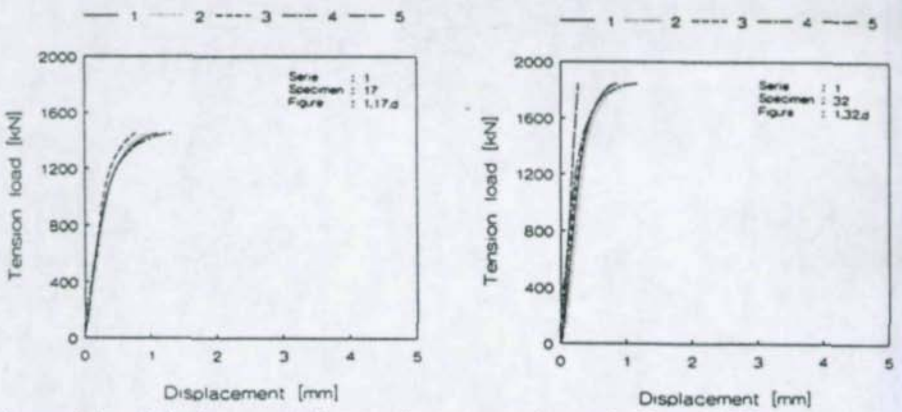

Figure 4: Load displacement diagram for test specimens 1.17 and 1.32.

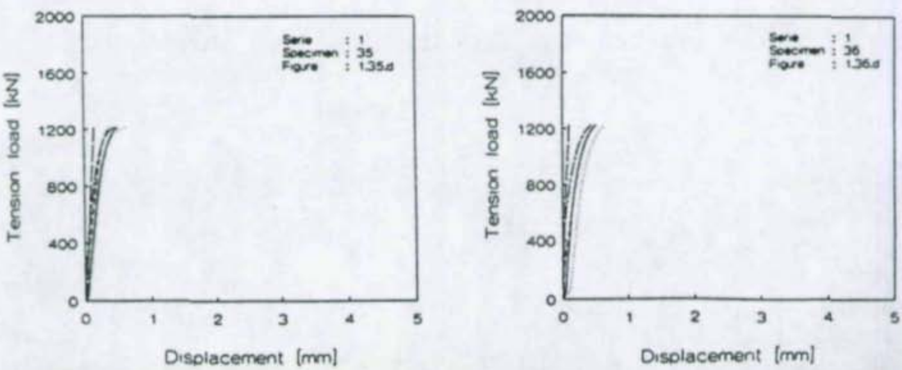

Figure 5: Load displacement diagrams for specimens 1.35 and 1.36.

Speci-men	Failure load (kN)	Plate thick-ness (mm)	Weld length side (mm)	Weld length end (mm)	Throat side (mm)	Throat end (mm)	Welding consumable
1.17	1451	28	55	55	7.5	7.3	Conarc 60 G
1.32	1847	40	40	80	10.1	9.3	Conarc 60 G
1.35	1216	40	40	80	7.5	7.4	Filarc PZ 6030
1.36	1226	40	40	80	7.1	11.2	Kardo

Table 4: Failure load, dimensions and welding consumables of some test specimens. The throat thickness data are the mean values of the measurements at 8 respectively 4 sections of the side and the end fillet.
The ultimate strength of the 28 mm plate was 702 N/mm² and of the 40 mm plate 725 N/mm².

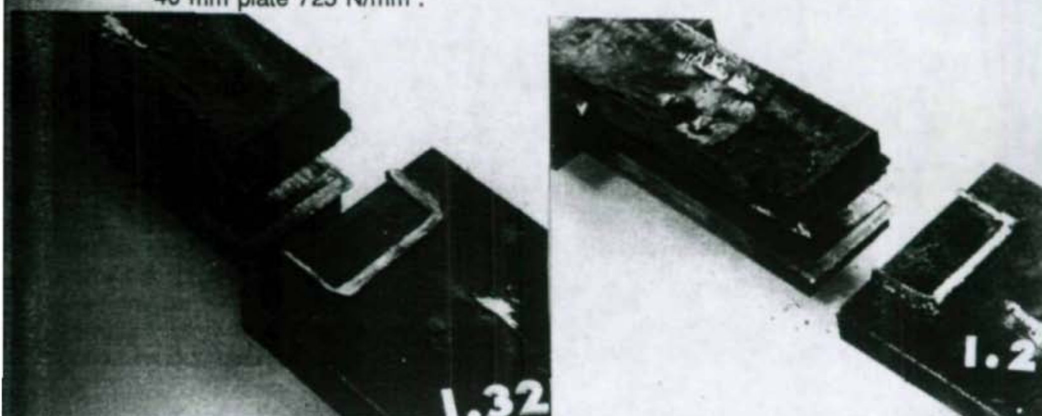

Figure 6: Some of the test specimens

3.3 Statistical evaluation

The statistical evaluation was carried out by the RWTH Aachen (Background documentation D.03, 1990). The evaluations were based on the two methods of Eurocode No 3: the mean stress method and the stress component method. From the evaluations, $\beta = 1.0$ was proposed for Fe E 460. For Fe E 235 and Fe E 355 Eurocode No 3 gives $\beta = 0.8$ and 0.9 respectively.

Figure 7 shows that also for Fe E 460, the stress component method gives the best results (Gresnigt, 1990).
Figure 8 gives an impression of the influence of the welding consumables.

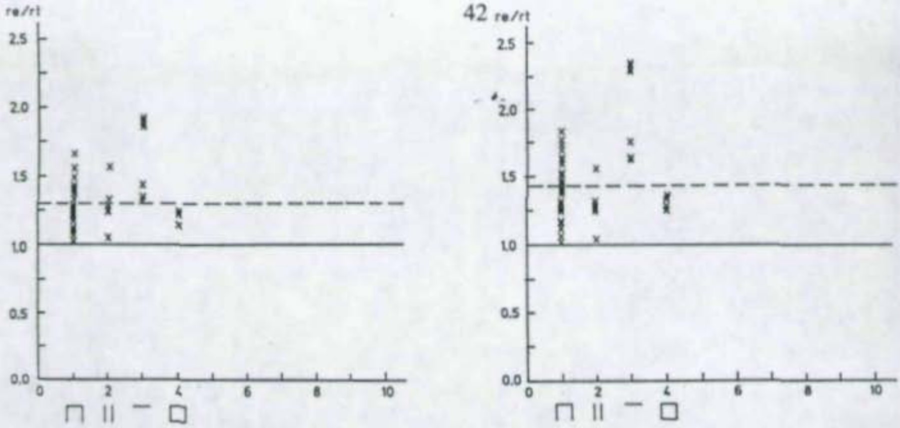

Figure 7 Sensitivity plots for various weld configurations: left the stress component method and right the mean stress method. The value re/rt is the quotient experimental resistance/theoretical resistance (without γ_M).

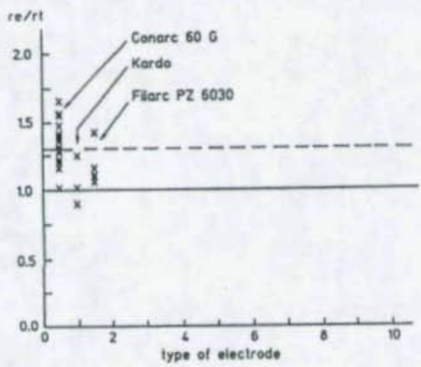

Figure 8. Influence of welding consumables.

3.4 Concluding remarks

- From figure 8 it appears that the undermatching electrode Kardo is not suitable for application when the rules of Eurocode No 3 are applied. Therefore, in Eurocode No 3 it is stated that the welding consumables must be at least matching. In the rules of Eurocode No 3 the ultimate strength of the parent metal is taken into account directly. The difference between the ultimate strength of the parent material and the weld material is accounted for by the ß-value (Gresnigt, 1990). In the Swedish rules, the average of the strength of both materials is taken as the basis for the calculation of the design strength (Treiberg, 1991). Especially for high strength steels this seems important. Application of the Swedish rules to the present test series gives less scatter.
- The test results indicate that the more ductile Kardo electrode does not give a better deformation capacity. The reason for this is probably that in an undermatched situation the strains mainly occur in the weld material, while in an

overmatched situation the strains mainly occur in the adjacent plate material. So, the advantage of a more ductile weld material could not be proven in these tests.
- Some test specimens failed with very little deformation, leading also to low failure loads. The present test specimens had a rather short weld length. It is well known that a longer weld gives rise to a more uneven strain distribution. The question arises whether the rules of Eurocode No 3 regarding the strength of "long welds" are also applicable for Fe E 460. It is felt that more research is necessary.

4. FINITE ELEMENT CALCULATIONS

4.1 Introduction

To evaluate the test results further and to obtain more insight in the influence of the various parameters on the strength and deformation capacity of fillet welded connections in high strength steels, finite element calculations are carried out at the Delft University of Technology. More in particular the research is concentrated on the influence of the following parameters:
- weld length,
- weld thickness compared to plate thickness,
- matching, overmatching, undermatching weld material and toughness properties,

on the:
- strain distribution,
- deformation capacity and
- strength

of fillet welded connections.

4.2 Calculations

At present, calculations are completed for the geometry of test specimen 1.17. Figure 9 gives the used finite element model. The element type is a solid element with 20 nodes. The computer programme is MARC.

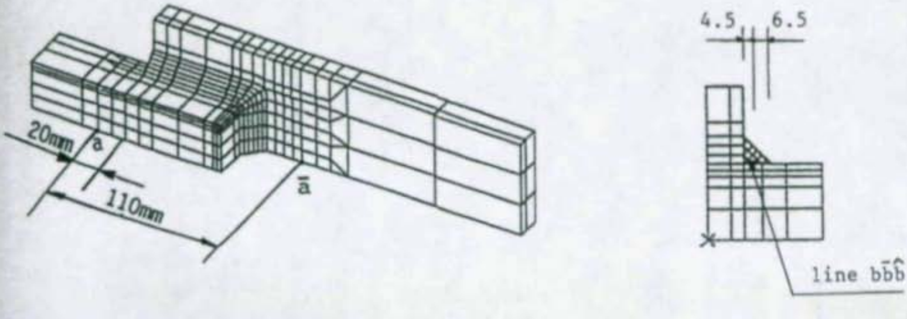

Figure 9. Finite element mesh

Figure 10 gives the applied stress strain relationships for the matching, the undermatching and the overmatching situation.
Figure 11 gives strain distributions for the side fillet and for the end fillet weld. The "rupture line" indicates where the largest strains occur. These lines are in accordance with the test results in figure 3.

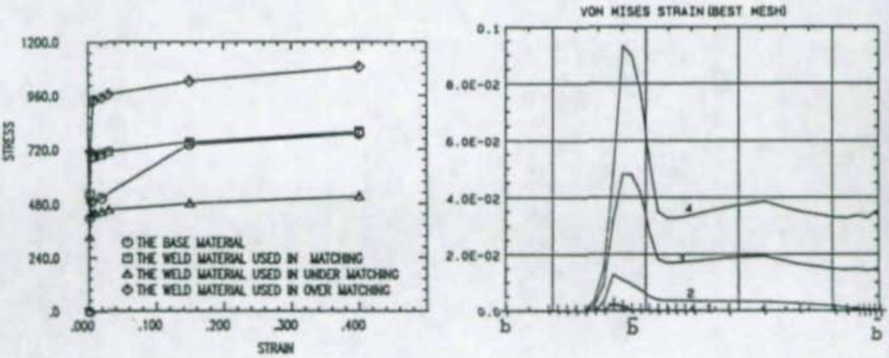

Figure 10. Stress-strain relationship in the finite element calculations and strain distribution along the side fillet weld on the edge between the fillet and the parent material 4.5 mm from the root. Point b̄ is the point where the tension plate ends; point b̂ is the point where the cover plate ends. The lines are given for various load levels.

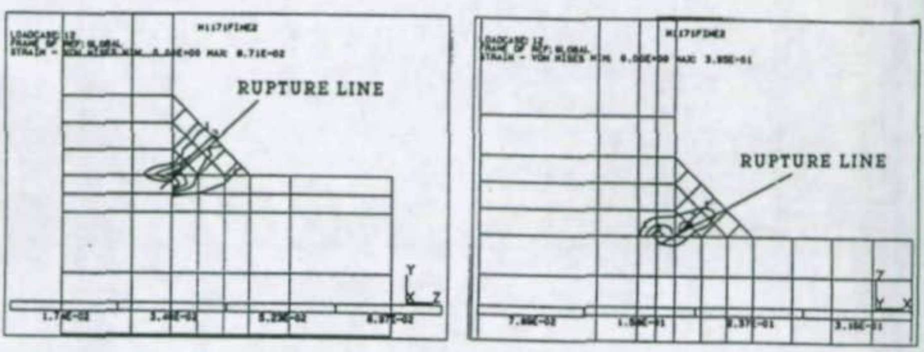

Figure 11. Von Mises strain distributions in the side fillet weld and in the end fillet weld (matching case).

4.3 Concluding remarks

Sofar, calculations have been carried out for undermatched, matched and overmatched weld material. In all three cases the dimensions are kept the same. From the results up till now, the following conclusions may be drawn.

- The undermatching conditions gives the highest strains in the weld, while the overmatching condition causes the highest strains in the adjacent parent material.
- Generally, an undermatching weld material has a better ductility. However, this ductility is (partly) necessary to accommodate for the higher strains in undermatched weld material.
- Further calculations are necessary to investigate the influence of the parameters mentioned before.

5. CONCLUSIONS

The conclusions regarding the test results are given in section 3.4 of this paper. Those regarding the finite element calculations are given in section 4.3.

6. REFERENCES

Annex D of Eurocode No 3, "The use of steel grade Fe E 460", January, 1991

Background Documentation D.03: "Evaluations of test results on welded connections made from Fe E 460 in order to obtain strength functions and suitable model factors", Eurocode No 3 Editorial Group, April 1990

Eurocode No 3: "Design of steel structures, Part 1 - General rules and rules for buildings", Volume 1, April 1990

Gresnigt, A.M.: "Calculation of fillet welds in Eurocode No 3", Rivista Italiana della Saldatura, 6-1990, pp 591-598, based on: doc IIS/IIW-XV-714-89, 1989

International Institute of Welding: "Recommended practice for testing the strength of fillet welds in structural steels and aluminium alloys", Doc IIS/IIW-685-81, Welding in the World, Volume 19, No 9/10, 1981

Snijder, H.H., De Koning, C.H.M., Gresnigt, A.M., Bijlaard, F.S.K.: "Tests on welded connections in Fe E 460", TNO-IBBC report B-89-241, 1989

Treiberg, T.: "Influence of base and weld metal strength on the strength of welds", Second International Workshop Connections in Steel Structures: Behaviour, Strength and Design, Pittsburgh USA, April 1991.

INFLUENCE OF BASE AND WELD METAL STRENGTH ON THE STRENGTH OF WELDS

Tom Treiberg[1]

Abstract

The strength of statically loaded welds is strongly influenced by the strength of the base material and the strength of the deposited weld metal. In national design codes both these variables are usually not taken into account explicitly in the design formulas. This paper presents some tests which indicate that the design strength for welds can be based on a mean value of the strength of the parent material and the weld metal, provided the difference is not too great. This approach seems to be safe for steel grades with ultimate tensile strengths up to about 800 N/mm^2, if a reduction factor of 0.9 on the mean value is applied.

1. INTRODUCTION

The strength of statically loaded welds depends on different factors of which the strength of the base material and the strength of the weld metal have the strongest influence. For extra high strength steel, the heat input from welding is also a very important variable.

Extensive research has been carried out around the world in the field of weld strength, in particular regarding calculation models, failure criterias etc. However, few investigations are made on the influence of weld metal strength on the strength of welds, above all where the actual strength of the welding consumables are measured.

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2. NATIONAL REGULATIONS

There is a big difference between various national codes with respect to calculation of the design weld strength. In the American Load and Resistance Factor Design Specification (1986) the strength of welds is governed by the strength of either the base material or the deposited weld metal. In some cases, undermatching weld metal is not allowed. The Canadian Standard for Limit States Design of Steel Structures (1989) has similar rules. Both these regulations are valid also for standardized extra high strength steel.

In Eurocode 3 (1989), a modification of the β-formula is adopted for fillet welds. The β-formula was originally presented by the International Institute of Welding (IIW, Commission XV, 1976). Eurocode 3 expresses the design resistance of the fillet weld as

$$f_{vw} = \frac{f_u/\sqrt{3}}{\beta_w \gamma_{Mw}} \tag{1}$$

where
f_{vw} = the design shear strength of the weld
f_u = the nominal ultimate tensile strength of the weaker part joined
$\gamma_{Mw} = 1.25$
β_w = correlation factor according to Table 1

The weld metal shall be compatible with the parent metal in terms of mechanical properties. Therefore, the design resistance of a full penetration butt weld may be taken equal to the design resistance of the base material.

The design rules in Eurocode 3 (1989) are applicable for steel grades with nominal ultimate tensile strengths up to 510 N/mm^2. At present, work is in progress extending Eurocode 3 to include also steel grades with higher strength values. In the first step Fe E 460, with an ultimate tensile strength of 580 N/mm^2, is likely to be included.

For common steel grades, the strength of the consumable is normally higher than the strength of the parent material. This is implicitly taken into account by the β_w-factor. The value of β_w is based on a lot of tests, of which the actual electrode strength in most cases were never measured.

If the β-formula shall be extended to include also welds in extra high strength steel, several questions arise. What β_w-factors can be used? Can the β_w-factor also apply to welds made with undermatching electrodes, or must the validity of the rule be limited to connections with matching weld metal? For extra high strength steel an undermatching "soft" welding consumable is preferable in many cases, since it decreases the risk of cracks in the weld.

One way to take the weld metal strength into consideration is adopted in the Swedish Regulations for Steel Structures (1987). The weld strength is based on the mean value of the nominal ultimate strengths of the base material and the electrode material, provided that the difference is not greater than 300 N/mm^2.

$$f_{wd} = \frac{\varphi\,(f_u + f_{ue})/2}{1.2\,\gamma_n} \tag{2}$$

where

f_{wd} = the design strength of the weld
f_u = the nominal ultimate tensile strength of the base material
f_{ue} = the nominal ultimate tensile strength of the electrode material
ϕ = 0.9 in most cases
γ_n = partial coefficient with respect to safety class

The factor ϕ takes into account the reduction of strength partly due to slag inclusions and porosity and partly due to high heat input from welding. Tests have shown that defects lower the static strength of a welded joint in proportion to the area they occupy (Biskup, 1968). The factor ϕ is normally taken as 0.9 but 1.0 may be used for butt welds of perfect quality.

Table 1 shows the value of β_W in Eq. (1) and the equivalent β_W-value derived from Eq. (2) above, when f_{ue} = 510 N/mm^2 and ϕ = 0.9. The most common standardized electrodes have a nominal ultimate tensile strength of 510 N/mm^2.

Table 1. Value of β_W in Eq. (1) and equivalent β_W-value derived from Eq. (2).

Steel grade	f_u (N/mm^2)	β_W (EC3)	β_W [Eq. (2)]*
Fe E 235	360	0.8	0.92
Fe E 275	430	0.85	1.02
Fe E 355	510	0.9	1.11

* f_{ue} = 510 N/mm^2 and ϕ = 0.9

3. TEST RESULTS

As pointed out before, the influence of the electrode strength on the weld strength is not very much investigated. For common steel grades the electrode material is normally at least as strong as the base material. Hence, one can assume that it is safe to base the weld strength on a mean value of the parent material strength and the weld metal strength, since the latter is likely to have the strongest influence. In the following some test results are presented which indicate that the mean value might be an acceptable basis for the design weld strength even with higher steel grades and moderately undermatching electrodes.

3.1 Butt welded connections

Figs. 1–3 show the results from three test series on statically loaded butt welded connections in extra high strength steel. The tests were carried out in the research department of the Swedish steel producer SSAB.

Each test specimen were made of two steel plates, butt welded together. Steel grades with ultimate tensile strengths of 728–1324 N/mm^2 were used. The plate thickness varied between 5 and 16 mm. Welding methods were MMA and MAG. The electrode strength was matching or undermatching the base material. In some cases the welds were machined, in other cases the specimens were tested as welded.

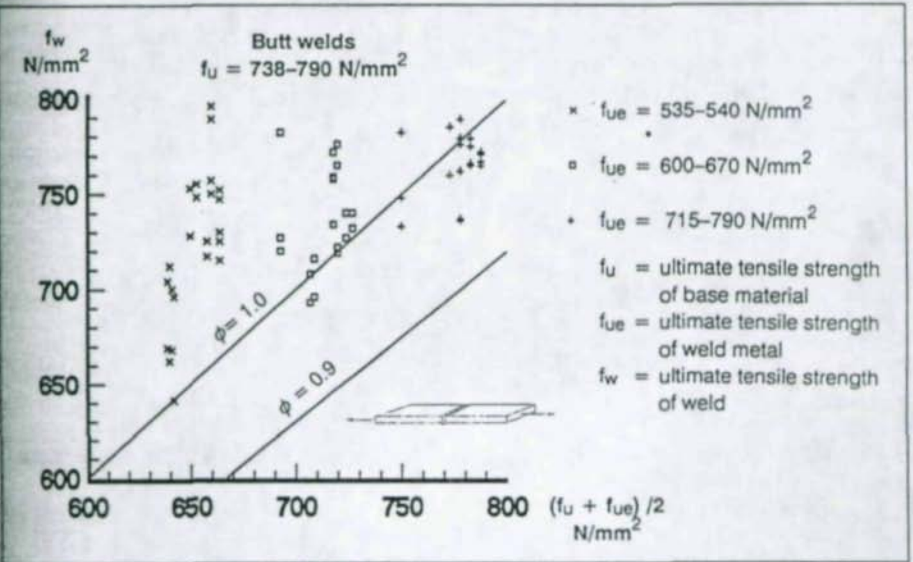

Figure 1. Ultimate tensile strength of butt welded connections versus the mean value of the ultimate tensile strength of the base material and the weld metal (Westerberg, 1981)

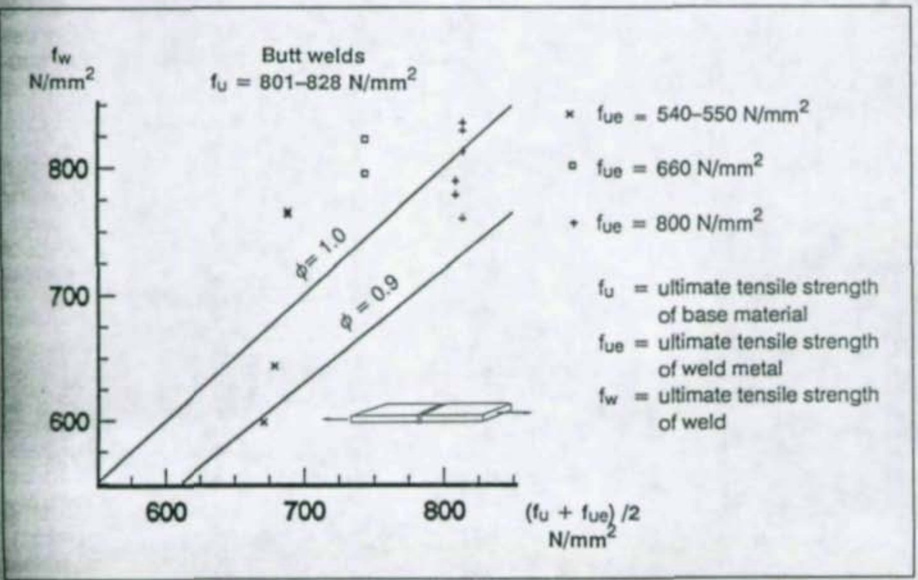

Figure 2. Ultimate tensile strength of butt welded connections versus the mean value of the ultimate tensile strength of the base material and the weld metal (Bergqvist et al., 1981)

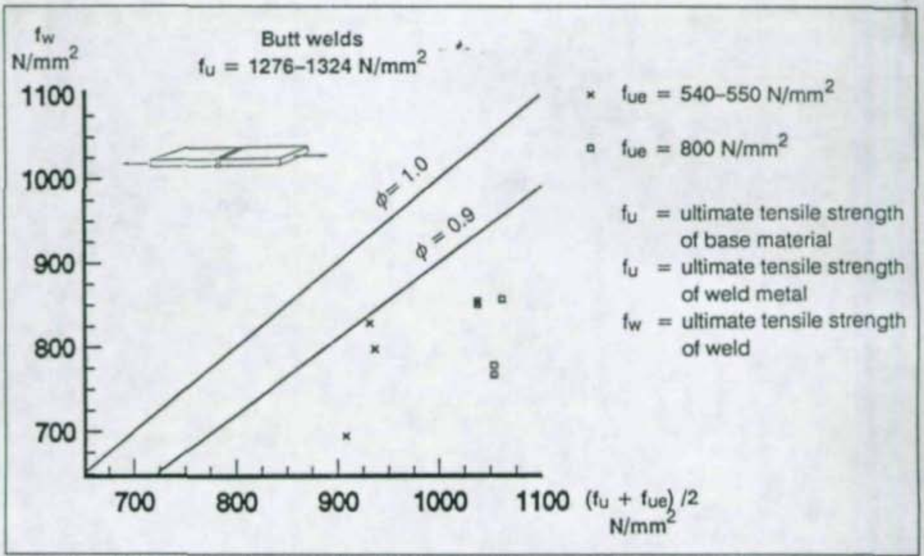

Figure 3. Ultimate tensile strength of butt welded connections versus the mean value of the ultimate tensile strength of the base material and the weld metal (Bergqvist et al., 1981)

In Figs. 1–3 the measured ultimate tensile strength is plotted versus the mean value of the ultimate tensile strength of the base material and the weld metal. Two lines, corresponding to ϕ = 0.9 and ϕ = 1.0 respectively in Eq. (2), are also shown in the diagram. Since the actual electrode strength was never measured, the mean value is based on the catalogue strength values of the electrodes and actual strength values of the base material. Experience has shown that the actual strength of these electrodes are often 0–10 % greater than the catalogue values. This means that the actual mean values are likely to be approximately 0–5 % greater.

The test results in Figs. 1 and 2 show that "the mean value method", with a reduction factor ϕ of 0.9, is in these cases on the safe side except for one specimen. In many tests, the specimens welded with low strength electrodes were as strong as specimens welded with matching electrodes. One reason for that is the beneficial effect of the triaxial stress state which is formed by the restricted contraction of the narrow zone of undermatching strength.

The tension tests according to Fig. 3 were made with extremely high strength steel, welded with consumables with an ultimate strength that is 500–800 N/mm^2 lower than the strength of the steel. In these cases all the test results fall below the line of ϕ = 0.9 in the diagram. It is obvious that traditional weld design rules could not be applied for welds whith these strength parameters of base and weld metal in combination.

There are many reasons for the big scatter of the test results in Figs. 1–3. In order to study the influence of heat input from welding, this parameter was varied in combination with different types of welding consumables and welding methods. A high heat input creates a wider zone of undermatching strength which showed to lower the static strength of the joint. Also machining of the welds was detrimental for the static strength.

3.2 Fillet welded connections

Tests on fillet welded connections in high strength steel have been conducted at Lappeenranta University in Finland (Niemi, 1988). Specimens with side fillet welds, end fillet welds and a combination of side and end fillet welds were tested.

The steel had a measured ultimate tensile strength of 738–759 N/mm^2. Welding methods were MMA and MAG. The ultimate tensile strength of the welding consumables were measured to be 739 and 849 N/mm^2 respectively. The throat thickness of the fillet welds were nominally 3 mm.

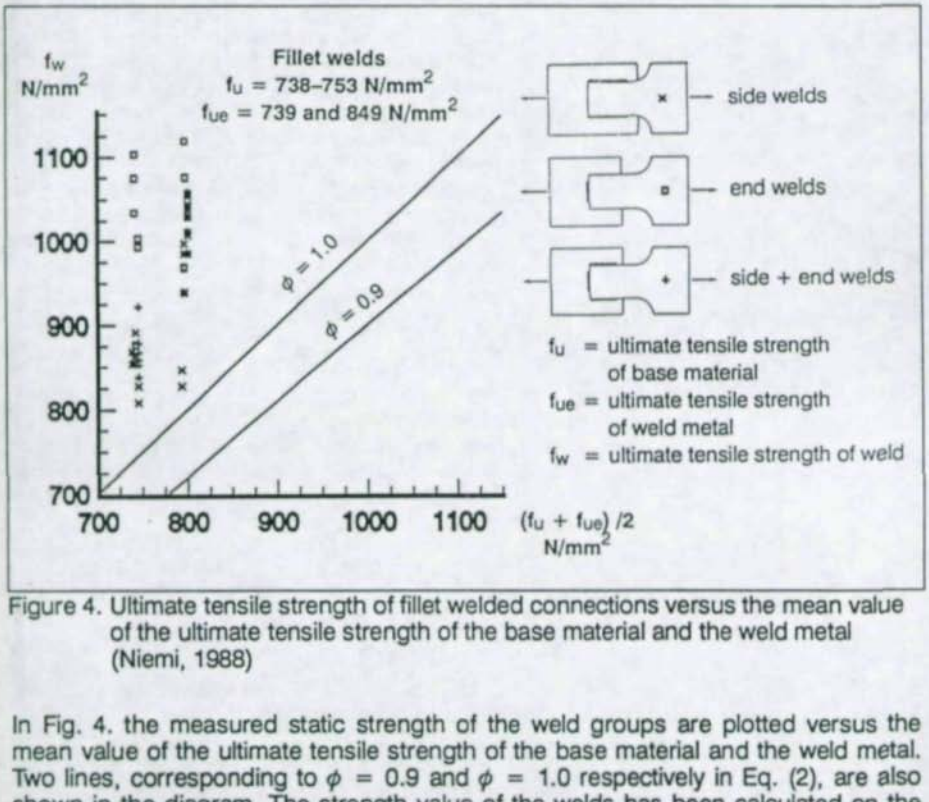

Figure 4. Ultimate tensile strength of fillet welded connections versus the mean value of the ultimate tensile strength of the base material and the weld metal (Niemi, 1988)

In Fig. 4. the measured static strength of the weld groups are plotted versus the mean value of the ultimate tensile strength of the base material and the weld metal. Two lines, corresponding to $\phi = 0.9$ and $\phi = 1.0$ respectively in Eq. (2), are also shown in the diagram. The strength value of the welds has been calculated on the basis of measured weld failure surface areas and von Mises equivalent stress formula on the throat section (Treiberg, 1989). This stress component model is given as an alternative design method in Annex M of Eurocode 3 (1989). For specimens with both side welds and end welds, it has been assumed that the load bearing capacity of the weld group is the sum of the load bearing capacity of each individual weld.

In the diagram in Fig. 4 all the test results fall safely above the $\phi = 0.9$ line. In fact, even $\phi = 1.0$ is safe with these combinations of base material and weld metal. One must bear in mind that only matching electrodes are used. Fig. 4 also shows that the calculated strength values for end weld specimens are higher than for side weld specimens. The reason for this is mainly that the design model with von Mises equivalent stress formula is very conservative for transversely loaded fillet welds.

4. SUMMARY AND CONCLUSIONS

The strength of statically loaded welds is strongly influenced by the strength of the base material and the strength of the deposited weld metal. Not many national design codes take both these variables into account in the design weld strength formulas.

Welding in common steel grades is normally done with an electrode material which is at least as strong as the base material. Hence, you can assume that it is safe to base the weld strength on a mean value of the parent material strength and the weld metal strength, since the latter is likely to have the strongest influence. This "mean value method" can be applied also for higher steel grades, provided the difference between the strength variables is not too great. Tests described above indicate that the method is safe for steel grades with ultimate tensile strengths up to about 800 N/mm^2 if a reduction factor of 0.9 on the mean value is applied.

More research is needed, especially on fillet welds made with undermatching welding consumables, to get a clear picture of the influence of the weld metal strength on the strength of welds.

5. REFERENCES

American Institute of Steel Construction
Load & Resistance Factor Design, Specification for Structural Steel Buildings, September 1986

Bergqvist L. et al.
Design with EHS and AR-plate, Design Manual E70, SSAB, Heavy Plate Division, Oxelösund, Sweden, 1981

Biskup J.T.
Investigation Related to Welding of Beam to Column Connections with Reference to Defect Evaluation Under Static Tension, Transactions of the Engineering Institute of Canada, Vol 11, No B-1, May 1968

Canadian Standards Association
Limit States Design of Steel Structures, CAN/CSA-S16.1-M89, December 1989

Commission of the European Communities
Eurocode No 3, Design of Steel Structures, Edited draft, Issue 2, November 1989

International Institute of Welding (IIW), Commission XV
Design rules for arc-welded connexions in steel submitted to static loads, Welding in the World, Vol 14, No 5/6, 1976

Niemi E.
Tests on Fillet Welded Joints with High Strength Steel Specimens, IIW Doc XV-696-89, Lappeenranta University of Technology, Lappeenranta, Finland, 1988

Treiberg T.
Analysis of the Tests of Niemi According to the Swedish Regulations, ECCS Doc TC10-89-TWG5-09, Swedish Institute of Steel Construction, Stockholm, 1989

Westerberg G.
Hållfasthet hos MAG- och MMA-svetsad 5 och 10 mm plåt i DOMEX 640 XP och 590 XP, Teknisk rapport SF 394/81, SSAB Svenskt Stål AB, Borlänge, 1981 (in Swedish)

FORCES IN BEAM-TO-COLUMN CONNECTIONS

by

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Abstract

From analysis of the geometry and stiffness of connection components, forces in those components may be computed. By generalizing the results, proportional force distributions can provide valuable rule-of-thumb estimates for connection design in either semi-rigid or rigid construction. Examples are treated for weak-axis beam-to-column connections, tee-stub connections, and top-and-seat- angle connections.

1 Introduction

Forces on the detail components of three types of beam-to-column connections are analyzed employing a new approach using a linear structural analysis program and beam-type elements. The key to the procedure is use of dummy rigid members to space angles or plates fastened to the outer surfaces of beam or column at the proper distance from member centerlines. Use of correct coordinate geometry results in better equilibrium calculations. Realistic values for stiffness of joint components as well as main beams and columns provide for determination of displacements. The dummy rigid members serve to invoke the plane-sections-remain-plane concept of typical mechanics of materials solutions.

For a weak axis beam-to-column moment connection, top-and-bottom horizontal connection plates provided for moment resistance are found to carry about 50 percent of the total vertical shear. As a result, serious bending occurs both out of the plane of the plate and in the plane of the plate, also adding to forces on edge welds.

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In a top-and-bottom tee-stub connection, bolts in the vertical legs of the tee-stubs are found to be pulled inward toward the tee stem in the top tee and pushed outward in the bottom (compression) tee.

Top angles of a top-and-seat-angle connection are found to carry from 50 percent to 84 percent of the vertical shear on the end of the beam, in contrast to some design methods which assume that only the seat angle carries vertical load.

From the results of this type of study, rules of thumb could be modified or developed to permit design of appropriate connections without requiring a computer analysis for every problem.

2 Background

As part of the preparation for an experimental program on connections, several analyses were conducted on three types of beam-to-column connection details. Schematic diagrams of the resulting force distributions are revealing.

3 Model of Connection Details

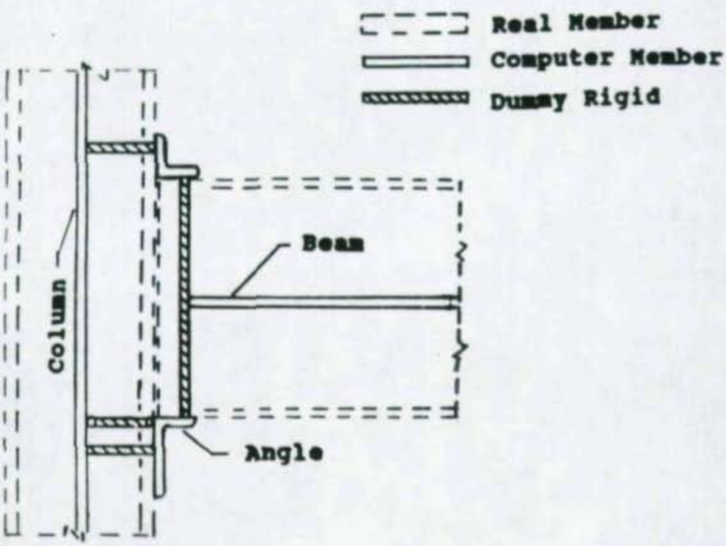

Figure 1: Dummy Rigid Members Space Connection Details for Proper Equilibrium

In the design of structural connections, it is valuable to have information on the forces in the detail material added to enable connecting the major members, i.e., continuity plates, shear tabs, angles, stiffeners and fasteners. Usually assumptions of forces are made in order to allow the application of routine mechanics of materials. This method, however, provides very little chance of discovering unexpected force paths. On the other hand, the application of finite element methods with large numbers of miniscule geometric shapes, amounts to overkill with reams of excess information.

A compromise can be made using ordinary linear structural analysis programs and modelling the added detail materials as complete beam or truss members having the appropriate geometry and stiffness for the complete member. Since the main structural members are modelled as one-dimensional "sticks", the trick is to position the connecting elements at the correct distance away from the centerlines of beams and columns so that their forces will be invoked with proper leverage (Fig. 1). Dummy rigid members inserted between beam or column centerline and the points of attachment of detail material will enable the proper equilibrium to be invoked while engaging the real stiffness of the detail material. Since the dummy members displace with the deflection or rotation of beams and columns, the concept of "plane sections remain plane" is preserved. Procedures for executing this concept were presented by (Driscoll, 1987).

4 Structural Assemblages

In order that the connected structural members will apply the proper combination of forces to a connection model, realistic structural assemblages must be assumed. Two assemblages were assumed in the examples studied: a single-bay, two-story frame, and a cantilever beam connected to a mid-height of a column fixed at its top and bottom. In the single-bay two-story frame, the connection to be studied is at each end of a beam located at the top of the first story and subjected to a central vertical concentrated load. Connections ranging from fully-rigid to pin-ended may be studied using this model. The model may be cut in half by the application of symmetry when only vertical loads are involved. The cantilever beam model stimulates a test setup recently used in cyclic loading of weak axis moment connections.

5 Weak-Axis Beam-to-Column Moment Connection

A weak axis beam-to-column connection tested by Heaton provides the first case (Heaton, 1987b). Field bolts connect the beam web to a web plate while full penetration groove welds connect the beam flanges to flange connection plates. All of the detail material is fillet welded to the column flanges and web. The linear forces V and the bending moments VL on the separate parts are shown in exploded view of the connection (See Fig. 2.). The top and bottom connection plates carry the expected large normal forces from the beam flanges to the column flanges and web. In addition, each of the connection plates carries a vertical force of about 36 percent of the vertical shear V of the beam. In each horizontal plate about two-thirds of its vertical shear is transmitted to the column flanges at the edges of the plate, and the remaining third passes into the web plate as a vertical bearing force. The net effect is that about half the beam shear passes to the column web from the web plate and the edges of the horizontal connection plates. The other half passes to the column flanges through the side edges of the horizontal connection plates.

This information about forces can provide some guidance in the proportioning of plates and welds for such connections.

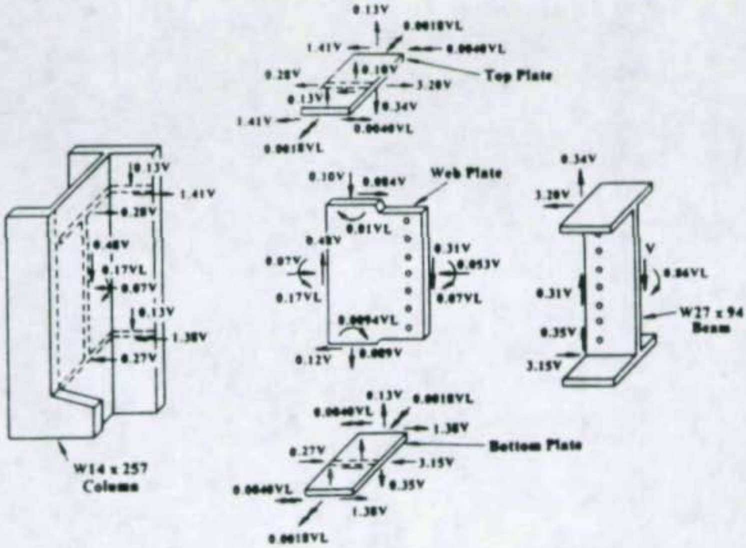

Figure 2: Exploded Diagram of Connection
Showing Member Forces

6 Tee-Stub Connection

As a semi-rigid connection, the tee-stub connection approaches a fully rigid moment connection. Tests on such connections were planned in the ATLSS Engineering Research Center ((Chasten, 1987)). The force distribution depicted in Fig. 3 represents a stage of loading at the limit of the elastic range. The overall moments for the structures are shown in the moment diagram and they are essentially the same as would be obtained with fully rigid joints. (Some obvious information is omitted to reduce clutter.)

The forces on the beam in the vicinity of the connection are carried out into the horizontal stems of the two tees. A large couple with horizontal forces 2.72 times the beam shear makes up the axial forces in the two tee stub stems. The vertical shears carried into the tee stems are each approximately half of the beam vertical shear. Very small bending moments are also imposed on the tee stems, but they approacn the maximum bending capacity of the stems.

The horizontal forces passed from the tees to the column pull at the top and push at the bottom as would be expected. However, an interesting combination is noted in the vertical forces. On the top tee, the vertical forces in the tee flanges are both in tension, with 0.73V upward and 0.23V downward with a net effect of 0.5V upward. The sketch labelled TEE BENDING shows that deformation could be expected to cause an inward pull against the fasteners. The reverse situation occurs on the bottom tee which is in

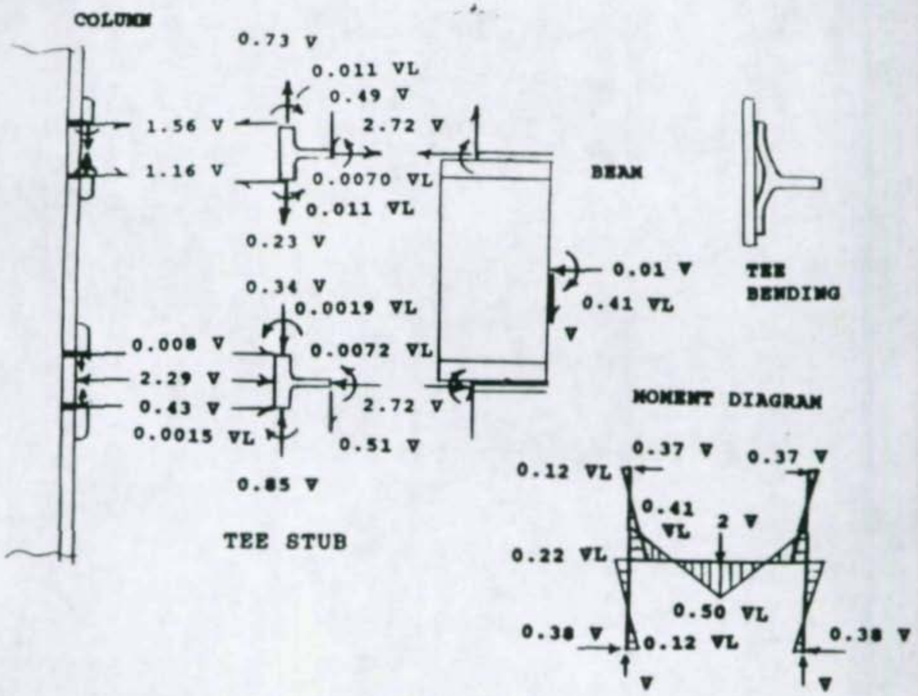

Figure 3: Forces on Tee-Stub Connection

compression. The vertical forces on the tee are both inward toward the stem, but their net vertical resultant is the appropriate half of the beam shear acting upward on the beam.

7 Top-and-Seat Angle Connection

The top-and-seat angle connection (without a stiffened seat) was analyzed in the elastic range and step-by-step up to ultimate load (Driscoll, 1987). It was studied in the two-story single-bay frame assemblage. A series of connections of similar type were tested in the ATLSS Center (Chasten, 1987) (Fleischman, 1991).

Surprisingly, in the elastic range analysis (See Fig. 4.), the top angle carried 84 percent of the beam vertical shear while the seat angle carried 16 percent. As the load increased, redistribution of forces increased until both top and seat angle each carried 50 percent of the vertical shear (Fig. 5). As the load increased, the axial force in the beam increased from 0.04V to 1.04V because of the bearing of the heel of the seat angle on the column flange caused by rotation of the beam ends. Comparison with test results gave general good agreement with the predictions, but showed a need for inclusion of fastener stiffness and boundary conditions in the model. This need also becomes apparent in infinitesimal finite element calculations.

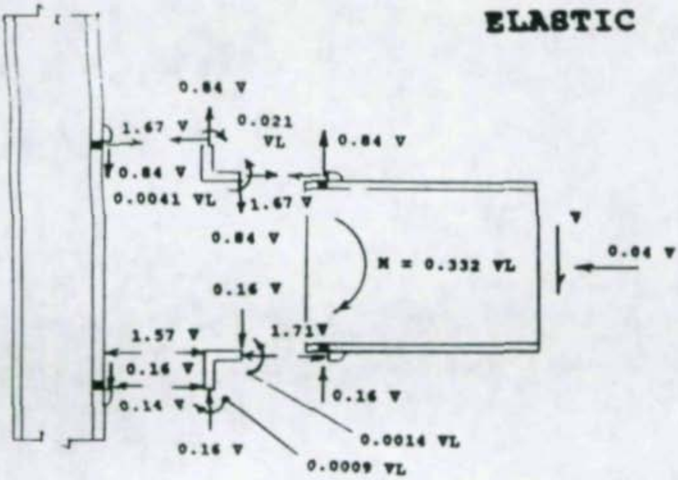

Figure 4: Elastic Forces on Seat-and-Top-Angle Connection

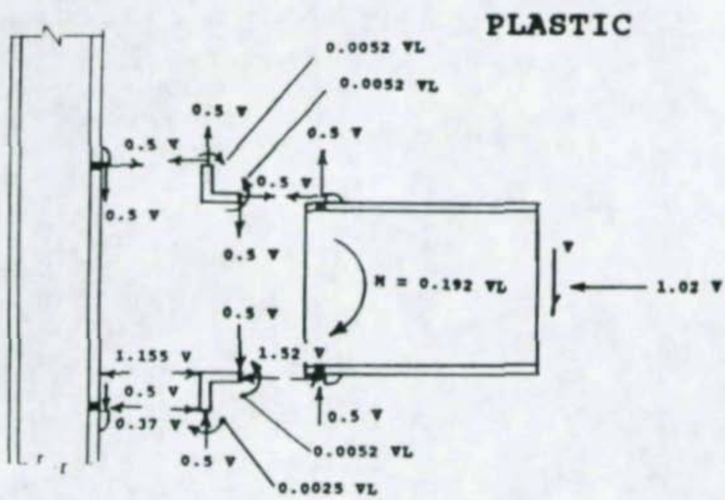

Figure 5: Plastic Forces on Seat-and-Top-Angle Connection

8 Conclusions

- The studies reported here provide information that is probably accurate enough for use in proportioning fasteners and details of connections for design.
- Rules-of-thumb could be developed or modified to permit design without conducting the computer analysis for every problem.
- On the other hand, the method requires few enough added node points in a line-type structure that it could be feasible to model an entire structure with some semi-rigid connections.
- This admittedly crude analysis can provide valuable guidance in the interpretation of results from "infinitesimal" finite element solutions.

Better knowledge of the behavior of connections will help in the understanding of the performance of structures and in the development of innovations for design of the structures of the future.

References

Chasten, C. P., Fleischman, R. B., Lu, L. W., and Driscoll, G. C. 1987
SEMI-RIGID STEEL CONNECTIONS AND THEIR EFFECTS ON STRUCTURAL STEEL FRAMES. ATLSS Report 87-02, Lehigh University.
Experimental and Theoretical Proposal.

Driscoll, George C. 1987
ELASTIC-PLASTIC ANALYSIS OF TOP-AND-SEAT-ANGLE CONNECTIONS. Constructional Steel Research Special Issue.
Fritz Engineering Laboratory Report No. 505.1.

Fleischman, Robert B., Chasten, Cameron P, Lu, Le-Wu, and Driscoll, G.C. 1991
TOP-AND-SEAT-ANGLE CONNECTIONS AND END-PLATE CONNECTIONS: SNUG VS. FULLY PRETENSIONED BOLTS. Engineering Journal, AISC 28(1):18-28.

Heaton, Kenneth A., and Driscoll, George C. 1987b
CYCLIC TESTS OF BEAM-TO-COLUMN WEB MOMENT CONNECTIONS. Fritz Engineering Laboratory Report 504.2, Lehigh University.

PLASTIC ANALYSIS AND SIMPLIFIED DESIGN OF THE COMPRESSION ZONE OF A BEAM-TO-COLUMN CONNECTION

Jean Marie ARIBERT

Abstract

The formulae of crushing and buckling resistances specified in Eurocode 3 and AISC are compared statistically with the experimental data of sixty tests on european and american steel members. To achieve a suitable adjustment, a theoretical modelling is developed, which is based on plastic hinge mechanisms taking account of second order geometrical effects and initial imperfections. In practice, it is possible to propose a simplified design approach leading to the same accuracy as the theoretical modelling.

1 - INTRODUCTION

This paper deals with the compression zone of the column web of welded or bolted beam-to-column connections **(fig. 1)**. For economical reasons, there is nowadays a tendency to use unstiffened column webs, but in return the resistance of the compression zone needs to be checked carefully. Failure can result from two different phenomena, namely crushing and local buckling of the column web.In the case of ordinary rolled I or H members, the crushing resistance often prevails ; but the more and more frequent use of welded built-up members, slender webs and high steel grades (for example, grade FeE 460 in Europe) requires one to pay particular attention to the local buckling resistance which may influence the crushing resistance. At first sight, it seems that a thorough analysis of this interaction is complicated because it implies taking account of the geometrical and material non-linear behaviour of the column. Moreover, although the column web is especially concerned by failure, participation of the column flanges must be introduced into the analysis if an accurate determination of the total resistance of the column is wanted.

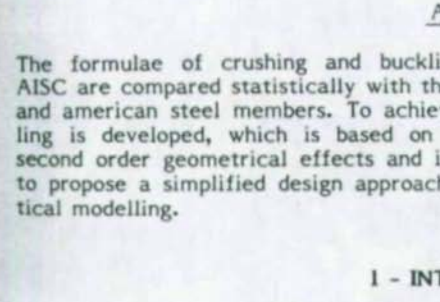

Fig. 1

In this paper, only the situation of symmetrically loaded beam-to-column connections

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is investigated and secondary effects caused by normal stresses in the column due to the frame behaviour are neglected ; nevertheless some generalizations compensating these aspects are suggested at the end. At first, we are going to examine how the topic is treated in recent design codes.

2. DESIGN SPECIFICATIONS IN EUROPEAN AND AMERICAN CODES

2.1 Eurocode 3 (paragraph 5.7 and annex J.3.5.1)

The last draft of Eurocode 3 (dated from November 1990) specifies that the design crushing resistance is given by :

$$F_{c.Rd} = f_{yw}\, t_w\, b^{(1)}_{eff} / \gamma_{Mo} \qquad (1)$$

where the effective width of the column web in compression (fig. 2) is equal to :

$$b^{(1)}_{eff} = t + 2a\sqrt{2} + 2\, t_p + 5\, k \quad ; \qquad (2)$$

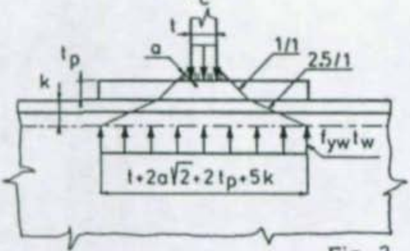

Fig. 2

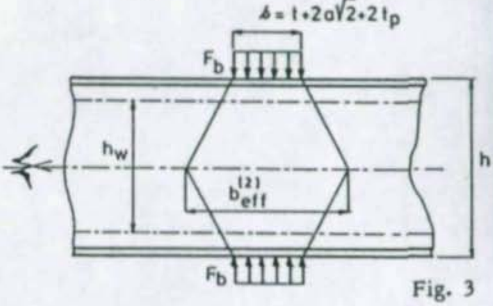

Fig. 3

k is the distance from the outer face of flange to web toe of fillet ; so :

$$k = t_f + r_c \qquad (3)$$

for a rolled I or H section column where r_c is the root radius.

Moreover, the design buckling resistance is calculated by considering the web as a virtual compression member with an effective breadth (fig. 3) equal to :

$$b^{(2)}_{eff} = [h^2 + (t + 2a\sqrt{2} + 2\, t_p)^2]^{1/2} \quad , \qquad (4)$$

and by using the european buckling curve C. So :

$$F_{b.Rd} = \chi\, f_{yw}\, t_w\, b^{(2)}_{eff} / \gamma_{M1} \qquad (5)$$

where χ is a reduction factor which depends on the non-dimensional slenderness $\bar{\lambda}$ corresponding to the web depth.

It should be noted in formulae (1) and (5) that γ_{M0} and γ_{M1} are partial safety factors.

Finally, the ultimate resistance of the compression zone should be taken as :

$$F_{u.Rd} = \text{smaller of } (F_{c.Rd}, F_{b.Rd}) \,. \qquad (6)$$

2.2 AISC Specification (chapter K)

In the LRFD Specification (dated from September 1986), the design crushing resistance is determined by :

$$F_{c.Rd} = f_{yw}\, t_w\, b_{eff}\, \phi \qquad (7)$$

where ϕ is a safety factor (equal to 1) and b_{eff} has now the simpler form :

$$b_{eff} = t + 2\, t_p + 5\, k \ . \qquad (8)$$

Therefore, these formulae are similar to the corresponding ones in Eurocode. On the contrary, AISC considers the buckling resistance based on concept of critical stress equal to the yield limit, so that the specified design formula is :

$$F_{b.Rd} = \phi\ 10\ 750\, \frac{t_w^3}{h_w} \sqrt{f_{yw}} \qquad (9)$$

with Newtons and mm as units and $\phi = 0.9$. Obviously, condition **(6)** must be applied again.

3. COMPARISON BETWEEN THE CODES AND EXPERIMENTAL DATA

Main characteristics and failure loads F_u^{exp} of sixty tests are plotted in **table I.** Thirty four of them were carried out in Europe, namely twenty nine tests in France [1] for steel grade going from FeE 235 to FeE 460, and five tests in the Netherlands [2] ; the other twenty six tests are extracted from the american literature [3, 4, 5, 6]. All the tests concern rolled H and I sections, except one test on a welded slender section (called I 775 x 200 in the table), with loading in symmetrical local compression. It should be noted that the mentioned yield limit of the column web, f_{yw}, is the measured value, not the nominal one.

In the same table, we can do a comparison between the experimental ultimate loads, F_{ui}^{exp}, and the theoretical resistances, F_{ui}^{th}, given by condition **(6)** and formulae **(5)**, **(7)** and **(9)** ; there is no column assigned to formula **(1)** in the table because of lack of information on the throat thickness, a, of the fillet welds used in the american tests. To make the comparisons easier, the following statistical quantities will be introduced :

- the mean value :

$$\bar{b} = \frac{1}{n} \sum_{i=1}^{n} \frac{F_{ui}^{exp}}{F_{ui}^{th}}$$

where n is the number of tests ;

- the standard deviation of the errors terms $\delta_i = \dfrac{F_{ui}^{exp}}{\bar{b}\, F_{ui}^{th}}$, i.e. :

$$s_\delta = \sqrt{\frac{1}{n-1} \left(\sum_{i=1}^{n} \delta_i^2 - n\, \bar{\delta}^2 \right)}$$

(with obviously $\bar{\delta} = 1$) ;

REF	SECTION	CHARACTERISTICS				EXPERIMENTAL RESISTANCE	CALCULATED RESISTANCES						
		f_{yw} (MPa)	t_w (mm)	t (mm)	t_p (mm)	F_u^{exp} (kN)	EC3 F_b (kN)	AISC F_c (kN)	AISC F_b (kN)	MODEL F_c (kN)	MODEL F_b (kN)	REL. (26)-(27) $\bar{\lambda}$	REL. (26)-(27) F_u (kN)
LAB. STRUCTURES INSA RENNES [1]	HEB 140	320	7	10	-	365	253	291	717	279	-	0,56	279
	HEB 200	320	9	10	-	770	439	504	1046	475	-	0,60	475
	HEB 260	320	10	10	-	870	571	696	1087	668	777	0,70	635
	HEB 140	320	7	20	-	375	255	314	717	301	-	0,58	301
	HEB 200	320	9	15	-	780	439	518	1046	489	-	0,61	489
	HEB 200	320	9	20	-	825	440	533	1046	504	-	0,62	504
	HEB 260	320	10	20	-	880	572	728	1087	700	792	0,72	677
	HEB 160	275	8	15	-	550	294	341	878	334	-	0,53	334
	HEB 200	263	9	15	10	760	383	477	952	462	-	0,59	462
	HEB 200	263	9	15	15	800	389	501	952	497	-	0,62	497
	HEB 200	263	9	15	20	840	393	525	952	543	534	0,64	543
	HEB 200	263	9	15	30	940	405	572	952	635	579	0,71	639
	IPE 140	303	5,1	10	-	175	127	125	219	105	158	0,61	105
	HEA 260	335	7,8	15	-	608	346	504	507	472	481	0,87	405
	IPE 220	284	6,2	10	-	300	177	209	245	189	218	0,76	177
	IPE 360	326	8,3	15	-	530	284	437	369	382	425	0,91	318
	HEA 140	484	5,7	10	-	365	230	312	463	311	297	0,84	273
	HEA 160	481	6,7	10	-	530	315	409	660	405	406	0,79	369
	HEA 160	475	6,6	10	-	322	305	401	629	386	403	0,79	332
	HEA 200	542	7,7	10	-	760	443	655	834	619	602	0,89	524
	HEA 200	542	7,8	10	-	740	458	674	871	632	620	0,88	540
	HEAA 200	610	5,7	10	-	402	233	525	369	495	376	1,25	326
	HEAA 300	544	6,8	10	-	588	267	747	379	722	498	1,41	433
	IPE 240	566	6,1	10	-	434	198	464	308	407	383	1,21	273
	IPEA 360	524	6,6	15	-	490	160	520	233	500	328	1,48	288
	HEA 160	481	6,7	10	10	380	322	479	662	472	418	0,86	410
	HEA 160	481	6,7	10	15	620	326	511	662	544	429	0,92	451
	HEA 160	481	6,7	10	20	664	332	543	664	628	439	0,98	496
	1775 x 200	410	7,5	15	-	314	91	303	122	378	236	1,67	197
STEVIN [2]	IPE 240	367	6,2	40	-	380	183	373	258	343	277	1,10	249
	IPE 240	425	6,2	40	-	320	190	432	278	398	300	1,18	273
	HEA 240	317	7,5	40	-	483	313	487	492	494	422	0,92	408
	HEA 300	337	8,5	40	-	630	411	743	600	737	380	1,06	364
	HEA 300	286	12	40	-	980	618	995	806	1030	837	1,02	805
FRITZ ENGIN. LAB. [3]	W8 X 48	237	10,3	12,7	-	610	419	361	1116	433	-	0,52	433
	W8 X 58	230	12,9	12,7	-	901	612	528	2258	601	-	0,44	601
	W 10 X 66	276	11,6	12,7	-	782	615	550	1391	631	768	0,59	631
	W 10 X 72	241	12,9	12,7	-	845	660	557	1787	646	-	0,50	646
	W 12 X 40	277	7,5	12,7	-	456	239	323	307	350	306	0,93	288
	W 12 X 65	257	9,9	12,7	-	636	436	417	675	504	497	0,74	480
	W 12 X 85	261	12,6	12,7	-	1101	712	616	1400	742	845	0,62	742
	W 14 X 61	250	9,6	12,7	-	612	386	411	519	472	452	0,81	426
	W 14 X 68	264	10,6	12,7	-	730	495	502	716	579	578	0,77	537
	W 14 X 84	271	11,5	12,7	-	383	602	585	928	720	699	0,76	673
	W 14 X 103	266	12,6	12,7	-	1112	723	656	1215	842	835	0,71	815
FRITZ ENGIN. LAB. [4,5]	W 10 X 39	841	8,7	12,7	-	1126	477	938	993	1010	717	1,17	699
	W 12 X 45	816	8,7	12,7	-	1157	417	1091	807	1206	720	1,41	721
	W 12 X 31	275	6,9	12,7	-	271	163	193	218	221	199	0,89	188
	W 10 X 29	287	7,8	12,7	-	401	253	236	382	296	295	0,78	272
	W 10 X 54	399	9,7	12,7	-	957	514	550	960	702	622	0,83	623
	W 8 X 67	213	14,6	23,6	-	1112	639	554	2914	640	-	0,39	640
	W 12 X 120	674	17,8	28,2	-	4360	2379	2732	6230	3200	3086	0,79	2921
	W 12 X 45	372	9,8	12,7	-	739	465	509	767	571	557	0,82	510
	W 12 X 36	763	8,2	12,7	-	1046	310	730	600	890	533	1,38	341
	W 10 X 29	291	7,9	12,7	-	423	261	242	400	292	301	0,76	272
	W 12 X 27	281	6,8	12,7	-	285	159	192	210	229	195	0,92	189
	W 12 X 45	392	9,8	12,7	-	748	472	537	787	602	574	0,84	528
WATER. [6]	W 8 X 17	344	5,8	7,8	-	265	151	174	222	184	182	0,88	161
	W 12 X 14	371	5,7	5,7	-	178	98	130	136	126	129	0,91	103
	W 14 X 22	357	6,5	8,1	-	274	134	237	178	220	203	1,04	166

Table I

- the correlation coefficient between experimental and theoretical values :

$$\rho = \frac{\sum_{i=1}^{n} F_{ui}^{exp} F_{ui}^{th} - n \overline{F}_u^{exp} \overline{F}_u^{th}}{n\, S_F^{exp} S_F^{th}} \; .$$

where $\overline{F}_u^{exp}$ and $\overline{F}_u^{th}$ are the mean values respectively of experimental ultimate loads and theoretical resistances, and S_F^{exp} and S_F^{th} the corresponding standard deviations. If ρ is greater than 0.9, the correlation may be considered sufficient for a statistical interpretation with the theoretical formulae concerned. It is pointed out that the safety factors γ_{M0}, γ_{M1} and ϕ have been taken equal to 1 because f_{yw} was measured in the tests and these factors do not affect the two main quantities ρ and s_δ for the statistical interpretation. The results of different comparisons are summarized in **table II**. The upper part of this table deals only with twenty three french tests where failure by crushing was demonstrated by experimental evidence and throat thicknesses, a, were known (all of the tests referred to [1] in **table I** except those concerning the following sections : IPE 360 - HEAA 200 - HEAA 300 - IPE 240 with f_{yw} = 566 N/mm² - IPEA 360 and I 775 x 220). It appears that adjustment by formula **(7)** - **(8)** is slightly better than the one by formula **(1)** - **(2)**, which confirms that the term $2a\sqrt{2}$ is not really significant, as it has already been demonstrated by a detailed experimental study [7]. Taking account of this fact and considering now all the tests (n = 60), EC3* formulation leads to a high standard deviation (s_δ = 0.227) in comparison with that for crushing failure (s_δ = 0.143). As explanation, it is sufficient to examine the **table I** ; for all the tests (except one), the buckling resistance, F_b, is systematically lower than the crushing one F_c (see the underlined numbers). Therefore, the buckling formula **(5)** is clearly far too conservative. On the other hand, AISC formulation gives a more satisfying adjustment to experimental data than EC3*, but there is now an opposite tendency for formula **(9)** to overestimate the buckling resistance F_b in some cases, as illustrated in **table I**. Finally, for both codes, it may be concluded that any research work to improve the analytical formulation of buckling resistance is fully justified. From this point of view, a contribution is proposed hereafter.

Table II

n	Code	Formulae	$\overline{b}$	s_δ	ρ
23	EC3	(1) - (2)	1.206	0.143	0.85
23	AICS	(7) - (8)	1.359	0.133	0.86
60	EC3*	(6) - (1) - (8) - (5)	1.858	0.227	0.96
60	AISC	(6) - (7) - (8) - (9)	1.469	0.185	0.96

(EC3* means EC3 where (2) is replaced by (8))

4. THEORETICAL MODELLING

The approach developed here is based on plastic hinge mechanisms and which permits *consideration of web-to-flange coupling at failure. Obviously, with such mechanisms,* analysis of web buckling requires considering second order geometrical effects, which may lead to using Plastic Analysis beyond rigorous conditions of applicability.

4.1 Modelling of crushing resistance

The crushing model is presented in **figure 4** with three plastic hinges in each column flange and distributed yielding along the web-to-flange fastening line. It looks like other models published in the literature [8, 9], but its originality lies in taking

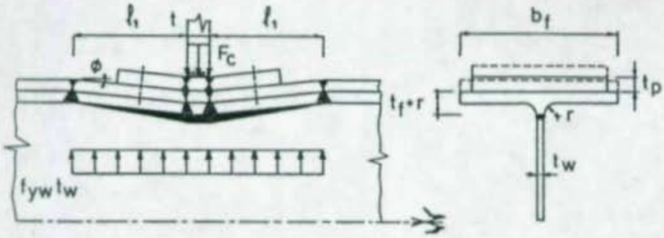

Fig. 4

account of the non-negligible effect of the root radius r of the profile, so that the crushing resistance is given by :

$$F_c = (t + 2\,\ell_1)\, f_{yw}\, t_w \qquad \text{with : } \ell_1 = 2\,(M_{pf}/f_{yw}\, t_w)^{1/2} \qquad \textbf{(10)}$$

where M_{pf} is the plastic bending moment of the flange including the part due to the profile roots :

$$M_{pf} = \left(\frac{b_f\, t_f^2}{4} + Z_r\right) f_{yw} \qquad \textbf{(11)}$$

with :

$$Z_r = \frac{r t_f}{2}(t_w + 0.43\, r) + \left(\frac{t_w}{2} + 0.1\, r\right) r^2 - \frac{r^2}{4 b_f}(t_w + 0.43\, r)^2 . \qquad \textbf{(12)}$$

In the presence of an intermediate plate of thickness t_p, the expression for ℓ_1 has to be multiplied by factor $(1 + 0.5\, M_{ptp}/M_{pf})$ where M_{ptp} is the plastic bending moment of the intermediate plate.

4.2 Modelling of buckling resistance

In addition to the above flange mechanism, a second plastic hinge mechanism in the web is considered, as shown in **figure 5**, which occurs only when the web has a certain transverse deflection characterized by angle θ_o in the figure, the web-to-flange fastening line now being partially yielded in compression along some length $(t + 2\,\ell_2)$. On the one hand, the ultimate load is equal to the partial resistance of the web :

$$F_b = (t + 2\,\ell_2)\, f_{yw}\, t_w \ ; \qquad \textbf{(13)}$$

on the other hand, it is given by the web mechanism :

$$F_b = \frac{4\, M_{pf}}{q} + \frac{2\, m_{pw}}{h_w \sin\theta_o}\left(4q + t + \frac{h_w^2}{q} - 2\,\ell_2\right) , \qquad \textbf{(14)}$$

where m_{pw} is the plastic moment of the web per unit length.

The size q of the web mechanism can be determined by minimizing the right-member of relationship **(14)** ; also, we have geometrically :

$$\sin \theta_o = 2\,(\delta / h_w)^{1/2} \qquad (15)$$

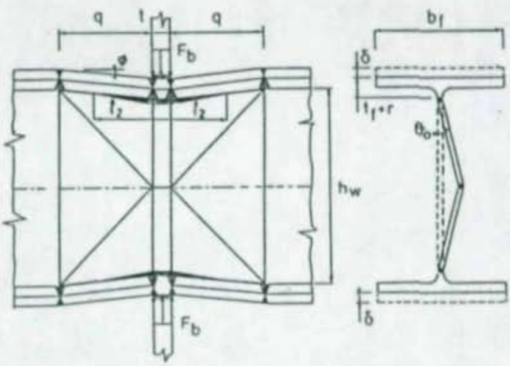

Fig. 5

where δ is the maximum flange deflection which can be evaluated by means of *the same assumption as* ROBERTS *and* ROCKEY **[10]** :

$$\delta = M_{pf}\, q^2 / (6\, E\, I_f)\,, \qquad (16)$$

where I_f is the second moment of area of the flange (including the roots). After calculation, it is found :

$$q = \frac{h_w}{2}\left(\sqrt{\alpha} + \sqrt{1+\alpha}\right) \quad , \quad \text{with : } \alpha = \frac{M_{pf}^3}{6\, EI_f\, m_{pw}^2\, h_w} \quad , \qquad (17)$$

and :

$$\ell_2 = \frac{4q + t}{1 + 0.5\, f_{yw}\, t_w\, h_w \sin\theta_o / m_{pw}} - \frac{t}{2}\ . \qquad (18)$$

Finally, the buckling resistance F_b is given by the relationships **(13)** and **(18)**.

In the presence of an intermediate plate, it is easy to demonstrate that only the expression of α must be modified ; in this case :

$$\alpha = \frac{(M_{pf} + 0.5\, M_{ptp})^2\, M_{pf}}{6\, EI_f\, m_{pw}^2\, h_w} \qquad (19)$$

4.3 Ultimate resistance

It is logical to propose again :

$$F_u = \text{smaller of } (F_c, F_b) \qquad (20)$$

where F_c and F_b are calculated respectively by **(10)** and **(13)**.

Physically, the web mechanism cannot occur if **(17)** and **(18)** lead to $\ell_2 > q$.

4.4 Validity of the web mechanism

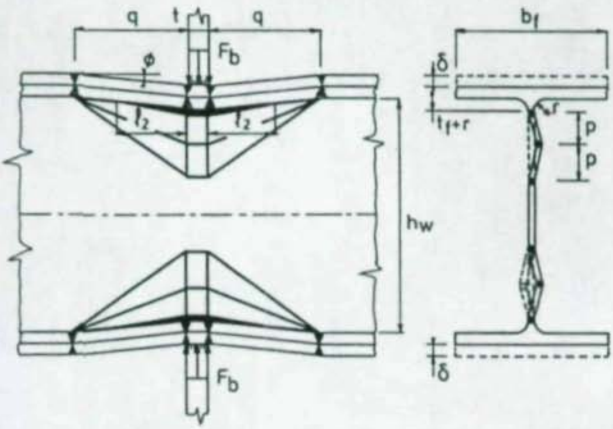

Fig. 6

Three other mechanisms of the web have been investigated thoroughly, but none of them led to values of F_b less than those deduced from **(13)** and **(18)**, at least for all the tests considered in **table I**. For example, one of these mechanisms was a double web crippling mechanism with a depth of penetration $2p \leqslant h_w/2$, as shown in **figure 6** ; its buckling resistance can be determined by **(13)** again, on condition that we use :

$$\ell_2 = \frac{4q + t}{0.5 + f_{yw}\, t_w\, p \sin\theta_o / m_{pw}} - \frac{t}{2} \qquad \textbf{(21)}$$

where :

$$q = p\,(\sqrt{\beta} + \sqrt{2+\beta}) \quad , \quad \text{with : } \beta = \frac{M_{pf}^3}{24\,EI_f\, m_{pw}^2\, p} \qquad \textbf{(22)}$$

and :

$$\sin\theta_o = q\,[M_{pf} / (6\; EI_f\; p)]^{1/2} \; ; \qquad \textbf{(23)}$$

(Note that F_b has a minimum value when $p = h_w/4$).

5. NEW COMPARISON WITH EXPERIMENTAL DATA AND IMPROVEMENT OF THE THEORETICAL MODELLING

Formulae **(10)** and **(13)** have been applied systematically to the sixty tests of **table I**, the numerical values being set in the two columns headed "Model". The absence of some values of F_b corresponds to cases where the condition $\ell_2 \leqslant q$ is not satisfied. Adopting the definition **(20)** of the ultimate resistance, the statistical quantities for adjustment to the experimental data are now :

$$(n = 60) \; ; \; \bar{b} = 1.404 \; , \; s_\delta = 0.126 \; , \; \rho = 0.97 \; .$$

It appears that this adjustment is clearly better than the ones using the code formulae. Moreover, the cases where the buckling resistance prevails are more in agreement with experimental observation.

Although these results point to a rather satisfactory model, the well-know effect of initial geometrical defects and residual stresses have not been included. To introduce a parameter to account for these imperfections, a possible solution consists in assuming that there is a transverse deflection η_i at the median depth of the column web before any loading ; but the parameter η_i may not have familiar values because it is just an artificial means introduced into a plastic model (and not into an elastic one). Consequently, the geometrical relationship **(15)** is replaced by :

$$\delta = \frac{h_w}{4}\left[\sin^2 \theta_o - 4\left(\frac{\eta_i}{h_w}\right)^2\right] \quad , \qquad \textbf{(24)}$$

the asumption **(16)** about the flange deflection being still valid. By using the same procedure as in **paragraph 4.2**, more general expressions of ℓ_2 and q can be obtained (which are not presented here to reduce the volume of the text). Coming back to the adjustment to the experimental data (n = 60), several values of η_i/h_w have been tested empirically in order to find the most significant one. As illustrated by the improved values of the statistical quantities ρ and s_δ in **table III** showing an asymptotic tendency when η_i/h_w increases, it seems reasonable to adopt $\eta_i/h_w = 0.04$ in practice. To assign a mechanical interpretation for this particular value, a parametric study has been carried out on different types of profile with the help of the theoretical model. By making the web depth vary, several curves $F_u^{th}\left(\frac{\eta_i}{h_w} = 0.04\right) / F_u^{th}\left(\frac{\eta_i}{h_w} = 0.\right)$ have been plotted against the ratio $\sqrt{F_c/F_b}$; all the curves approximately showed a constant specific shape whose interpretation has direct relation to the well-known WINTER formula, as that is justified again in the following paragraph where a simplified approach is proposed using just this formula.

η_i/h_w	$\bar{b}$	s_δ	ρ
0	1.404	0.126	0.970
0.02	1.429	0.120	0.971
0.04	1.510	0.110	0.973
0.06	1.645	0.110	0.973

(n = 60)   Table III

6. PROPOSAL FOR A SIMPLIFIED DESIGN

The theoretical model with parameter η_i may be estimated too laborious for usual design. To try attaining a simplified formulation which will be so much accurate, we have proceeded as follows :

a/ the crushing resistance is interpreted as a maximum plastic resistance of the web alone and prevails if elastic buckling collapse does not occur before ;

b/ the critical buckling resistance of the web, $F_{cr,w}$, in the case of symmetrical loading, is given by the familiar relationship :

$$F_{cr,w} = \frac{\pi E t_w^3}{3(1-\nu^2) h_w} \quad . \qquad \textbf{(25)}$$

c/ If $F_{cr,w}$ is less than F_c, advantage can be reaped from the favourable effect of post-critical resistance after buckling by means of analogy with VON KARMAN or WINTER concept. According to the necessity of introducing imperfections as demonstrated in **paragraph 5**, it is better to adopt the point of view of WINTER leading to the simplified formulation :

$$F_u = F_c \left[\frac{1}{\bar{\lambda}} \left(1 - \frac{0.22}{\bar{\lambda}} \right) \right] \leq F_c \qquad \textbf{(26)}$$

where $\bar{\lambda}$ is the non-dimensional web slenderness equal to :

$$\bar{\lambda} = [F_c / F_{cr,w}]^{1/2} \qquad \textbf{(27)}$$

and F_c is given by relationship **(10)**.

Formulation **(26)** has been applied to the sixty tests of **table I**, the numerical values of $\bar{\lambda}$ and F_u being set in the two right-columns of the table. The adjustment to the experimental resistances F_u^{exp} gives now :

$$(n = 60) \quad ; \quad \bar{b} = 1.477 \quad ; \quad s_\delta = 0.112 \quad ; \quad \rho = 0.973$$

which are very near those of **table III** (when $\eta_i / h_w = 0.04$). In addition to this satisfactory result, it is pointed out that the tests corresponding to values of $\bar{\lambda}$ clearly greater than 0.7 have effectively collapsed by buckling.

In order to bring formulation **(26)** to take a design form, a specific statistical treatment has been carried out **[1]** according to the reliability index adopted in Eurocode 3 ($\beta = 3.8$) ; the corresponding partial safety factor so deduced from the thirty three french tests on rolled members has been found exactly equal to $\gamma^*_{M1} = 1.0$. Nevertheless, in order to keep the usual value $\gamma_{M1} = 1.1$ specified in Eurocode, we like better the following proposal as final form of design formula :

$$\boxed{F_{ud} = 1.1\, F_c \left[\frac{1}{\bar{\lambda}} \left(1 - \frac{0.22}{\bar{\lambda}} \right) \right] / \gamma_{M1} \leq 1.1\, F_c / \gamma_{M1}} \qquad \textbf{(28)}$$

where F_c and $\bar{\lambda}$ are now calculated from nominal values of steel strength and geometrical characteristics. Besides, for usual rolled european members, we can note that F_c may be also determined by formulae **(7)** and **(8)** which are simpler than **(10)**, **(11)** and **(12)** ; indeed, these two sets of formulae are numerically equivalent,as we can see in **table I** by comparing the values of F_c in the columns headed "AISC" and "Model" (so, for the thirty three french tests, the ratio between the two calculated values has a mean equal to 1.034 with standard deviation 0.065).

7. POSSIBLE GENERALIZATIONS

There are good reasons for believing that formulation **(28)** could be generalized to welded built-up columns ; in this case, we suggest only to take Z_r equal to zero in formula **(11)** used to calculate F_c.

For one-sided beam-to-column connections, formulation **(28)** could be still valid provided that the value of $\bar{\lambda}$ is deduced from another formula than **(25)**, probably :

$$F_{cr,w} = \frac{\pi^2 E\, t_w^3}{6(1-\nu^2) h_w} \qquad \textbf{(29)}$$

Finally, when there are non-negligible compressive normal stresses in the web of columns, the value of F_c in formulation **(28)** should be reduced, for example by multiplying f_{yw} by the reduction factor specified in Eurocode 3 :

$$r = 1.25 - 0.5\,\sigma_n / f_{yw} \leq 1 \qquad \textbf{(30)}$$

where σ_n is the maximum stress in the web. But introducing a corrective factor to $F_{cr,w}$ in that particular case seems neither proved nor justified.

8. LITERATURE REFERENCES

[1] **J.M. ARIBERT, A. LACHAL** and **M. MOHEISSEN** - Interaction between buckling and web plastic resistance of a rolled beam submitted to symmetrical load compression. Construction Métallique n° 2, 1990, p. 3 - 23.

[2] **P. ZOETEMEIJER** - The influence of normal, bending and shear stresses on the ultimate compression force exerted laterally to European rolled sections. Report 6-80-5, Stevin Laboratory, Delft, Feb. 1980.

[3] **J.D. GRAHAM, A.N. SHERBOURNE, R.N. KHABBAZ** and **C.D. JENSEN** - Welded interior beam-to-column connections. WRC Bulletin n° 63, 1960.

[4] **W.F. CHEN** and **J. OPPENHEIM** - Web buckling strength of beam-to-column connections. Fritz Engineering Laboratory, Report n° 333, Sept. 1970.

[5] **W.F. CHEN** and **E. NEWLIN** - Column web strength in beam-to-column connections. Journal of the Structural Division, Sept. 1973.

[6] **A.N. SHERBOURNE** - Behaviour and design of moment connections. Report submitted to the Canadian Steel Industries Construction Council, Toronto, 1975.

[7] **J.M. ARIBERT** and **A. LACHAL** - Elasto-plastic analysis of the local compressive stresses on a web. Construction Métallique n° 4, 1977, p. 51 - 66.

[8] **A. BERGFELT** - Patch loading on a slender web, influence of horizontal and vertical web stiffeners on the load carrying capacity. Sweden,1979.

[9] **J. WARDENIER, P. STOLLE** and **P.W. DOOL** - The strength and behaviour of plate to I chord connections. The Netherlands, 1982.

[10] **T.M. ROBERTS** and **K.C. ROCKEY** - Method for predicting the collapse load of a plate girder when subjected to patch loading in the plane of the web. Construction Métallique, n° 3, 1978, p. 3 - 13.

REVIEW OF INTERNATIONAL DESIGN CRITERIA FOR FILLET WELDS IN HOLLOW STRUCTURAL SECTION TRUSS CONNECTIONS

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George S. Frater[2]

Abstract

Design recommendations pertinent to fillet welds in Hollow Structural Section truss connections from Europe (Eurocode 3, International Institute of Welding and British Steel), the U.S. (AWS D1.1 and AISC-LRFD) and Canada (CAN/CSA-S16.1 and Stelco) are reviewed and shown to result in quite different weld proportions for most applications. Laboratory testing on large-scale Hollow Structural Section fillet-welded trusses has accordingly been undertaken, to produce sequential failure of the fillet weldments around the tension web members at the chord face connection. The results of these tests have been used to evaluate the international design recommendations and thereby produce preferred design approaches.

1. INTRODUCTION

In predominantly statically-loaded Warren and Pratt trusses fabricated from Hollow Structural Sections (HSS), fillet weldments are typically used between rectangular web and chord members, for web to chord width ratios (β') up to about 0.8. International recommendations for the design of HSS truss connections have been established by the International Institute of Welding (IIW, 1989). The applicability of such connection strength rules is based upon the connector - the weldment - being non-critical. Due to the uneven distribution of load around HSS truss connections, and the need for the weldment to be able to withstand significant localized bending moments - in addition to the web member axial loads - prior to joint ultimate strength being attained, IIW (1989) has specified that fillet weldments should develop the strength of the connected web member. To ensure such strength, and to provide ductile behaviour of the joint, IIW (1989) has made the recommendation that the fillet weld throat thickness (a) be greater than or equal to 1.07 times the thickness (t) of the adjoining web member (both tension and compression) for HSS with a yield strength of 350 MPa or greater. This weldment

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design rule is likely conservative, leading to overwelding, and not consistent with the design approach of most national welding and structural codes since the weld size requirement is related to the full strength of the web member wall, with no consideration of actual member forces.

The American Welding Society (1990) has a more logical approach for proportioning the weldment by requiring that it be capable of developing, at its ultimate breaking strength, the lesser of the web member yield strength or local (connection) strength of the main member. The means of achieving this requirement in fillet-welded HSS or Rectangular Hollow Section (RHS) truss joints (AWS D1.1, 1990, Section 10.5.3.2) is to comply with certain "prequalified" fillet weld details (allowed when $\beta' \leq 0.8$; AWS D1.1, 1990: Section 10.13.3), which require that the throat thickness (a) be $\geq 0.71t$ for a weldment at the toe and side of the joint while at the heel of the joint the leg length (S) be $\geq 1.5t$. However, the AWS connection strength formulae for RHS trusses differ from the IIW recommendations, which have an extensive experimental and theoretical basis, and so one is still left with the problem of establishing the minimum fillet weld sizes necessary to develop a particular connection strength where the latter meets the criteria of the IIW (1989) recommendations. These specifications given in Chapter 10 of AWS D1.1 (1990) are primarily based on research on circular tubular connections conducted at the University of Texas, the University of California and by several large oil companies.

Another recent guide by BSC (1988) for proportioning weldments in HSS trusses attempted, (like AWS D1.1, 1990), to relate the required weld throat thickness to the lesser of the connection strength and member strength, but it is worth noting that the former typically governs in RHS truss design due to a "weak connection" phenomenon of most HSS connections. Accordingly, BSC recommends a weld size of:

$$a \geq f(L) \cdot f(W) \cdot t \qquad .....(1)$$

where $f(L)$ is a load function and is equal to the higher value of

$$\frac{\text{Actual applied load}}{\text{Allowable connection load}} \quad \text{or} \quad \frac{\text{Actual applied load}}{\text{Allowable member load}}$$

where the allowable connection load is based on CIDECT (1984)/IIW (1981) recommendations. $f(W)$ is the ratio of allowable HSS stress to allowable weld stress, and takes on values dependent on the steel grade and the electrode grade.

Specific advice on the design of HSS truss fillet weldments in Canada, namely by the Canadian Institute of Steel Construction (1985, Table 3-41) and Stelco (1981), has also been very conservative, with both of these specifying fillet weld sizes that would develop the tensile strength of the connected web member walls.

Experimental work at the University of Toronto was conducted to assess the validity of using national welding standards for proportioning fillet welds in HSS trusses, as opposed to the current design guidelines discussed above. Initially, isolated connection tests (29 in total) were performed (Packer and Frater, 1987), with the latter part of this test program consisting of connections where weldments were unevenly-loaded and subject to similar restraint to that experienced in an RHS truss. Based on this work a tentative proposal for weldment design, using an effective length concept, was made by applying traditional national and international code provisions for weldment design. To validate this proposal, two carefully-designed, large-scale, fillet-welded, RHS Warren trusses were then tested (Frater and Packer, 1990). This program involved the testing of 12.0 m and 12.2 m span, simply-supported, HSS Warren trusses, comprised of 60° gap or overlap K-connections, and was undertaken in a controlled manner to produce sequential failure of the fillet weldments around the tension web members at the chord face joint. A total of 15 tests were performed on truss gap connections and a further 2 on truss overlap connections.

2. FILLET WELD DESIGN - FACTORED RESISTANCE MODELS

2.1 General

Although a fillet weld is in concept extremely simple, the internal stress systems by which it transmits loads are highly complex. For design, the strength of fillet welds is often described by simplifying the force system, assuming critical failure surfaces and distributing a mean stress over them, although the stresses through sections of the fillet weld are highly irregular due to stress raising effects at the root and toe of the fillet weld. For example, IIW (1974, 1976) and Eurocode 3 (1989) adopt a simple technique by neglecting the moments that produce equilibrium in a weld, assuming a uniform stress distribution on a 45° section and using a von Mises et al equivalent stress formula, or σ_c formula, as discussed below.

North American weld design philosophy (e.g. CAN/CSA-S16.1, 1989) differs from the IIW/EC3 approach and is more conducive to design, (forces need not be resolved into components along an effective throat area), since only two simple resistance equations are involved; i.e., one for the ultimate shear resistance of weld metal along an effective throat area and one for the shear yield resistance of the base metal. Listed below is a summary of some international and national code provisions for weldment design.

2.2 European Design Methods

2.2.1 IIW - σ_c Formula

From 1974 the accepted IIW design approach (IIW 1974,1976) for checking the strength of fillet weldments, under static loads, has been to show that the stresses $\sigma_\perp$, $\tau_\perp$ and $\tau_\parallel$ (see Figure 1), satisfy the following:

$$\beta\sqrt{\sigma_\perp^2+3(\tau_\perp^2+\tau_\parallel^2)} \quad \leq \quad \sigma_c \qquad .....(2)$$

where $\beta = 0.85$ for 345 MPa yield strength steel (Fe 510).

and $\sigma_\perp \leq \sigma_c$, ...(3)

with σ_c being a "comparison stress". IIW defines the comparison stress as a "permissible tensile stress" in the base material, and the Dutch limit states steelwork code (NSI, 1977) has used Eqs. (2) and (3) with σ_c set equal to the yield stress of the base material.

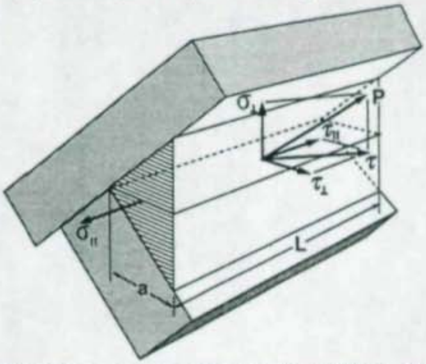

Figure 1: Fillet Weld Stresses according to IIW (1974, 1976) and EC3 (1989)

2.2.2 EUROCODE 3 - Modified σ_c Formula

In the 1984 edition of Eurocode 3 the drafting committee adopted the IIW (1974, 1976) σ_c - formula for weld design given by Eq. (2). In the 1989 Eurocode 3 draft, two versions of weld design were permitted: a simple version (the "mean stress method") in the main text, plus a modified σ_c - formula (the "alternative method for fillet welds" or "stress component method") in Annex M. The "mean stress method" ignores the dependence of weld orientation to the applied force and is analogous to the North American design approaches presented below.

In Annex M of Eurocode 3 (EC3) the σ_c formula is modified to:

$$\sqrt{\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_\parallel^2)} \quad \leq \quad \frac{F_u}{\beta^* \gamma_{Mw}} \qquad .....(4)$$

and
$$\sigma_\perp \quad \leq \quad \frac{F_u}{\beta^* \gamma_{Mw}} \qquad .....(5)$$

where γ_{Mw} is a partial safety factor and taken to be 1.25. The coefficient β^* was determined by reliability studies, with a minimum reliability index of 3.8, and for steels having an ultimate strength of F_u = 510 MPa (Fe 510) β^* = 0.90.

2.2.3 Vector Addition Method

The vector addition method is used in a number of countries such as Britain (BS 5950; BSI, 1985) and Germany (DIN 18 800; DIN, 1981). For example, BS 5950 which is in

Limit States Design format states that the vector sum of the design stresses due to all forces and moments transmitted by the weld should not exceed the design resistance, p_w. The design stress, $\overline{\sigma}$, should be calculated on a thickness equal to the effective throat size, a, as follows:

$$\overline{\sigma} = \sqrt{\sigma_x + \sigma_y + \sigma_z} \leq p_w \qquad .....(6)$$

where $\sigma_{x,y,z} = \dfrac{\text{applied load in x,y,z co-ordinate axis directions}}{\text{throat area}}$

2.3 North American Design Methods

2.3.1 CAN/CSA-S16.1-M89 (CSA, 1989)

In the Canadian Standards Association (CSA) Standard CAN/CSA-S16.1-M89 (1989), the ultimate factored resistance (V_r) of fillet welds loaded in tension or compression normal to the axis of the weld, or loaded in shear, is given by the smaller of:

(i) For Base Metal: $V_{r1} = \phi 0.67 F_{y1} A_{m1}$ .....(7)

or $V_{r2} = \phi 0.67 F_{y2} A_{m2}$, and .....(7a)

(ii) For Weld Metal: $V_r = \phi_w 0.67 X_u A_w$ .....(8)

where ϕ and ϕ_w are resistance factors for base and weld metals, and equal to 0.9 and 0.67, respectively.

2.3.2 AWS D1.1-90 (1990)

The American Welding Society (AWS D1.1, 1990) also has the same criteria as CAN/CSA-S16.1 (1989) except it uses Allowable Stress Design rather than a Limit States Design resistance format, and the allowable loads are given by the smaller of:

(i) For Base Metal: $V_1 = 0.4 F_{y1} A_{m1}$ .....(9)

or $V_2 = 0.4 F_{y2} A_{m2}$, and .....(9a)

(ii) For Weld Metal: $V = 0.3 X_u A_w$ .....(10)

2.3.3 AISC-LRFD (1986)

The American Institute of Steel Construction's Load and Resistance Factor Design Specification for Structural Steel Buildings (1986) gives the factored resistance of a fillet weld by the smaller of:

(i) For Base Metal: $V_{r1} = \phi 0.60 F_u A_{m1}$ .....(11)

or $V_{r2} = \phi 0.60 F_u A_{m2}$, and .....(11a)

(ii) For Weld Metal: $V_r = \phi 0.60 X_u A_w$ .....(12)

where $\phi = 0.75$ for all of Eqs. (11), (11a) and (12). The inclusion of this resistance factor value means that Eq. (12) simplifies to $0.45 X_u A_w$ which is identical to the CAN/CSA-S16.1 (1989) Eq. (8). However, whereas both CAN/CSA-S16.1 (1989) in Eq. (7), and AWS D1.1 (1990) in Eq. (9), determine the base metal strength on the basis of the yield stress, AISC-LRFD (1986) uses the base metal ultimate stress in Eq. (11).

3. CONCLUSIONS

On the basis of the previously mentioned experimental studies, it was concluded that satisfactory performance can be obtained from the fillet welds around web members in RHS trusses, such that the joint will attain the mean ultimate "connection strength" as defined by IIW (1989), if the weldments are designed in accordance with the factored resistance models discussed above, but providing an appropriate effective length is used for the weldment (Frater and Packer, 1990).

To illustrate the range of prequalified weldments and design techniques for sizing fillet weldments, a design example from Frater and Packer (1990: Section 3.5) is cited. Table 1 shows the weld sizes of a 90° fillet weld along the side of a web member, for a typical gap-jointed Pratt truss having diagonal web members inclined at 34°. This joint also includes a fillet weld along the heel of the joint, and a groove weld along the toe of the joint. From Table 1 one can observe that EC3 (1989; see Eqs. (4) and (5)), CAN/CSA-S16.1 (1989; see Eqs. (7) and (8)), AWS D1.1 (1990; see Eqs. (9) and (10)), and AISC-LRFD (1986; see Eqs. (11) and (12)) weld design equations can give smaller weld throat sizes relative to the prequalified, $a = 1.07t$ rule given by IIW (1989). Although it is cumbersome to use in design, the EC3 (1989) Stress Component Method leads to the most significant potential for down-sizing of weldments relative to the $a = 1.07t$ rule. The prequalified weld sizes given in Chapter 10 of AWS D1.1 (1990) are also small, however the recommendations for the weld size are questionable for square/rectangular tube joints, since they are based on research on circular tube joints. Guidelines by the two steel producers, Stelco (1981) and BSC (1988), are conservative as they predict weld throat sizes greater than $1.07t$. These recommendations by Stelco and BSC could now be considered out of date relative to the new IIW (1989) rule.

The IIW (1989) rule is based on the wall thickness of web members (hence the yield load), without any consideration of the actual weld length, effective weld length and the actual force to be transferred between the web and chord member. Regarding the welding of overlap joints, the $a = 1.07t$ rule is based on fillet welds and does not consider the special aspects of an overlapped welded connection; i.e., % overlap, length of flare bevel weldments or the resistance of the flare bevel weld itself. Hence, this rule can lead to overwelding, which likely will increase fabrication costs. This condition can become more acute in instances where compression web members, carrying the same load as tension web members, might require a larger wall thickness, or in the case of a truss having low web member angles where the increase in weld length results in a significant increase in joint strength (e.g. example in Table 1).

Design Method	Weld throat thickness, a, in mm
IIW (1989) Prequalified Rule (a=1.07t)	8.5
Stelco (1981): Table 3.3	9.9
AWS D1.1 (1990) Prequalified Rule	5.6
BSC (1988)	10.9
AWS D1.1 (1990) by A.S.D.	8.2
CAN/CSA-S16.1 (1989)	7.4
AISC-LRFD (1986)	7.4
EC3 (1989) Stress Component Method	4.9

Table 1: Weld Sizes for Side Fillets in Example Pratt Truss

4. LIST OF SYMBOLS

a = effective throat thickness of weld
A_m = effective area of fusion in base metal no. 1 (A_{m1}) or no.2 (A_{m2}) = effective size of fillet leg x length of weld
A_w = effective throat area of weld = effective throat thickness of weld x length of weld
F_y = yield strength of base material no. 1 (F_{y1}) or no. 2 (F_{y2})
F_u = ultimate strength of base metal
p_w = design resistance of a fillet weld according to BS5950 (BSI, 1985)
t = thickness of hollow section web member
V = shear resistance of weld metal according to AWS D1.1 (1990)
V_1, V_2 = shear resistance of base metal according to AWS D1.1 (1990)
V_r = factored resistance of weldment according to CAN/CSA-S16.1 (1989), IIW (1974, 1976) and EC3 (1989) design methods
V_{r1}, V_{r2} = shear resistance of base metal according to CAN/CSA-S16.1 (1989)
X_u = ultimate strength of electrode material
β = parameter used by IIW to relate the properties of a parent or base metal to the properties of a "matching" weld consumable, and also the observed experimental strength of such weldments
β^* = same as above but values modified as shown in EC3 (1989)
β' = average width ratio between web member(s) and chord
$\sigma_\perp$ = normal stress along effective throat, perpendicular to axis of weld
$\sigma_{x,y,z}$ = normal stress (subscripts x, y and z indicate direct stresses parallel to the coordinate axes)
$\bar{\sigma}$ = addition of individual stresses $\sigma_{x,y,z}$ according to Eq. (6)
σ_c = comparison stress used in IIW (1974, 1976) and EC3 (1989) design equations

$\tau_{\perp}$ = shear stress along effective throat, perpendicular to axis of weld
$\tau_{\parallel}$ = shear stress along effective throat, parallel to axis of weld

5. REFERENCES

1. AMERICAN INSTITUTE OF STEEL CONSTRUCTION, "Manual of Steel Construction - Load & Resistance Factor Design", 1st. ed., Chicago, U.S.A., 1986.
2. AMERICAN WELDING SOCIETY, "Structural Welding Code-Steel", ANSI/AWS D1.1-90, 12th ed., Section 10, Miami, U.S.A., 1990.
3. BRITISH STANDARDS INSTITUTION, "Structural Use of Steelwork in Building", BS 5950: Part 1, London, England, 1985.
4. BRITISH STEEL CORPORATION TUBES DIVISION, "Design of SHS Welded Joints", Publication No. TD 297/3E, Corby, England, May, 1988.
5. CANADIAN INSTITUTE OF STEEL CONSTRUCTION, "Handbook of Steel Construction", 4th ed., Willowdale, Canada, 1985.
6. CANADIAN STANDARDS ASSOCIATION, "Limit States Design of Steel Structures", CAN/CSA-S16.1-M89, Rexdale, Canada, 1989.
7. CIDECT, "Construction with Hollow Steel Sections", British Steel Corporation Tubes Division, 1st. ed., Corby, England, December, 1984.
8. DEUTSCHER AUSSCHUSS FÜR NORMUNG, "DIN 18 800, Teil 1 Stahlbauten, Bemessung und Konstruktion", (DIN 18 800, Part 1: Steel Structures, Design and Construction), Beuth-Vertieb GmbH, Berlin und Koln, West Germany, March, 1981.
9. EUROCODE 3 Editorial Group, "Design of Steel Structures: Part 1 - General Rules and Rules for Buildings", November, 1989.
10. FRATER, G.S. and PACKER, J.A., "Design of Fillet Weldments for Hollow Structural Section Trusses", CIDECT Report No.5AN/2-90/7 and IIW Doc. No. Sc.-XV-E-90-166, October, 1990.
11. INTERNATIONAL INSTITUTE OF WELDING, Subcommission XV-A, "Design Rules for Arc-Welded Connections in Steel submitted to Static Loads", Doc. No. XV-358-74, Welding in the World, Vol. 14, No. 5/6, 1976, pp. 132-149.
12. INTERNATIONAL INSTITUTE OF WELDING, Subcommission XV-E, "Design Recommendations for Hollow Section Joints - Predominantly Statically Loaded", 1st. ed., Doc. No. XV-491-81 (Revised), I.I.W. Annual Assembly, Oporto, Portugal, 1981.
13. INTERNATIONAL INSTITUTE OF WELDING, Subcommission XV-E, "Design Recommendations for Hollow Section Joints - Predominantly Statically Loaded", 2nd. ed., Doc. No. XV-701-89, I.I.W. Annual Assembly, Helsinki, Finland, 1989.
14. NETHERLANDS STANDARDS INSTITUTION, "ARC WELDING - Calculation of Welded Joints in Unalloyed and Low-Alloy Steel with up to and Including Fe 510 (Fe 52) which are Predominantly Statically Loaded", NEN 2062, February, 1977.
15. PACKER, J.A. and FRATER, G.S., "Weldment Design for Hollow Section Joints", CIDECT Report No. 5AN-87/1-E and IIW Doc. No. XV-644-87, 1987.
16. STELCO, Inc., "Hollow Structural Sections Design Manual for Connections", 2nd. ed., Hamilton, Canada, 1981.

Technical Papers on

PREDESIGNED AND SPECIAL CONNECTIONS

RECENT DEVELOPMENTS IN CONNECTION RESEARCH AND DESIGN IN THE U.S.A.

Robert O. Disque[1]

Abstract

This paper will describe some recent developments in research and design of both shear and moment connections in the United States. New design procedures are given for shear tabs, tee connections and heavy bracing connections. Criteria for the behavior of a tube wall supporting a shear tab and a column web supporting a stiffened seat are reported. Recent research results on eight-bolt end plates are discussed.

1. SHEAR TABS

Shear tabs as shown in Fig. 1 are one of the most popular "pinned" connections in the country. Even though the bolts are in single shear and, therefore, require twice as many fasteners as in the traditional double angle connections, their ease of erection usually results in overall economy.

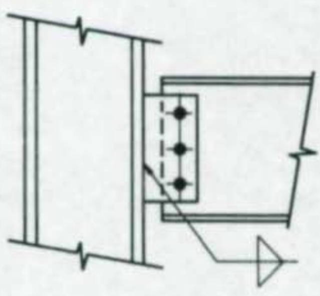

Figure 1

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In order to develop a design procedure, Professor Hassan Astaneh at the University of California-Berkeley, built a special testing device illustrated in Fig. 2 (Call, et al., l989). Astaneh recognized that the behavior of a "simple" connection could not be modeled by the usual cantilever test because the behavior of a real connection includes a specific rotation that is related to the shear or end reaction. It was also recognized that the connection should be tested to its ultimate load. The design of the connection would therefore be carried out under the behavior of the connection at ultimate load. An appropriate factor of safety could then be applied when, as in the case of Allowable Stress Design, the design is made with service loads. In Fig. 2 the beam end rotation is control led by Actuator R and the beam reaction is controlled by Acuator S.

The research demonstrated that the moment diagram of a beam connected by a "pinned" connection (including double angles) is significantly different at ultimate load than at service load. Figure 3a shows a typical moment diagram for a simple beam at the working load, W. The end connection at this loading may have some significant stiffness resulting in an end moment equal to the reaction, W/2 x e. But, as shown in Fig. 3b, the eccentricity decreases considerably as the load increases to the factored load, W' = LF x W. This occurs because the connection distorts and accommodates the beam end rotation by inelastic deformation. In the case of the shear tab, this deformation is by shear yielding. At the factored load, which may be considered the ultimate load, the end moment is W /2 x e' which is considerably smaller than the product: load factor x W/2 x e. This means that the eccentricities on the fasteners (bolts and welds) are smaller that would be expected at factored service loads. As can be seen in Fig. 4. the eccentricity approaches zero as the load increases. The design procedure reflects this small eccentricity. This phenomenon also helps explain why the traditional practice of ignoring eccentricities, in connections assumed to be "simple", has historically resulted in safe designs.

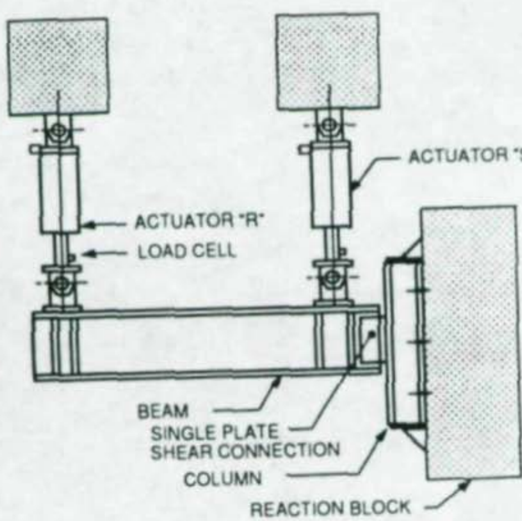

Figure 2

The new AISC design procedure for shear tabs, based on Astaneh's research, apply to composite or noncomposite beam and for standard or short slotted holes. Snug-tight rather than tensioned bolts are highly recommended for these connections. This is because snug-tight bolts will allow the connection to slip during construction negating the possibility of a sudden, pronounced, and often very loud, slip subsequent to building occupancy.

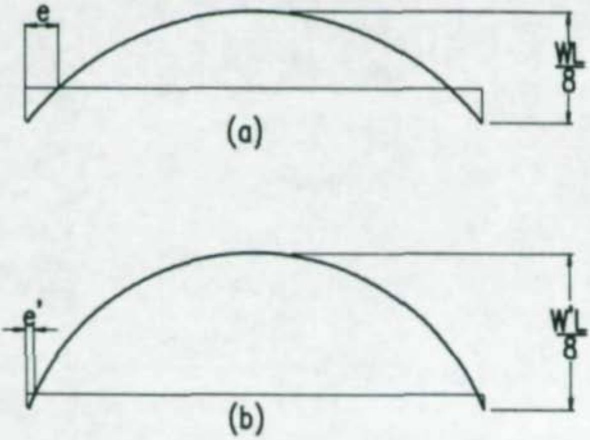

Figure 3

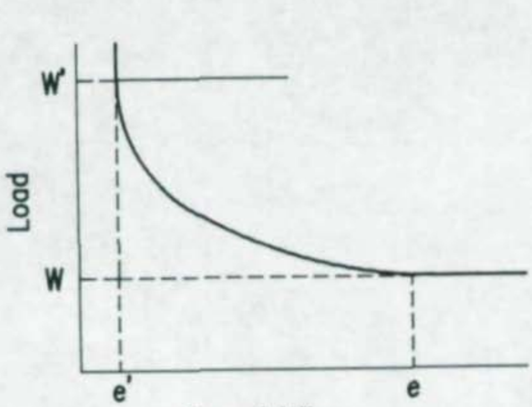

Figure 4

2. STRUCTURAL TEES

In addition to shear tabs, Astaneh has done extensive testing on welded-bolted structural tee connections (Fig. 5) and design procedures have been developed (Nader and Astaneh, 1989). In this case the required rotation capacity is achieved by bending of the flanges. As a result, only certain tee sections can qualify; those with heavy flanges are excluded.

3. TUBE WALLS WITH SHEAR TABS

Figure 6 illustrates a shear tab connected to a tube. This connection has been used for some years with no reported distress to the tube wall. Nevertheless, because some designers have expressed concern, research was undertaken at the University of Wisconsin-Milwaukee by Professor Donald Sherman and design guidelines developed. (Sherman, 1991).

4. STIFFENED SEATS CONNECTED TO COLUMN WEBS

Probably the safest and best way to connect a beam into the web of a column is with a stiffened seat as shown in Fig. 7. It is the favorite of most erectors and had been used for probably 100 years without reported distress to the column web. In spite of its excellent record, as with shear tabs on tubes, some engineers have expressed concern. Professor Duane Ellifritt at the University of Florida recently completed an extensive testing program and has found that, with few exceptions, the column web is more than adequate to support a stiffened seat (Ellifritt, 1991).

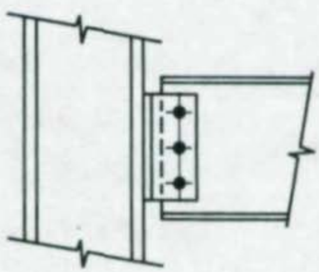

Figure 5

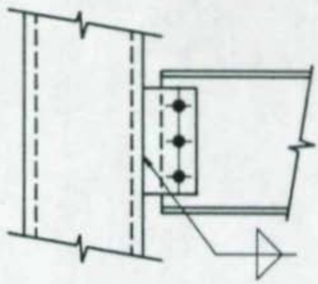

Figure 6

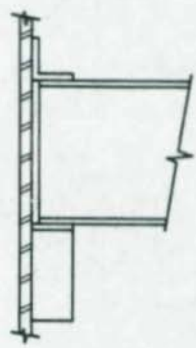

Figure 7

It is interesting to note that Ellifritt's research demonstrated that, as the load increases and the beam rotates, the reaction shifts toward the column web, decreasing moment on the column. This helps to justify the traditional practice of ignoring any moment on the column design. Another reason is that, if there is any moment on the column, there is also an accompanying rotational restraint, reducing the column K-Factor from the usual design assumption of unity. Also, a moment on the weak axis of a column is resisted by the flanges which, as rectangles, have large shape factors.

With both stiffened and unstiffened seats, it has always been the practice in the United States to provide a "stabilizing" angle on the top of the beam or on its web to assist in safe erection. Ellifritt's research, along with work by Charles W. Roeder at the University of Washington (Roeder, 1989), indicates that this angle is also important to the actual strength of the connection. It should not be omitted.

5. HEAVY BRACING CONNECTIONS

Heavy bracing connections as illustrated in Fig. 8 have been a matter of concern to the fabricating industry and the design profession for many years. The problem is not so much one of safety as it is of lack of uniformity in design philosophy. There has been a considerable amount of controversy as how these connections should be designed and it very difficult for detailers to learn all the different methods preferred by different engineers. For this reason AISC and the National Institute of Standards and Technology undertook a major research program. Analytic work was done by Professor Ralph M. Richard at the University of Arizona (Richard, 1983). Testing was carried out by Dr. John L. Gross at the NIST laboratory in Gaithersburg, Maryland (Gross, 1991) and Professor Reidar Bjorhovde at the University of Arizona (Hardash and Bjorhovde, 1985). The results from this research were studied by a joint task group from the American Society of Civil Engineers (ASCE) and AISC under the chairmanship of Dr. William A. Thornton of CIVES Steel Company and Dr. Gross. Although many different design procedures would probably be satisfactory, four were chosen to be recommended by the ASCE/AISC task group. These will be published in by Thornton (Thornton, 1991). All four methods are based on the Lower Bound Theorem and its Corollaries.

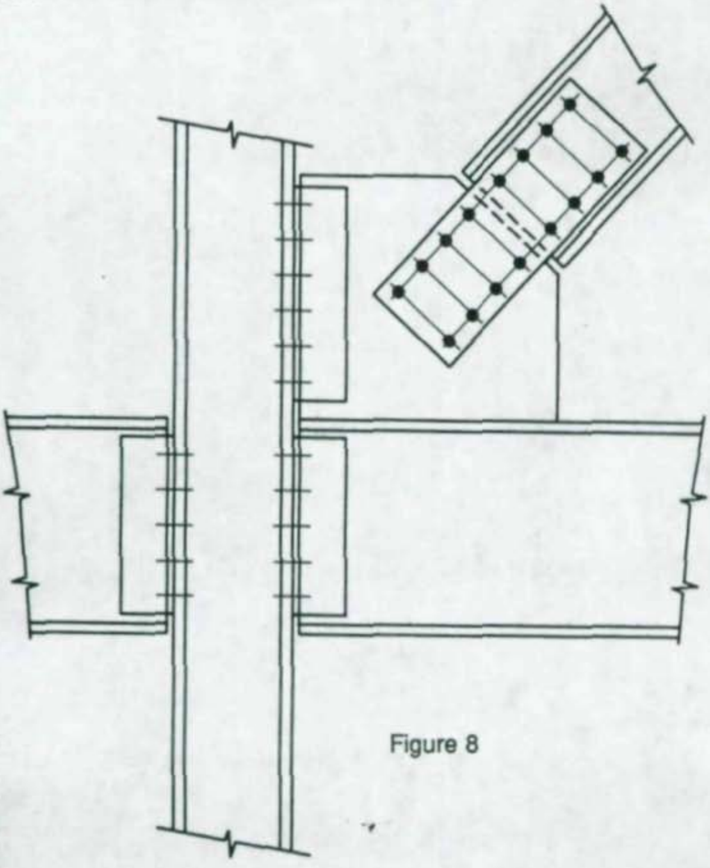

Figure 8

Lower Bound Theorem: If an equilibrium distribution of stress can be found which balances the applied load and is everywhere less than or equal to the yield stress, the structure will not fail. At most it will just have reached the plastic limit load.

Corollary One: Initial stresses, deformations, or support settlements have no effect on the plastic limit load provided the geometry is essentially unaltered.

Corollary Two: Except for its effect on dead load, addition of material without any change in the position of the applied load cannot lower the plastic limit load.

Corollary Three: Increasing the yield strength of the material in any region cannot weaken the structure. Conversely, decreasing the yield strength cannot strengthen it.

6. REFERENCES

Call, Steven M., Abolhassan Astaneh and Kurt M. McMullin "Design of Single Plate Shear Connections" Engineering Journal, American Institute of Steel Construction, 1 qtr 89, p.21.

Dalley, Ronald H. and Charles W. Roeder "The Results of Experiments on Seated Beam Connections" Engineering Journal, American Institute of Steel Construction, 3 qtr 89, p.90.

Ellifritt, Duane S. and Thomas Sputo "Strength of Supports of Seated Connections" Proceedings, National Steel Construction Conference, American Institute of Steel Construction, 1991.

Gross, John L. "Experimental Study of Gusseted Connections" Proceeding, National Steel Construction Conference, American Institute of Steel Construction, 1991.

Hardash, Steve G. and Reidar Bjorhovde "New Design Criteria for Gusset Plates in Tension" Engineering Journal, American Institute of Steel Construction, 2 qtr 85, p.77.

Nader, Marwan N. and Abolhassan Astaneh, "Design of Tee Framing Shear Connections" Engineering Journal, American Institute of Steel Construction, 1 qtr 89, p.9.

Rabern, Donald A., George C. Williams and Ralph M. Richard "Stress, Strain and Force Distributions in Gusset Plate Connections" Research Report, The University of Arizona, March, 1983.

Sherman, Donald R. "Simple Connections in Tubular Connections" Proceedings, National Steel Construction Conference, American Institute of Steel Construction, 1991.

Thornton, William A. "Heavy Bracing Connections" Proceedings, National Steel Construction Conference, American Institute of Steel Construction, 1991

BOLTED FRAMING ANGLE CONNECTIONS DESIGN AIDS: PAST AND PRESENT

Cynthia J. Zahn[1]

Abstract

AISC has developed new design aids for bolted framing angle connections now included in the AISC publication *Allowable Stress Design of Simple Shear Connections*. A historical review demonstrates the transitions this design aid has undergone, ranging from fully designed "Standard Beam Connections" in the Fifth Edition *Manual of Steel Construction*, to no real condensed design aid in the Ninth Edition ASD *Manual*. The new version is very similar in format to the Sixth and Seventh Editions with values given for A36 and A572 Gr. 50 steel beams. For a specific steel strength, bolt size, and number of horizontal bolt rows, three tables are given: Allowable Bolt and Angle Capacity, Allowable Beam Web Capacity, and Allowable Supporting Member Capacity. The allowable end reaction is the lower of the three resulting capacities.

1. INTRODUCTION

As in the past, the 9th Edition of the AISC *Manual of Steel Construction* (AISC, 1989) contains standard details and design aids for the different connection types often used in building design. This newest version of the Manual also contains design aids for shear tabs for the first time. One connection type for which there have been no design tables since the 7th Edition is bolted framing angle connections. To help fill this gap, AISC has developed a new design aid to be used for the design of this connection type. Although it is not the first of its kind, as the following historical review will demonstrate, its methodology is unique. The design checks considered will

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also be discussed.

The new tables have been published in two new AISC publications. These publications also include additional new material along with existing connection design aids currently found in the AISC Manuals of Steel Construction.

2. FRAMING ANGLE CONNECTION DESIGN AIDS

1.1 History

The format of the AISC *Manual of Steel Construction* design aid for bolted framing angle connections has undergone several transitions with varying degrees of design flexibility. Some early editions gave allowable end reactions for a particular number and type of fasteners and beam size with no calculations required, while the 8th and 9th Editions provide no comprehensive design aid.

The first AISC *Manual of Steel Construction* appeared in 1927, including the first tables for framing angle connections (see Table 1). "Connection Values" were given for shear in the web material and shear in the 3/4 in. rivets or unfinished bolts in the outstanding legs. The allowable load then could be determined as the lower of the two values.

CONNECTION ANGLES FOR CARNEGIE BEAMS

DIMENSIONS, WEIGHTS AND WORKING LOADS — 3/4" POWER DRIVEN RIVETS

Beam: Depth	Beam: Weight per foot	Connection Value: Web	Connection Value: Outstanding Single Shear: Power Driven Rivets	Connection Value: Outstanding Single Shear: Unfinished Bolts	Framing Distance c	Connection Angles: A.I.S.C. Mark	Connection Angles: Gage g	Connection Angles: Size and Length	Connection Angles: Weight inc. Web Rivets
8"	24.0	21510	23860	17670	3/16	1C.13.10	2 5/8	6" × 4" × 3/8" × 0' — 5 1/2" Long	13 lbs.
	27.0	24120	23860	17670	3/16	1C.13.10	2 5/8		
	30.0	26820	23860	17670	3/16	1C.13.10	2 5/8		
9"	29.0	25110	23860	17670	3/16	1C.13.10	2 5/8		
	32.0	27630	23860	17670	3/16	1C.13.10	2 5/8		
	35.0	30150	23860	17670	1/4	1C.13. 9	2 9/16		
10"	21.0	20700	23860	17670	3/16	1C.13.10	2 5/8		
	23.0	20700	23860	17670	3/16	1C.13.10	2 5/8		
	26.0	23310	23860	17670	3/16	1C.13.10	2 5/8		
	30.0	26820	23860	17670	3/16	1C.13.10	2 5/8		
12"	25.0	21600	23860	17670	3/16	1C.13.10	2 5/8		
	28.0	21600	23860	17670	3/16	1C.13.10	2 5/8		
	32.0	24660	23860	17670	3/16	1C.13.10	2 5/8		
	34.0	33750	23860	17670	1/4	1C.13. 9	2 9/16		
	36.0	27720	23860	17670	3/16	1C.13.10	2 5/8		
	40.0	26120	23860	17670	3/16	1C.13.10	2 5/8		
	45.0	29340	23860	17670	1/4	1C.13. 9	2 9/16		
	50.0	32490	23860	17670	1/4	1C.13. 9	2 9/16		

Connection Details

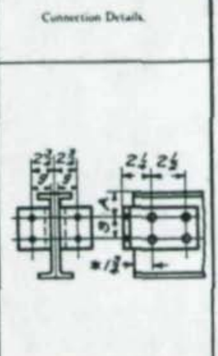


Table 1. Standard Connection Angles from 1st Edition AISC *Manual of Steel Construction*

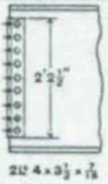
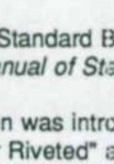
The 2nd Edition (1934) introduced "Standard Beam Connections" (Table 2). These were fully designed riveted framing angle connections; fully designed meaning allowable end reactions were given directly with no required calculations. Values were given for 3 different series of connections with 7/8 in. diameter rivets. By the time of the 4th Edition (1941), 3 more series were added for 3/4 in. diameter rivets. Six different series, designated as A, B, H, HH, K, and KK, were included based on the number of vertical rows and rivet diameter. Some fabricators fabricated A and B Connections by symbol without detail drawings. The heavier H, HH, K, and KK series with 2 vertical rows and special connections required detailing. Within each series, specific beam sizes were assigned a maximum end reaction based on rivet shear and material bearing in the web angle legs. The downfall of the "Standard Beam Connections" was their lack of design flexibility. The combination of number of bolt rows and beam size were limited by assigning each beam size only one possible end reaction within each series; meaning each beam size had only 6 possible framing angle connection arrangements. Possibly for this reason, the design aid was revamped in the 6th Edition.

STANDARD BEAM CONNECTIONS

"A" SERIES

ALLOWABLE LOADS IN KIPS

RIVETS 7/8" HOLES 15/16"

	Rivets in Outstanding Legs		Rivets in Web Legs		Maximum Reaction	
	No.	Shear	Bearing	Shear	Section	R
A 10  2'5½" 2∠ 4×3½×7/16	20	162.4	262.5 t (t = thickness of web)	162.4	36 WF (all weights)	162.4
A 9  2'2½" 2∠ 4×3½×7/16	18	146.2	236.3 t	146.2	33 WF 240 to 152 141 132 125	146.2 142.9 137.0 134.7

...ctive beams, and should in general be used for re- ..."H" or "HH" connections, pages 216 and 217, or ...

Table 2. Standard Beam Connections from 2nd Edition AISC *Manual of Steel Construction*

In 1963, the 6th Edition was introduced with new tables for "Framed Beam Connections-Bolted or Riveted" and "Heavy Framed Beam Connections-Bolted or Riveted". The new tables permitted a fine tuned design, although a minor calculation was required. As before, the tables (Table 3) were given for 1 through 10 horizontal rows. One vertical row of fasteners was used except in the case of the "heavy" connections where 2 vertical rows were employed. Values were tabulated for 3/4, 7/8, and 1 inch diameter fasteners of the ASTM designations listed in Table 3 for both friction- and bearing-type connections. All angle material was A36, while values were given for beam material with yield strengths of 33, 36, 46, or 50 ksi. The design checks included were: shear capacity of the fastener group, and bearing (parallel to the line of force) and gross shear capacity of the framing angles. The 7th Edition (1970) version of the framing angle design aid was essentially a repeat of the 6th Edition version with updated beam material strengths.

FRAMED BEAM CONNECTIONS

Bolted or riveted

TABLE I Allowable loads in kips

10 ROWS

WF 36

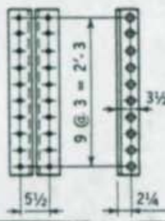


TABLE I-A Total Shear, kips

Fastener Diameter	¾	⅞	1
Angle Thickness, t	5/16	⅜	7/16
ASTM A307 Bolts	88.4	120	157
ASTM A141 Rivets *ASTM A325 HS Bolts	133	180	236
*ASTM A325 HS Bolts	194	265	346

TABLE I-B Total Bearing, kips

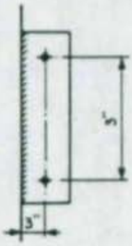
F_y ksi	Fastener Diameter ¾	⅞	1	
33	338	394	450	Bearing is on 1" thick material. Use decimal thickness of enclosed web as a multiplying factor for these values.
36	364	424	485	
46	465	543	620	
50	506	591	675	

2 ∠ 4 × 3½ × t × 2'-5½ ASTM A36. See Table I-A for t.

Table 3. Framed Beam Connections (Bolted or Riveted) from 6th Edition AISC *Manual of Steel Construction*

The 1978 *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings* introduced new provisions that complicated the production of the framed beam connection design aid. The applicable new considerations were block shear, material bearing perpendicular to the line of force, and net shear. Because of these revisions, the design aid was deleted in the 8th Edition Manual and replaced by individual tables for bolt shear, material bearing, connection angle shear, and block shear. This deletion was not well received and the AISC Manual Committee (a committee consisting of member fabricators) directed the staff to produce a new design aid. The result was a series of tables referred to as "Predesigned Bolted Framing Angle Connections" which were printed in both the 1st Quarter, 1982, AISC *Engineering Journal*, and *Detailing for Steel Construction* (AISC, 1983). (See Table 4). These tables gave allowable end reactions for 2 to 9 horizontal rows of bolts in only one vertical row for 36 and 50 ksi yield strength beam material, 3/4- and 7/8-in. dia. A325 bolts, bearing-type connections only, and uncoped or single coped beams, without requiring calculations by the user. Despite the demand for these tables, they experienced limited use. This could be attributed to two reasons: the design aid was very complicated and possibly difficult to employ, and/or few designers were aware of their existence. At the time of publication of the 9th Edition Manual, no new format had been devised, so the tables included for framed beam connections closely resemble those in the 8th Edition. AISC continued to get requests for another easy-to-use design aid for framed beam connections.

Table B-3A. Maximum Permissible Beam End Reaction (kips)



No. of Bolts: 2
Plate Depth: 6 in.

Min. Plate t, in.	Max. Moment M, kip-in.	Fillet Weld Size (E70XX), in.									
		1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16
3/16	30	9	16								
	33	3	16								
	37		16								
	41		16								
1/4	43		17	21							
	47		11	21							
	50		2	21							
	54			21							
5/16	57			23	27						
	60			19	27						
	64			12	27						
	68				27						
3/8	70				30	32					
	74				25	32					
	77				21	32					
	81				13	32					
7/16	84					36	37				
	87					33	37				
	90					29	37				
	95					22	37				

Max. Permissible Plate Thickness, in.		
Bolt		Plate Max. t
Type	Diam.	
A325	3/4	3/8
	7/8	7/16
	1	9/16
A490	3/4	1/2
	7/8	5/8
	1	11/16

Note: For standard holes only; restriction does not apply for bolts in slotted holes.

Table 4. Predesigned Framing Angle Connections from *Detailing for Steel Constuction*

1.2 Current Practice

After completion of the 9th Edition *Manual of Steel Construction*, the next priority of the AISC Manual Committee was to produce a new design aid for framing angle connections that would be useful to the beginning detailer as well as engineers. A format very similar to the 6th and 7th Edition Manuals was chosen as it provides more flexibility for fabricators and designers in comparison to the earlier one of the 2nd through 5th Editions. It follows that the resulting connections will be more economical and efficient. The new tables, simply titled "Framed Beam Connections" (Table 5), have been developed using the Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD) methods. Producing both simultaneously was straightforward, as both methods require similar design checks, except the LRFD values must be used with factored loads. The design equations and procedures were based on the 1989 AISC ASD Specification for the Design, Fabrication and Erection of Structural Steel for Buildings or the 1986 LRFD version of the Specification. The design parameters, ie. bolt size, angle thickness, hole type, edge distance, were stipulated by the AISC Manual Committee.

The design parameters and procedures employed will be described by looking at a sample page (Table 5) of the ASD version of the tables. The LRFD version is identical to the ASD tables but is labelled conspicuously to avoid confusion. Tables are given based on the following:

1. Beam material: F_y = 36 or 50 ksi
2. Angle material: F_y = 36 ksi
3. Bolts: 3/4, 7/8, 1, and 1 1/8-in. diameter
4. 2 through 9 horizontal rows of bolts in 1 vertical row in all cases

Additional parameters were assumed when calculating capacities for sub-Tables A, B, and C, as labelled here for reference only, in Table 5. Sub-Table A gives the controlling capacity between the bolts and angles. The bolt shear allowable loads were calculated for the given diameter for A325 and A490 high-strength bolts, slip-critical (Class A and B) and bearing-type connections, with either standard or short-slotted holes. Angle allowable loads are based on material bearing, gross and net shear. Values are tabulated for angle thicknesses 1/4, 5/16, 3/8, and 1/2 in. Vertical edge distance is assumed to be 1 1/4 in. and pitch is 3 inches. Net shear is based on hole reductions of fastener diameter plus 1/16, so hole type does not effect that calculation.

Sub-Table B tabulates Beam Web Capacities or allowable loads for 1 in. thick material. For uncoped beams, the loads are based on material bearing and assuming the edge distance is greater than 1.5 multiplied times the bolt diameter (d_b). In the case of beams with the top flange coped, the values given are the smaller due to block shear and material bearing. When both flanges are coped, the overall depth can be calculated and gross and net shear can be checked in addition to block shear and material bearing. Horizontal edge distance is assumed for two cases: 1 1/2 in. and 1 3/4 in. Because of possible mill underrun or other uncertainties of fabrication, these values have been reduced for calculation purposes to 1 1/4 in. and 1 1/2 in., 5/8 in., 2 in., and 3 in. The effect of deep or long copes on beam

SAMPLE PAGE

F_y=36 ksi — BOLTS - 3/4 in.

FRAMED BEAM CONNECTIONS

Bolted

Allowable loads in kips

2 ROWS

W 12, 10, 8
S 12, 10, 8
C 12, 10, 9, 8
MC 13, 12, 10, 9, 8

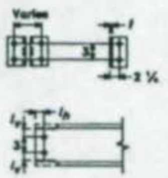


Table A

Allowable Bolt and Angle Capacity*, kips

ASTM Designation	Connection Type	Class	Hole Type	F_v ksi	Angle Thickness t, in.			
					1/4	5/16	3/8	1/2
A325	SC	A	STD	17	30.0	30.0	30.0	30.0
A325	SC	A	SSL	15	26.5	26.5	26.5	26.5
A325	SC	B	STD	28	33.7	42.1	49.5	49.5
A325	SC	B	SSL	24	33.7	42.1	42.4	42.4
A325	N	-	-	21	33.7	37.1	37.1	37.1
A325	X	-	-	30	33.7	42.1	50.6	53.0
A490	SC	A	STD	21	33.7	37.1	37.1	37.1
A490	SC	A	SSL	18	31.8	31.8	31.8	31.8
A490	SC	B	STD	34	33.7	42.1	50.6	60.1
A490	SC	B	SSL	29	33.7	42.1	50.6	51.2
A490	N	-	-	28	33.7	42.1	49.5	49.5
A490	X	-	-	40	33.7	42.1	50.6	67.4

* Includes bearing, and gross and net shear.

Note: SSL (short-slotted) values are with the load perpendicular to the slot.
SSL values also apply to oversized holes.

Table B

Allowable Beam Web Capacity, 1 in. material, kips

	No Cope	l_v, in.											
		Coped, Top Flange						Coped, Both Flanges					
		1 1/4	1 3/8	1 1/2	1 5/8	2	3	1 1/4	1 3/8	1 1/2	1 5/8	2	3
STD HOLES l_h = 1 1/2	104	77.2	79.4	81.6	83.7	90.3	104	67.4	71.8	76.1	80.5	90.3	104
l_h = 1 3/4	104	84.5	86.6	88.8	91.0	97.5	104	67.4	71.8	76.1	80.5	93.5	104

Table C

Allowable Supporting Member Capacity, 1 in. material, kips

F_y, ksi	
36	50
209	234

Table 5. Sample page of new Framed Beam Connections Tables (Bolted)

performance has not been included and must be checked independently (AISC, 1984 and Cheng, 1988 and Cheng, 1988).

Allowable bearing loads are tabulated in sub-Table C for the supporting member to which the outstanding legs are attached. The edge distance is again assumed to be at least 1.5 x d_b and the material to be 1 in. thick for F_y = 36 and 50 ksi.

These are not fully designed connections, however each page lists in the upper left-hand corner the applicable beam depths based on a maximum and minimum angle length of *T* and *T*/2, respectively, where *T* is the depth of the web minus the fillets. The allowable end reaction for a particular beam is the smaller value resulting from the three sub-tables (A, B, C). The number can be taken directly from sub-Table A, while B and C require multiplication of the beam web thickness times the table value.

3. CONCLUSION

Despite the increasing use of computer software in design offices and fabricating shops, there continues to be a demand for printed design aids, particularly for connection design. The new AISC publications, *Allowable Stress Design of Simple Shear Connections* and the corresponding Load and Resistance Factor Design version, include only shear connection design aids. It is a combination of new material, and existing material from the Connection Chapters of the AISC Manuals (ASD and LRFD). New material includes the Framed Beam Connection Tables, design aids for welded single angles, and tee framing connections. The design aids for seated beam, end-plate shear, single-plate, and bolted single-angle connections are repeated from the AISC Manual.

REFERENCES

American Institute of Steel Construction (1989) Manual of Steel Construction *9th ed., Chicago, IL, 1989.*

American Institute of Steel Construction (1983) Detailing for Steel Construction *Chicago, IL, 1983.*

American Institute of Steel Construction (1984) Engineering for Steel Construction

Chicago, IL, 1984.

Cheng, J.R., J.A. Yura and C.P. Johnson (1988) Lateral Buckling of Coped Steel Beams *ASCE Journal of the Structural Division, Vol. 114, No. 1, January 1988.*

Cheng, J.R. and J.A. Yura (1988) Lateral Buckling Tests on Coped Steel Beams *ASCE Journal of the Structural Division, Vol. 114, No. 1, January 1988.*

SIMPLE BEAM-TO-COLUMN CONNECTIONS.

John W. Pask [1]

Abstract

This paper discusses four commonly used types of simple beam-to-column connection, viz: full depth and short flexible end plates, web angle cleats and fin plate connections and outlines general guidelines as used in British practise for ensuring appropriate rotational flexibility. The stability during erection of framing with simple connections is also discussed.

1. INTRODUCTION.

Design codes usually require that beam end connections in simple framing should be capable of transmitting the design end shear and accepting the resulting rotations but should not develop end moments likely to adversely affect the design of the main members.

In the design of simple connections the optimum means of implementing these requirements varies from one type of connection to another and for this reason the aformentioned types of connection, typical details of which are given in Figs. 1-4, are discussed separately.

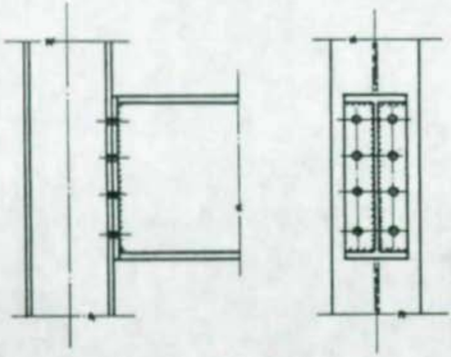


Fig 1.
Full depth flexible
end plate connection.

[1] Representing the British Construction Steelwork Association Ltd., London.

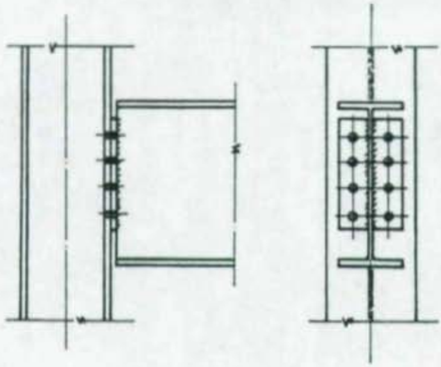


Fig. 2
Short flexible
end plate connection.

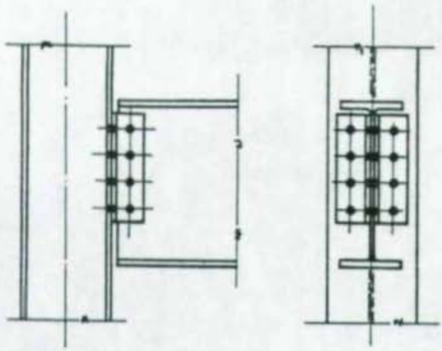


Fig. 3
Angle web cleat connection.

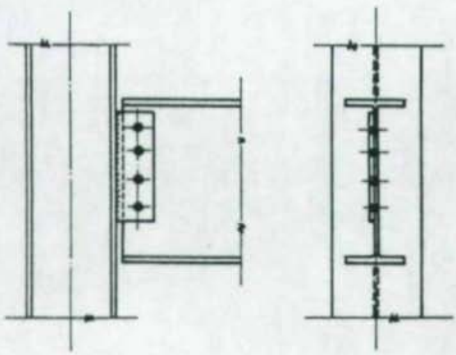


Fig. 4
Fin plate connection.

2. FULL DEPTH FLEXIBLE END PLATE CONNECTIONS.

Full depth flexible end plates are significantly more rigid than short flexible end plates and provide greater stability during erection. For this reason full depth plates are usually preferred in the U.K. In addition to being welded to the beam web they are also fillet welded to the insides of the beam flanges. The following rules for ensuring appropriate flexibility are given with reference to Fig. 5.

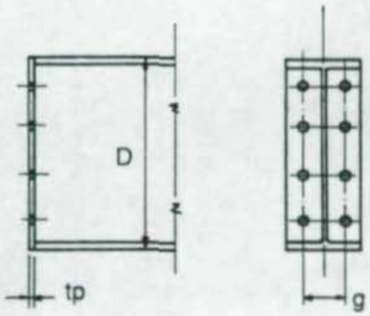


Fig.5

- tp = 8mm when D ≤ 450 mm
  tp = 10mm when D > 450 mm
- g ≥ 90mm when tp = 8mm
  g ≥ 140 mm when tp = 10mm
- Use non-preloaded bolts.

The extensive use in the U.K. over many years of the full depth flexible end plate is verified by Eurocode 3: Part 1, which permits connections to be classified as nominally pinned on the basis of significant experience of previous satisfactory performance in simple framing.

3. SHORT FLEXIBLE END PLATE CONNECTIONS.

The short flexible end plate is used in preference to the full depth plate when it is decided to maximise on rotational flexibility. The subsequent rules, applied with reference to Fig. 6, ensure that within practical limitations the connection is as flexible as possible.

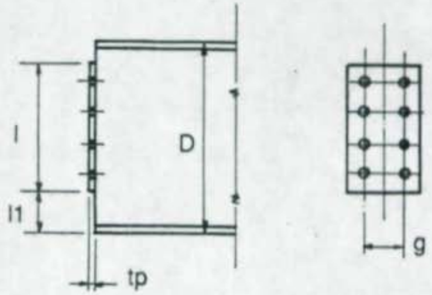


Fig. 6

- l ≤ depth between beam web fillets
  l ≥ 0.6 D

- tp = 8 mm when D ≤ 450 mm
  tp = 10 mm when D > 450 mm

- $\frac{l1}{tp}$ ≤ 33

- g ≥ 90 mm when tp = 8mm
  g ≥ 140 mm when tp = 10mm

- Use non-preloaded bolts

The third item ensures that the beam tension flange does not bear against the supporting column and is based on the adoption of a maximum design rotation of 0.03 radians and research (Kennedy, 1969) indicating that the connection rotates about a point close to the lower edge of the end plate.

As the short end plate is less rigid than the full depth plate it is obviously suitable for use in simple framing.

4. ANGLE WEB CLEAT CONNECTIONS

Research (Munse et al., 1959) indicates that in the context of rotational flexibility the behaviour of flexible angle cleats of the type shown in Fig. 3, is similar to that of short flexible end plates. The tests indicated that when a beam is supported on a stiff member such as a column, the centre of rotation is at or near the lowest bolts in the supported member web. Resulting from this similarity, many of the subsequent rules for achieving flexibility in angle web cleat connections are similar to those short flexible end plates.

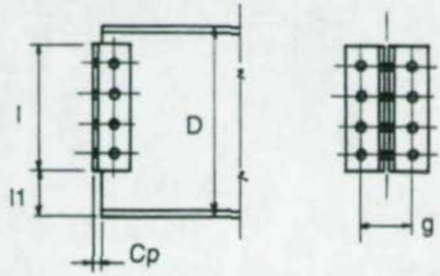


Fig. 7

with reference to Fig. 7:

- l $\leq$ depth between beam web fillets
  l $\geq$ 0.6 D

- Cleat thickness:
  8mm when D $\leq$ 450mm
  10mm when D $>$ 450mm

- $\frac{l1}{Cp} \leq 33$
  where Cp is the cleat projection

- g $\geq$ 125mm

- Use non-preloaded bolts

As in the case of full depth flexible end plate connections the use of angle web cleat connections in simple framing is verified by previous extensive and satisfactory performance.

5. FIN PLATE CONNECTIONS

Recent tests in the U.K. indicate that the main contribution to rotational flexibility in fin plate connections is derived from hole elongation in the fin plate and/or beam web, and consequently failure modes such as bolt shear, weld rupture and shear failure of the fin plate or beam web which exhibit insufficient ductility should be avoided to enable the necessary hole elongation to take place.

The tests established that sufficient rotation exists when non-preloaded 8.8 bolts are used in conjunction with Fe E 275 steel provided that the fin plate and/or beam web thickness is $\leq$ 0.5 d where d is the nominal diameter of the bolt and also provided that all end and edge distances are $\geq$ 2.0 d.

Extrapolating beyond the tests indicates that in the case of Fe E 355 steel, the thickness of fin plate and/or beam web should be ≤ 0.42 d.

The research was restricted to beams not greater than 610 mm in depth.

Rules for generating flexibility are given with reference to Fig. 8.

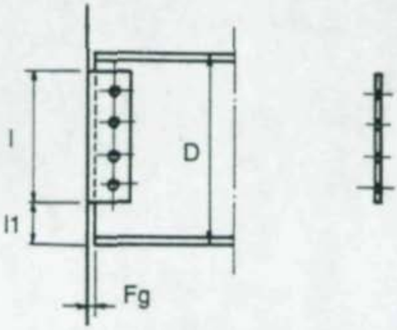


Fig. 8

- l ≤ depth between beam fillets
  l ≥ 0.6 D

- Use 8.8 non-preloaded bolts

- Either fin plate and/or beam web thickness
  ≤ 0.5d for Fe E 275 steel
  ≤ 0.42d for Fe E 355 steel

- All edge and end distances
  ≥ 2.0d

- $\frac{l1}{Fg} \leq 33$
  where Fg is the fin-plate projection

- Avoid low ductile failure modes as previously discussed.

6. ERECTION OF SIMPLE FRAMING.

In simple framing such as when used in multi-storey office block constuction, due to the lack of appreciable joint rigidity in the beam-to-column connections, appropriately designed floors and roofs are normally used for the purpose of transmitting horizontal wind loading and other lateral forces from the intermediate frame positions to points of lateral support such as shear walls, lift shafts and vertical braced frames etc.

During erection this facility is not available and it is necessary to decide by calculation *whether the connections are capable of stabilising the structure and resisting all forces* likely to be experienced during erection, such as those due to wind loading on the bare steelwork, lack of verticality in the structure and the stacking of building materials on the floor beams.

In the event of the connections not being capable of meeting these requirements then temporary lateral restraint must be provided. This usually involves the appropriate use of cables, props or bracings etc, in locations where necessary. These stay in place for plumbing purposes and are not removed until the floors and roof are cast and are capable of acting as stiff diaphragms and until the permanent bracings are installed, or until such time as erection is sufficiently advanced so as to allow their safe removal. These arrangements are preferably carried out in conjuntion with the provision of columns with substantial bases and four holding down bolts into the concrete. This imparts considerable stability to the structure during the early stages of erection.

Clearly these procedures are somewhat onerous and tend to delay erection but subject to design confirmation can usually be avoided by using full depth flexible end plates, or by using short flexible end plates, angle web cleats or fin plates of depth equal to the depth between web fillets of the supported beam. However, in the case of the three latter mentioned types of connection, when the depth of the connection is equal to or approaching the minimum value of 0.6 D and particularly when lower values as used in some countries are adopted, then the provision of temporary lateral restraint may be crucial. Unfortunately due to the large number of parameters involved it is not practical to classify simple connections as being capable or otherwise of stablilising structural framing during erection and consequently every application should be considered separately.

REFERENCES.

Kennedy, D.J.L., 'Moment-Rotation Characteristics of Shear Connections', American Institute of Steel Construction, Engineering Journal, Vol. 6 No. 4, Oct. 1969, p.105.

Munse, W.H., Bell, W.G. and Chesson, E., 'Behaviour of Riveted and Bolted Beam-to-Column Connections', *Journal of the Structural Division, ASCE, Vol. 85, No. ST3, March* 1959, p.29.

A DESIGN APPROACH FOR SEMI-RIGID CONNECTIONS IN COLD-FORMED STEEL INDUSTRIAL RACKS

Teoman Peköz[1]

ABSTRACT

Design of cold-formed steel industrial rack frames involves dealing with semi-rigid connections. Design procedures have been included in the Commentary to the Design Specifications for Racks since 1972. These procedures are described in this paper.

1. INTRODUCTION

The edition of "Specification for the Design, Testing and Utilization of Industrial Storage Racks", (Ref. 1) adopted in 1972, recognizes the need for semi-rigid frame design. Rational analysis is required by the Specification (Ref. 1) and the Commentary to the Specification (Ref. 2) provides simple procedures primarily for hand-calculations. These design procedures were developed based on analytical and experimental studies on full scale structures and components. These studies are described in Refs. 3, 4 and 5.

Work is presently under way to refine or modify these design approaches with the aid of computer programs that account for non-linear geometric behavior and non-linear semi-rigid connections.

2. RACK TYPES

Though there are several types of racks, the discussion here will be restricted to the design of pallet and stacker racks. Standard or pallet racks shown in Fig. 1 consist of upright frames and pallet beams. Upright frames usually consist of two lipped channel

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columns tied together by horizontal and diagonal braces. The goods to be stored are usually on pallets placed on pallet beams. The pallet beams are connected to the columns of the upright frames by mechanical connectors which have to facilitate quick assembly and changes depending on the goods stored. There are many different types of connectors and an example is shown in Fig. 2. At present, the maximum height of pallet racks is about 35 feet determined by the vertical reach of fork-lift trucks.

When storage requirements necessitate a higher rack system a stacker rack may be used. The pallets are placed on the rack by means of automated stacker cranes. Currently, stacker racks in excess of 120 feet are being built. The building walls and roof may also be supported by the rack.

3. STRUCTURAL BEHAVIOR

Along the aisle direction, namely parallel to the pallet beams, the framing consists of columns, beams and diagonal bracing in the rear vertical plane if any is provided. Frame action perpendicular to the aisle also involves semi-rigid joints. However, the discussion here will be restricted to the frame action parallel to the aisle.

Racks are designed for horizontal loads resulting from earth-quake effects, initial out-of-plumbness or floor irregularities and vertical pallet loads. Additional loads due to forklift collision and impact of the placement of loads need to be considered.

Semi-rigid behavior of the joints between the upright columns and the pallet beams is primarily due to the distortion of the walls of the columns at the joints and the distortion of the beam end plates. The connection details vary very widely. It is impossible to establish general procedures for computing joint stiffness and strength. It is, therefore, necessary to determine these characteristics by simple tests. Two types of tests are used to characterize the behavior of joints. These tests are cantilever and portal tests.

4. CANTILEVER TEST

A cantilever test provides a simple means of determining the connection moment capacity and rigidity. The test set-up according to Ref. 2 is shown in Fig. 3.

The relationship between the moment and the angular change at a joint is not linear as shown in Fig. 4 for typical joints. However, a linear idealization of the behavior is used in Ref. 2. The change in angle between the column and the connecting beam (in radians) is idealized as follows:

$$\theta = M / F$$

where M is the moment at the joint between the connecting members and F is the spring constant relating the moment to the rotation.

Cantilever tests can be used to determine the value of F with the following equation (Refs. 2 and 3):

$$F = \frac{R_f}{\dfrac{\delta_{0.85}}{P_{0.85} L_{b2}} - \dfrac{L_c}{16EI_c} - \dfrac{L_b}{3EI_b}}$$

where $P_{0.85}$ is 0.85 times the ultimate load and $\delta_{0.85}$ is the deflection of the free end of the cantilever at load $P_{0.85}$. L_c, L_b, I_c, I_b are the lengths and moments of inertias of the column and the beam, respectively. E is the modulus of elasticity. R_f is a reduction factor to provide safety considering the scatter of test results. Since a lower F means a higher design moment for the beam, an R_f = 2/3 may be taken in the design of the beam. However, in determining the bending moments for the columns, a higher F leads to a more conservative value of the bending moment. It is, therefore, recommended to take R_f = 1.0 for this case.

It is suggested that the spring constant F be calculated on the basis of the average results of two tests on identical specimens, provided that the deviation from the average does not exceed 10%: if the deviation from the average exceeds 10%; then a third specimen is to be tested. The average of the two higher values is to be used in design.

5. PORTAL TEST

The portal test illustrated in Fig. 5 is desirable when the value of F is to be used in a sidesway analysis either for lateral deflections or stability. Under vertical loads the connections in general "tighten up". Under sidesway, the connection at one end of the beam "tightens up" while the connection at the other end "loosens". The portal test gives an approximate average value of the spring constants involved in this process, which should be used to determine the effective lengths and horizontal deflections.

The following expression (Refs. 2 and 3) can be used to determine F:

$$F = \frac{R_f}{\frac{2\delta}{Hh^2} - \frac{h}{3EI_c} - \frac{L}{6EI_b}}$$

R_f is a reduction factor to be taken equal to 2/3, H is the horizontal load per beam. The dimensions L and h are shown in Fig. 5. Sway deflection δ corresponds to a lateral load of 2H.

Since the behavior at both the design and the ultimate load is of interest, portal tests are to be conducted at both load levels. Multiple portal tests are recommended as in the case of cantilever tests.

6. BEAM DESIGN

Designing beams as simply supported would result in about 10-15% conservatism for the beams. The following equation is derived in Ref. 3 for determining the maximum midspan moment M_{max} of a pallet beam considering semi-rigid end connections:

$$M_{max} = \frac{WL}{8}\left(1 - \frac{LF}{3EI_b\lambda}\right) \quad \text{and} \quad \lambda = \frac{F}{E}\left(\frac{h}{12I_c} + \frac{L}{2I_b}\right) + 1$$

where h is the vertical distance between the inflection points in the column segments above and below the beam in question, L is the span (centerline to centerline of the columns) and W is the total load on each beam. The joint spring constant F is to be determined by cantilever tests. The load is assumed to be uniformly distributed.

Again, if one considers semi-rigid joints, the following expression for maximum deflection is given in Refs. 1 and 3:

$$\delta_{max} = \frac{5WL^3}{384EI_b}\left(1 - \frac{2FL}{5EI_b\lambda}\right)$$

7. FRAME STABILITY ANALYSIS

As in Refs. 6, 7 and 8, the effective length concept is used determine the load carrying capacity of columns subjected to either an axial load or a combination of an axial load and bending moments. The effective length factor accounts for the restraining effect of the end conditions or the effect of the members framed into a particular member.

Effective length coefficients can be determined (Ref. 2) using alignment charts given in Refs. 7 and 8 provided that the stiffness of the pallet beams is reduced to $(I_b/L_b)_{red}$ due to the semi-rigid nature of the joints. For racks not braced against sidesway, Ref. 2 gives the following expression:

$$\left(\frac{I_b}{L_b}\right)_{red} = \frac{I_b/L_b}{1+6(EI_b)/(L_bF)}$$

where I_b and L_b are the actual moment of inertia and the actual span of the pallet beams, respectively. The joint rigidity F is to be determined by portal tests.

For racks braced against sidesway, the above expression becomes

$$\left(\frac{I_b}{L_b}\right)_{red} = \frac{I_b/L_b}{1+2(EI_b)/(L_bF)}$$

Partial base fixity of the columns is accounted for (Refs. 2 and 3) by using fictitious beams to represent the floor. The expression used for the moment of inertia, I_f and length, L_f of this beam is based on an interpretation of Ref. 9 by Refs. 10 and 11.

The effective length factor is found directly from the alignment charts on the basis of G_a and G_b of Refs. 7 and 8. For the portion of the column from the floor to the first beam level:

$$G_a = \frac{I_c(1/L_{c2}+1/L_{c1})}{2(I_b/L_b)_{red}} \quad \text{and} \quad G_B = \frac{I_c/L_{c2}}{I_f/L_f}$$

where I_c is the column moment of inertia, L_{c1} is the distance from the first beam level to the second beam level and L_{c2} is the distance from the floor to the first beam level.

8. ACKNOWLEDGEMENTS

The original work and the work in progress are sponsored by the Rack Manufacturers Institute. The support of the RMI Specification Advisory Committee and its Chairman Mr. H. Klein and Mr. J. Nofsinger of the RMI Staff is gratefully acknowledged.

REFERENCES

1. "Specification for the Design, Testing and Utilization of Industrial Storage Racks", Rack Manufacturers Institute, Charlotte, NC, June 1990 Edition.

2. "Commentary on Specification for the Design, Testing and Utilization of Industrial Storage Racks", Rack Manufacturers Institute, Charlotte, NC, June 1990 Edition.

3. Peköz, T., "Interpretation of Pallet Rack Test Results", Report for the Rack Manufacturers Institute, Charlotte, NC, February 1978.

4. Peköz, T., "Pallet Rack Design Criteria", Report for the Rack Manufacturers Institute, Charlotte, NC, February 1978.

5. Peköz, T., "Pallet Rack Test Results", Department of Structural Engineering Report, Cornell University, September 1975.

6. "Specification for the Design of Cold-Formed Steel Structural Members", American Iron and Steel Institute, Washington, DC, 1986 Edition.

7. "Manual of Steel Construction - Load and Resistance Factor Design", AISC, Chicago, IL, First Edition, 1986

8. "Manual of Steel Construction - Allowable Stress Design", AISC, Chicago, IL, Ninth Edition, 1989

9. Timoshenko, S. P. and Goodier, J. N., "Theory of Elasticity", McGraw-Hill Book Company, New York, NY., Third Edition

10. Galambos, T. V., "Influence of Partial Base Fixity on Frame Stability", J. of the Structural Division, ASCE, May 1960.

11. Salmon, C. G., Schlenker, L. and Johnston, B. G., "Moment-Rotation Characteristics of Column Anchorages", Proceedings, ASCE, April 1955.

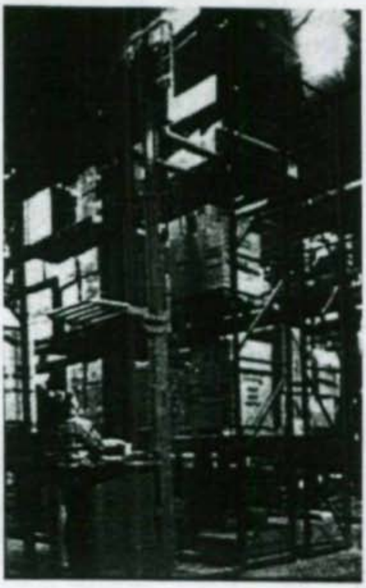

Fig. 1 A Typical Pallet Rack Installation

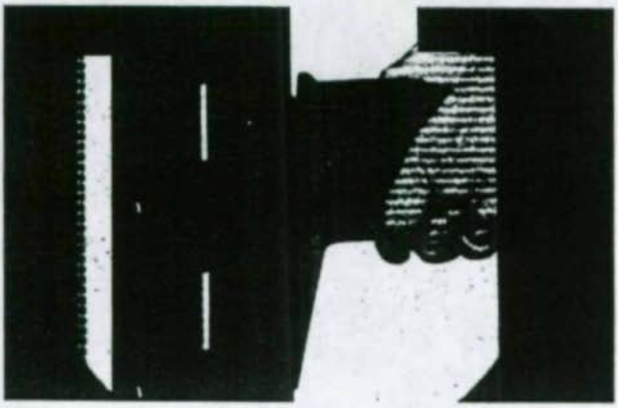

Fig. 2 A Pallet Rack Joint

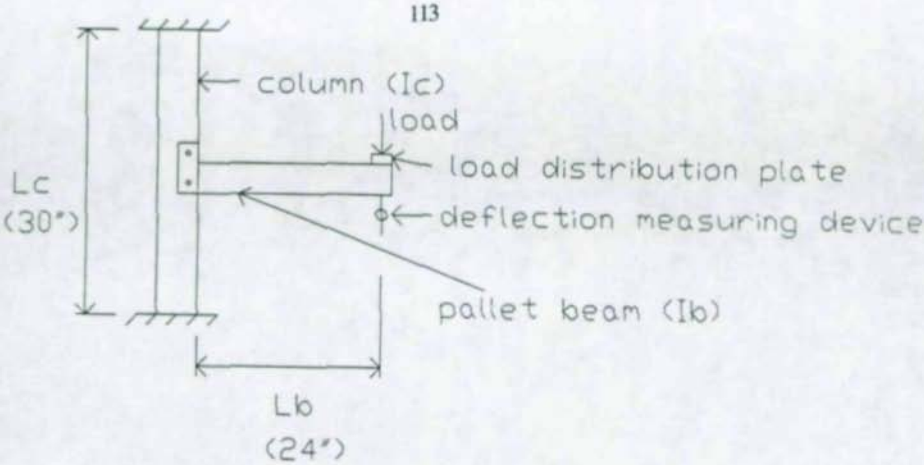

Fig. 3 Cantilever Test Set-up

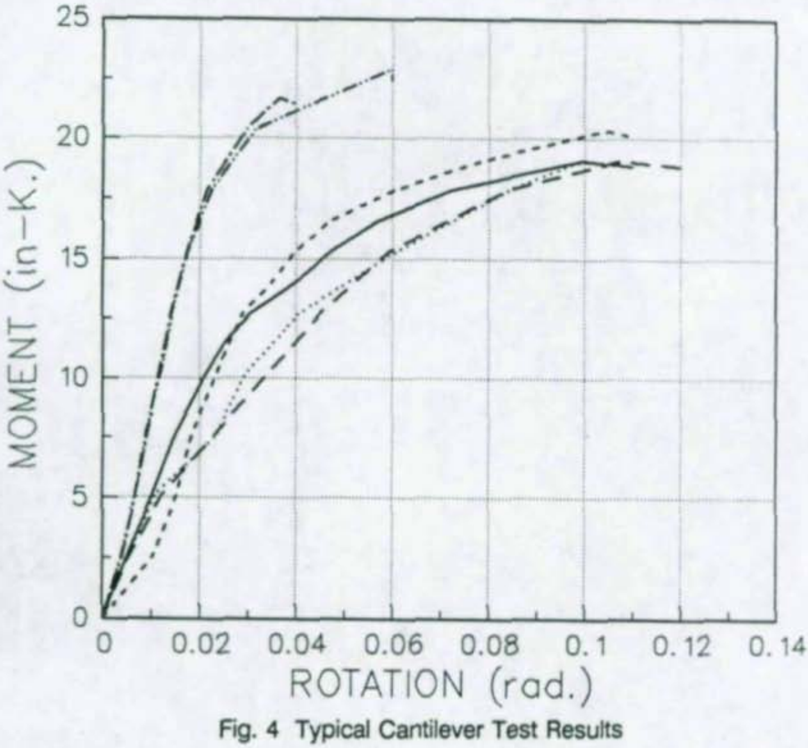

Fig. 4 Typical Cantilever Test Results

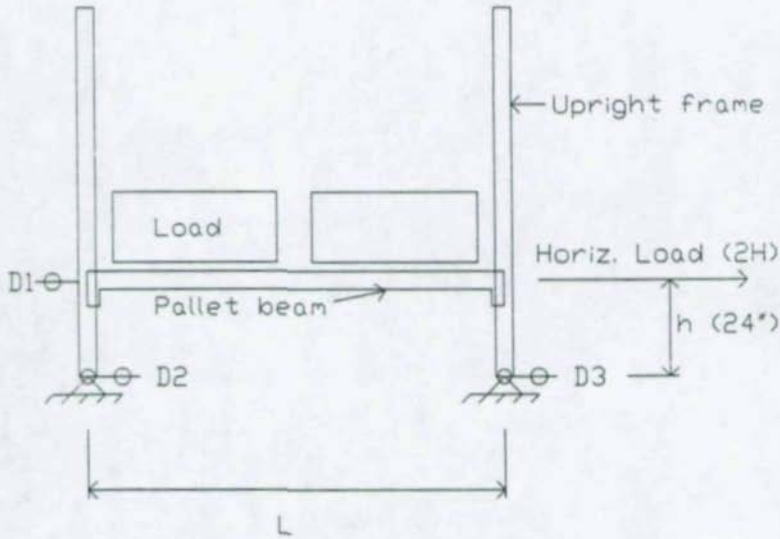

Fig. 5 Portal Test Set-up

CONNECTIONS OF THIN WALLED MEMBERS

G. Sedlacek[1]

K. Weynand[2]

Abstract

In the frame of the background studies for the Annex A of Eurocode 3 the strength functions for connections in thin walled members have been calibrated with test results. Available test results for all mechanical fasteners were taken into account. The test evaluation was based on a procedure for the determination of characteristic and design values for resistances which was used to develope the design rules of EC 3. As a result of the evaluations some of the strength functions proposed by the ECCS were improved and simplified. The new strength functions are compatible with existing strength functions for connections of members with normal plate thicknesses. A unique safety factor $\gamma_M{}^* = 1.25$ can be applied to all connection types.

1. INTRODUCTION

Connections of thin walled members are used for connecting steel sheets to the supporting structure e.g. to a purlin or to interconnect two or more sheets, e.g. in joints of corrugated sheets. For such connections a lot of different types of fasteners and joining methods are used in practice.

In comparison with normal plate connections with plate thicknesses $t > 3$ mm the behaviour of connections of thin walled members is characterized by the small plate stiffness of the sheet. Therefore additional effects may appear in the ultimate limit state. Such effects are for example the tilting of the fastener in hole bearing failure or the big distortion of the sheet when the fastener is loaded in tension and the sheet is pulled over the head of the fastener. Therefore particular investigations were conducted to determine the load capacity of connections of thin walled members.

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A typical way to check the strength of a connection is to carry out big test series and to prepare tables with design strengths obtained from these tests. The disadvantage of this method is that a big number of tests is necessary to get design strength values with sufficient reliability and that the design tables are limited to cases where tests were made. A more efficient method for the designer is the use of strength functions based on a physical model that includes all significant parameters and is calibrated with tests. This method gives more possibilities to design a connection.

2. DESIGN RULES IN EUROCODE 3

2.1 General

In the frame of the development of a common European Standard for the design of Steel Structures (Eurocode 3), unified design rules are being developed. These design rules are based as far as possible on recommendations of the international technical scientific organizations as the ECCS. They shall also be founded on a standardized safety concept and meet a certain target reliability.

Eurocode 3, part 1 contains general rules and rules for buildings. Design rules for normal connections are presented in the main document. Additional rules that are less frequently used and special information are given in Annexes.

The Annex A of Eurocode 3 in particular deals with cold formed steel sheeting and members. This Annex A has been drafted by a Eurocode-Drafting Panel together with the ECCS TWG 7.5.

2.2 Annex A of Eurocode 3

Annex A of EC 3 provides design rules for the ultimate limit state and serviceability limit state verification of thin walled sheeting and members. The chapter 8 of Annex A presents strength functions and characteristic values for bolted and welded connections.

In order to achieve the required reliability these functions and characteristic values had to be calibrated with test results. The procedure for the evaluations that was harmonized across different materials by CEB and ECCS and some results are presented in this paper.

The statistical evaluations which are documented in a background report to Annex A are based on results of tests with mechanical connections. The types of fasteners for which strength functions had to be calibrated are given in table 1. The different failure modes that were dealt with in the test reports are given in table 2.

On the basis of these tests the ECCS had proposed particular design rules, the validity of which were limited to the following thickness:

$0.5 \leq t \leq 4.0$ mm for sheeting

$1.0 \leq t \leq 8.0$ mm for members.

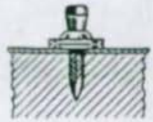
blind rivets	bolts with nuts	cartridge fired pins	screws
			

table 1: types of fastener

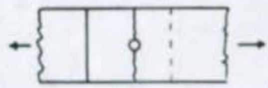
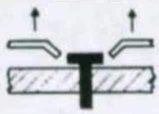
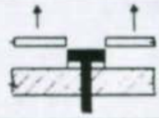
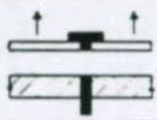
shear force	hole bearing (including tilting)			
	failure of net section			
	shear of fastener			
tension force	pull through	pull over		
	pull out			
	tensile of fastener			

table 2: failure modes of connections in thin walled members

By the calibration described in this paper the ECCS-proposals had to be checked and further developed by taking into account the following aims:

- Compatibility with EC 3, part 1. The transition from thin sheeting to members with normal plate thicknesses should be continuous.
- The design rules should be based on mechanical models.
- The range of validity of the design rules should be as large as possible.

3. BACKGROUND STUDIES

3.1 General Procedure

The procedure for the calibration of the characteristic strength functions with test results is based on a statistical evaluation that is carried out in the following steps, figure 3.

Step 1: A strength function is proposed, that should be based on a simple mechanical model and contain all relevant parameters.

Step 2: All available test results are collected. The documentation of these tests should contain measured data for all parameters taken into account in the strength function; otherwise the tests cannot be considered.

Step 3: Comparison of the experimental test results with the results yielding from the strength functions with the measured parameters.

Step 4: Check of the sensitivity of the strength function in view of the different parameters and subsets.

Step 5: Determination of the mean value corrections and coefficients of variation.

Step 6: Determination of the characteristic values, design values and the partial safety factors γ_M from the statistical parameters obtained in step 5.

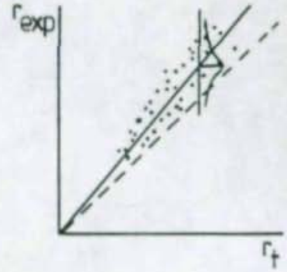


figure 1: comparison of theoretical strength with test results

The results of the sensitivity checks (step 4) as shown in the diagrams below (chapter 3.2) may be used

- to show the influence of the different parameters in the strength function,
- to find out whether additional parameters should be included in the strength function or subsets of the test population, for which parameters are almost constant, should be provided in the evaluation.

A parameter may be taken as correctly taken into account when the ratio of the experimental results to the results of the strength function is constant versus this parameter.

3.2 Test Evaluations

For the statistical evaluations all available test reports were collected and evaluated to form a database. This database contains nearly 6000 test results for all types of fastener and all failure modes. *Only test results which are completely described in the test reports were taken into account.*

The statistical evaluations were carried out for each fastener type and for each failure mode. The modes shear failure and tensile failure of the fastener were not checked because these modes were completely checked in the frame of the background work for connections of members with normal plate thicknesses.

Most of the characteristic strength functions proposed by ECCS could be verified with the statistical evaluations. In two cases the proposals could be improved significantly. The results of these evaluations are given below as examples.

3.2.1 Example 1: Bolts with nuts, hole bearing failure

The strength function for the hole bearing failure as originally proposed by ECCS is given in equation (1). It contains an α-value which is dependent on the distance e/d and on the sheet thickness t as shown in figure 2.

$$F_b = \alpha \cdot t \cdot d_n \cdot f_u \qquad (1)$$

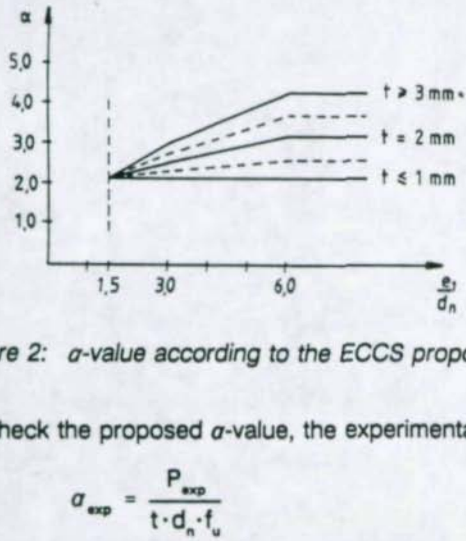


figure 2: α-value according to the ECCS proposal

To check the proposed α-value, the experimental α-factor α_{exp} was calculated as

$$\alpha_{exp} = \frac{P_{exp}}{t \cdot d_n \cdot f_u} \qquad (2)$$

where
- P_{exp} : ultimate load of the connection obtained from the experiment
- t : measured thickness of the thinnest sheet
- d_n : nominal diameter of the bolt
- f_u : measured ultimate tensile strength of the steel sheet that failed.

The α_{exp}-values are plotted versus the sheet thickness t in figure 3. To check the compatibility with the design rules in EC 3 for normal plate thicknesses, the results of test specimen with thicker sheets are also included in this sensitivity diagram.

The functions $\alpha(t)$ as proposed by the ECCS and as specified in EC 3, part 1 for normal plate thicknesses are given in this figure. It is evident from figure 3, that the dependence on the sheet thickness t as proposed by ECCS is not justified and the proposal could be easily changed to get compatibility with EC 3. The revised formula

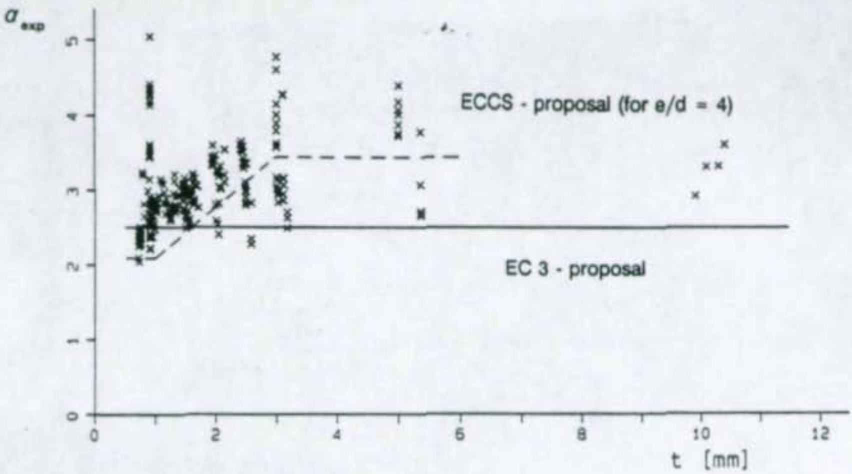


figure 3: sensitivity diagram

in Annex A therefore reads:

$$F_b = 2.5 \cdot \alpha \cdot t \cdot d_n \cdot f_u \qquad \text{where} \quad \alpha = \min\left[\frac{e}{3\,d}\,;\,1\right] \tag{3}$$

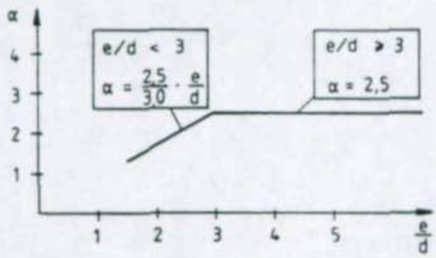


figure 4: α-value as specified in EC 3

All test results were then compared with this revised strength function. The partial safety factor γ_M^* obtained from it is smaller than the EC 3 - value $\gamma_M^* = 1.25$.

3.2.2 Example 2: Screws, pull through/pull over failure

The strength function for this failure mode proposed by the ECCS is given in the following equation.

$$F_p = 15 \cdot t \cdot f_u \qquad \text{and} \quad d_w \geq 14\text{mm} \tag{4}$$

where d_w: maximum of the nominal diameter of the fastener head or washer, if any.

The figure 5 shows the sensitivity diagram for

$$\sigma_{exp} = \frac{P_{exp}}{t \cdot f_u} \tag{5}$$

which is plotted versus the parameter d_w.

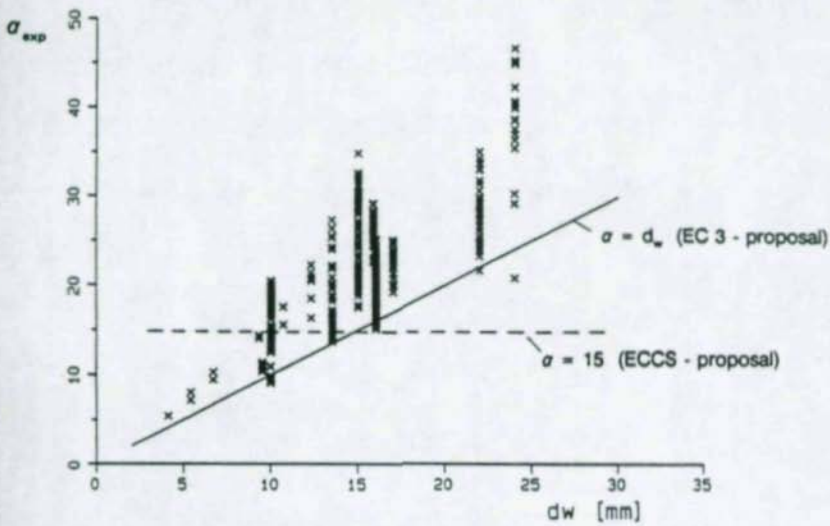


figure 5: sensitivity diagram

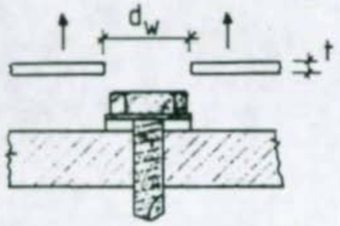
As can be seen from figure 5, the σ_{exp}-value is depending on the parameter d_w, that means that the washer diameter d_w can be included in the strength formula. The application of a simple mechanical model as shown in figure 6 leads to the formula given in equation (6)

$$F_p = \sigma \cdot t \cdot d_w \cdot f_u \quad \text{with} \quad \sigma = 1{,}0 \tag{6}$$

The results of the statistical evaluation carried out with the strength function in equation (6) lead to a safety factor γ_M^* of 1.25. The strength function is safe for all checked diameters (see figure 5). The limitation for d_w in equation (4) does not apply for equation (6).

3.2.3 Results

The checked strength functions for all types of fasteners and for all failure modes which were checked with test results are summarized in table 3.



$$F_p = \alpha' \cdot \pi \cdot d_w \cdot t \cdot f_u / \sqrt{3}$$
$$= \alpha' \cdot 1.8 \cdot d_w \cdot t \cdot f_u$$

where α' takes into account the bending and membrane effects

$$F_p = \alpha \cdot d_w \cdot t \cdot f_u$$

where $\alpha = 1.8 \cdot \alpha'$

figure 6: mechanical model of the pull over failure

fastener type	failure mode		range of validity
	hole bearing (including tilting)	failure of net section	
blind rivets	$F_b = \alpha\, t\, d_n\, f_u$ where: - for $t_1 = t$: $\alpha = 3.6\sqrt{\frac{t}{d_n}}$ but $\alpha \leq 2.1$ - for $t_1 \geq 2.5t$: $\alpha = 2.1$ - for $1 < t_1/t < 2.5$: linear interpolation	$F_n = A_n\, f_u$	$e_1 \geq 3d_n$ $e_2 \geq 3d_n$ $u_2 \geq 3d_n$ $u_1 \geq 1{,}5d_n$ $2.6 \leq d_n \leq 6.4$ mm
bolts with nuts	$F_b = 2.5\, \alpha\, t\, d_n\, f_u$ where α is the minimum of $\frac{e_1}{3d_n}$ and 1.0	$F_n = A_n\, f_n$ where: $f_n = (1 - 0.9r + 3r\frac{d}{u})f_u$ with a maximum of $f_n = f_u$ u = minimum of $2u_1$ or u_2	$e_1 \geq 1.5d_n$ $e_2 \geq 3d_n$ $u_2 \geq 3d_n$ $u_1 \geq 1.5d_n$ bolt size: > M6 Bolt classes: 4.6 up to 10.9 $t \geq 1.25$ mm
cartridge fired pins	$F_b = 3.2\, f_u\, d_n\, t$	$F_n = A_n\, f_u$	$e_1 \geq 4.5\, d_n$ $e_2 \geq 4.5\, d_n$ $u_2 \geq 4.5\, d_n$ $u_1 \geq 4.5\, d_n$ $3.7 \leq d_n \leq 6.0$ $d_n = 3.7 \rightarrow t_1 \geq 4$ $d_n = 4.5 \rightarrow t_1 \geq 6$ $d_n = 5.2 \rightarrow t_1 \geq 8$

table 3: characteristic shear strength

faste-ner type	failure mode		range of validity
	hole bearing (including tilting)	failure of net section	
screws	$F_b = \alpha\, t\, d_n\, f_u$ where: - for $t_1 = t$: $\alpha = 3.2\sqrt{\frac{t}{d_n}}$ but $\alpha \leq 2.1$ - for $t_1 \geq 2.5t$: $\alpha = 2.1$ - for $1 < t_1/t < 2.5$: linear interpolation	$F_n = A_n\, f_u$	$e_1 \geq 3d_n$ $e_2 \geq 3d_n$ $u_2 \geq 3d_n$ $u_1 \geq 1.5d_n$ $3.0 \leq d_n \leq 8.0$

table 3 (continuation): characteristic shear strength

faste-ner type	failure mode		range of validity
	Pull through pull over	Pull out	
Screws	$F_p = t\, d_w\, f_u$, or $F_p = 0.5\, t\, d_w\, f_u$ for repeated load	$F_o = 0.65\, t_1\, d_n\, f_u$	$0.5 < t < 1.5$ mm $t_1 > 0.9$ mm
Cartrid-ge fired pins	$F_p = t\, d_w\, f_u$, or $F_p = 0.5\, t\, d_w\, f_u$	Characteristic pull out strength larger then F_p	$0.5 < t < 1.5$ mm $t_1 > 6$ mm

table 4: characteristic tension strength

4. REFERENCES

[1] Eurocode No. 3, part 1: "Common Unified Rules for Steel Structures", Commission of the European Communities, Edited draft, July 1990.

[2] Annex A of Eurocode No. 3: "Cold Formed Steel Sheeting and Members", Commission of the European Communities and ECCS, draft March 89 and draft Jan. 1991.

[3] Bryan, E.R.; Sedlacek, G.; Tomà, A.W., Weynand, K.: "Evaluation of test results on connections in thin walled sheeting and members in order to obtain strength functions and suitable model factors", Background Document A.01 to Annex A of Eurocode No. 3:, Institute of Steel Construction, RWTH Aachen, 1990.

[4] European Convention for Constructional Steelwork (ECCS): European Recommendations for Steel Construction: "Mechanical Fasteners for use in Steel Sheeting and Sections", ECCS Committee TC7, TWG 7.2, Brussels, June 1983.

[5] Stark, J.W.B.; Tomà, A.W.: Connections in thin-walled structures, reprint from "Development in thin-walled structures-1" by Rhodes, J. and Walker, A.C., Applied Science Publishers Ltd

[6] Institut zur Förderung zum Bauen mit Bauelementen aus Stahlblech e.V. (IFBS) - Richtlinien für die Prüfung von mechanischen Verbindungsmitteln für Stahlprofilblech-Konstruktionen, Teil A I, November 1972/2.

WELDED HOLLOW SECTION CONNECTIONS

UNDER PREDOMINANTLY STATIC LOADING

Dipak Dutta

ABSTRACT

The complexity of the load transfer due to the non-uniform stiffness of the intersection in the hollow section joints make them behave quite differently than the joints of open sections. Numerous research programmes had to be absolved in many parts of the world in order to develop the design strength formulae and recommendations. This paper gives an up-to-date state-of-the-art for the design of uniplanar and multiplanar hollow section joints in lattice structures in Europe.

1. GENERAL

Welded joints are mostly applied in lattice girder constructions and that is why emphasis is placed in this area. Structural hollow sections, circular and rectangular, are particularly suited for framed structures as a solution formed by connecting one hollow section to another directly without the addition of gusset plates, stifeners or other encumbrances. Wellknown is the fact that the design of the connections in welded latticed structures of structural hollow sections requires not only the knowledge about proper welding but also special insight into the connection behaviour mainly dependent on the connection configuration governed by the geometrical parameters, such as diameter ratio d_i/d_o or width ratio b_i/b_o, thickness ratio t_o/t_i, chord diameter or width to thickness ratio d_o/t_o or b_o/t_o, gap g between bracing toes, overlap ov of bracings and angle of inclination θ_i between chord and bracing axis. In order to secure the structural integrity of a hollow section connection, it is of vital importance that the dimensions of the constructional members as well as the configuration of the connection result in adequate deformation and rotation capacity. It was necessary to carry out extensive experimental investigations besides theoretical analysis to come to proper understanding of the solution. Over the last twenty-five years research programmes have been performed in many countries to come to the design formulae and constructional rules for hollow section structures using the results obtained by these research works. Besides the researches done by many universities and research institutes all over the world, special mention has to be made of the sponsorship of CIDECT (Comité International pour le Developpement et l'Etude de la Construction Tubulaire - International Committee for the Development and Study of Tubular Structures), an

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association of major hollow steel section manufacturers, which has devoted considerable resources to the investigation of virtually all aspects of the hollow section design including static and fatigue strength of joints, buckling behaviour of empty and concrete-filled columns, aerodynamic properties, corrosion resistance and workshop fabrication. The international nature of CIDECT has enabled to have access to relevant material from worldwide sources and get direct contact with many of the world's leading experts. This has facilitated to formulate design rules and formulae for hollow section constructions, which are accepted in the most countries of the world. However, an internationally agreed standard has not yet been created.

2. STATE-OF-THE-ART

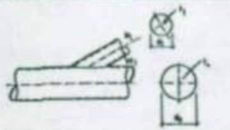
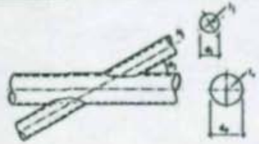
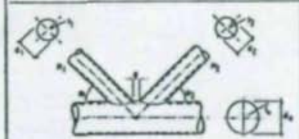
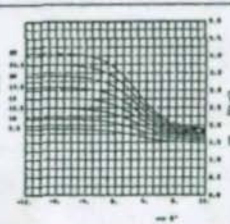
2.1 Design of uniplanar hollow section joints

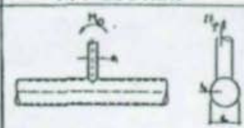
Lattice structures are usually designed assuming that the joints are pinjointed. When designing HSS trusses, the paramount rule the engineer must follow is to select the members and to detail the joints simultaneously.

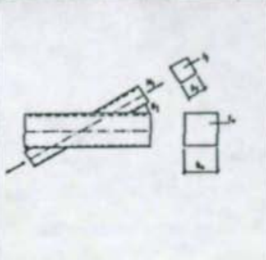
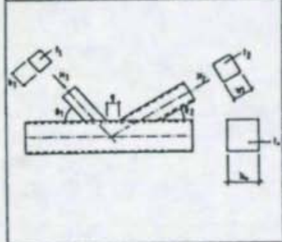
The joint strengths can be calculated basing on the semi-empirical formulae derived from the experimental investigations together with theoretical analysis. The development of these design recommendations are based largely on CIDECT/ECSC work (Wardenier 1982) (Giddings and Wardenier 1986) (IIW-XVE, 1989) (Sedlacek et al 1990) as far as RHS joints and SHS to I-section joints are concerned and largely upon Japanese works (Kurobane et al, 1980), (Kurobane, 1981) as far as circular hollow section joints are concerned. Although the joint strength formulae given in these publications are of the same form manifesting the effects of the geometrical parameters, yield strength, member dimensions and preloads in the chord, they are slightly different from one another showing the different stages of development. Final design strength formulae, which are accepted currently after an extensive discussion and re-analysis by the experts, are given in (Sedlacek et al 1990) and (Wardenier et al 1991) (IIW-XVE, 1989).

The final design formulae for uniplanar circular and rectangular hollow section joints are shown in the Tables 1 and 2. They have been incorporated in the lates draft of Eurocode 3 (Sedlacek et al, 1990).

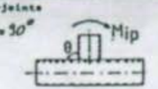
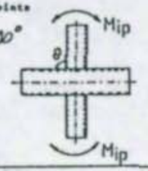
While designing an welded lattice structure of hollow sections, secondary bending moments due to the actual joint stiffness can be neglected for static design if the joints have sufficient rotation capacity. Within the

TYPE OF JOINT	DESIGN RESISTANCE (i=1,2)
T- and Y-joints	CHORD PLASTIFICATION
	$N_{1Rd} = \frac{f_{y0} \cdot t_0^2}{\sin\theta_1} \cdot (2{,}8 + 14{,}2 \cdot \beta^2) \cdot \gamma^{0,2} \cdot f(n') \cdot \frac{1{,}1}{\gamma_{Mj}}$
X-joints	CHORD PLASTIFICATION
	$N_{1Rd} = \frac{f_{y0} \cdot t_0^2}{\sin\theta_1} \cdot \left(\frac{5{,}2}{1 - 0{,}81 \cdot \beta}\right) \cdot f(n') \cdot \frac{1{,}1}{\gamma_{Mj}}$
K- and N- GAP or OVERLAP joints	CHORD PLASTIFICATION
	$N_{1Rd} = \frac{f_{y0} \cdot t_0^2}{\sin\theta_1} \cdot (1{,}8 + 10{,}2 \cdot \beta) \cdot f(\gamma, g') \cdot f(n') \cdot \frac{1{,}1}{\gamma_{Mj}}$ $N_{2Rd} = N_{1Rd} \cdot \frac{\sin\theta_1}{\sin\theta_2}$
T-, Y- and X-joints; K-, N- and KT joints with a gap if: $d_1 \le d_o - 2t_o$	PUNCHING SHEAR $N_{1Rd} = \frac{\pi \cdot f_{y0}}{\sqrt{3}} \cdot d_1 \cdot t_o \cdot \frac{1 + \sin\theta_1}{2 \cdot \sin^2\theta_1} \cdot \frac{1{,}1}{\gamma_{Mj}}$
FUNCTIONS	
	$f(n') = 1{,}0$ for $n' \ge 0$ (tension) $f(n') = 1 + 0{,}3 \cdot n' - 0{,}3 \cdot n'^2$ for $n' < 0$ (compression) $\le 1{,}0$ $f(\gamma, g') = \gamma^{0,2} \cdot \left(1 + \left(\frac{0{,}024 \cdot \gamma^{1,2}}{\exp(0{,}5 \cdot g' - 1{,}33) + 1}\right)\right)$

TYPE OF JOINT	DESIGN RESISTANCE (i=1,2)
T-, Y- AND X-JOINTS	
	$M_{ip1Rd} = 4{,}85 \cdot f_{y0} \cdot t_o^2 \cdot \gamma^{0,5} \cdot \beta \cdot d_1 \cdot \frac{f(n')}{\sin\theta_1} \cdot \frac{1{,}1}{\gamma_{Mj}}$
T-, Y-, X-, K- AND N-JOINTS	
	$M_{op1Rd} = f_{y0} \cdot t_o^2 \cdot d_1 \cdot \frac{2{,}7}{1 - 0{,}81 \cdot \beta} \cdot \frac{f(n')}{\sin\theta_1} \cdot \frac{1{,}1}{\gamma_{Mj}}$
GENERAL: Punching shear check for: $d_i \le d_o - 2 \cdot t_o$	$M_{ip1Rd} \le \frac{f_{y0}}{\sqrt{3}} \cdot t_o \cdot d_1^2 \cdot \frac{1 + 3 \cdot \sin\theta_1}{4 \cdot \sin^2\theta_1} \cdot \frac{1{,}1}{\gamma_{Mj}}$ $M_{op1Rd} \le \frac{f_{y0}}{\sqrt{3}} \cdot t_o \cdot d_1^2 \cdot \frac{3 + \sin\theta_1}{4 \cdot \sin\theta_1} \cdot \frac{1{,}1}{\gamma_{Mj}}$
$0{,}2 \le \frac{d_i}{d_o} \le 1{,}0$	Table 1 Design resistance of welded joints of circular hollow sections and range of validity
$\frac{d_i}{2t_i} \le 25$	
$\gamma \le 25$ $\gamma \le 20$ (X - joints)	
$O_v \ge 25\%$ $g \ge t_1 + t_2$	

TYPE OF JOINT	DESIGN RESISTANCE (i=1,2)	
T-, Y- and X- joints	$\beta \le 0{,}85$ CHORD FACE YIELDING	
	$N_{iRd} = \frac{f_{yo}\cdot t_o^2}{(1-\beta)\cdot\sin\theta_i}\cdot(\frac{2\eta}{\sin\theta_i} + 4(1-\beta)^{0,5})\cdot f(n)\cdot\frac{1,1}{\gamma_{M1}}$	
	$\beta = 1{,}0$ CHORD SIDE WALL FAILURE 1)	$0{,}85 \le \beta \le 1$
	$N_{iRd} = \frac{f_k\cdot t_o}{\sin\theta_i}\cdot(\frac{2h_i}{\sin\theta_i} + 10t_o)\cdot\frac{1,1}{\gamma_{M1}}$	linear interpolation of chord face yielding and chord side wall criteria
	$\beta > 0{,}85$ EFFECTIVE WIDTH	
	$N_{iRd} = f_{yi}\cdot t_i\cdot(2h_i - 4t_i + 2b_e)\cdot\frac{1,1}{\gamma_{M1}}$	
	$0{,}85 \le \beta \le 1 - 1/\gamma$ PUNCHING SHEAR	
	$N_{iRd} = \frac{f_{yo}\cdot t_o}{\sqrt{3}\cdot\sin\theta_i}\cdot(\frac{2h_i}{\sin\theta_i} + 2b_{ep})\cdot\frac{1,1}{\gamma_{M1}}$	
K- and N- GAP joints	CHORD FACE YIELDING	
	$N_{iRd} = \frac{8{,}9\cdot f_{yo}\cdot t_o^2}{\sin\theta_i}\cdot(\frac{b_1+b_2+h_1+h_2}{4\cdot b_o})\cdot\gamma^{0,5}\cdot f(n)\cdot\frac{1,1}{\gamma_{M1}}$	
	CHORD SHEAR	
	$N_{iRd} = \frac{f_{yo}\cdot A_v}{\sqrt{3}\cdot\sin\theta_i}\cdot\frac{1,1}{\gamma_{M1}}$ $N_{oRd} = [(A_o - A_v)\cdot f_{yo} + A_v f_{yo}(1 - (\frac{V_{Sd}}{V_{plRd}})^2)^{0,5}]\cdot\frac{1,1}{\gamma_{M1}}$	
	EFFECTIVE WIDTH	
	$N_{iRd} = f_{yi}\cdot t_i\cdot(2h_i - 4t_i + b_i + b_e)\cdot\frac{1,1}{\gamma_{M1}}$	
	$\beta \le 1 - 1/\gamma$ PUNCHING SHEAR	
	$N_{iRd} = \frac{f_{yo}\cdot t_o}{\sqrt{3}\cdot\sin\theta_i}\cdot(\frac{2h_i}{\sin\theta_i} + b_i + b_{ep})\cdot\frac{1,1}{\gamma_{M1}}$	
K- and N-OVERLAP joints	SIMILAR TO JOINTS OF SQUARE HOLLOW SECTIONS (TABLE K.8.1	
CIRCULAR BRACES	additional check on chord shear if $h_o/b_o < 1{,}0$	

FUNCTIONS		
tension: $f_k = f_{yo}$ compression: $f_k = f_{kn}$ (T-,Y-joints) f_{kn} according to buckling curve a	$f_k = 0{,}8\sin\theta_i\cdot f_{kn}$ (X-joints) with: $\lambda = 3{,}46(h_o/t_o - 2)(1/\sin\theta_i)^{0,5}$	
$f(n) = 1{,}0$ $n \ge 0$ (tension) $f(n) = 1{,}3 - \frac{0{,}4}{\beta}\cdot\lvert n\rvert$ $n < 0$ (compression) $\le 1{,}0$	$A_v = (2h_o + \alpha b_o)\cdot t_o$ $V_{plRd} = f_{yo}A_v/\sqrt{3}$	$\alpha = (\frac{1}{1 + \frac{4g^2}{3t_o^2}})^{0,5}$ for a circular brace section: $\alpha = 0$
$b_e = \frac{10}{b_o/t_o}\cdot\frac{f_{yo}\cdot t_o}{f_{yi}\cdot t_i}\cdot b_i \le b_i$	$b_{ep} = \frac{10}{b_o/t_o}\cdot b_i \le b_i$	$b_{e(ov)} = \frac{10}{b_j/t_j}\cdot\frac{f_{yj}\cdot t_j}{f_{yi}\cdot t_i}\cdot b_i \le b_i$
1) For X-joints with angles $\theta < 90°$ the chord side walls must be checked for shear		

TYPE OF JOINT	DESIGN RESISTANCE (i=1,2)	
T-joints $\theta \approx 30°$ 	$\beta \le 0{,}85$	CHORD FACE YIELDING
	$M_{ipiRd} = f_{yo}\cdot t_o^2\cdot h_i\cdot(\frac{1}{2\eta} + \frac{2}{\sqrt{1-\beta}} + \frac{\eta}{1-\beta})\cdot f(n)\cdot\frac{1,1}{\gamma_{M1}}$	
X-joints $\theta \approx 30°$ 	$0{,}85 \le \beta \le 1{,}0$	WEB CRIPPLING
	$M_{ipiRd} = 0{,}5\cdot f_{yk}\cdot t_o\cdot(h_i + 5t_o)^2\cdot f(n)\cdot\frac{1,1}{\gamma_{M1}}$ f_{yk}: $f_{yk} = f_{yo}$ for T-joints ; $f_{yk} = 0{,}8f_{yo}$ for X-joints	
	$0{,}85 \le \beta \le 1{,}0$	EFFECTIVE WIDTH
	$M_{ipiRd} \le 1{,}5\cdot f_{yi}\cdot b_e\cdot t_i\cdot h_i\cdot\frac{1,1}{\gamma_{M1}}$	
T-joints $\theta \approx 30°$ 		WEB LOCAL BUCKLING
	$M_{opiRd} = f_k\cdot t_o\cdot(h_i + 5t_o)\cdot b_i\cdot f(n)\cdot\frac{1,1}{\gamma_{M1}}$	
		EFFECTIVE WIDTH
	$M_{opiRd} \le f_{yi}\cdot(W_{pi} - 0{,}5\cdot(1 - \frac{b_e}{b_i})^2\cdot b_i^2\cdot t_i)\cdot\frac{1,1}{\gamma_{M1}}$	

TYPE OF JOINT	JOINT PARAMETERS (i = 1 or 2, j = overlapped brace)					
	b_i/b_o h_i/b_o	$b_i/t_i, h_i/t_i, d_i/t_i$ compres.	tension	h_i/b_i	b_o/t_o h_o/t_o	gap/overlap b_i/b_j t_i/t_j
T, Y, X	≥ 0.25	$\le 1.25\sqrt{\frac{E}{f_{yi}}}$	≤ 35	$0.5 \le \frac{h_i}{b_i} \le 2$	≤ 35	
K, N GAP	$\ge 0.1 + 0.01\frac{b_o}{t_o}$ $\beta \ge 0.35$	≤ 35			≤ 35	$0.5(1-\beta) \le \frac{g}{b_o} \le 1.5(1-\beta)$ $g \ge t_1 + t_2$
K, N OVERLAP	≥ 0.25	$\le 1.1\sqrt{\frac{E}{f_{yi}}}$			≤ 40	$25\% \le Ov \le 100\%$ $\frac{t_i}{t_j} \le 1.0$, $\frac{b_i}{b_j} \ge 0.75$
CIRCULAR BRACE	$0.4 \le \frac{d_i}{b_o} \le 0.8$	$\le 1.5\sqrt{\frac{E}{f_{yi}}}$	≤ 50	limitations as above for $d_i = b_i$		

Table 2 Design resistance of welded joints of rectangular, square or circular bracings and a rectangular or square hollow section chord and range of validity.

geometric limits of validity shown in the tables this will be the case.

Hollow section trusses have noding eccentricities, (Fig. 1), which are sometimes necessary for the ease of fabrication. As the investigations demonstrate, the resulting bending moments can be neglected for joint design in the case $-0.55 \leq \dfrac{e}{d_o \text{ or } b_o} \leq 0.25$ and for chord members loaded in tension. Chord members loaded in compression, however have always to be checked for the bending effect distributing the bending moment due to noding eccentricity to the two chord members at the joint based on the stiffness of each (Fig. 2).

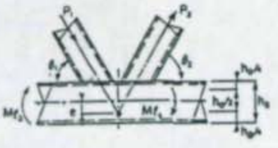
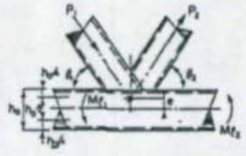


Fig. 1 Noding eccentricities



$M_{TOTAL} = e(P_1 \cos\theta_1 + P_2 \cos\theta_2)$

$Mf_1 = M_{TOTAL}\left(\dfrac{I_1/\ell_1}{I_1/\ell_1 + I_2/\ell_2}\right)$

$Mf_2 = M_{TOTAL}\left(\dfrac{I_2/\ell_2}{I_1/\ell_1 + I_2/\ell_2}\right)$

I = Moment of Inertia

Fig. 2 Moment distribution

Extensive research has been done to determine the effective buckling lengths of members of welded hollow section trusses (Mouty, 1980). A current evaluation of all test results (Rondal 1988, 1989) led to the formulae shown in the Table 3. This recommendation has also been incorporated in the latest Eurocode 3 draft (Sedlacek et al, 1990).

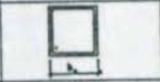
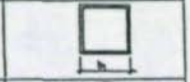
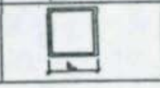
ℓ/L	chord member	bracing member
$2.20 \cdot [d_1^2/(L \cdot d_o)]^{0,25}$		
$2.35 \cdot [d_1^2/(L \cdot b_o)]^{0,25}$		
$2.30 \cdot [b_1^2/(L \cdot b_o)]^{0,25}$		
ℓ - buckling length L - system length d_1 - bracing diameter d_o - chord diameter b_1 - bracing width b_o - chord width		

Table 3 Calculation formulae for in and out of plane buckling lengths of bracing members

As regards the interaction between the axial loading and bending moments the investigations have shown that in-plane bending is less severe than out-of-plane bending. Eurocode 3 draft proposes the following lower bound interaction function:

$$\frac{N_i}{N_i^*} + \left(\frac{M_{ip}}{M_{ip}^*}\right)^2 + \frac{M_{op}}{M_{op}^*} \leq 1.0$$

N_i, M_{ip} and M_{op} are the loads acting

N^*_i, N^*_{ip} and N^*_{op} are design strengths.

2.2 Design of multiplanar hollow section joints

Multiplanar joints are frequently used in tubular structures, especially as triangular or quadrangular girders.
Investigations carried out in this sector until now are supplemented by further tests. RHS joints for triangular girders have been tested (Fig. 3) and theoretically analysed in the framework of a CIDECT research programme (Bauer and Redwood, 1988). A software programme for the joint design determining the joint strength is also available. (Makino et al, 1984) tested CHS KK-connections (Fig. 4) for triangular girders to determine the ultimate load bearing capacity of the said joints. (Paul et al, 1989) investigated into the static behaviour and strength of CHS multiplanar X-joints.

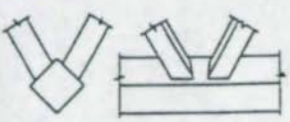


Fig. 3 Triangular RHS truss connection

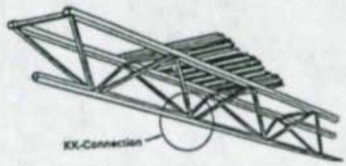


Fig. 4 Triangular girder with KK-connections

Based on the evidence obtained from the above mentioned programmes, design recommendations for multiplanar joints using the formulae for the strength of the uniplanar joints with the correction factors are given in Table 4. They have been incorporated in the Eurocode 3 draft.

Sponsored by the European Community and CIDECT two further research projects dealing with investigation of multiplanar joint behaviour are currently running:

1. Study of the behaviour under static loads of welded triangular and rectangular lattice girders made with circular hollow section, CIDECT programme 5AS (Liége University, Belgium). See Fig. 5.

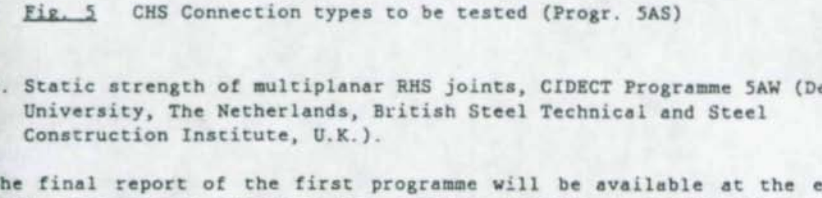


Fig. 5 CHS Connection types to be tested (Progr. 5AS)

2. Static strength of multiplanar RHS joints, CIDECT Programme 5AW (Delft University, The Netherlands, British Steel Technical and Steel Construction Institute, U.K.).

The final report of the first programme will be available at the end of 1991. The second programme has started shortly. A modification of the design recommendation (Sedlacek et al, 1990) has been planned after the results of the research programmes are available.

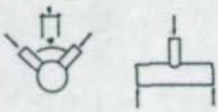
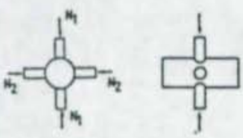
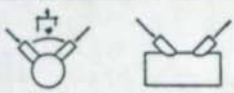
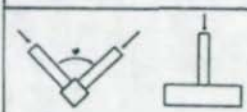
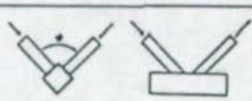
TYPE OF JOINT	CORRECTION FACTOR TO UNIPLANAR JOINTS
	$60° \leq \varphi \leq 90°$ 1.0
	$1 + 0.33 \cdot \frac{N_{2Sd}}{N_{1Sd}}$ take account of the sign of N_{2Sd} and N_{1Sd} $\lvert N_{2Sd} \rvert \leq \lvert N_{1Sd} \rvert$
	$60° \leq \varphi \leq 90°$ 0.9
	$60° \leq \varphi \leq 90°$ 0.9
	$60° \leq \varphi \leq 90°$ 0.9

Table 4 Correction factors for multiplanar joints

3. REFERENCES

1. Bauer, D. and Redwood, R.G.: Triangular truss joints using rectangular tubes. ASCE Journal of Structural Engineering, Vol. 114, No. 2, February 1988

2. Bauer, D. and Redwood, R.G.: A spreadsheet for HSS triangular truss joint design. CIDECT report no. 5W2-88/1-E, McGill University

3. Giddings, T.W. and Wardenier, J.: CIDECT Monograph No. 6, The strength and behaviour of statically loaded welded connections in structural hollow sections, 1986

4. IIW-XVE: Design recommendations for hollow section joints - predominantly statically loaded. 2nd Ed., IIW-Doc. XV-701-89, September 1989

5. Kurobane,Y. Makino,Y., Mitsui,Y.: Re-analysis of ultimate strength data for truss connections in curcular hollow sections. IIW-Doc. XV-461-80.

6. Kurobane, Y.: New developments and practices in tubular joint design (+ addendum). IIW-Doc. XV-488-81.

7. Makino,Y., Kurobane,Y., Ochi,K.: Ultimate capacity of tubular double K-joints. Proceedings IIW Conference "Welding of Tubular Structures", Boston, Pergamon Press, 1984.

8. Mouty, J.: CIDECT Monograph No. 4, Calcul des longueurs de flambement des éléments des poutres à treillis, 1980.

9. Paul, J.C., Valk, C.A.C. v/d, Wardenier, J.: The static strength of circular multiplanar X-joints. Proceedings of the International Symposium on Tubular Structures, Lappeenranta, Finland, September 1989, Elsevier, 1990.

10. Rondal, J.: Effective lengths of tubular lattice girder members - statistical tests. CIDECT report 3K-88/89, August 1988.

11. Rondal, J.: Addendum to the CIDECT report 3K-88/9, Report 3K-89/9, August, 1989.

12. Sedlacek,G., Wardenier,J., Dutta,D., Grotmann,D.: Annex K - Hollow section lattice girder connections to Eurocode No. 3 "Design of Steel Structures", Document 16.11.90, Eurocode 3 Editorial Group.

13. Wardenier, J.: Hollow Section Joints. Delft University Press, 1982.

14. Wardenier, J., Kurobane, Y., Packer, J.A., Dutta, D., Yeomans, N.: Design guide for circular hollow section (CHS) joints under predominantly static loading. CIDECT, TÜV Rheinland, 1991 (to be published).

STEEL BASEPLATE–FOOTING–SOIL BEHAVIOUR

Robert E. Melchers[1]

Abstract

An overview of recent experimental work and preliminary mathematical modelling for the moment–rotation behaviour of two and four bolt "pinned" and "fixed" bases respectively is presented. Baseplate thickness and bolt behaviour are the main factors governing behaviour of the steel baseplate–concrete footing interface. However considerable deformation can occur in the footing–soil interface and the relative moment–deformation behaviour of two interfaces is important in modelling columns as "pinned" or otherwise at foundation level. This matter is of interest in predicting (mainly) lateral deflections and in stability analysis of steel frames.

1. INTRODUCTION

The behaviour, as distinct from only the strength (e.g., Thambiratnam and Paramasivam, 1986; Penserini and Colson, 1990), of the bases of columns in rigid (and other) frames is mainly of interest for the accurate prediction of frame deflections under working loading conditions and to a lesser extent for buckling load predictions. Earlier work has shown that connection behaviour can have an influence on the magnitude of the predicted deflections, and, of course, it is well–known that the assumptions of a "fixed" or a "pinned" base condition can affect the economy of the structure for given ground conditions. Yet there is surprisingly little information about these matters available in the literature.

A review to 1987 of steel baseplate behaviour under large moments is available (Melchers, 1988). Since then details of tests and of modelling of nominally "pinned" bases for the rather large column sizes (460 UB and 310 UC and UC) employed in factory–type structures common in Australia have become available (Hon and Melchers, 1987; 1988) and further tests (unpublished) have been performed on both "pinned" and "clamped" bases for smaller column bases (200 UB 25 columns). Some aspects of the latter will be reported herein. The behaviour and strength of

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baseplates under mainly column axial forces will not be considered (but see, for example, Krishnamuthy and Thambiratnam, 1990).

2. TESTS ON BASEPLATE CONNECTIONS

Laboratory tests generally similar to those described earlier (e.g., Hon and Melchers, 1988) but for column stubs and baseplates manufactured from 200 UB 25 and welded with 6 mm fillet weld all round to various thickness of 300 × 200 mm steel baseplate, were conducted both for nominally "pinned" and "fixed" type connections. For the "pinned" connections only two holding down bolts were used, placed symmetrically about the column web and along the column minor axis. For the "clamped" connections, four bolts were used, off–set from the column minor axis (see Figure 1). As for the earlier tests (Hon and Melchers, 1988), the connection was packed, grouted and tightened as in field practice. The column stubs were loaded laterally by a double acting jack so as to simulate applied moment under load–reversal conditions. Axial load was not applied, since for the factory–type portal frames of interest, it is not significant for the column size.

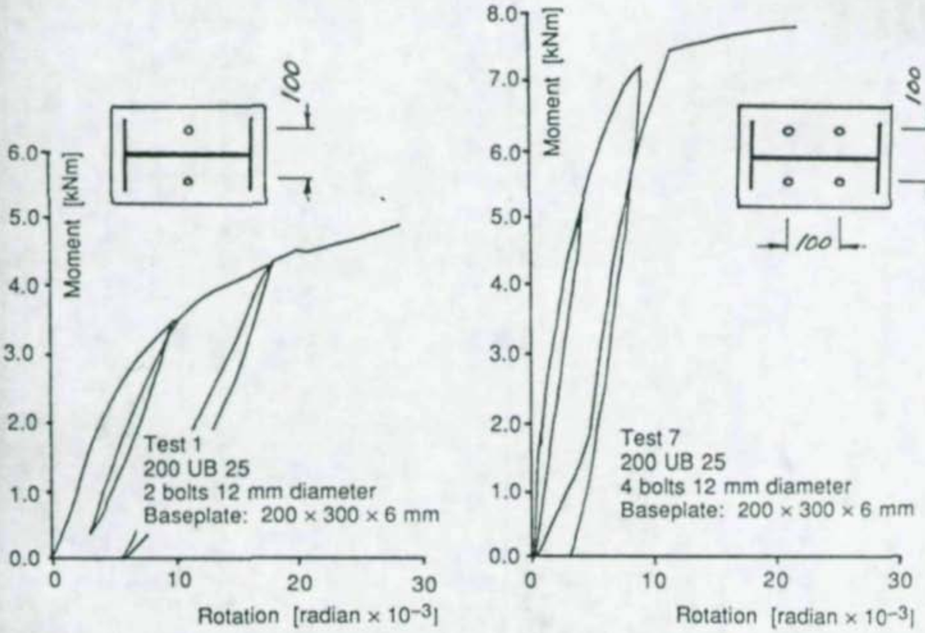

Figure 1. Column–Base Behaviour (Typical)
(a) "Pinned"; (b) "Fixed" Type

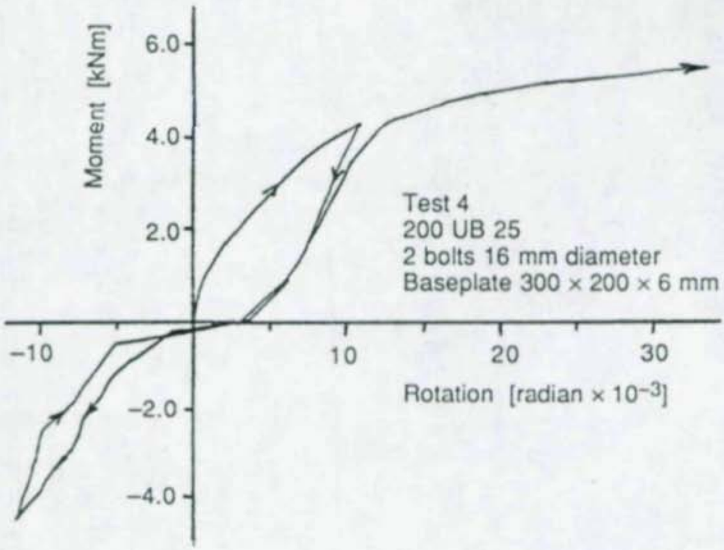

Figure 2. Column–Base Behaviour Under Load Reversal

The rotation of the column relative to the concrete foundation block was measured using inclinometers and digitally recorded together with applied loading. Typical plots of applied moment, measured at the level of the column baseplate, versus relative rotation are given in Figure 1. It is evident that the addition of two holding down bolts but otherwise using the same connection details has a considerable influence on the connection stiffness.

As reported earlier, the connection behaviour depends principally on baseplate thickness and to a lesser extent on bolt size. [Connection strength depends, of course, on material strength parameters as well]. Deformation of the baseplate was found to be the critical mode of behaviour except when exceptionally small diameter bolts were used relative to baseplate thickness. The modes of deformation observed for the baseplate generally corresponded to those observed earlier (e.g., Hon and Melchers, 1988).

Most of the tests were conducted under full moment reversal (see Figure 2). This revealed that even for nominally "fixed" type base connections the base can become essentially "pinned" once sufficient permanent deformation of the baseplate occurs. As seen in Figure 2, this required loading beyond about 0.5 the ultimate capacity – a load level which might well be exceeded in the base area of a column such as might occur in structures under exceptional loading conditions and with long–term stress re–distribution within the frame as a whole. This aspect requires further investigation if attempts are to be made to reduce baseplate thickness.

3. TESTS ON FOUNDATIONS

Response curves of the type given in Figures 1 and 2 do not represent adequately the behaviour of the support conditions for the column. Allowance should be made also for possible rotation deformation of the foundation. Unlike the baseplate, however, the design of the foundation is much less standardised and depends on site conditions and on the inter–relationship of the column with the floor and other details of the structure. For the factory–type portal frames of interest, typically a reinforced concrete floor slab is employed. This may be (i) laid over the foundation, abutting the columns but usually separated from them by a filler material, (ii) laid abutting the foundation block and level with it at the top and again usually separated, and (iii) laid at a lower level (more common in older structures).

The foundation block is generally constructed in reinforced concrete and of a size determined by the downward–acting axial force and by soil capacity, or more commonly in Australia, the need to use the block self–weight to hold down the frame against uplift wind forces. With these imponderables, it was decided to use a foundation block comparable in size to that observed on typical factory structures for comparable column size. It follows that the results obtained have only indicative value, in particular, in relation to the overall behaviour of the column–foundation assembly.

A foundation block 1100 mm long × 300 mm wide × 580 mm high was cast into a prepared foundation hole without formwork (except at the top) (see Figure 3). It was surmounted by and fixed to a steel column against which a double–acting jack could act at various levels and angles. This arrangement was selected to allow for the possibility of varying axial force with applied moment.

The rotation of the foundation block under increasing and decreasing jacking load was recorded for a number of different jacking arrangements. Details of this work will be presented elsewhere, suffice it to note here that a typical set of increasing moment–rotation curves for a given jacking arrangement were found to be as given in Figure 4. Also shown in this figure is the curve obtained to predict the deformation using a somewhat modified version of a mathematical model due to Xiong Jianguo et al., (1990).

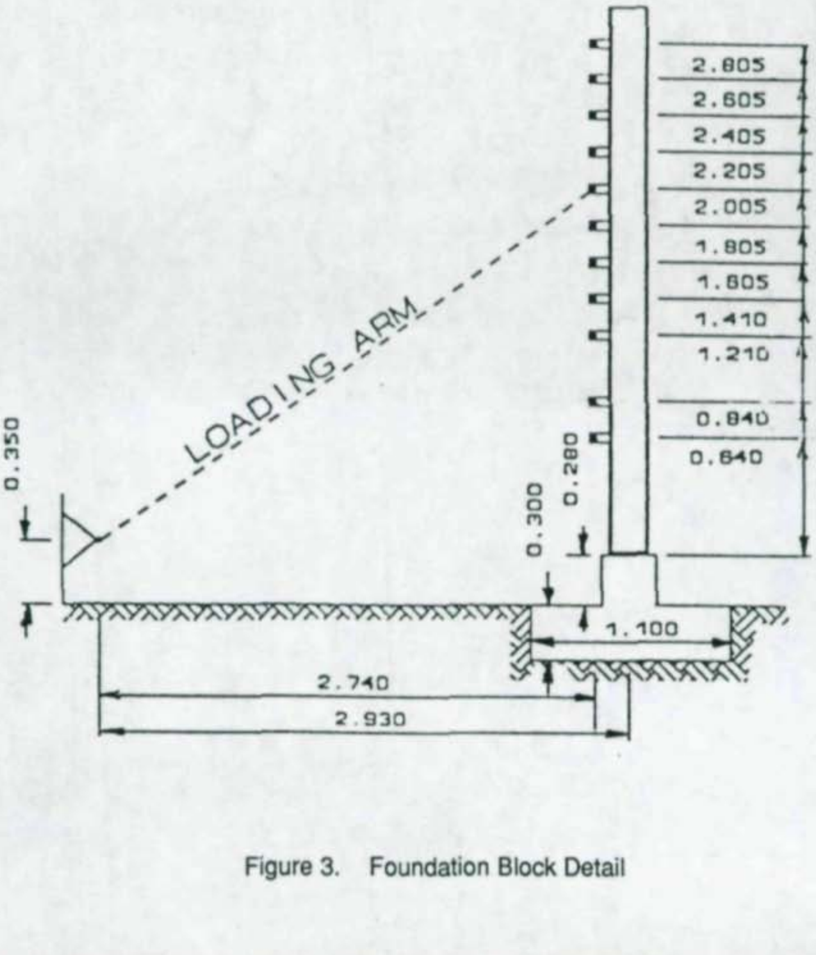

Figure 3. Foundation Block Detail

The moment–rotation curves were obtained by plotting the moment M_c at the baseplate level (i.e., the top of the foundation block) against the measured rotation. Thus the moment M_c is directly comparable to that used in the description of the moment–rotation curves at the steel baseplate (see Figure 1). However, it is clear from elementary considerations that the centre of rotation is approximately at the base of the foundation block for the particular experimental conditions encountered. This was confirmed by the analysis of the test results. It means that at the level of the steel baseplate (i.e., top of foundation block) there is both rotation and horizontal translation. It also means that the self–weight of the foundation block (and the steel column above) are important ingredients in the capacity of the foundation to sustain deformation.

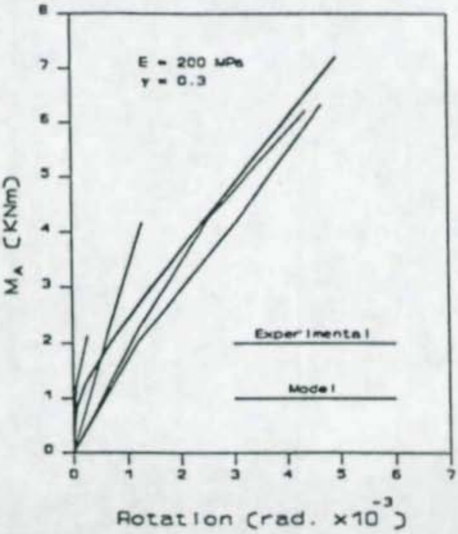

Figure 4. Typical Experimental and Model Moment–Rotation Curves (Positive Moment and Increasing Loading Only)

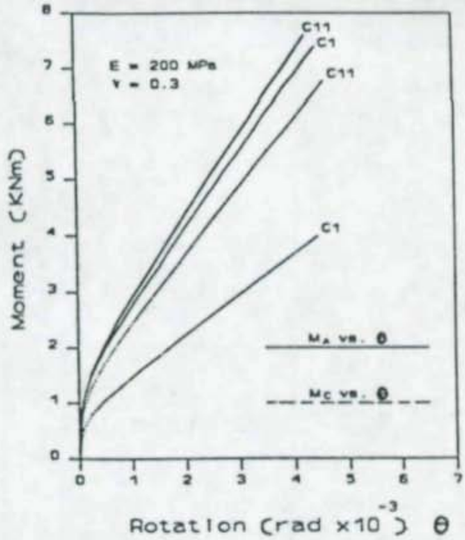

Figure 5. Moment–Rotation Curves for Moment Measured at Foundation Block Base

This is illustrated in Figure 5 in which all moment–rotation curves for the tests become almost coincident when the moment (M_A) is measured at the foundation base. Any remaining difference must be attributed to the effect of the induced vertical force due to jack inclination and to experimental error.

4. DISCUSSION

In comparing the relative importance of foundation rotation and baseplate effect, it is important to note the differences in accuracy of the two. The foundation rotation estimates have considerable uncertainty. Nevertheless, it is clear from comparing Figure 1 to Figure 4 that for a given applied moment the foundation–soil interface can contribute more than half the total rotation for nominally "pinned" bases, and considerably more for nominally "fixed" bases, even in the case of the very stiff soil used in the experiments. The difference would be larger for less stiff soils and for foundation blocks of lower self–weight.

It must be emphasised again that the investigation of the foundation block rotation in soil noted above is very preliminary. The modelling of the behaviour of the block in the soil is based on limited understanding (cf. Xiong Jianguo et al., (1990)) and further work, both experimental and analytical, will need to be undertaken. The present set of results indicates that such work is necessary for the complete modelling of column base behaviour.

5. CONCLUSION

Some current research work aimed at modelling the behaviour of the column–base–footing system has been outlined and an indication given of the likely relative importance of the two interfaces: column baseplate–footing and footing–soil. For factory–type portal frames of large span as commonly used in Australia, the footing–soil interface appears to make a substantial contribution to the rotation of the system under an applied moment. However, further research work is required before work can commence on design guides.

6. ACKNOWLEDGEMENTS

The author is indebted to W Jordan who analysed test results and carried out preliminary modelling, K H Fan who carried out the foundation soil tests, and N S Lai who carried out the column stub tests, all as part of their final year undergraduate work.

7. REFERENCES

HON, K.K. and MELCHERS, R.E., (1988), Experimental Behaviour of Steel Column Bases, J. Const. Steel Res., 9 ( ), 35–80.

HON, K.K. and MELCHERS, R.E., (1987), Moment–Rotation Curves for "Pinned" Column Bases, The Structural Engineer, 65B (3), 54–58.

KRISHNAMUTHY, N. and THAMBIRATNAM, D.P., (1990), Finite Element Analysis of Column Base Plates, Computers and Structures, 34 (2), 215–224.

MELCHERS, R.E., (1988), Modelling of Column–Base Behaviour, (In) Connections in Steel Structures, (Ed) Bjorhovde et al., Elsevier Applied Science, 150–157.

PENSERINI, P. and COLSON, A., (1990), Ultimate Limit Strength of Column–Base Connections, J. Cons. Steel Res., 14 (4), 301–320.

XIONG JIANGUO, GAO WEIJIAN, GONG ZHENPENG and DING ZHAOKUI, (1989), Overturning Resistance and Rotation of Foundation in Soil Under Lateral Loading, Computers and Geotechnics, 8 (3), 157–174.

THAMBIRATNAM, D.P. and PARAMASIVAM, P., (1986), Baseplates Under Axial Loads and Moments, J. Struct. Engg., ASCE, 112 (5), 1166–1181.

Technical Papers on

COMPOSITE CONNECTIONS

SEMI-CONTINUOUS COMPOSITE FRAMES IN EUROCODE 4

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Abstract

The approach adopted in EC4 for semi-continuous composite frames is explained. Difficulties were encountered in writing rules for frames with semi-rigid joints and research needs are identified. Rules given in EC3 for calculation of steel connections have been extended to include composite connections. Recent tests on composite connections are reported and compared with these rules. The tests indicate that careful detailed design of connections is necessary to provide sufficient rotation capacity for plastic global analysis.

1. COMPOSITE CONNECTIONS IN EUROCODE 4

1.1 Introduction

Part 1 of Eurocode 4 (1991) gives general rules for the design of composite steel and concrete structures, and detailed rules for buildings. It is to be used in conjunction with Part 1 of Eurocode 3 (1990). EC3 gives a classification system for beam-to-column connections, related to rotational stiffness and moment resistance; that applicable to braced frames is shown in Fig.1. This enables the type of connection required for different kinds of framing (simple, continuous, semi-continuous) to be specified, depending on the method of global analysis to be used. This is to ensure that assumptions made in analysis of the structure are in

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accordance with anticipated behaviour of the connections. As EC4 has to be consistent with EC3, it has been necessary to extend the classification of connections to composite construction.

1.2 Definition of Composite Connection

Design that relates only to steelwork components within composite structures is not treated by EC4. Hence in EC4 a composite connection is defined as one in which reinforcement is intended to contribute to the resistance. This will exclude connections such as that in Fig. 2, because concern over lack of ductility usually causes welded mesh to be omitted from the effective section. With the concrete cracked due to hogging bending and the profiled steel sheeting neglected, the connection reduces to one between steel sections and is therefore within the scope of EC3.

1.3 Classification of Composite Connections

To non-dimensionalize the classification limits, the properties of the connection are compared with those of the connected beam (Fig. 1). For composite beams, the design plastic resistance moments, $M_{p\ell.Rd}$, in sagging and hogging bending are generally different. Similarly, the flexural rigidity of the beam, EI_b, depends on whether the "cracked" or "uncracked" section is considered.

For classification by moment resistance, the appropriate value of $M_{p\ell.Rd}$ is that of the composite beam's cross-section adjacent to the connection. As in EC3, a connection is then classified as full strength or partial strength, depending on whether the resistance of the connection is greater than or less than $M_{p\ell.Rd}$.

In elastic global analysis for ultimate limit states, EC4 permits flexural stiffnesses of beams to be taken as the "uncracked" values E_aI_1 throughout the length of a beam, where E_a is the modulus of elasticity for structural steel and I_1 is the second moment of area of the equivalent steel section, assuming concrete in tension is uncracked. Alternatively, flexural stiffnesses are taken as the "cracked" values E_aI_2 over 15% of the span on each side of each internal support and as the uncracked values E_aI_1, elsewhere.

For classification of connections by rotational stiffness, it is desirable that calculation of E_aI_2 is not required when the designer has chosen uncracked global analysis. In classification, EC4 permits flexural rigidity of the beam to be either the cracked or uncracked value, consistent with the approach used in global analysis. As $E_aI_2 < E_aI_1$ it is more likely that a connection will be classified as rigid if the cracked rigidity is used; with this classification, the connection flexibility is ignored. This is appropriate as the cracked approach is the more accurate model of beam behaviour and therefore greater approximation can be tolerated in representation of the connection.

1.4 Semi-continuous Framing in EC4

Following earlier design recommendations (ECCS, 1981), EC4 permits moments given by elastic global analysis to be redistributed by reducing support moments such that equilibrium is maintained. Such redistribution away from supports accounts for yielding and cracking in regions subject to hogging bending. Limits to redistribution have been established by studies on continuous composite beams.

Semi-rigid joints could similarly be treated by analysing the structure as continuous and then redistributing moments. To do this, though, requires research to find appropriate degrees of redistribution, which would be dependent in part on the M-ϕ characteristics of connections. Even if the connection is rigid, the established degrees of redistribution may not be appropriate unless the connection is also full-strength. This is because yielding will occur in the connection rather than the member if the former is only partial-strength. Behaviour will be dependent again on the M-ϕ characteristic of the connection. Methods to predict such characteristics are so far not well enough established to justify inclusion in EC4. Rules for elastic global analysis with redistribution are therefore only given for situations in which the beam is either continuous over internal supports or is jointed by full-strength and rigid connections.

A rigid-plastic approach is an alternative to elastic global analysis. The important characteristics of the connection are now limited to the moment resistance and the rotation capacity, rather than the complete moment-rotation characteristic. Such analysis of semi-continuous framing is included within the contents of EC4. It is necessary to demonstrate that the connections have sufficient rotation capacity. No attempt has been made to quantify this, nor are detailed rules given for the calculation of moment resistance and available rotation capacity. Attention is drawn though to the possibility of making some use of the detailed rules in Annex J of EC3 for steel beam-to-column connections, supplemented by consideration of the yielding of slab reinforcement.

1.5 Conclusion

To use rigid-plastic global analysis for semi-continuous framing, it is necessary to determine the moment resistance of connections and to ensure that adequate rotation capacity is available. Section 2 below shows that the rules in EC3 can be readily extended to predict moment-resistance of composite beam-to-column connections. Adequate rotation capacity requires careful detailed design of the connection.

2. RECENT TESTS ON COMPOSITE BEAM-TO-COLUMN CONNECTIONS

2.1 Introduction

A series of tests are in progress at the University of Warwick, all involving end-plate connections. The first three tests, on major axis connections, are reported herein. Each test is on a symmetrical cruciform arrangement comprising a column stub and two connected beams. Steelwork details are shown in Fig. 3, including the arrangement of the profiled steel sheeting used to form the slabs. Reinforcement details are given in Fig. 4. Common details are as follows:

Structural Steelwork	Grade 43 (minimum yield strength 275 N/mm^2)
Bolts	20 mm diameter Grade 8.8
Deck	Precision Metal Forming CF46 0.9 mm thick
Slab	1100 mm wide x 120 mm deep overall
Concrete	Normal weight, designed as Grade 30 (cube strength)
Reinforcement	A142 mesh (0.142 mm^2/m) plus additional bars as given below
Shear connectors	19 mm stud x 100 mm long before welding

The variable parameters are the form of the steelwork connection and the additional slab reinforcement (of characteristic yield strength 460 N/mm^2). The choice of variables for Tests 1-3 is given in Table 1:

Table 1 - Summary of tests on composite connections

Test No.	End Plate	Reinforcement	Rebars only	Rebars and mesh
1	Flush	A142 + 8 no. T12	1.11%	1.31%
2	Extended	A142 + 8 no. T12	1.11%	1.31%
3	Flush	A142 + 4 no. T12	0.56%	0.75%

For comparison with recent fire tests (Lawson, 1990), compression stiffeners were provided to the web of the column section, thus eliminating any possibility of failure in the compression zone.

2.2 Test rig and Instrumentation

Loading was applied to each beam by an independent jack placed 1.4 m from the face of the column. Each jack acted on the slab through an arrangement of rollers and a knife-edge. The jacks reacted against a supporting rig attached to the laboratory floor. The base of the column was supported on a ball joint seated on a concrete footing. The base was thereby held in position but was free to rotate.

Some of the instrumentation is shown in Fig. 5. The rotation of the steel beam was measured by means of displacement transducers acting on a length of angle section attached to the upper flange of the beam. Rotation was measured relative to the vertical axis through the centroid of the column section. In addition, inclinometer readings were taken on the lower lip of a zed-shaped bracket, the upper lip being attached to the upper flange of the beam.

2.3 Testing Procedures

Actual dimensions of test specimens were measured and materials tests performed. The ductility of the reinforcing bars met the requirements of BS4449 (1988), the elongation at fracture being of the order of 17%.

In Tests 1 and 2, the specimen was loaded to 50% of the calculated ultimate resistance of the composite connection (defined below), in increments of 10 kN. The development of cracking in the slab was traced. The specimen was then unloaded, before being loaded to failure.

A similar procedure was followed in Test 3, except that two thirds of the design ultimate load was applied before the specimen was unloaded. Crack widths were also measured.

2.4 Test Observations

Fig. 6 shows the M-ϕ curve for the second stage of each test i.e. loading to failure. In each case, the behaviour shown is that for the connection on the side which failed. Moments are calculated at the face of the column. Moments at failure are given in the bottom row of Table 2, including 4 kNm for the self weight of the beam.

Test 1: Within the tension zone of the steelwork connection, significant deformation arose in the column flange and, to a lesser extent, the end plate. Failure occurred due to fracture of reinforcing bars and mesh in one composite beam, over half the width of the slab. The associated crack was immediately outside the column section. At failure, slight deformation associated with local buckling was visible in the lower flange of the beam. Negligible slip occurred at the steel beam - slab interface. Negligible deformation occurred in the column stiffener.

Test 2: Much less deformation occurred in the steelwork connection. The most noticeable deformation was to the column flanges, which were pulled apart relative to one another. Failure occurred by local buckling in the lower flange and in the lower part of the web of the steel beam.

Test 3: Behaviour was similar to Test 1 except that fracture of the reinforcement occurred at a much smaller rotation. The cracking pattern in the slab was more limited than in previous tests, being restricted to regions near the column.

2.5 Analysis of Results

In calculating theoretical values, the yield strength of structural steel, f_y, and reinforcement, f_{yr}, have been taken as measured values. Cross-sectional properties, e.g. area of reinforcement, A_r; breadth B and thickness T of the steel flange, have been taken as tabulated values.

The resistance moment of each steel connection was calculated using Annex J of EC3. The resistance moments of each composite connection were determined from the resistance of the tension zone of the steelwork connection, R_b (determined using

Annex J) and the resistance of the reinforcement, $R_r = f_{yr}A_r$. If the total resistance $(R_b + R_r)$ exceeded that of the lower steel flange, $R_f = BTf_y$, a depth of web adjacent to this flange was assumed to be stressed to yield. This depth was determined by the equilibrium and account was taken of this region when calculating the resistance moment. Calculations were made (i) excluding mesh, (ii) including mesh.

The negative resistance moments of the composite beam were calculated by plastic analysis of the section, neglecting sheeting and the tensile strength of concrete. Due to the position of the plastic neutral axis, the web was classified as Class 3, with one exception. Plastic analysis could still be employed, by using an effective section for the web in calculation. The exception was for Test 3, considering rebars only, which resulted in the section being in Class 2.

Table 2 Resistance Moments

Structural Component	Resistance Moment			Non-dimensional Resistance M_{hog}/M_{sag}		
	Test 1 (1)	Test 2 (2)	Test 3 (3)	Test 1 (4)	Test 2 (5)	Test 3 (6)
Steel beam	194	194	194	0.51	0.51	0.51
Steel connection	65.4	133	65.4	0.17	0.35	0.17
Composite beam:						
Rebars only	263	263	240	0.70	0.70	0.64
Rebars and mesh	273	273	258	0.72	0.72	0.68
Composite connection:						
Rebars only	227	266	151	0.60	0.71	0.40
Rebars and mesh	255	282	191	0.68	0.75	0.51
Test	262	291	179	0.69	0.77	0.47

The positive moment of resistance of the composite beam, assuming full shear connection, has been calculated as 377 kNm, based on Grade 30 concrete. Table 2 also presents the negative resistance moments in non-dimensional form, relative to the calculated positive resistance.

The lower of the resistances for the composite beam and the composite connection (both including the mesh) has been taken as the calculated resistance. This is compared with the experimental value in Column 1 of Table 3. In practice, mesh would be excluded from the design moment of resistance because of lack of ductility. Excluding mesh, *the lower calculated resistance is compared with the experimental* value in Column 2 of Table 3. The critical component (connection or beam) was correctly predicted in each case.

Table 3 Resistance Moment (Calculated/Test)

Test	Resistance Moment (including mesh) (1)	Resistance Moment (excluding mesh) (2)	Failure Mode (3)
1	0.97	0.87	Fracture
2	0.94	0.90	Local buckling
3	1.07	0.84	Fracture

2.6 Classification of Connections

The M-ϕ curves from the tests have been compared with the classification limits given in EC4. As explained in 1.3 above, these follow EC3 (Fig. 1), but with definitions of $M_{pl.Rd}$ and EI_b appropriate to composite beams. In the comparisons made by the authors, the cracked section was used to determine EI_b. The beam span L_b was taken as 7.5 m. Fig. 7 shows that all these connections are rigid.

2.7 Rotation Capacity

Two tests failed by fracture of the reinforcement, the last rotations recorded before failure being 36 x 10^{-3} rad. (Test 1) and 15.6 x 10^{-3} rad. (Test 3). In both cases, fracture resulted from the flexibility of the steelwork, particularly from the column flange. The connections utilised flush end plates and were partial-strength relative to the negative moment resistance of the beam.

A tentative design method (Lawson, *1990 (2)*), *based on earlier work on steel* structures (Lawson, 1988), has been proposed in which the minimum rotation capacity for plastic global analysis should be 25 x 10^{-3} or 30 x 10^{-3} radians, for structural steel of Grade 43 and Grade 50 respectively. This would exclude Test 3 (with approximately 0.5% reinforcement) but permit the use of the flush end plate connection which included approximately 1% reinforcement (Test 1).

Test 2 utilised an extended end plate, the connection being full-strength relative to the negative plastic moment of the effective section. As the section is in Class 2, plastic global analysis is not permitted because of susceptibility to local buckling.

2.8 Conclusion

The resistance moment of the composite connections has been calculated by extending the method given in Annex J of Eurocode 3. The experimental resistance moments were predicted to within -6% and +7%, including mesh within the effective section. This is not usual practice in design. If mesh is excluded, the calculated resistance is between 84% and 90% of the test result.

The flush end plate connections, as well as the extended end plate connection, were classified as rigid according to EC4.

To achieve sufficient rotation capacity in flush end plate connections to permit plastic global analysis, small amounts of additional reinforcement should be avoided or neglected. To define limits, it is proposed to consider the reinforcement relative to both the area of concrete and the resistance of steelwork components.

Acknowledgements

The drafting of Eurocode 4 has been undertaken by a panel comprising Prof. R.P. Johnson (Chairman), M. H. Mathieu, Prof. K-H. Roik, Prof. J.W.B. Stark and Dr. D. Anderson (Technical Secretary).

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References

1. BS4449 Specification for carbon steel bars for the reinforcement of concrete, British Standards Institution, 1988.
2. ECCS (European Convention for Constructional Steelwork), Composite Structures, The Construction Press, London, 1981.
3. Eurocode 3 Editorial Group, Eurocode 3 Design of Steel Structures: Part 1 General Rules and Rules for Buildings, prepared for the Commission of the European Communities, Edited draft, Issue 3, April 1990.
4. Eurocode 4 Editorial Group, Eurocode 4 Design of Composite Steel and Concrete Structures: Part 1 General Rules and Rules for Buildings, prepared for the Commission of the European Communities, Revised draft, Issue 2, Jan. 1991.
5. Lawson, R.M., A proposal for the design of steel frames with semi-rigid bolted connections, Steel Construction Today, 1988, 2, 109-116.
6. Lawson, R.M., Behaviour of steel beam-to-column connections in fire, Structural Engineer, 1990, 68, 263-271.
7. Lawson, R.M., Rotation capacity of composite connections, Private Communication from Steel Construction Institute, Ascot, England, Dec. 1990.

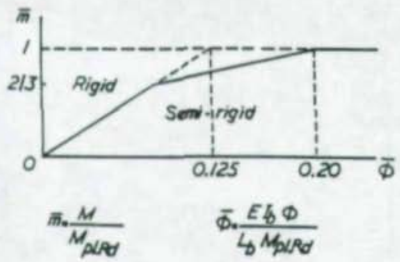

Fig. 1 Classification of connections

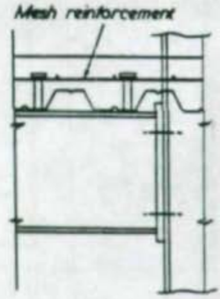

Fig. 2 Non-reinforced effective section

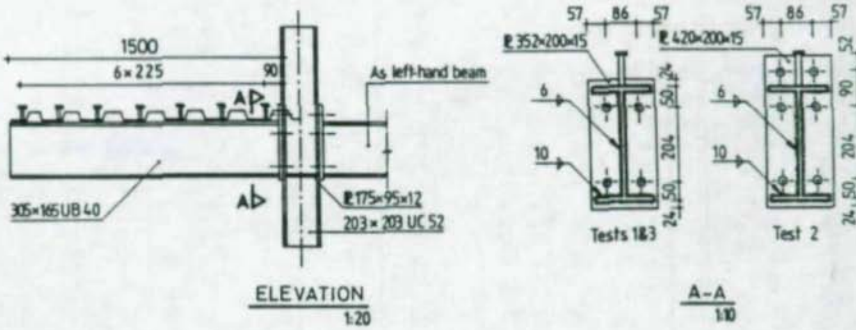

Fig. 3 Steelwork details

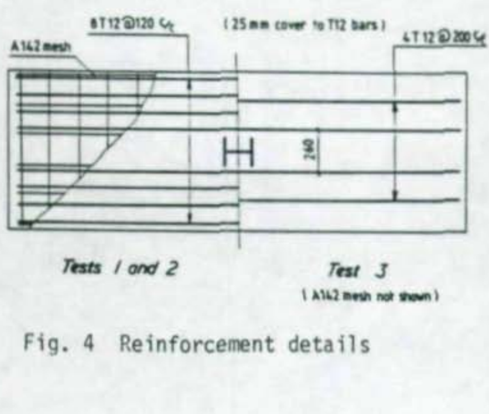

Fig. 4 Reinforcement details

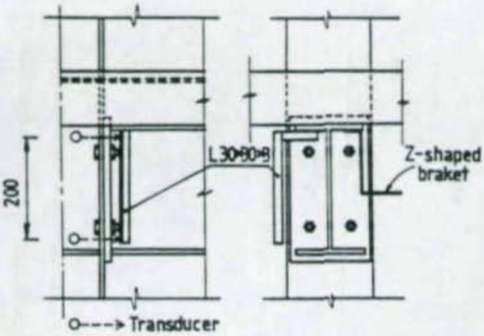

Fig. 5 Measurement of rotation

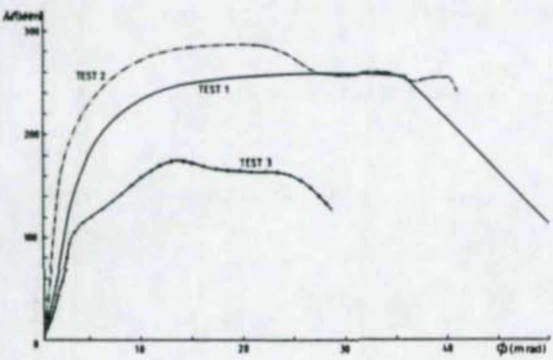

Fig. 6 Moment-rotation curves

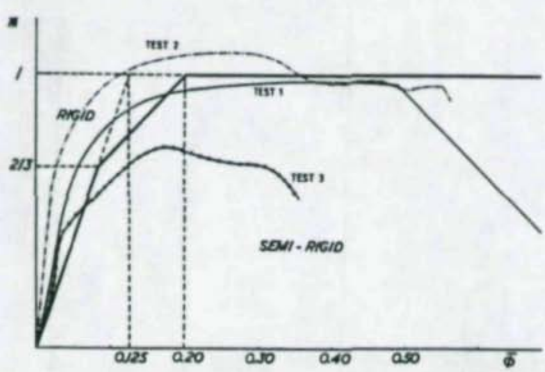

Fig. 7 Classification of test results

PARAMETRIC STUDY OF COMPOSITE FRAMES

Roberto T. Leon[1]

Gabriel P. Forcier[2]

Abstract

A series of 27 three-bay, fixed-base frames utilizing semi-rigid composite connections were designed to satisfy the LRFD specification utilizing a procedure proposed by Leon and Ammerman. The frames were then analyzed using an existing analysis program modified to take into account the non-prismatic nature of the girders and the effect of initial dead load stresses. In this paper the behavior of these frames is compared to the behavior of geometrically similar rigid frames. The results show that for the majority of the frames examined the use of semi-rigid composite connections combined with composite girders is a valid structural alternative.

1. INTRODUCTION

Unbraced frames provide much freedom for the subdivision of space in a building, but generally result in the use of fully restrained (FR) or rigid (AISC, 1986) connections to provide moment resistance. The main disadvantages of FR connections are economical since they require considerable fabrication and a high standard of fit-up. A simple alternative to fully rigid connections for frames up to ten stories is the use of composite semi-rigid connections. These are semi-rigid or partially restrained (PR) connections that are simple to detail and to construct (Leon, 1990).

In this study the performance of a whole class of unbraced composite frames will be evaluated to determine the validity of a simplified version of the design approach proposed by Leon and Ammerman (Leon and Ammerman, 1989a, 1989b).
The simplified composite frame design process is as follows: (1) design the frame as being rigid non-composite, (2) use the same column sizes, (3) replace the steel girders

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by steel girders capable of resisting the factored construction loads without yielding, (4) detail the composite girder to carry all the factored live loads, (5) provide enough shear connectors for 100% composite action, and (6) replace the rigid connections by semi-rigid composite connections.

The main objective of this study was to demonstrate that composite semi-rigid connections can be used in medium sized frames without increasing the member sizes relative to a rigid design. In fact, main girder sizes can probably be scaled back if careful attention to serviceability criteria is given.

2. DESIGN OF SEMI-RIGID COMPOSITE FRAMES

In earlier studies Leon (Leon and Ammerman, 1989a) has pointed out some of the difficulties encountered when devising a design procedure for flexibly-connected frames and their connections. Amongst the most important were:

(1) The determination of a moment-rotation curve for a particular type of connection cannot be achieved without a large combined experimental-analytical program because of the many geometrical parameters and material properties that are of interest. Databases containing data from past tests should be used with caution. For example, there are several ways to measure moment and rotation, and it is not always possible to convert them to the same baseline.

(2) In general the data generated in experiments refers primarily to the strength of the connections, and the initial stiffnesses and their degradation are seldom well-reported. Initial stiffness is a crucial item for semi-rigid frames where drift considerations will likely govern design.

(3) Even a careful experimental study may not clarify all relevant variables. For example, most tests are carried out with fixed values of moment-to-shear at the connection, but some recent studies indicate that for weak steel connections the interaction between the two should be taken into account to develop a yield surface.

(4) For composite semi-rigid connections the moment-rotation curves are not symmetrical, and exhibit non-linear behavior relatively early in the load history (at loads less than 25% of ultimate).

(5) If cyclic loads (such as wind and earthquake) are involved in the design, the degradation of the connection behavior needs to be accounted for in design. Little is known of the degradation characteristics of semi-rigid composite connections except for some very specific types.

(6) The typical construction sequence in the U.S. calls for unshored construction. This requires that the dead load of the wet concrete and other construction loads

be carried by the steel beam alone when the end connections are relatively weak. This requires a historical analysis, where the end restraints for the girders and beams change as the construction progresses.

(7) In the analysis of frames it is typical to assume prismatic sections. When looking at frames with semi-rigid composite connections, the section properties for the girders change from positive to negative moment regions.

(8) The contribution of panel zone yielding to the overall lateral deformation can be very large, particularly if the panel zones begin to yield. Thus panel zone deformations should be included in drift calculations unless their contribution is shown to be small.

(9) Serviceability criteria calculations need to account for both time-dependent effects such as creep and shrinkage of the concrete and effects of end restraint.

In the parametric study to be described next, the authors addressed items (1) through (7) directly, item (8) indirectly, and ignored the time effects for item (9) since the design was based on ultimate strength and drift.

3. PARAMETRIC STUDY

Twenty-seven fixed-base, three bay frames having 4, 6, and 8 stories, story heights of 3.65 m, 4.27 m, and 4.87 m, and bay width (W) to story height (H) ratios of 2, 2.25, and 2.5 were studied. They were meant to replicate to some degree the frames studied by Ackryod (Ackroyd, 1981) for the Type 2 design of unbraced frames under the old allowable stress design specification.

Each of the individual frames was analyzed for response to lateral loads using an available second order program (Leon and Ammerman, 1989b) that takes into account the non-linear partial rigidity of the connections. Two modifications were implemented into this computer program. The first one permits the analysis of the frames taking into account the non-prismatic nature of continuous composite floors. The second accounts for the unshored construction practice, where the weight of the fresh concrete has to be borne by the steel alone when the connections are essentially pinned.

The selection of member sizes for the composite frames was largely based on the sizes of similar rigid frames whose members were selected by a commercially available structural optimization software (SODA, 1989) in a way that ensures that the rigid frames respect the LRFD specification. The columns for composite frames were identical to those in the rigid frames having the same geometric proportions and same applied loads. The differences were in the connections and girders. In the case of the semi-rigid frames the fully composite girders of least weight were selected. For rigid frames the steel girders were assumed to have no interaction with the floor slabs. For composite frames it was assumed that composite connections can be detailed at the outside columns with no extensive overhang on the outside of the building.

The frames were all assumed to be spaced 9.1 m apart, a span selected based on the building's intended office occupancy. This spacing determined the number of beams that frame into the girders. The selection of the transverse beam spacing was based on the allowable effective width, typically governed by the span divided by four. All floors consisted of a 127 mm lightweight concrete slab placed on 50 mm steel decking. The concrete had a f'c of 24.1 Mpa, all structural steel had a yield stress of 248 MPa, and steel reinforcement had a yield stress of 413 MPa.

The total permanent load including concrete, steel, ceilings, ducts, etc. was evaluated as 2.87 kPa. The uniformly distributed floor live load was chosen to be 4.79 kPa. Using the ANSI code (ANSI, 1982) allowable girder live load reduction factor, the applied live loads reduced to values close to 2.87 kPa for all the frames. The roof live load was chosen as 2.39 kPa. This includes any snow or rain load and no girder live load reduction was permitted. The wind and earthquake loads were also specified in conformity with the ANSI code. The wind loads selected were for wind speeds of 145 kph and suburban (Type B) exposures. The earthquake loads were chosen for low to moderate (Zone 2) intensities and the foundation conditions were chosen as being poor.

The imposed serviceability constraints were: for girder deflection due to live loads the girder span divided by 360, for frame sway due to wind loads the frame height divided by 400, and for frame sway due to earthquake loads the frame height divided by 200. The principal design constraints for the rigid frame were that for each individual frame geometric configuration the floor girder shapes are forced to be the same and that any selected column shape must be continuous through at least two stories.

The connections for the design of the unbraced semi-rigid frames were all detailed for an ultimate strength approximately equal to half of the plastic capacity of the steel girders. The behavior of the connections is predicted by using the parametric equations proposed by Kulkarni (Leon and Ammerman, 1989b), an improvement over similar ones proposed by Lin and Ammerman (Ammerman and Leon, 1989a). Since the program only permits the analysis of structures having symmetrical connections, only one curve was used for input. Furthermore the procedures to account for connection stiffness degradation during cyclic loading need to have the connection curves defined in a piecewise linear fashion. The analysis was carried out using the negative moment-rotation proposed by Kulkarni (Ammerman and Leon, 1989b) and reduced to a tri-linear curve to reduce computation time.

A total of 162 frame analyses were performed. Since lateral sway was the main concern with semi-rigid composite frames, only load cases involving lateral loads were considered. The lateral load cases were: (1) constant vertical load plus increasing lateral load till collapse, and (2) increasing both vertical and lateral loads till collapse. For each load case the 27 different frame configurations were analyzed with (1) rigid connections and steel girders, (2) composite connections and prismatic composite girders, (3) composite connections and non-prismatic composite girders. The two types of composite analysis (2 and 3) were carried out to see if there were major differences between considering the composite girders as prismatic as Ammerman and Leon (Ammerman and Leon, 1989b) proposed and doing the analysis with girder stiffnesses that are closer to reality.

4. RESULTS

The computed load-deflection curves for a typical rigid, composite-prismatic and composite non-prismatic analyses appear in Figure 1. In the figure the normalized lateral load (applied lateral load divided by the total ANSI wind load) is plotted as a function of the normalized drift (calculated top story drift divided by the allowable value of H/400). The main conclusion is that accounting for the non-prismatic nature of the composite girders does not make a significant difference for this type of loading. The only difference is that using non-prismatic sections results in slightly higher collapse loads. The composite girders for this frame have relatively small loads and appear to be acting mostly as stiff links transferring moment from column stack to column stack.

To summarize the information from all the frames figures showing (1) the drift at a lateral load factor of 1, and (2) the collapse load factors plotted versus the frame aspect ratio (B/H) were developed. Figures 2 and 3 are typical results for the frames with 4.27 m story heights. Figure 2 and similar plots indicate that (1) for story heights of 3.66 m, rigid and composite frames have the same drift at a load factor of 1 except for frame aspect ratios smaller than 0.8, (2) as the story height increases the difference begins to occur at higher values of B/H, and (3) only 3 composite frame configurations were unserviceable ($H = 4.87$ m and $B/H < 1$).

The lateral load factors at collapse (Figure 3 and similar) showed that (1) for story heights of 12 feet composite frames can resist practically the same lateral load intensity as the rigid frames, (2) all the frames collapsed at lateral load factors greater than 2.25 which can permit the assumption that all the frames would have passed the standard LRFD ultimate lateral load case which is $1.2D + 0.5L + 0.5L_r + 1.3W$.

The previous results come from load case 1 where the gravity loads were held constant as the lateral ones increased till collapse. For the usual case (load case 2, Figure 4) where all the loads are increased proportionally, it was assumed that if a frame could resist a load factor of 1.3 then its behavior was satisfactory. The plots for the collapse global factor led to the conclusions that (1) the composite frames almost all attained a higher collapse load factor than the rigid frames, and (2) only one frame did not achieve a load factor of 1.3, in both composite and rigid configurations.

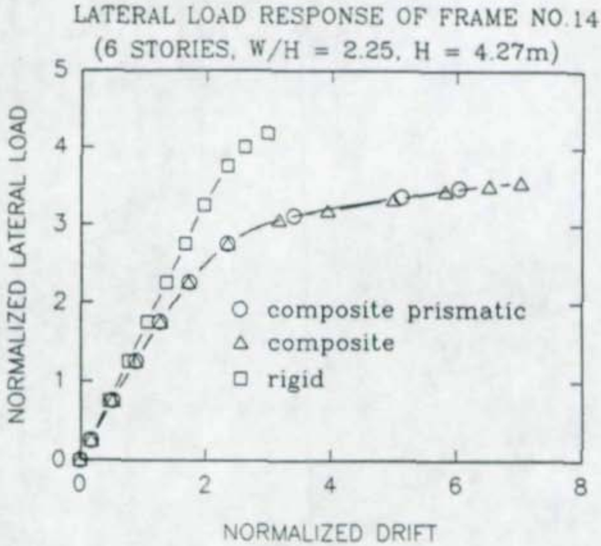

Figure 1.- Typical *lateral load response (Load Case 1).*

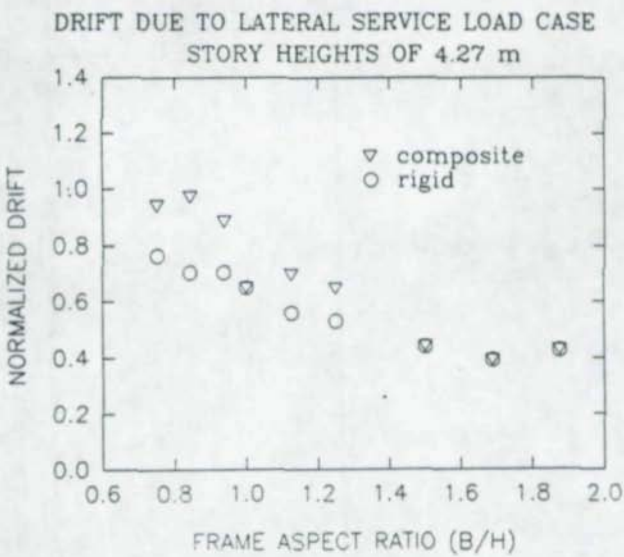

Figure 2.- Drifts at service loads for frames with H = 4.27 m.

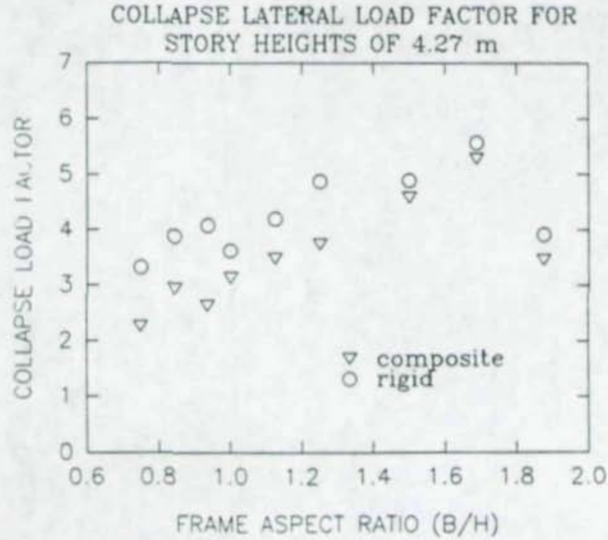

Figure 3.- Collapse load factors for frames with H = 4.27 m.

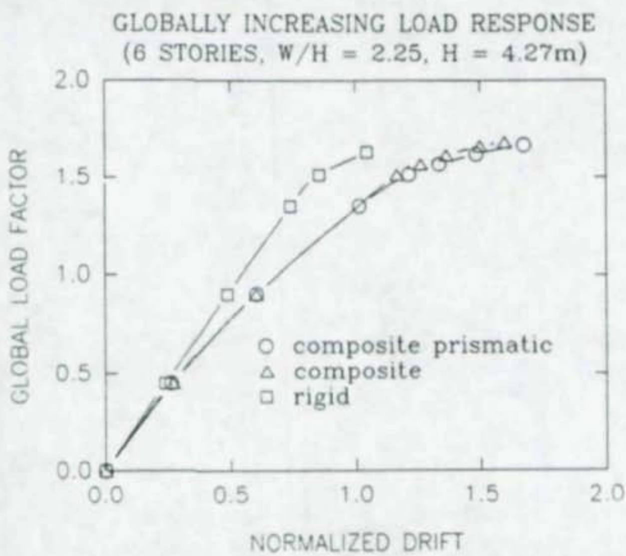

Figure 4.- Typical lateral load response (Load Case 2)

6. CONCLUSIONS

The simplified composite frame design process proposed here proved itself to be reliable for the majority of the three bay frames that were studied. Only frames with extreme geometries, particularly those with both short bays and large story heights, showed marginal behavior. Limited studies with two-bay and four-bay frames led to similar conclusions.

ACKNOWLEDGEMENTS

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REFERENCES

Ackroyd, M.H., 1985, Design of Flexibly-Connected Steel Building Frames, Final Report, Project No. 333, American Iron and Steel Institute, Washington, D.C.

AISC, 1986, Manual of Steel Construction: Load and Resistance Factor Design, American Institute of Steel Construction, First Edition, Chicago, Ill. 1986.

ANSI A58.1-1982, Minimum Design Loads for Buildings and Other Structures, American National Standards Institute, New York, NY.

Leon, R.T, and Ammerman, D.J., 1989a, Semi-Rigid Composite Connections for Gravity Loads, Engineering Journal, AISC, Vol. 27, No. 1, First Quarter 1989, pp. 1-11.

Leon, R.T, and Ammerman, D.J., 1989b, Unbraced Frames with Semi-Rigid Composite Connections, Engineering Journal, AISC, Vol. 27, No. 1, First Quarter 1989, pp. 12-21.

Leon, R.T, 1990, Semi-Rigid Composite Construction, Journal of Constructional Steel Research, Vol. 15, No. 1 & 2, pp 99-120.

SODA, 1989, Structural Optimization Design and Analysis Software for Structural Engineering, Waterloo Engineering Software, Waterloo, Ontario.

SLAB AND BEAM LOAD INTRODUCTION IN COMPOSITE COLUMNS

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Michel Crisinel[1]

Abstract

The objective of the current research programme is the development of a full understanding of the force transfer between steel and concrete at points of load application in composite columns. Tests have indicated the influence of parameters on the force transfer and the mechanism has been identified by analyses. A comparison between tests and analyses has enabled the establishment of a numerical model which can be used to carry out a parametric study.

1. INTRODUCTION

Horizontal elements in a building are often connected to the steel part of a composite column in a way similar to structural steel frames, causing forces from slabs and beams to be introduced directly to the steel section. Column loads applied to the steel section may produce yielding before the concrete part of the column can participate, and therefore needs to be strengthened locally to prevent yielding, which can have an influence on the design and rigidity of the beam-column connection. Force transfer mechanisms are to be relied upon for transferring force between the steel and concrete, these can be either the bond between steel and concrete or mechanical transfer mechanisms.

Current design codes allow a certain amount of force to be transferred by bond between the steel and concrete in specifying an allowable bond stress. The shear resistance on the interface is given as two different values for concrete encased and concrete filled steel sections respectively (Eurocode, 1990) (DIN, 1984) (SG + CUR-VB, 1983). Several parameters could however play a role in determining the shear resistance between the steel and concrete, and quantitative shear resistance values that take into account the configuration of the composite section do not exist. The design of connections to composite columns requires these quantitative data regarding the mechanism of shear transfer between the steel and concrete (Furlong, 1980, 1988) (Griffis, 1986). By taking into account the cross sectional configuration in

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determining the shear resistance on the interface, savings can be made on the number of mechanical shear transfer mechanisms that are needed to transfer the force.

2. PURPOSE OF STUDY

Research is conducted on composite construction at the Swiss Federal Institute of Technology, with the emphasis on composite slabs, columns and their connections. In a study of the force transfer between steel and concrete at points of load application in composite columns, the influence of parameters on the force transfer is being identified. The purpose of the study is to develop guidelines for calculating the force that can be transferred between the steel and concrete in a composite column, whereby the influence of parameters can be taken into account.

The study is based on an experimental and theoretical approach. For the theoretical part, use is made of the finite element method to study the stress distribution in a composite column at points of load application. The method allows the normal interface pressures between the steel and concrete to be calculated, and the influence of parameters on the interface pressure can therefore be studied. Tests are being carried out to identify and investigate the influence of parameters on the force transfer and to evaluate and improve the finite element model. The study is currently restricted to axially loaded concrete encased steel I-sections.

3. PRELIMINARY FINITE ELEMENT ANALYSES

A series of preliminary analyses were carried out with the finite element program ADINA to investigate the stress distribution in a column at the point of application of a vertical load on the steel profile. The purpose of the preliminary analyses using a simple finite element model, was to understand the behaviour of a column section. The analyses helped to define the boundary conditions between the steel and concrete for a more complete analysis and served as a basis for proposals concerning tests to be carried out on short column specimens. They also helped to define the areas in which concrete cracking could be expected.

4. TEST SERIES

After the preliminary analyses, a series of tests was proposed by which the transfer of force in a composite column could be studied, and which was used to verify the results of a finite element model (Wium and Lebet, 1990b). The tests were carried out on 5 short composite columns with the force being applied to the steel part of the column. The steel and concrete were both supported at the bottom of the column. Strain gauges were placed along the length of the column on the steel profile to supply information for calculating the force which was transferred to the concrete. Strains were also measured on the horizontal binding reinforcement. The thickness of concrete cover and the spacing of hoop reinforcing were varied as shown in TABLE 1. Tests were also carried out on 6 push-out specimens and results were compared with those for the short columns. Two groups of three push-out specimens each had similar concrete cover and spacing of hoop reinforcing to short columns 2 and 4 respectively. The dimensions of short column and push-out specimens are shown in FIGURE 1.

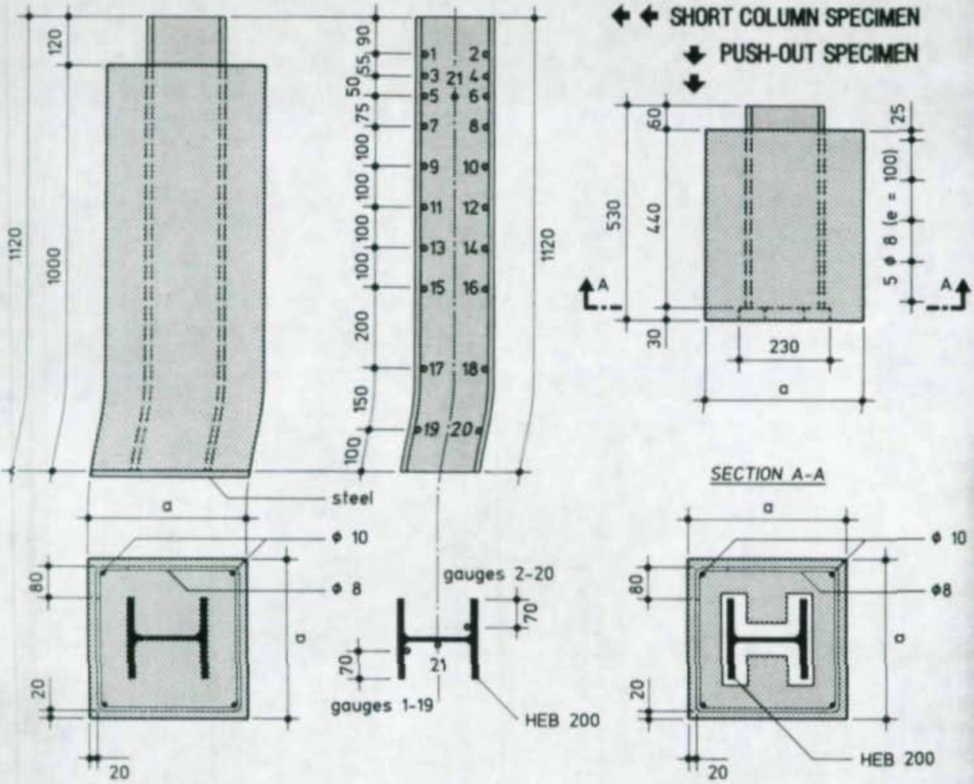

FIGURE 1 Short column and push-out test specimens.

TABLE 1
Details and numbering of specimens for tests on short columns.

Spacing of hoop reinforcement [mm]	Concrete cover [mm]		
	50	75	100
50	1		
100	2	3	4
200	5		

TABLE 2
Percentage of force transferred over first 600 mm at an applied load of 1500 kN as related to column 1.

Spacing of hoop reinforcement [mm]	Concrete cover [mm]		
	50	75	100
50	100		
100	78	89	125
200	61		

1.1 Test Results

The results of the tests on short columns 2 and 4 are shown in FIGURE 2 where the bond stress along the length of the column is presented. The bond stress is defined as the force transferred from steel to concete divided by the steel surface area between the top of the column and the level under consideration. Bond slip (breaking of the chemical bond), and longitudinal cracking in the flange cover concrete, appeared between 1000 kN (0.37 N_{pl}) and 1500 kN (0.55 N_{pl}). The bond stress at the top of the column specimens is therefore a function of the normal interface pressure between the steel and concrete at loads of 1500 kN (0.55 N_{pl}) and higher. N_{pl} is the theoretical force required to plastify the steel section at a nominal yield strength of 355 N/mm^2. The influence of mechanical interaction is small.

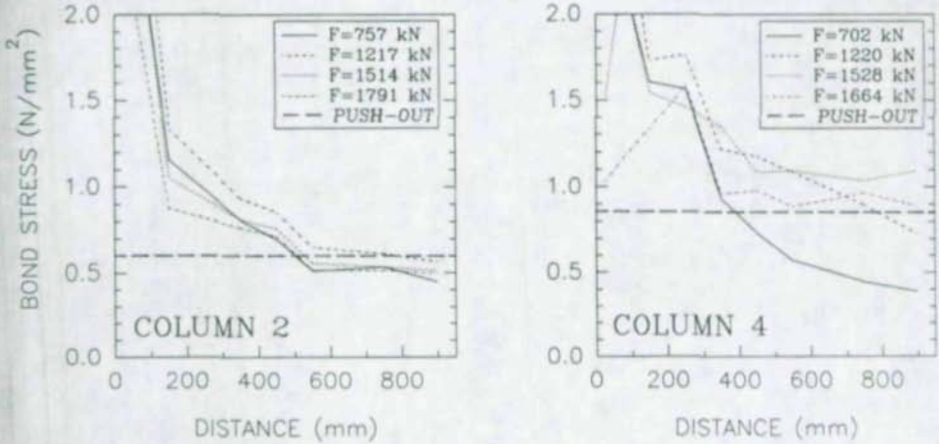

FIGURE 2 Bond stress values for columns 2 and 4.

There is not a big difference, before bond slip, in the force transfer between the different short column specimens. The two parameters under investigation, namely the concrete cover thickness and the spacing of horizontal hoop reinforcement, therefore do not play any significant role before bond slip. After bond slip, the transfer of force is influenced by the thickness of concrete cover and the spacing of horizontal hoop reinforcement at a distance further than 300 mm from the top of the column. The percentage of force transferred in the columns, in relation to column 1, is presented in TABLE 2.

The shear resistance on the interface given by some design codes (Eurocode, 1990) (DIN, 1984) (SG + CUR-VB, 1983), is 0.6 N/mm^2 and is to be taken over a length of twice the outside dimension of the composite section. Considering applied forces after bond slip, it can be seen from FIGURE 2 that the average bond stress for column 2 at a load of 1500 kN (0.55 N_{pl}) and higher, is less than 0.6 N/mm^2 over 500 mm. This was also the case for column 5.

The average bond stresses after bond slip for the push-out tests, at 5 mm slip, are 0.6 N/mm^2 and 0.86 N/mm^2 for the two cases respectively. These two values are also shown in FIGURE 2 for comparison with the short column tests. FIGURE 2 shows that the bond stresses from the push-out tests are less than the bond stresses in the column tests at high loads over a transfer length of 440 mm (length of push-out specimen). This may be due to an increased interface pressure in the column tests, which is a result of the higher dilation of the steel webs at this load level. However, at 1500 kN the applied column loads are 3.4 and 5 times higher than the push-out test loads, and the differences in the bond stresses are not nearly as big. It is possible that the increased dilatation of the profile in the column tests caused separation between the steel and concrete on the inside of the profile.

5. NUMERICAL ANALYSES

After the tests had been carried out, a final finite element model was established by comparing it with the results of the tests. The program ADINA was used for all analyses.

All concrete and steel elements were assumed to have a linear material behaviour. Some concrete elements were defined along the entire length of the column with a rigidity 100x less than the other concrete elements. These are 'soft' elements defined to model the position of concrete cracking. Both concrete and steel consisted of 8-noded three-dimensional brick elements. Horizontal and vertical reinforcing bars were defined by truss elements connected rigidly to the nodes of concrete elements.

Uni-directional horizontal spring elements were placed on the inside of the profile between the steel and concrete to allow separation between the two materials. Horizontal spring elements were also placed between the steel and concrete on the outside of the steel flange and at the flange tip. The horizontal spring elements were defined with high axial rigidities.

In the absence of a continuous bond element in the ADINA program, vertical spring elements were placed between the steel and concrete along the entire perimeter of the steel profile. The vertical spring elements had an elastic perfectly-plastic material behaviour. The stiffness values used for the vertical spring elements were obtained by earlier tests carried out on steel plates embedded in concrete (Wium and Lebet, 1990a). The maximum stress value which was used for the vertical spring elements corresponds to a normal interface pressure of 0.6 N/mm^2.

The analyses were carried out by using 20 load increments to apply the load necessary to plastify the steel section. The dimensions of the columns and the spacing of secondary hoop reinforcement along the length of the column were varied to be in agreement with the parameters of columns tested.

6. COMPARISON OF TEST AND NUMERICAL RESULTS

The cummulative force, normal to the surface of the steel profile, between the top of the column and a specific level on the column, can be calculated by using the results of the finite element analysis. After the chemical bond has been broken, this cummulative normal force at a specific level should differ from the force transferred between the top of the column and that level, by a factor which is equal to the coefficient of friction μ. The only difference can be due to the part played by the mechanical interaction.

The results of the analyses at applied loads of approximately 1500 kN (0.55 N_{pl}) and 1750 kN (0.64 N_{pl}) are presented in FIGURE 3 for columns 2 and 4 respectively. The force per unit length, normal to the steel surface, multiplied by a factor μ of 0.8 (Wium and Lebet, 1991), is shown against the distance along the length of the column. The figure also presents the normal force per unit length as calculated by assuming no concrete cracking in the flange concrete, and one can clearly see the important influence of the concrete cracking on the results. Presented in the same figure are the forces transferred per unit length as measured during the tests. There is a slightly better agreement between the test results and the finite element calculations at higher loads.

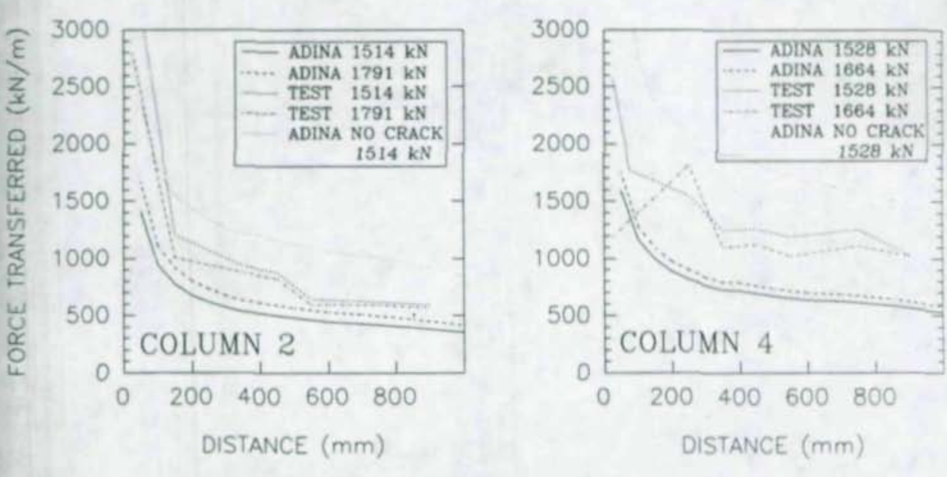

FIGURE 3 Force transferred per metre compared to calculated normal interface force per metre for columns 2 and 4.

The *ADINA* calculated stresses in the reinforcement also compare well with measured stresses at loads after concrete cracking has occurred. The test and analysis results for column 2 are shown in FIGURE 4.

During all analyses it was found that separation occurs between the steel profile and concrete on the inside of the profile, due to the dilatation of the steel. It may however be possible that force is being transferred in this area during the tests, and this can explain the difference between the test and analyses results. A future series of tests will be carried out to determine the role played by the inside of the steel profile during the force transfer.

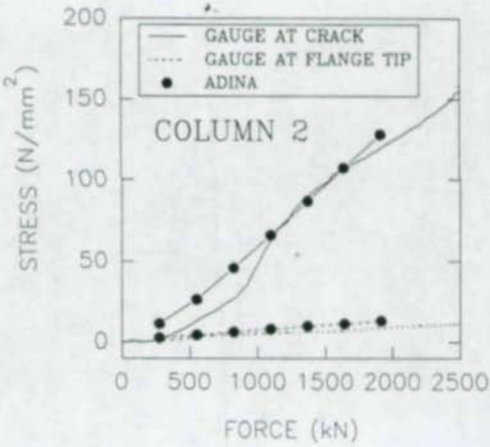

FIGURE 4 Stresses on horizontal hoop reinforcement.

7. CONCLUSIONS

Tests on short column specimens have shown that results of push-out tests, for studying the load application on composite columns, do not always represent the actual behaviour. Shear resistance values presented in design codes need to be defined in terms of different cross sectional parameters and applied loads. The tests have also identified the influence of the thickness of concrete cover and spacing of hoop reinforcement on the force transfer characteristics. Design of composite connections must therefore include load introduction criteria.

Finite element analyses have identified that the force transfer between the steel and concrete can be mainly attributed to the dilatation of the steel section under load application. The greatest part of the force is therefore transferred by bond at the outside of the steel flange and the flange tip.

The results of the finite element analyses have been compared with tests results on short column specimens. A reasonable agreement was found between test and finite element results at loads higher than those at which the chemical bond is broken. The results of the analyses have shown the important influence which concrete cracking has on the normal pressure between the steel and concrete and therefore on the transfer of force between the two materials. Stresses in the horizontal reinforcing showed a good agreement between tests and analyses after bond slip.

Further tests will be carried out to identify the importance of the inside of the steel profile in the force transfer, and the influence of parameters on force transfer will be studied with the finite element model.

References

DIN 18806 : Verbundkonstruktionen, Verbundstützen. Berlin, Deutches Institut für Normung, 1984.

EUROCODE No. 4 : Design of Composite Steel and Concrete Structures, Part 1. Revised draft, Issue 1, Oct. 1990. Commission of the European Communities, Brussels, 1990.

FURLONG, R.W. Strength Implications of Connection Details to Composite Columns. In : Developments in Composite and Mixed Construction. Proceedings of the U.S.A.-Japan Seminar on Composite Structures and Mixted Structural Systems. Edited by B. Kato and L.-W. Lu. Tokyo, Gihodo Shuppan, 1980, pp. 212-215.

FURLONG, R.W. *Steel-Concrete Composite Columns - II. In : Steel-Concrete* Composite Structures. Edited by R.Narayanan. London, Elsevier Applied Science, 1988, pp. 195-220.

GRIFFIS, L.G. Some Design Considerations for Composite-frame Structures. Engineering Journal, AISC, Chicago, Vol. 23, No. 2, 1986, pp.59-64.

SG + CUR-VB : Staal-Beton Kolommen. Theorie en richtlijnen. Rotterdam, Staalbouwkundige Genootschap, 1983 (rapport 3).

WIUM, J.A., LEBET, J-P. Push-out tests on embedded steel plates. Lausanne, Ecole polytechnique fédérale, 1990a (Publication ICOM 240).

WIUM, J.A., LEBET, J-P. Tests for the application of forces on short composite columns. Lausanne, Ecole polytechnique fédérale, 1990b (Publication ICOM 241).

WIUM, J.A., LEBET, J-P. Load introduction in composite columns: Numerical approach and comparison with test results. Lausanne, Ecole polytechnique fédérale, 1991 (Publication ICOM in preparation).

ENERGY-BASED PREDICTION FOR
COMPOSITE JOINTS MODELING

František Wald

Abstract

The composite steel-concrete structures have a widespread use in building frames. For frame computer modeling the joint behavior is as important as member behavior . The paper deals with the simple design response of the joint model as a concentrated rotational spring and the effect of joint energy dissipation. It is shown that the use of the initial stiffness, ultimate carrying capacity and shape parameter is well suited for the prediction of the spring characteristics. The comparison with experiments are presented to demonstrate the possibility of shape parameter establishing.

1. INTRODUCTION

The frequent use of the composite steel-concrete construction in steel building frames makes the applying of semi-rigid composite connections an economically attractive alternative for redistributing moments and resisting moderate horizontal forces. The major obstacle to its use was the lack of experimental verification and analytical models of the moment-rotation behavior.

The research projects in the 70's and mid 80's were aimed at extensively reviewing and discussing the main nonlinear factors influencing the composite beam behavior. Their results were summarized by ZANDONINI (1989). An intensive experimental research was carried out and presented in the last few years in order to single out each particular factor affecting the response of the beam-column composite joint.

The aim of joint modeling is to establish the design tools

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which are simple and reliable for frame design. It is common practice to make the first applicable parametric study from experimental results thru regression analysis of the main parameters (LEON, 1990). The curves can be used for design purposes in the boundaries determined by experimental results only. The mechanical models for the composite joints predict the behavior in a wider range (JASPART at al.,1990). Only a limited number of FE models (LEON, 1990), (PUHALI at al.,1990) was used for this very complex problem.

On the basis of good knowledge of the behavior of each component and the influence on the overall response, it is possible to develop an analytical model as a rotational spring. For this goal JOHNSON and LAW (1972) proposed to compute the initial stiffness C_i as the sum of component flexibility. They adopted the slip in connectors from Newmark's theory and used the simple expression for the ultimate moment summarizing the steel part moment resistance and reinforced bars influence. To predict the shape of the curve WALD and PARIK (1990) adopted a power model and established the slip as a beam internal problem. The shape parameter n was calibrated against experiments (BENUSSI at al., 1989). IGARASHI at al.(1990) evaluate analytically and experimentally the ultimate compressive force in the slab under the positive bending moment by the bearing strength at the column face and by the shear strength at the column sides. This study enable to involve the nonproportional loading into the prediction model.

Several prediction models have been developed to represent steel connection flexibility. We used for a simple connection a polynomial expression (FRYE and MORRIS, 1975) with Mu at level $\theta=0.02$. For semi-rigid connections we got more precise results from analytical prediction models (CHEN and KISHI, 1990) and (YEE and MELCHERS, 1986).

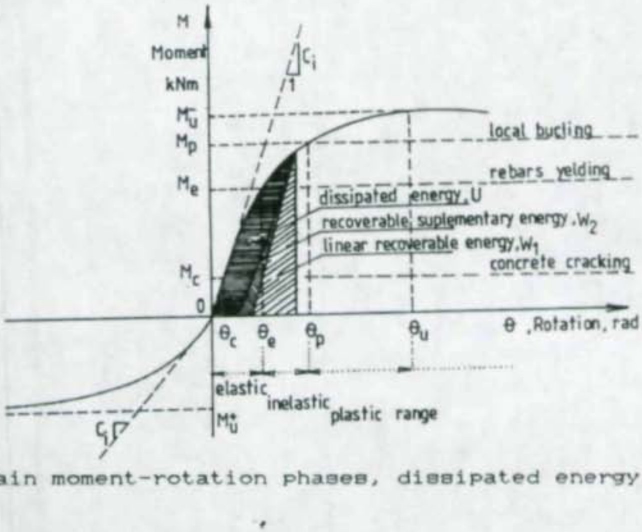

Fig.1. Main moment-rotation phases, dissipated energy.

The thermodynamics of irreversible processes was first proposed by KUCZYNSKI and GOSZYNSKI (1980) to represent the moment-curvature relationship in reinforced concrete beams. Later, it was used by COLSON at al. (1984) to predict the behavior of steel joints. Finally, El-METWALLY and CHEN (1989) developed the R/C beam-to-column connection model summarizing the joint dissipated energy, Fig.1. The model is easy to adopt for the cycling of a load.

2.JOINT MODEL

We idealized the connection between the beam and the joint as a concentrated rotational spring. We expect that the spring characteristics will be affected by all beams and columns in the joint according to moments redistribution, Fig.2.

$$k=(F-F_B)/F=(M-M_B)/M \qquad (1).$$

The spring is defined by the three parameters, Fig.1.: (i) We determine the initial stiffness of the connection C_1 from linear behavior of the materials. (ii) We compute the ultimate moment capacity M_u of the connection using the limit analysis. (iii) We establish the internal variable n summarizing the dissipated energy . The thermodynamics field theory of slow processes is used to define these variables and to develop the general form of the spring model (COLSON,1990) as a power formula for the moment-rotation relationship:

$$\Theta=M(1/(1-(M/M_u)^n))/C_1 \qquad (2).$$

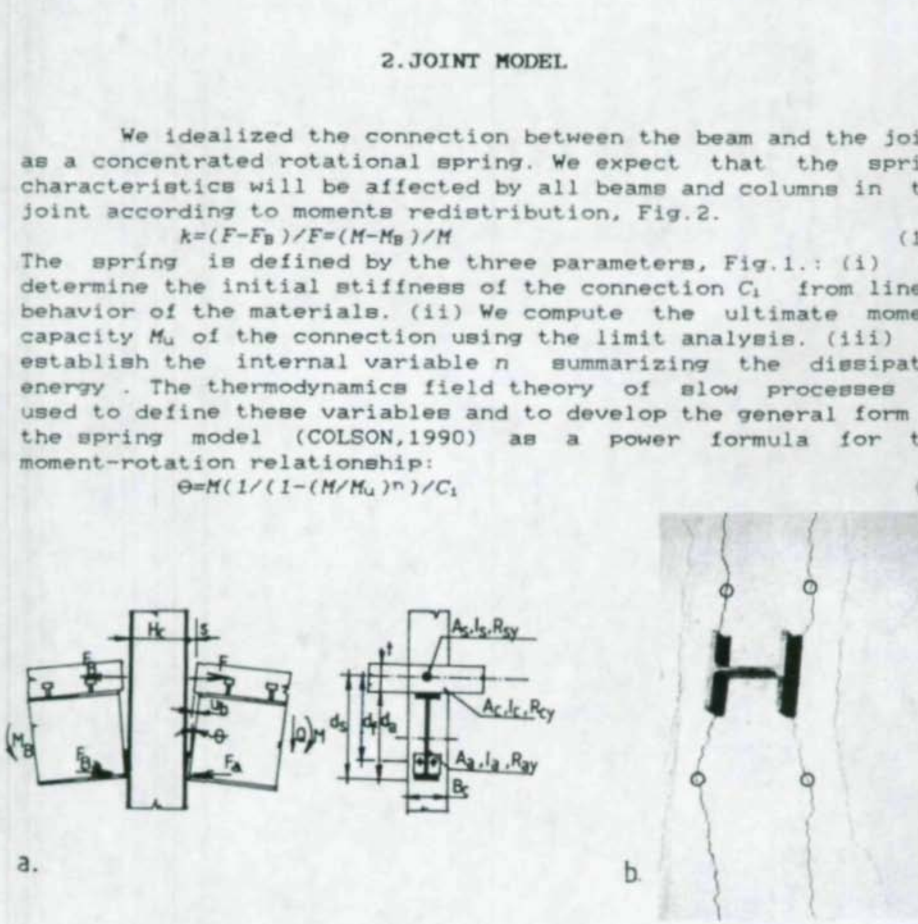

Fig.2. a- Joint moment-rotation modeling; b- The failure mode of a concrete slab, o leading crack

2.1. INITIAL STIFFNESS

The initial stiffness of the cracked composite connection is influenced prominently for the negative moment by (i) the reinforcement behavior, (ii) a slip in the shear connectors of the beam, (iii) the steel connection, (iv) the concrete slab cracking and (v) the slab-column action under unsymmetrical loading. For the simplified composite joint shown in Fig.2.a., the equilibrium and compatibility conditions at the column face are

$$M = F.d_s + C_{a1} . \Theta \qquad (3)$$

and $$\Theta.ds = s.da/ds + uc , \qquad (4)$$

where, C_{a1} is the initial stiffness of the steel part of the connection. The beam slip, which substantially affects the initial stiffness, we expect to behave linear. The contribution of slip action is connected with composite beam behavior. We can calculate this slip at the column face with the finite difference integration of the moment-shear-curvature relation affected with shear forces along the composite beam (ZAREMBA,1988), (WALD and PARIK, 1990).

We expect the leading initial cracking parallel to the column face, Fig.2b. The slippage at the column face is

$$s = s_{bo} + s_{co} \qquad (5).$$

where sbo is caused by bound deterioration of the reinforcement on both sides of crack. A very simple model for bound deterioration (MORITA and KAKU, 1984) was applied by El-METWALLY and CHEN (1989)

$$s_{bo} = F.d/(4.a.A_s) \qquad (6)$$

where, d is a reinforcement bar diameter and a parameter (a=2730MPa). From experimental research (IGARASHI at al.,1990) we estimate the contact initial stiffness a using bilinear model (shape $3/4F_{uco};0.001$)

$$C_{co} = 3.F_{uco}/(4000.H_c) \quad [kN/m] \qquad (7)$$

and $$s_{co} = F.k/C_{co} \qquad (8).$$

The equation (4) can be rewritten as follows

$$\Theta.da = F/(E_s.A_s)*(E_s.d/(4.a)+E_s.A_s.k/C_{co}).d_a/d_s+u_c) \qquad (9)$$

and we can express the reinforcement bars forces

$$F = (\Theta - u_c/d_a)\ A_s.E_s.d_s/B \qquad (10).$$

From the substitution (10) into (3) initial stiffness can be obtained.

$$M = (A_s.E_s.d_s^2/B + C_{a1})\ \Theta - u_c.A_s.E_s.d_s^2/(d_a.B) \qquad (11).$$

For the very stiff shear connector we can neglect the slip

$$C_1 = dM(0)/d\Theta(0) = A_s.E_s.d_s^2/B + C_{a1} \qquad (12).$$

The parameter B can be estimated as $1.5\ H_c$.

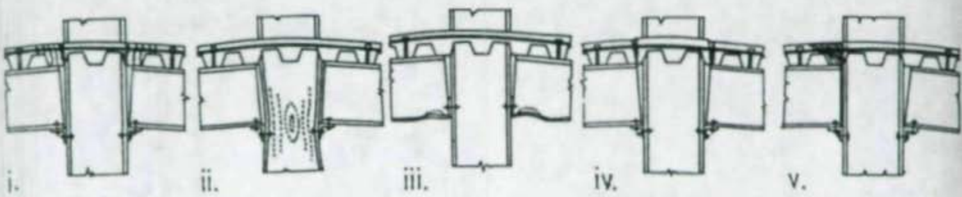

Fig.3. The major failure modes.

2.2. ULTIMATE MOMENT

The major local failures, Fig.3., affecting the failure modes are: (i) the reinforcement yielding, (ii) the column web failure, (iii) the local beam failure, (iv) the connection bolt failure and (v) the beam-column contact failure. (i) The ultimate moment capacity of the joint can be determined by adding up the moment capacity of the steel connection M_{ua} to the moment of resistance given by the yield strength of the bars (JOHNSON and LAW, 1981).

$$M_u = M_{ua} + A_s . R_{sy} . d_1 \qquad (13).$$

(ii) We have to check the resistance of the column compression zone (EUROCODE 3,1989) for an ultimate force. (iii) The steel section web slenderness should be less than 60 (ZANDONINI,1989) to prevent the beam compression as a leading failure. (iv) The shear resistance of connecting bolts should satisfy the design requirements (EUROCODE 3,1989). (v) The ultimate shear stress along the column side is given by following equation (IGARASHI at al.,1990).

$$F_{csu} = t . H_c . R_{cb} / 3 \qquad (14)$$

where, R_{cb} is a cylinder strength of concrete and $t.H_c$ is an area of a column side. The ultimate bearing strength at the column face (on an area of $t.B_c$) is derived from

$$F_{ccu} = t . B_c . R_{cb} (1 + t / (1.5 * B_c) + A_s . R_{sy} / (R_{cb} . t . B_c) \qquad (15),$$

$$k . F < F_{csu} + F_{ccu} \qquad (16).$$

2.3. INTERNAL PARAMETER

The establishing of the internal parameter n will require the calculation of the energy dissipation in the joint. Value of n can be assumed constant for simplicity. This will enable to calculate at a loading point when the tension steel reinforcement of the beams starts to yield. On the assumption that the free energy and the dissipated energy are unbound at moment of failure, we derive the equation for the dissipated energy U for slow processes according to the thermodynamics field theory (EL-MET-

WALLY and CHEN,1990) :

$$U=M_u^2/C_i * \sum_{1}^{n} \{[na(na+2)(\ln(M/M_u)+1)+2](M/M_u)^{(na+2)}/(na+2)^2 \qquad (17)$$

The energy results from the inelastic behavior of each joint components: (i) We denote the loss in the energy due to bar slippage on the crack boundaries

$$U_{r1}=F.A_s.s/2 \qquad (18)$$

and summarize for members coming into the joint.

$$U_r=\Sigma \{\pi.d^3.F_s/(32.A_s.a)\} \qquad (19).$$

(ii) We expect no energy loss due to a slip between the concrete plate and girder. (iii) For the steel connection we summarize dissipated energy from actual moment rotation curve (FRYE and MORRIS, 1975):

$$U_s = a_1 M_a^2/2 + a_2 M_a^5/5 + a_3 M_a^7/7 - M_a{}^\wedge 2/(2.C_{a1}) \qquad (20),$$

where M_a, C_{a1} are steel connection moment and initial stiffness and a_1, a_3, a_5 polynomial constants.

(iv) A concrete slab cracking is the important part of energy contribution. The hypothetical crack is assumed in front of the column face. The loss of energy is equivalent to energy density U_{c1} times the hypothetical volume V, where the crack spread thru

$$V = b.t.s.E_s.A_s/F \qquad (21).$$

The energy density was established (EL-METWALLY and CHEN,1990)

$$U_{c1}= 371.5(0.15*R_{ct})^{4/3}/E_c \qquad [kN/m^2] \qquad (22),$$

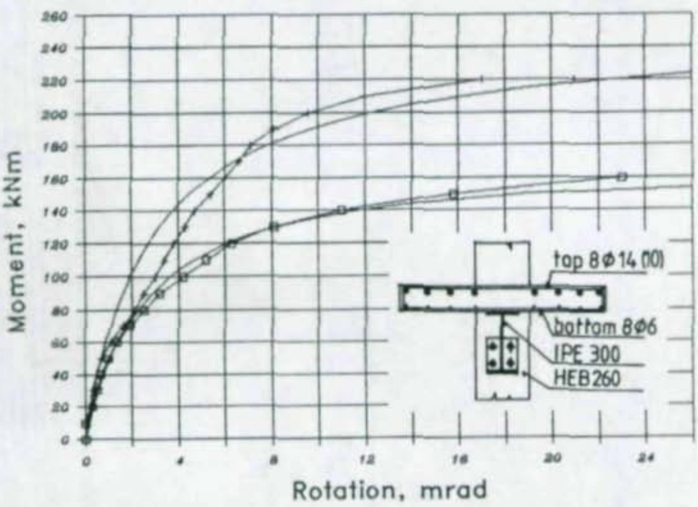

Fig.4. Comparison of tests (BENUSSI at al.,1987) ▮ SJA10, + SJA14 with - the predicted curves.

where, R_{ct} is a concrete cylindrical compressive strength [MPa] and E_c is a Young's modulus of concrete in compression [MPa].
(v) For the unsymmetrical loading we have to estimate the lost energy density from push out test results (IGARASHI at al.,1990). If we neglect the shear stress along the column side, we can from bilinear expression of force-strain relation assumed as

$$F_{co} = 3.75E{-}4.F_{ccu} + k.F(1.E{-}3{-}s/H_c) - (aF)^2/(2.C_{co}) \quad (23).$$

(vi) We have no evidence about the energy overall contribution of steel connection in tension under the positive moment. The estimate is possible to calculate thru analytical force-strain expression (YEE and MELCHERS,1986).

2.4.EXPERIMENTAL EVALUATION

The proposed model has been checked against some published tests. It has shown its reasonable agreement with the experimental data. Two examples are presented here to show the moment-rotation comparison. On Figs.4,5. the curves the experimental data vs the analytical M-Θ relation are compared.

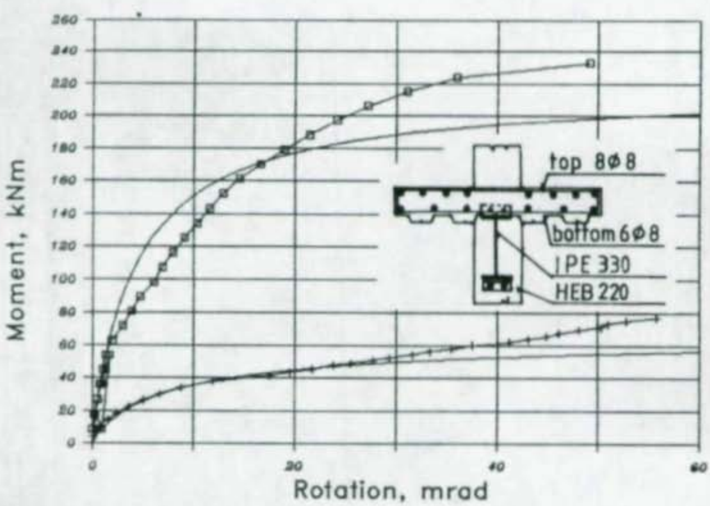

Fig.5. Comparison of tests (WALD,1991) ▮ composite joint, + steel top and seat angles connection with - the predicted curves.

2.5.DESIGN APPLICATION

The semi-rigid composite connections are relatively stiff compared with the steel ones. The connection behavior should be predicted with the ultimate moment M_u, Fig.1., and the real initial stiffness C_i. The special computer program for composite frames being developed at the Czech Technical University is taking into account the slip of beam and shape parameters of connections. The design value of connection moment M_{cd} limits the connection moment. Within a simplified frame design we calculate the initial stiffness C_i for stability and serviceability calculations and the secant stiffness C_s according to the beam line theory for the ultimate ones. The neglecting of slip, for deformable shear connectors, could lead to unpleasant over-estimate of initial stiffness.

3.CONCLUSIVE REMARKS

- The analytic results from the proposed model were possible to check only against a very limited number of tests. The results are in reasonable agreement with the experimental ones in case of a monotonic load. The author hope to calibrate the model against more tests to prepare it as a practical design tool.
- The model is possible to simplify for preliminary design purposes. The use is limited for joints with the steel connection as a semi-rigid (EUROCODE 3,1989).

Acknowledgement

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References

-BENUSSI F., PUHALI R., ZANDONINI R.,(1989): Semi-Rigid Joints in Steel-Concrete Composite Frames, Construcioni Metalliche, No.5, p.28

-COLSON A.,(1990): Theoretical Modelling of Semi-Rigid Connections Behaviour, in Stability of Steel Structures, Budapest, Vol.II, pp.143-153

-COLSON A., GALEA Y., PILVIN P.,(1984): Modeling of the Beha-

viour of Joints in Structures, in 12th Congress Vancouver, IABSE Final Report, Zurich, pp.55 - 62
-El-METWALLY S.E., CHEN W.F.,(1989): Moment-Rotation Modeling of Reinforced Concrete Beam-Column Connections, ASCII Structural Journal, July-August, pp.384-394
-FRYE M.J., MORRIS G.A.,(1975): Analysis of Flexibly Connected Steel Frames, Canadian Journal of Civil Engineering, Vol.2, pp. 280 - 291
-IGARASHI S., INOVUE K., TSUJIOKA S., ARAI T.,(1990): Strength of Concrete Slab at Composite Beam-to-Column Connection, Mixed Structures, Including New Materials, IABSE Symposium, Brussels, pp. 341-346
-JASPART J.P., MAQUOI R., ALTMANN R., SCHLEICH J.B.,(1990) : Experimental and Theoretical Study of Composite Connections, Mixed Structures, Including New Materials, IABSE Symposium, Brussels, pp.407-412
-JOHNSON R.P., LAW C.L.C.,(1981): Semi-Rigid Joints for Composite Frames, Joints in Structural Steelwork, Howlett J.H. at al. editors, John Wiley and Sons, New York, pp. 3.3 - 3.19
-KUCZYNSKI W., GOSZCZYNSKI S.,(1980): Behaviour of Hyperstatic Reinforced Concrete Beams Subject to Increasing Loads, Archiwum Inzynerii Ladowej, Warsaw, V-26, pp. 79 - 84
-KISHI N., CHEN W.F.,(1990), Moment-Rotation Relations of Semi-Rigid Connections with Angles, Journal of Structural Engineering, ASCE, Vol.116, No.7, July, pp.1813-1834
-LEON R.T.,(1990) : Semi-Rigid Composite Construction, Journal of Constructional Steel Research, 15, pp.99-120
-LEON R.T., AMMERMAN D.J.,(1990): Semi-Rigid Composite Connections for Gravity Loads, Engineering Journal, First quarter, pp.1-11
-PUHALI R.,SMOTLAK I., ZANDONINI R.,(1990) : Semi-Rigid Composite Action: Experimental Analysis and a Suitable Model, Journal of Constructional Steel Research, 15, pp.121-151
-Wald F.,(1991): Composite Beam-to-Column Joint Tests, in Czech, CVUT, Prague, in printing
-WALD F., PAŘÍK J.,(1990): Semi-Rigid Joints in Composite Frames, Acta Polytechnica, Prace CVUT v Praze, No.5, pp.107-124
-YEE Y.L., MELCHERS R.E.,(1986): Moment-Rotation Curves for Bolted Connections, Journal of Structural Engineering, Vo.112,No.3, March, pp.615-635
-ZANDONINI R.,(1989): Semi-Rigid Composite Joints - Strength and Stability Series, Vol.8, Connections, Ed. Narayanan R., Elsevier Applied Science
-ZAREMBA CH.J.,(1988): Strengths of Steel Frames using Partial Composite Girder, Journal of Structural Engineering, Vol.114, No.8., August, pp.1741-1759

TESTS ON COMPOSITE CONNECTIONS

D A Nethercot[1]

Abstract

It is suggested that one of the major obstacles to the application of simple plastic theory to the design of composite frames – ensuring the necessary form of behaviour in the hogging regions – may be removed through the use of partial strength connections. The ability of various forms of composite connection to meet the necessary performance criteria is assessed with reference to existing test data; comments on the type of systematic research study necessary as the foundation for use of the design approach with modern forms of composite construction are provided.

INTRODUCTION

Multi-storey steel frame buildings are often constructed using composite floors. Although the beams are then designed compositely, they are usually assumed to be simply supported – except in special circumstances (Brett et al, 1987) – thus beam to column joints are designed on the basis that the whole of the load transfer is achieved via the steel detail; since the beams are assumed to function as simply supported, simple shear connections are normally employed.

The true structural behaviour of such arrangements is, of course, rather different. The ability of all practical forms of steel detail to transfer at least some small degree of moment and to provide some degree of rotational restraint is well established (Nethercot, 1989). Over and above this there is a clear potential to utilise the concrete, acting in conjunction with any reinforcement, metal decking etc., to assist in load transfer. Although this may well have some effect on the shear capacity, it is likely to be of far greater significance in improving both the moment capacity and the rotational stiffness of the connection.

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Thus the concept of semi-rigid, partial-strength joints is even more relevant to composite construction than to bare steelwork. The potential advantages are also greater since elastic design of continuous composite beams (assuming full continuity) leads to large support moments at that point within the beam where its moment capacity is low (being effectively that of the steel section plus the reinforcement only), whereas plastic design requires significant rotation capacity from the support region where compression in the lower flange and the web of the steel section make premature buckling failures the likely controlling factor.

By limiting the joint moments, the demands on the steelwork in the support region and thus the likelihood of premature instability failure, can be reduced. At the same time the generation of some (limited) degree of hogging moment at the supports means that the high sagging moment capacity of the composite section at mid-span can be better utilised. Since engineering intuition suggests that this type of behaviour must actually be occurring with the sort of arrangements presently employed, the potential exists for demonstrable gains in structural performance with little or no change to current practice. The key clearly lies in a better understanding of the role of the composite connections themselves.

It is not, therefore, too surprising to find an increase in interest recently in the possibility of exploiting composite action in steelwork connections. However, a comprehensive review of all known test data available as of summer 1987 (Zandonini, 1989) showed that surprisingly little of it was directly relevant to popular, present day forms of construction.

The basic concept discussed herein – the use of semi-rigid/partial strength connections between composite beams and steel columns – is not new. It has previously been considered by several authors (Barnard, 1970), (Johnson and Hope-Gill, 1972) , (Kemp and de Clercq, 1985).

However, separate studies within the last 5 years of several key aspects have resulted in significant improvements in understanding. The present paper attempts to synthesise these and to identify outstanding issues, paying particular attention to the role of the physical testing of composite connections.

Behaviour of hogging moment regions

The hogging moment capacity, assuming the plastic stress block type of analysis shown in Figure 1, of a composite section is typically of the order of two thirds of the sagging capacity. Elastic analysis of continuous beams, however, leads to much larger support moments than mid-span moments as the series of cases of Figure 2 illustrates. Thus if plastic design of continuous composite beams is to be employed, the plastic hinges that form early at the supports must be capable of sustaining considerable rotation without unloading if the mid-span hinges and thus the full collapse mechanism is to develop. Behaviour of the type illustrated by curve A in Figure 3 is required; curve B, which demonstrates adequate strength but insufficient rotation capacity is clearly unsuitable.

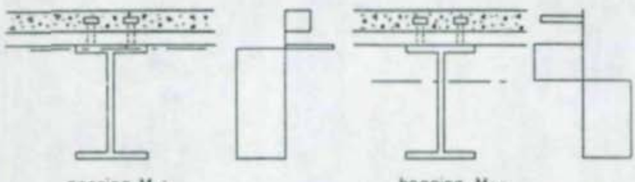
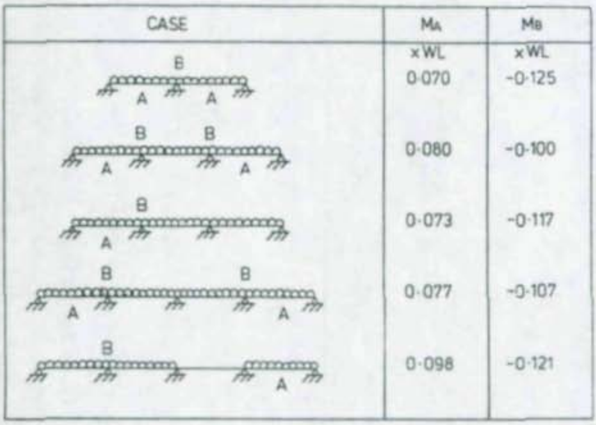


Figure 1 Moment Capacities for Composite Beam



CASE	M_A $\times WL$	M_B $\times WL$
	0·070	-0·125
	0·080	-0·100
	0·073	-0·117
	0·077	-0·107
	0·098	-0·121

Figure 2 Mid-span and Support Moments in Continuous Beams - Elastic Analysis

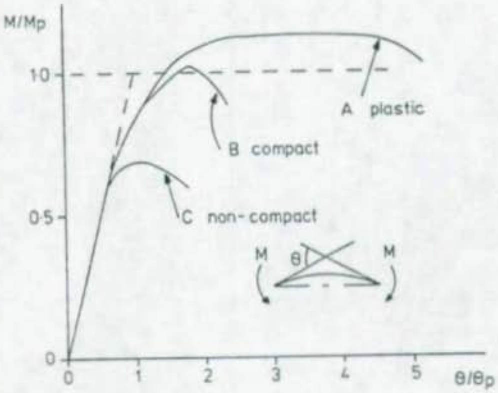


Figure 3 Beam Behaviour in Plastic Hinge Region

The feasibility of applying simple plastic theory as the basis for a rapid determination of the collapse load of a composite structure has been demonstrated by a number of tests on continuous beams (Hope-Gill and Johnson, 1976), (Daniels and Fisher, 1967), (Davison and Longworth, 1969) as well as by numerical simulation (Johnson and Hope-Gill, 1976), (Ansourian, 1987). In order to do this it was, however, necessary for certain undesirable forms of behaviour to be precluded. The most important of these is any form of buckling of the steel beam in the support region that leads to the type B or possibly even the type C behaviour in Fig. 3. Systematic studies (Bradford and Johnson, 1987) have shown that local buckling of the web and/or the lower flange are the key phenomena. For conventional rolled sections the special form of distortional lateral - torsional buckling associated with more slender plate girders (Weston et al, 1991) is not likely to be a factor.

Local buckling may be "designed out" and type A behaviour ensured providing the steel cross-section meets certain geometrical restrictions. Table 1 lists these for two recent codes (British Standards Institution, 1990), (Commission of the European Communities, 1990) as well as giving those recommended by Johnson and Climenhaga on the basis of their original tests (Climenhaga and Johnson, 1972)). For the web limit both codes give values as a function of the amount of compression present; Johnson and Climenhaga use the ratio of slab reinforcement to account for the position of the neutral axis and thus the severity of the compression. Figures 4 and 5 show respectively how, using the full list of

Basis	Material strength(N/mm^2)	Flange B/T	Web bending	d/t compression
Climenhaga & Johnson	275	17	70*	43*
	355	14	58*	36*
EC4 (draft)	275	18.4	66+	30+
	355	16.2	58+	27+
BS 5950: Part 3.1	275	17	64+	32+
	355	15	56+	28+

* Rule works in terms of ratio of reinforcement area to slab area
\+ Rule works in terms of level of compression in web.

Table 1 Geometric Limits for Plastic Cross-sectional Behaviour

British rolled sections (Steel Construction Institute, 1987), the flange and web buckling limits are likely to rule certain sections out of consideration. For high levels of web compression comparatively few sections, generally at the smaller end of the range, are suitable.

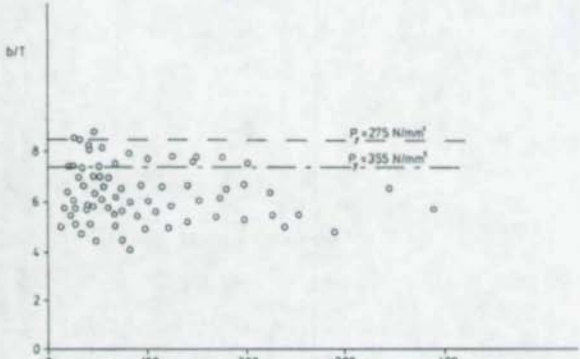


Figure 4 Comparison of Flange Properties of U.K. UB Rolled Sections with Plastic Cross-Sectional Limits of BS 5950: Part 3.1 - Flanges

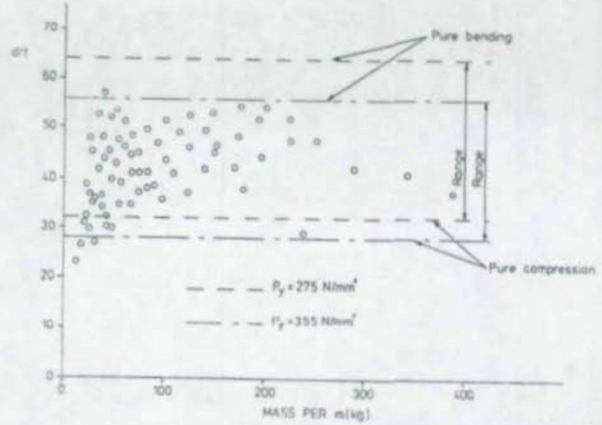


Figure 5 Comparison of Web Proportions of U.K. UB Rolled Sections with Plastic Cross-Sectional Limits of BS 5950: Part 3.1 - Webs

Use of Partial Strength Connections

Figure 6 illustrates three different types of end connection between a composite beam and a steel column. In Figure 6a sufficient reinforcement and slab depth are provided to develop the full hogging moment capacity of the composite beam. The arrangement of Figure 6b uses rather less reinforcement so that the neutral axis falls within the steel section, whilst Figure 6c shows a detail designed to restrict the points of load transfer to tension in the rebars and compression (principally) through the beam's lower flange. In all cases premature failure of the column is assumed to be prevented e.g. by suitable stiffening. It is clear from the associated stress diagrams that the demands on the web in terms of resisting possible local buckling decrease from cases a - c.

Of course, the moment capacity of the joint will also reduce. However, in terms of an efficient design of the whole system, the main requirements are to utilise the large sagging moment capacity within the span, not to place severe geometrical restrictions on the steel section and to utilise easily produced and therefore inexpensive end connections. It is clear that properly selected partial strength connections provide the designer with the means to achieve this.

At serviceability it is customary to calculate deflections using elastic analysis. The difference in mid-span deflection of a uniformly loaded fixed end and simply supported beam is 1:5. Thus a high connection stiffness at working load level offers the potential for very large reductions in beam flexibility. If little cracking – leading to loss of rotational stiffness (Davison et al, 1990) – is present at serviceability, it seems likely that behaviour approaching fully continuous may be achieved. *Thus a second set of requirements* concerns the connection characteristics at working load, where high stiffness up to an adequate level of moment is necessary. The exact moment level will, of course, depend upon the load factors being used; a figure of around 60% of the ultimate capacity is suggested.

Combining these requirements leads to the sort of connection performance in terms of shape of M-ϕ characteristic indicated by Figure 7.

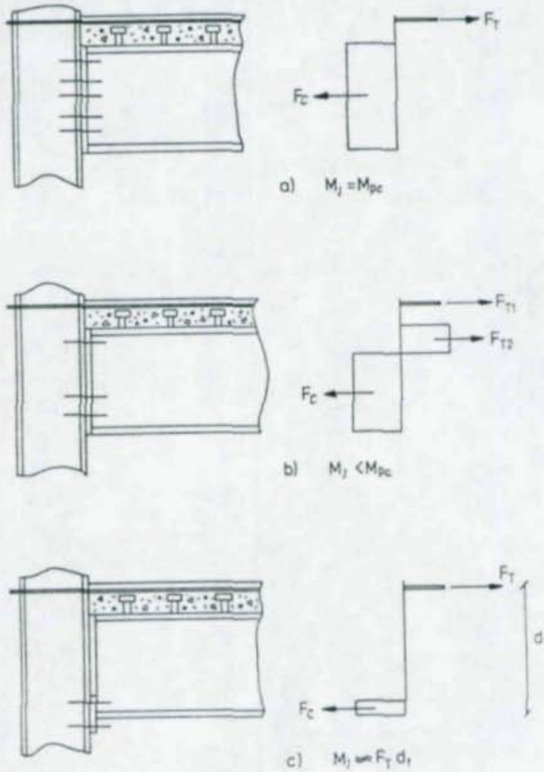


Figure 6 Composite Joints

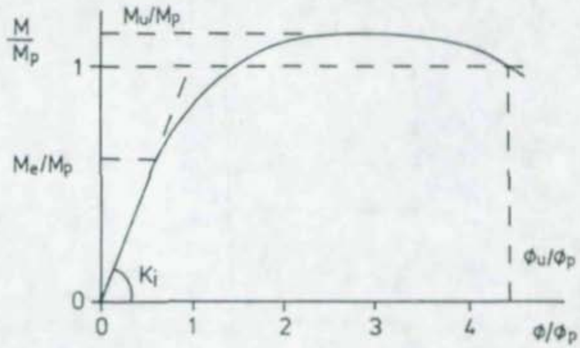


Figure 7 Required Joint M-φ Characteristic
(Lower Moment Capacities Acceptable)

Behaviour of Composite Connections

A review of all known test data for composite connections as of summer 1987 is available (Zandonini, 1989). Since then several other test series have been completed or are still in progress. Results from some of these for tests on cruciform specimens representing an internal column, usually under a balanced load condition, are summarised in Table 2. The key parameters in terms of joint behaviour are the ratio of joint moment capacity to beam hogging moment capacity M_J/M_{pc} and the available rotation capacity ϕ_u. The levels of both M_J/M_{pc} and ϕ_u necessary for a quasi-plastic design of the type illustrated in Figure 8 will, of course, depend upon the particular arrangement of spans, load levels, ratio of M_{pc}/M_{pt} etc. Calculations (Kemp, 1987) for a series of continuous beams assuming full continuity and full strength joints, as well as the indicative calculations for certain practical strength joints (Davison et al, 1990), suggest that requirements will be quite sensitive and a comprehensive study therefore needs to be undertaken.

These requirements should then be matched to the sort of performance achievable with different joint arrangements indicated in Table 2. For this to be done properly full M-φ curves, see Fig. 7, should be used so that behaviour at both ultimate and serviceability conditions may be assessed. Clearly even the summary provided by Table 2 suggests a wide range of available performance just for M_J/M_{pc} and ϕ_u.

Test	Joint Type	Column orientation	% reinforcement	M/M_p	M/M_{pc}	ϕ_u (m Rad)	ϕ_m (m Rad)	Failure mode	Column web stiffener	Decking	Reference
C6	f	F	0.20	0.159	0.104	10.0	26.0	F	-	perp.	
C7	f	F	0.60	0.707	0.461	31.0	38.0	I	-	perp.	
C8	f	F	1.00	0.914	0.365	32.5	38.0	I	-	perp.	
C9	f	F	0.60	0.808	0.526	15.0	28.0	F	-	perp.	Davison et al.,1990
C10	f	W	0.20	0.506	0.372	33.0	23.0	A	-	perp.	
C11	f	W	0.80	0.60	0.426	12.0	12.0	A*	-	perp.	
30x3c.2	g	F	0.68	0.99		39.3		A,H		-	
30x3c.3	g	F	1.28	0.98				H		-	
30x3c.1	g	F	2.12	0.94				H		-	
30x3c.6	g	F	0.68	1.14		51.5		A,(H)		-	
30x3c.8	g	F	1.28	0.95				H		-	Maquoi & Jaspart, 1990
30x3c.7	g	F	2.12	0.95				D		-	
30x2c.2	f	F	0.68	0.81		34.4		A		-	
30x2c.1	f	F	1.28	0.89				E		-	
30x2c.3	f	F	2.12	0.94				H		-	
30x2c.5	f	F	0.68	0.79		43.5		A			
30x2c.6	f	F	1.28	0.99				H		-	
30x2c.7	f	F	2.12	0.94				H			
36x3c.1	g	F	0.68	0.77		42.0		H			
36x3c.2	g	F	1.28	0.80				H		-	
36x3c.3	g	F	2.12	0.79				H		-	
36x3c.5	g	F	0.68	0.79		43.2		H		-	
36x3c.6	g	F	1.28	0.85				H		-	
36x3c.7	g	F	2.12	0.84				F		-	
36x2c.2	f	F	0.68	0.67		47.8		H		-	
36x2c.1	f	F	1.28	0.82				H		-	
36x2c.3	f	F	2.12	0.78				A		-	
36x2c.7	f	F	0.68	0.67		65.9		H		-	
36x2c.6	f	F	1.28	0.83				H		-	
36x2c.5	f	F	2.12	0.77				H		-	
SJAM/1	h	F	1.21	1.36	0.86	34	34+	D	-	-	
SJAM/4	h	F	1.21	1.34	0.85	47	47+	D	-	-	Puhali et al.,1990
SJAM/5	h	F	1.21	1.33	0.84	24	24+	D	-	-	
RJ/4	h	F	1.21	1.52	0.96	18	31	D	-	-	
SJA10	h	F	0.71	0.91	0.67	24+		A	Y		
SJA11	h	F	1.71	1.27	0.81	21+		A	Y		Howard et al.,1989
SJB10	d	F	0.71	1.15	0.84	25+		A	Y		
SJB18	d	F	1.21	1.44	0.96	22		D	Y	-	
SRCC1MR	g	F	1.03	1.22	0.84	39.0	-	E	-	-	
SRCC1ML	g'	F	1.03	0.94	0.64	30.0	-	F	-	-	
SRCC3M	g'	F	1.25	0.97	0.60	12.3	17.0	F	-	perp.	
SRCC5M	g'	F	1.25	0.92	0.51	39.0	43.3	F	-	perp.	Lam 1990
SRCC9M	g'	F	1.25	0.86	0.61	45.0	-	F	-	perp.	
SRCC10M	g'	F	1.25	0.79	0.56	48.3	-	F	-	perp.	

Failure modes
- A - Test terminated due to excessive joint deformation
- D - Local buckling of the steel beam (flange and/or web)
- E - Shear fracture of the bolts connecting bottom cleat and beam flange
- F - Fracture of slab reinforcement
- H - Buckling in compression of column web
- I - Beam failure in span

Table 2 Summary of Selected Recent Test Data for Cruciform Specimens

When planning and conducting future tests the eventual use to which the results will be put needs to be kept firmly in mind.

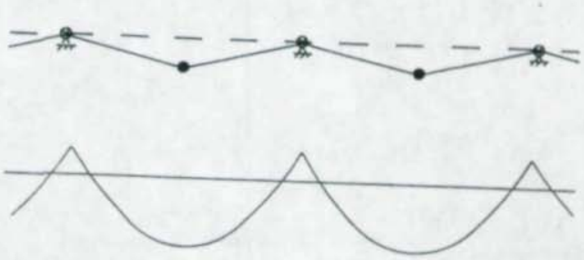


Figure 8 Quasi-plastic Design

CONCLUSIONS

The basis for the application of simple plastic theory to the design of composite frames has been reviewed. Buckling of the support regions has been identified as a major problem area. Restrictions on cross-sectional proportions may be greatly relaxed, thus rendering more standard beam shapes acceptable, if partial strength connections are used. Available test data have been used to assess the capabilities of several types of composite joint against the required performance. This has identified the approach necessary in future work of this type.

ACKNOWLEDGEMENTS

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REFERENCES

Ansourian, P., "On the Design of Continuous Composite Beams", Engineering Foundation Conference on Composite Construction, Henniker, 1987, pp. 1-12.

Barnard, P.R., "Improvements in Composite Floor Systems", Canadian Structural Engineering Conference, 1970, 13pp.

Benussi, F., Puhali, R., and Zandonini, R., "Semi-Rigid Joints in Steel - Concrete Composite Frames", Costruzioni Metalliche, No. 5, 1989, pp. 1-28.

Bradford, M.A., and Johnson, R.P., "Inelastic Buckling of Composite Bridge Girders near Internal Supports", Proc. ICE, Part 2, Vol. 82, March 1987, pp. 143-160.

Brett, P.R., Nethercot, D.A., and Owens, G.W., "Continuous Construction in Steel for Roofs and Composite Floors", Structural Engineer, Vol. 65A, No. 10, November, 1987, pp. 355-368 and discussion Vol. 66, No. 14, July 1988, pp. 216-227.

British Standards Institution, "BS 5950: Part 3.1, Code of Practice for Design of Simple and Continuous Composite Beams", BSI, London, 1990.

Climenhaga, J.J., and Johnson, R.P., "Local Buckling in Continuous Composite Beams", Structural Engineer, Vol. 50, No. 9, September 1972, pp. 367-374.

Commission of the European Communities, "Eurocode No. 4, Design of Composite Steel and Concrete Structures", draft Oct. 1990.

Daniels, J.H., and Fisher, J.W., "Static Behaviour of Continuous Composite Beams", Fritz Engineering Laboratory Report No. 324.2, March 1967.

Davison, J.B., Lam, D., and Nethercot, D.A., "Semi-Rigid Action of Composite Joints", Structural Engineer, Vol. 68, No. 24, December, 1990, pp. 489-499.

Davison, J.H., and Longworth, J., "Composite Beams in Negative Bending", Struct. Eng. Report No. 20, University of Alberta, May 1969.

Hope-Gill, M.C., and Johnson, R.P., "Tests on Three-Span Continuous Composite Beams", Proc. ICE, Part 2, Vol. 61, June 1976, pp. 367-382.

Johnson, R.P., and Hope-Gill, M.C., "Applicability of Simple Plastic Theory to Continuous Composite Beams", Proc. ICE, Part 2, Vol. 61, March 1976, pp. 127-144

Johnson, R.P., and Hope-Gill, M.C., "Semi-Rigid Joints in Composite Frames", Preliminary Report 9th Congress IABSE, Amsterdam, 1972, pp. 133-144.

Kemp, A.R., "Quantifying Ductility in Continuous Composite Beams", Engineering Foundation Conference on Composite Construction, Henniker, 1987, pp. 107-121.

Kemp, A.R., and de Clercq, H., "Influence of Ductility Requirements in Support Regions of Continuous Composite Beams on Economic Design", IABSE-ECCS Symposium on Steel in Buildings, Luxembourg, 1985.

Leon, R., "Semi-Rigid Composite Construction", Journal of Constructional Steel Research, Vol. 15, 1990, pp. 99-120.

Maquoi, R., and Jasport, J-P., "Experimental Study of the Non-Linear Behaviour of Beam-to-Column Composite Joints", Preliminary Report, Colloquium on the Stability of Steel Structures, Budapest, 1990, pp. N/3-11.

Nethercot, D.A., "Frame Analysis and the Link between Connection Behaviour and Frame Performance", Frame and Slab Structures, ed. G.S.T. Armer and D.B. Moore, Butterworths, 1989, pp. 57-74.

Puhali, R., Smotlak, I., and Zandonini, R., "Semi-Rigid Composite Action: Experimental Analysis and a Suitable Model", Journal of Constructional Steel Research, Vol. 15, 1990, pp. 121-151.

Steel Construction Institute, "Steelwork Design Guide to BS 950: Part 1: 1985. Volume 1 Section Properties and Member Capacities", 2nd edition, SCI, 1987.

Weston, G., Nethercot, D.A., and Crisfield, M.A., "Lateral Buckling in Continuous Composite Bridge Girders", Structural Engineer, 1991 (in press).

Zandonini, R., "Semi-Rigid Composite Joints", Structural Connections: Stability and Strength, ed. R. Narayanan, Elsevier Applied Science, 1989, pp. 63-10.

SEMI-RIGID COMPOSITE JOINTS: EXPERIMENTAL STUDIES

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Abstract

The paper summarizes the main results of a largely experimental research study of the rotational behaviour of semi-rigid composite joints. The key behavioural characteristics are presented, and design aspects related to joint classification and rotation capacity are discussed, also with reference to Eurocode 4. Finally the outcomes of cyclic tests are highlighted.

1.0 INTRODUCTION

The interest in semi-rigid joint action in composite steel-conrete frames is quite recent. The results of the first experimental studies (Zandonini, 1988; Davison et al., 1990; Zandonini and Leon, 1991) showed a very satisfactory performance under both monotonic and cyclic loads. A high degree of continuity can be achieved at a very low cost just providing the slab with rebars running along the column. Furthermore, stiffness and ultimate resistance can be controlled in a rather simple way by a suitable selection of steel connection detailing and amount of reinforcement; if the slab is not kept in direct contact with the column, continuity may be limited to adjacent spans, thus preventing that significant bending moments from being transferred to these elements. This solution may prove advantageous for non sway frames (SZS, 1989: Puhali et al., 1990).
Semicontinuous framing was included also in Eurocode 4 (1990), although no specific recommendations were provided. The present knowledge of joint behaviour, and of its influence on frame behaviour (in particular on moment redistribution), is not broad enough to allow for joint

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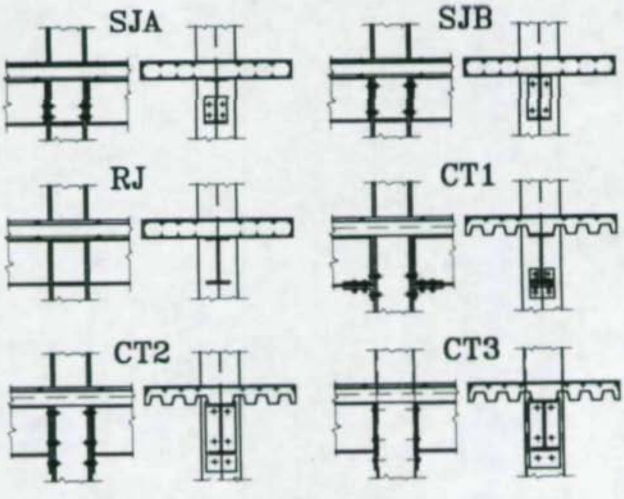
design and detailing at the required degree of reliability.
A research project aiming at investigating composite joint behaviour, and developing numerical and design models is in progress at the Universities of Trento and Trieste. The first experimental phase, which comprised several series of tests under monotonic as well as cyclic loads, has just been completed. This paper presents the main results, and discusses them with reference to the classification of Eurocode 4 and to the effect of joint action on the response of beams in non sway frames. The most important outcomes of the cyclic tests are finally illustrated, thereby contributing to the understanding of the possible role of semi-rigid composite joints in sway frame design.

2. MONOTONIC TESTS

2.1 THE SPECIMENS

The purpose of the research was to investigate the behaviour of composite joints, in which the continuity on the tension side was primarily provided by the slab. This makes possible to simplify the steel connection detailing as well as the key response mechanism of the joint. The steel connections studied are shown in figure 1:



STEEL BEAM: IPE 300 (SPECIMENS SJ AND RJ)
IPE 330 (SPECIMENS CT)

Figure 1

(1) connection SJA consists of a header plate welded to the bottom part of the beam web; (2) connection SJB is a flush end plate welded to both the beam web and flanges; (3) connection CT1 uses cleats connected to the beam lower flange; (4) connection of specimens CT2 and CT3 is an end plate extended beyond the beam lower flange; (5) connection RJ, finally, is a "rigid" fully welded connection. Connections SJA and CT1 are simple steel connections, while connections SJB, CT2 and CT3 are semi-rigid. Besides the steel connection, other parameters were investigated (see also Table 1): (1) the slab reinforcement ratio; (2) the type of slab (solid and with metal deck); (3) the type of shear connector: (4) the column type (specimen CT3 has a tubular column section filled with concrete); (5) the slab detailing around the column (permitting or not permitting direct contact); (6) the loading (balanced and unbalanced with respect to column axis).

The shear connection was in all cases designed so that full interaction and full shear transfer at collapse were ensured.

A total of 13 cruciform specimens were tested under increasing monotonically loads up to collapse. Table 1 presents some interesting parameters and the material properties of the specimens tested under symmetric loading.

TABLE 1

SPECIMEN	TOP BARS ϕ [mm]	$\rho_{\%}$	ρ_F	$f_{y,r}$ [MPa]	$f_{y,s}$ [MPa]	f_c [MPa]	SHEAR CONN.
SJA10	10	0.71	0.91	495	288	42	A
SJA14	14	1.21	1.30	413	288	42	A
SJB10	10	0.71	0.91	495	288	42	A
SJB14	14	1.21	1.30	413	288	42	A
SJA14/1	14	1.21	1.23	427	316	30	A
SJA14/2	14	1.21	1.23	427	316	31	A
SJA14/4	*14*	*1.21*	*1.23*	*427*	*316*	*21*	*B*
RJ14	14	1.21	1.23	427	316	24	A
CT1	14	1.10	0.98	483	329	36	A
CT2	14	1.10	0.99	473	319	39	A
CT3	14	1.10	1.02	478	312	41	A

$\rho_{\%}$ = Slab reinforcement ratio (%)

ρ_F = Semi-rigid force factor = $A_r f_{y,r} / A_f f_{y,s}$

A_r = Area of reinforcement

A_f = Area of a single flange of the steel beam

$f_{y,r}$ = Yielding stress of reinforcing bars

$f_{y,s}$ = Yielding stress of steel beam

f_c = *Concrete cubic compressive strength*

SHEAR CONNECTORS

A – Welded studs

B – Mechanically fastened (HILTI)

Measurements were taken of displacements and strains in order to monitor the behaviour of the nodal zone, and single out the primary sources of resistance and deformation. The joint rotation ϕ was not measured directly, but deduced from the rotation, relative to the column, of a steel beam cross section located 290 mm away from the column face. Different methods were considered for determining joint rotation either from the beam free end deflection or through the use of the steel beam curvature measured in the vicinity of the connection. Results were fully consistent and equivalent for practical purposes (Puhali et al., 1990).

2.2 THE RESULTS

(i) Specimens with solid slab.
The first two series of tests were conducted on specimens with solid concrete slabs, and end plate steel connections. The experimental M - ϕ relations for these joints (SJ types in fig.1) are plotted in figure 2.

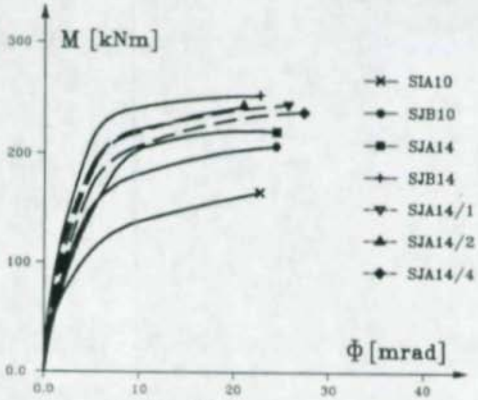


Figure 2

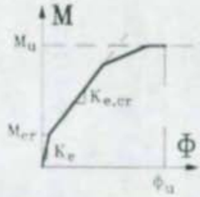
Four phases of behaviour can be identified (1) elastic with uncracked slab; (2) elastic with cracked slab; (3) inelastic with progressive deterioration of stiffness and (4) plastic with moderate hardening mainly due to the steel connection contribution and to the strain hardening of the rebars. The rotation capacity was always very high, considering that tests were generally stopped not because of joint failure, but of overly large displacements for the testing apparatus. A wide range of joint stiffness and strength values can be covered by properly selecting the steel connection and the amount of slab reinforcement, as is apparent from Table 2, which shows the key response parameters.
On the other hand specimen RJ confirms that the use of rigid steel connections leads to composite joints having a rotation capacity lower than that of joints with semi-rigid connections.
Evaluation of the test data also showed that:

(1) the shear transfer capacity of the slab is sufficient to ensure that all the rebars are yielding, thus contributing to the joint ultimate resistance;
(2) failure may occur due to local buckling, in particular buckling of the bottom flange of the steel beam was observed in some SJ specimens. However, buckling involved only the adjacent part of the web, and did not lead to an immediate loss of resistance, as in specimen RJ;
(3) mechanically fastened connectors seem fully equivalent to welded headed studs: indeed, no major difference in beam behaviour was found, but in the vicinity of the ultimate resistance, when uplift of the slab was observed in beams with Hilti connectors.

TABLE 2

SPECIMENT	K_e	$K_{e,cr}$	M_{cr}	M_u	Φ_u	FAIL MODE	$\frac{M_u}{M^-_{p,c}}$	$\frac{M_u}{M^+_{p,c}}$	$\frac{M_u}{M_{p,s}}$
SJA 10	68000	18700	49	165	21	A	0.66	0.43	0.91
SJA 14	112000	25900	51	221	23	A	0.81	0.58	1.22
SJB 10	140000	29800	51	208	22	A	0.84	0.54	1.15
SJB 14	302700	62100	51	261	24	C	0.96	0.68	1.44
SJA 14/1	116300	31700	61	246	27	A	0.86	0.61	1.24
SJA 14/2	113300	31900	47	242	21	A	0.85	0.60	1.22
SJA 14/4	104300	25500	47	240	28	A	0.84	0.59	1.21
RJ 14	202200	103200	47	287	13	C	1.01	0.71	1.45
CT1	131200	19000	53	298	31	B	0.80	0.56	1.18
CT2	173900	75200	55	356	13	B	0.98	0.69	1.40
CT3	176300	61200	55	300	11	B	0.83	0.59	1.19

Moments in kNm , Rotations in mrad and Stiffnesses in kNm/rad



FAILURE MODE

(A) Excessive joint deformation

(B) Fracture of the slab in shear

(C) Local buckling of the steel beam

(ii) Specimens with metal decking composite slabs.
Tests on CT specimens (fig. 1), having metal decking composite slabs (the most popular solution in many countries) revealed the importance of adequate transverse reinforcement of the slab. In the area adjacent to the columns in simple frames a nominal amount of reinforcement is used (often a wire mesh). The design philosophy adopted in the first series of tests (with solid slabs) involved the extension of this practice to semicontinuous frames. The limited transverse reinforcement in figure 3 was then selected. Due to the satisfactory behaviour of the tests, the same rebar layout was employed also for the specimens with metal decking.

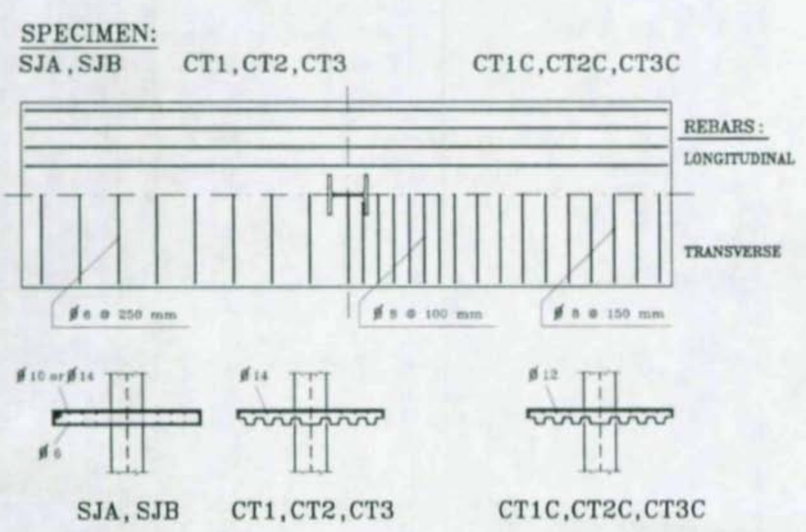


Figure 3

For all these three specimens collapse was however associated with longitudinal shear failure. Collapse occurred at a rather high moment, 80% greater than the negative bending resistance of the composite beam; the rotation capacity of joints CT2 and CT3 is, on the otherhand, significantly reduced (fig. 4). Only specimen CT1 with its very flexible angle connection, was able to achieve a rotation of the same order of magnitude as SJ joints. Due to the contribution of the slip between the angles and the beam flange, the stiffness of this joint is remarkably lower, in the cracked phase, than the equivalent joint in the first series, i.e. SJA14, whereas ultimate resistance is only slightly lower.
Specimens CT2 and CT3 are different in terms of column type: a HE section in the former, and a concrete filled tubular section in the latter. The thickness of the tubular section was 8mm; the forces transmitted by the upper tension bolts caused earlier nonlinearity due to inelastic deformation of the column wall. The stress state of the slab was consequently higher, inducing shear failure at a lower joint moment.

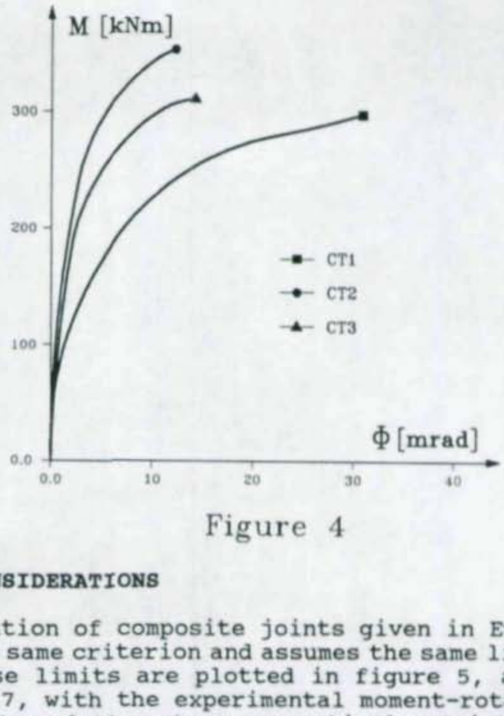


Figure 4

2.3 DESIGN CONSIDERATIONS

The classification of composite joints given in Eurocode 4 (1990) is based on the same criterion and assumes the same limits as Eurocode 3 (1991). These limits are plotted in figure 5, and compared, in figures 6 and 7, with the experimental moment-rotation curves for beam spans of 6. and 10. meters respectively. Joints are grouped by the steel connection type, whether simple (tests SJA and CT1) or semi-rigid (SJB,CT2 and CT3). Although interpretation of the Eurocode criterion is not straightforward for several of the tested joints, composite joints with simple steel connections may be classified as semi-rigid for the shortest beam (Fig. 6a), and rigid for the longest beam considered if reference is made to non sway frames (Fig. 6b). On the other hand joints with semi-rigid steel connections are "rigid" for both beam spans and non sway frames; joints CT2 and SJB14 may be classified as rigid also for sway frames when the beam span is 10 metres. It is also interesting to note that joint RJ14 with a fully welded connection with column stiffeners practically lies on the semi-rigid upper limit for a sway frame with L_b= 6m. This stresses the severity of the classification limits: many sway frames traditionally analysed as rigid should be presently considered semi-rigid.

When designing is based on plastic analysis, rotation capacity is another important characteristic, in addition to stiffness and strength.

Rotation capacity requirements in sway frames are rather difficult to be defined in an adequately general way, as they are heavily dependent upon frame configuration, loading condition, and beam to column relative stiffness and strength. An appraisal of the rotation capacity required by non sway frames can be obtained, through full nonlinear analysis of the internal beams, modelled as partially restrained members (Benussi et al, 1989; Puhali et al., 1990). Analyses were conducted assuming a piecewise linear behaviour of the joints, defined according to the different phases of their responses. The same two values were considered for the beam span as in the classification analysis. Figures 8 and 9 present some of the load versus midspan deflection curves obtained. The ideal cases of beams restrained by ideal joints the moment-rotation curve of which is represented by the Eurocode upper limits of the semi-rigid range (for sway and non sway frames respectively) were also considered, together with the limit conditions of fully fixed and simply supported beam. All the beams restrained by SJ joints collapsed due to the formation of a plastic mechanism: the first hinge formed at midspan, and joint rotation capacity was then enough for the beam to achieve the plastic collapse condition. Joint rotation at collapse varied between 14 to 23 milliradians, values significantly lower than those associated to joint failure. The beams with joint CT1 showed a similar behaviour. This seems to indicate that composite joints with simple steel connections tend to possess rotational ductility adequate for plastic design.

The stiffness of joints CT2 and CT3 was enough to make the sequence of activation of "plastic hinges" reverse for the beam with span L_b = 10m: i.e. the joint achieved its ultimate capacity first. Due to the lack of further rotation capacity of these joints, it is not possible to form the plastic hinge at beam midspan, and the attainment of their ultimate resistance corresponds to the attainment of the beam load carrying capacity. Their rotation capacity was hence inadequate for plastic analysis. The failure mode associated with the transfer of longitudinal shear can, however, be simply controlled by a proper sizing of the transverse reinforcement. The response of "rigid" joints, such as RJ, which tend to fail because of beam web and flange instability, sets more severe limits to design, though in the specific case of joint RJ rotation at the ultimate moment resistance was enough for the midspan plastic hinge to form.

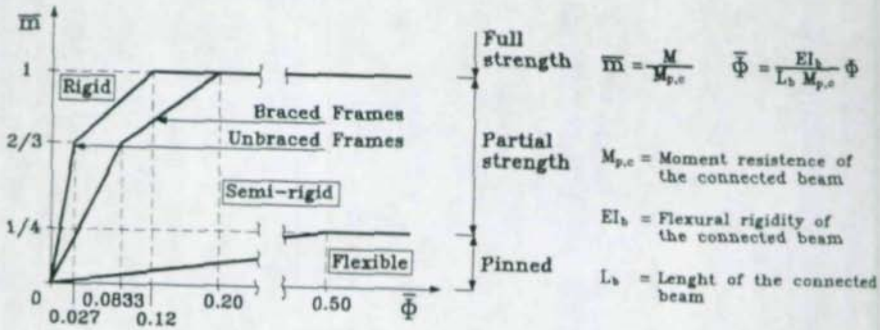
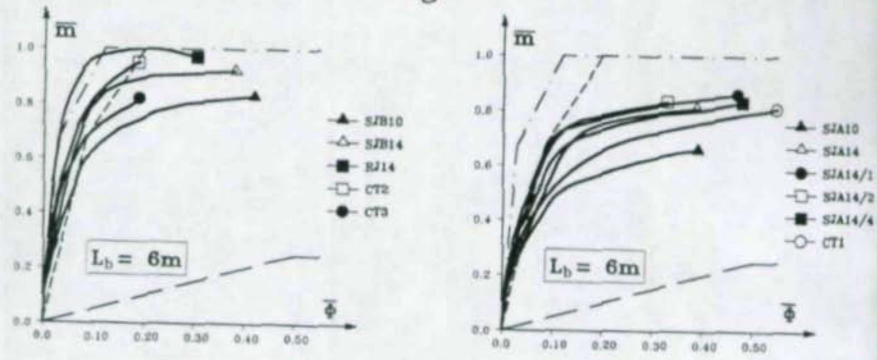
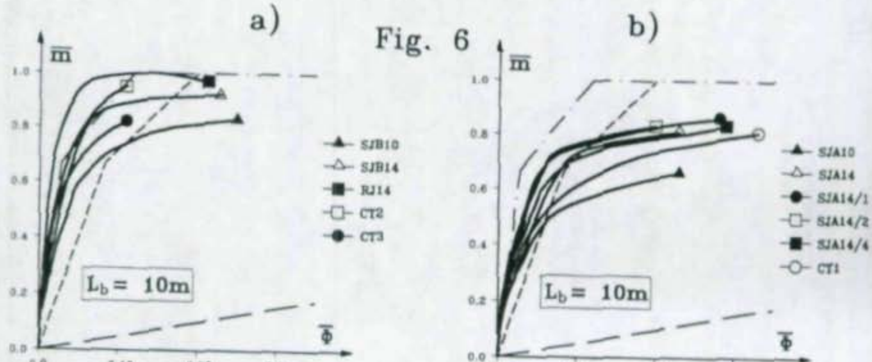


Fig. 5



a) Fig. 6 b)



a) b)

Fig. 7

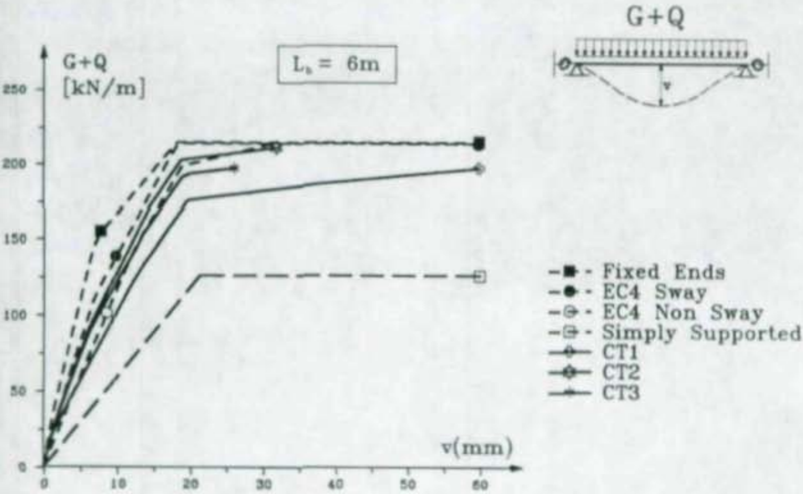


Figure 8

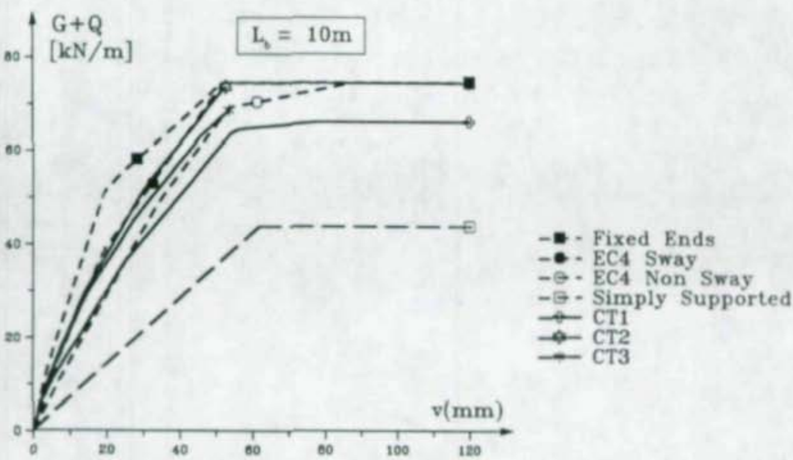


Figure 9

With reference to the ideal cases of beam with joints having as their M-Φ curves the upper boundaries of the semi-rigid Eurocode region, it should be noted that:
(1) Differences in deflection under service loads (obtained

assuming a common partial safety factor of 1.43 for both dead and live loads) with respect to the fully fixed beam were in average 54% and 85% for the sway and non sway limit curves respectively; this implies that a rather wide range of flexibility would be associated to the same design model of the joint, and stresses the need for reconsideration of the meaning of serviceability deflection limits.

(2) as to the non sway limit joints, the first hinge to form was for both spans the one in the middle of the beam. The same occurred for sway limit joints and beam length equal to 6m. Therefore sequence of formation of the plastic mechanism was in these cases opposite to that of a fully fixed beam.
These results suggest that a critical review of the adopted criterion of classification is necessary.

3. THE CYCLIC TESTS

Cyclic tests were conducted on four specimens: three CT joints and a joint with solid slab and steel connection with web and flange angles. This joint (CT4C) was nominally identical with a joint tested under monotonic loads in the framework of a coordinated research project carried out in Liege (Altman et al., 1990).
The cyclic loading history was in accordance with the testing procedure recommended by the European Convention for Constructional Steelwork (ECCS, 1986).
Although detailed appraisal of the results falls outside the scope of this paper, it is neverthless work reporting some interesting points:

(1) A fairly stable behaviour was observed, in spite of some pinching of the hysteresis loop due the slip of bolted connections and lack of contact due to gaps in certain phases of the inelastic response (see Fig. 10).

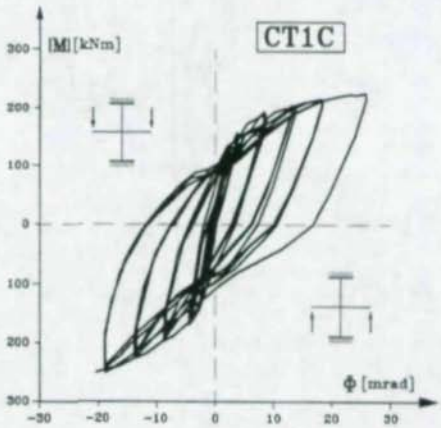


Figure 10

(2) Failure was associated with the collapse of the steel connection under positive moments (i.e. under moments compressing the slab); low cycle fatigue cracking of the steel connecting element (angle or end plate) occurred in specimens CT1C, CT2C and CT4C, while joint CT3C experienced the cracking of the threaded bars external to the beam lower flange, used in cyclic tests in lieu of the less effective bolts.
(3) Although collapse was attained under positive moments, the results indicate that the increase in the transverse reinforcement was adequate to improve the rotation capacity under negative moments.

4. CONCLUDING REMARKS

The experimental results of both monotonic and cyclic loading tests were shortly described in the paper, which were carried out in a research work on composite joint action and its influence on frame performance. The main outcomes can be summarized as follows:
(1) Semi-rigid joints represent a fairly important design option in order to improve the cost effectiveness of composite non sway frameworks. Their inherent stiffness and strength can be obtained also with rather simple steel detailing, and may be controlled in a straightforward way.
(2) The rotation capacity is generally sufficient for plastic beam design. It may, however, be limited by local buckling and longitudinal shear failure of the slab. Avoidance of the first phenomenon requires a proper selection of the steel beam section; however, it imposes less severe restraints than rigid joints. A simple conservative criterion was proposed by one of the Authors (Zandonini,1988), which is based on the semi-rigid force factor defined in Table 1. An adequate transverse reinforcement should be provided in order to prevent the second failure mode far from occurung. Further research studies are needed in order to define the minimum amount of transverse rebars to the adopted.
(3) Cyclic behaviour confirmed the suitability of semi-rigid composite joints for use in low rise frames (Leon, 1990), even in moderately seismic areas.
(4) The classification criterion adopted by Eurocode 4 should be critically reviewed with reference to the inelastic response of the joint-beam system. Furthermore, the meaning of the serviceability checks should be reconsidered, and possibly related to the analysis model.

A refined finite element program, which accounts for the main sources of nonlinearity, has been developed for the analysis of semi-rigid composite joints and frames; its calibration against the experimental data is currently in progress. An extensive numerical investigation will be then conducted, aimed at a comprehensive understanding of the role of semi-rigid action in composite frames. This will form the background to simple design approaches.

5. ACKNOWLEDGEMENTS

The research study was made possible by funds granted by the Italian Ministry of University and Scientific and Technological Research (M.U.R.S.T.), and by the European Community of Steel and Coal (ECSC, Grant 7210 SA/413).

6. REFERENCES

Zandonini R. (1988), "Semi-Rigid Composite Joints", Stability and Strength series, Vol. 8 'Structural Connections', Ed. Narayanan R., Elsevier Applied Science Publ., London.

Davison, Lam D. and Nethercot D. A. (1990), "Semi-rigid action of composite joints", The Structural Engineer, Vol. 68, No. 24, December.

Zandonini R. and Leon R. (1991), "Composite Joints", in 'Constructional Steel Design: An International Guide', Eds Harding J., Dowling P., Bjorhovde R., Elsevier Apl. Science, in pubblication.

SZS (Schweizerische Zentralstelle fur Stahlbau) (1987), "Rahmentragwerk in Stahl under besonderer Berucksichtigung der steifelosen Bauweise", Zurich.

Puhali R., Smotlak I. and Zandonini R. (1990), "Semi-Rigid Composite Action: Experimental Analysis and a Suitable Model", Journal of Constructional Steel Research, Special Issue on Composite construction, Vol. 15 Nos 1 & 2.

Eurocode No. 4 (1990), "Common Unified Rules for Composite Steel and Concrete Structures - Part 1 General Rules and Rules for Buildings", Commission of the European Communities.

Eurocode No. 3 (1991), "Common Unified Rules for Steel Structures - Part 1 General Rules and Rules for Buildings", Commission of the European Communities.

Altmann R., Maquoi R. and Jaspart J.P. (1990), "Experimental Study of the Nonlinear Behaviour of Beam-to-Column Composite Joint", Int. Colloquium 'Stability of Steel Structures', Budapest, Hungary, April.

European Convention for Constructional Steelwork (1986) - Technical Committee 1 - Technical Working Group 1, "Recommended Testing Procedure for Assessing the Behaviour of Structural Steel Elements under Cyclic Loads".

Leon R. (1990), "Semi-Rigid Composite Construction", Journal of Constructional Steel Research, Special Issue on Composite Construction, Vol. 15 Nos 1 & 2.

THE NONLINEAR BEHAVIOUR OF COMPOSITE JOINTS

Ferdinand Tschemmernegg

Abstract

This paper describes the tests to get the non linear behaviour of composite joints. Extensive test series were performed at the Institute of Steel and Timber Constructions, University Innsbruck. As bases of the test, the macromechanical modell of steel joints - developed at the institute - was used. At the moment only hinges are used in composite construction to connect the slabs and beams to the column. New possibilities for rigid or semi-rigid, full or partical strength composite joints are shown, which can be used in composite non sway composite frames.

1. INTRODUCTION

Elements of buildings are slabs (S) and beams (B), columns (C) which are connected by joints (J). The frames of the buildings consist of elements in steel reinforced concrete or composite. Also mixed structures of steel, reinforced concrete and composite elements become more interesting, Fig. 1.

To study the non-linear behaviour of composite joints the spring modell according (Tschemmernegg and Humer, 1988) for steel joints is used with the load introduction, shear, connection and overall springs.
Also a composite joint can be classificed according to EC 3 in view of stiffness strength and rotation capacity, Fig. 2.

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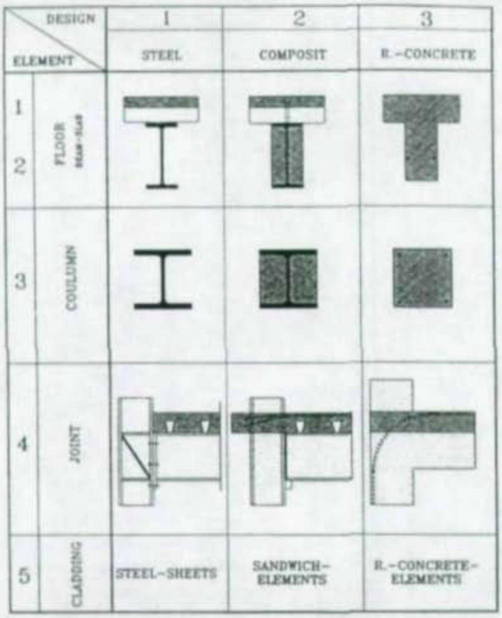
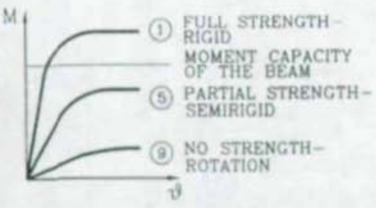


Fig. 1 Frame elements



		STRENGTH		
		FULL	PARTIAL	NO
STIFFNESS	RIGID	1	2	3
	SEMIRIGID	4	5	6
	PINNED	7	8	9

UNTIL NOW

Fig. 2 M-δ curves of composite joints

Until now only hinges were used in composite non sway frames, Solution 9, Fig. 2. All other solutions 1 - 8 have not been studied. So the aim of the research programm was, to find solutions for rigid and semi rigid partial or full strength joints and the according Moment-Rotation-Curves.

2. DEFINITIONS AND BACKGROUND

Simple solutions for composite joints are as follows. The beam is supported hinged by a small steel-block welded to the column flange. By concreting the slabs the joint gets rigid or semirigid. Only a tension reinforcement both sides of the columns in the slab is necessary, Fig. 1 row 4 column 2. The compression forces are going through the column in the region of lower flanges of the beams. So the construction can be erected with hinges which is very simple, and concreting the slab, the joint gets without bolting or welding rigid or semi rigid. 18 different tests with end- oder inner joints, different slabs, different beams, columnes and connections were tested. Fig. 3

TEST	SYSTEM		SLAB		BEAM		COLUMN		CONNECTOR	
NR.	+	⊣							T	L
1	×		×		×		×			×
2	×			×		×	×			×
3	×		×			×	×		×	
4	×		×			×	×		×	
5	×		×			×	×			×
6		×	×			×	×			×
7	×		×		×			×	×	
8		×	×		×			×	×	
9	×		×		×		×		×	
10		×	×		×		×		×	
11	×			×	×			×	×	
12	×		×		×			×		×
13	×			×	×		×		×	
14		×	×		×		×			×
15	×		×		×			×		×
16		×		×	×			×	×	
17	×		×		×		×		×	
18		×		×	×		×		×	

Fig. 3 Testing programm

3. TESTS

The test programm was developed in view of the spring modell according (Tschemmernegg and Humer, 1988).

3.1 Load introduction - compression

The load introduction in rectangular and circular composite columns were tested in comparison to steel columns (Wiesholzer 1991). Fig. 4 a, b, c.

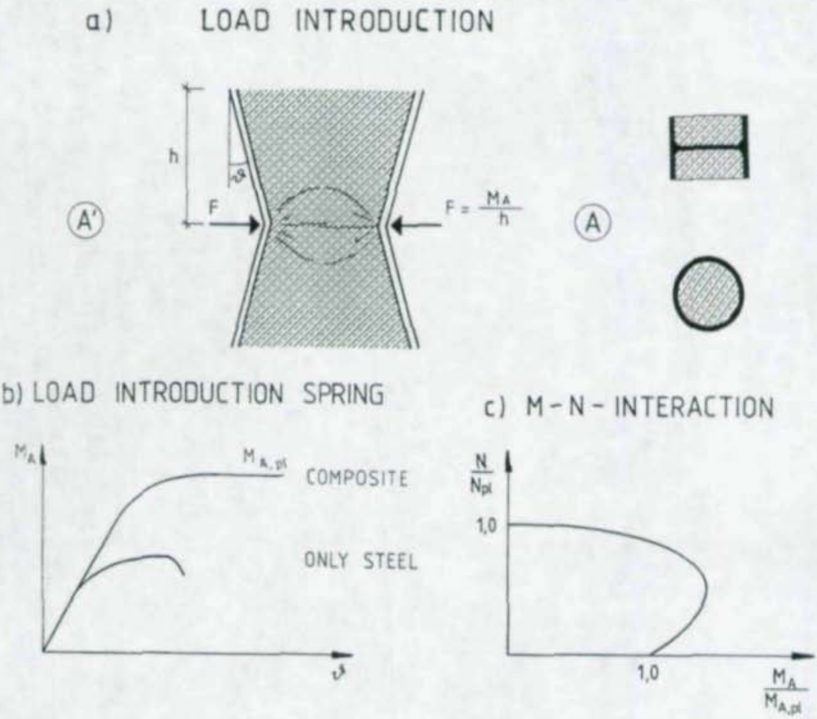


Fig. 4 Load introduction

The result was that the stiffness of the load introduction spring is not very much influenced by the concrete but the strength and deformation capacity is much better, Fig. 4 b.
The strength is at low load levels positively influenced by the normal force in the composite columns, because the cracks are closed by compression out of normal-forces in the columns, see Fig. 4 c.

3.2 Shear

The shear behaviour in composite rectangular and circular composite columns are tested in comparison to steel columnes in (Brugger, 1991). Fig. 5 a, b, c.

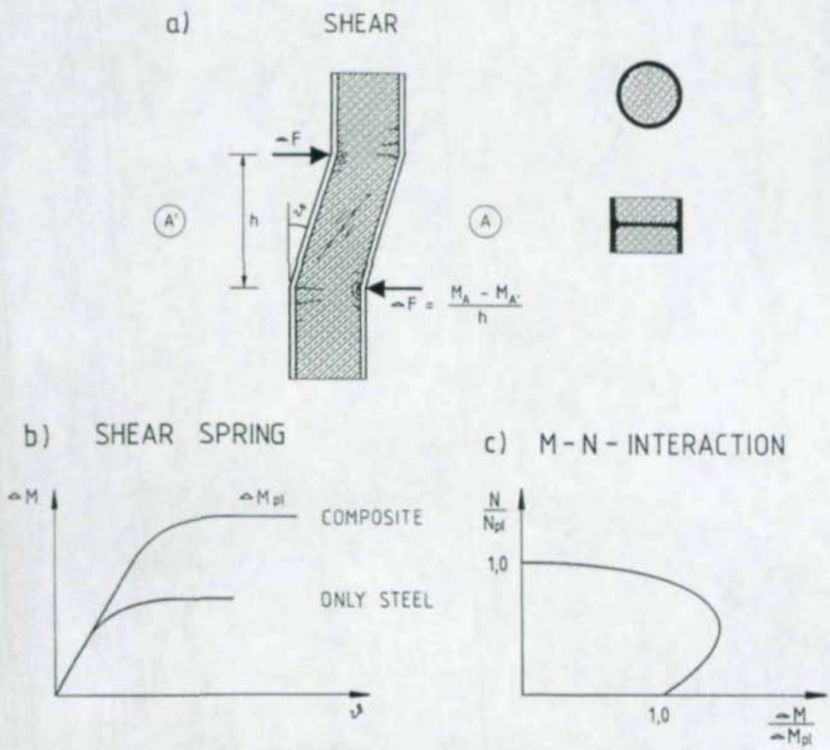


Fig. 5 Shear behaviour

The result again was, that the stiffness of the shear-spring is not very much influenced by the concrete, but the strength and deformation capacity is better, see Fig. 5b.
Also is to be seen the positive influence of compression out of normal forces in the column, see Fig. 5c.

3.3 Connection - tension

The problem of introducing the tension forces out of the moment differences in the beams is shown in Fig. 6.

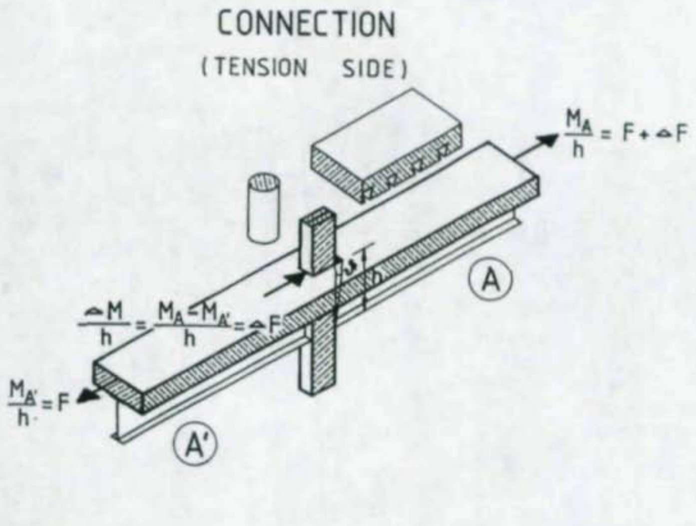


Fig 6 Connection

In (Hittenberger, 1991) the problem will be studied out of 18 full scale tests on composite joints with rectangular and circular columns.
The problem is equal to a bolted connection. The bolt corresponds to the column, the steel sheet to the slab. The main problem is here the compression between the slab and column and the design of the reinforcement in the slab for the tension forces.

3.4 Full scale tests

Full scale tests were described in (Tschemmernegg, 1990).
The measurment allows to get separately the load introduction, shear and connection spring in relation to the test load. For controll also the reactions for the test-specimens were measured.
First studies show very good agreement with the measurments according chapter 3.1 to 3.3.
So it is possible to get the main parameters for load-introduction, shear and connection separately.
The test results of full cate test are analysed now by Finite-Element-Methods (Brugger 1991), (Hittenberger 1991), (Wiesholzer 1991) to get a modell and simplified methods for design, and at the end the overall spring for composite joints. Fig. 7 shows the test arrangement of a full scale test.

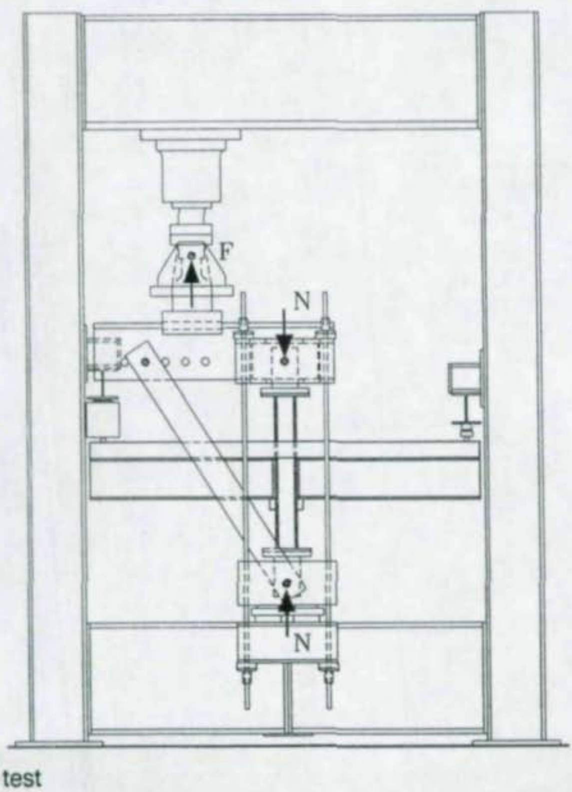


Fig. 7 Full scale test

4. CONCLUSIONS

Different possibilities of composite joints had been tested to get the nonlinear moment-rotation-behaviour of composite joints.
Using the computer programm for frames including this moment rotation curves it is possible to design composite frames.

References

Brugger, R.: The Beharvior of Composite Joints Due to Shear Loads. Institute for Steel and Timber construction, University of Innsbruck, Thesis (1991)

Hittenberger, R.: The Intersection of Columns and Plates in Composite Joints. Institute for Steel and timber construction, University of Innsbruck, Thesis (1991)

Tschemmernegg, F., Humer, C.: The design of Structural Steel Frames under Consideration of the Nonlinear Behavior of Joints. J. Construct. Steel Research 11 (1988) page 73-103.

Tschemmernegg, F.: The Moment Rotation Behavior of Composite Joints. IABSE symposium, Brussels (1990), volumne 60, page 83-88.

Wiesholzer, J.: The Load Introduction into Composite Joints. Institute fo Steel and Timber Construction, University of Innsbruck, Thesis (1991)

CYCLIC LOAD ANALYSIS OF COMPOSITE CONNECTION SUBASSEMBLAGES

SEUNG-JOON LEE[1]

LE-WU LU[2]

Abstract

The behavior of composite beam-and-column connection subassemblages subjected to cyclic loading has been analyzed by applying the finite element method. Special emphasis is on connections with a weak panel zone. The analysis incorporates the inelastic properties of steel and concrete and includes the effects of shear stud deformation and slip between the steel beam and concrete slab. The concept of effective width is used to account for the participation of the slab in resisting load. The analytical results have been compared with the previously completed tests and reasonable agreement is found.

1. INTRODUCTION

The behavior of a composite beam-and-column connection in a steel building structure, when subjected to seismic loading, is very complex, and only very limited research has been carried out. The effectiveness of reinforced concrete slab on the rigidity of the beam is markedly different from that specified in the design codes in practice. The moment transfer from beam to column is influenced by the presence of torsional members. Opening and closing of concrete slab cracks, nonlinearity of the material, slip across the interface between the steel beam and slab, and the interaction of the various structural elements at the joint further complicate the behavior.

A series of connection subassemblages were tested to study experimentally behavior under repeated and reversed loading (Lee and Lu, 1989). Two of these subassemblages

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designated as EJ-FC and IJ-FC and shown in Figs. 1 and 2, had beam-to-column-flange connections and exhibited very significant panel zone shear deformations, which became a major source of energy absorption. Analytical studies of such connections were subsequently performed to predict the observed behavior and the present paper is a summary of this work.

2. ANALYTICAL APPROACH

A two-step approach has been adopted to analyze these subassemblages with emphasis on their overall behavior:

- Three-dimensional elastic analysis of the composite beam to determine the effectiveness of slab.
- Two-dimensional inelastic analysis of the connection subassemblage with interest in the effect of slab on composite beams and panel zones.

2.1 Three-Dimensional Analysis of Composite Beams

The test results show that one of the parameters significantly affected the behavior of the composite beam is the effectiveness of the slab, commonly referred to as shear lag. It is evident that application of a mathematical formulation of the problem for an isolated T-beam to the composite beam considered in this investigation is difficult, because of its complex boundary conditions. The effective width variation along the beam under lateral loading condition is investigated using the linear finite element analysis program SAP IV.

In this analysis, the slab is represented by a network of thin plate elements to produce the flexural and membrane stresses. The thin plate element is SAP IV is a four-node quadrilateral element of arbitrary geometry formed from four compatible triangular elements. The element has six interior degrees of freedom which are eliminated at the element level prior to assembly. The resulting total degrees of freedom are twenty four per plate element. In order to obtain good accuracy, relatively fine mesh is used after some trials. Plate elements near the steel beam and the column are smaller in order to compute the effective width more accurately. The steel beam is represented by the beam elements to produce its axial, flexural, shear and torsional deformations. In the assembly of the composite system, the slab, represented by thin plate elements at the mid-surface, is attached to the steel beam elements by vertical link beams which enforced the eccentricity of the steel beam axis. To simulate full-composite action, very rigid link beams are employed.

2.2 Effective Width of Composite Beams

It is clear that the effective width of concrete slab of composite beam under the force condition assumed in this investigation is markedly different from that of an isolated T-beam. Figure 3 shows variation of the effective width along the beam of the tested subassemblage EJ-FC. Near the column where high slab stresses exist, the effective width is rather small. The deflection of the cantilever composite beam is close to that computed with the assumption that effective width is uniform and about 3/10 of the span L (length of beam between maximum and zero moments). On the other hand, the stresses in the composite beam, in the section adjacent to the column, are close to those computed with an effective slab width equal to the column flange width.

2.3 Two-Dimensional Analysis of Subassemblages

The finite element program, ADINA, is used to perform inelastic cyclic analysis of the connection subassemblages. The program performs an incremental nonlinear static and dynamic analysis of three dimensional structures and can separately treat material nonlinearity and kinematic nonlinear effect. Many material property models are available including elasto-plastic models with different yield criteria, concrete type material, a curvilinear model, etc. A truss nonlinear model can be employed to simulate opening and closing of gaps. It is believed that the composite connection subassemblages can be treated as a two-dimensional body if the main interest is in the overall behavior of the composite beam and the panel zone and if the subassemblages are loaded two-dimensionally as in the tests. Figure 4 shows the 2-D discretization of specimen EJ-FC. The slab is represented by two-dimensional plane stress concrete elements. The original slab thickness remains same, and the width of slab (which is the thickness of plane stress elements in Fig. 4) is the effective width from the 3-D elastic analysis and remains same throughout the analysis. It is known that the concept of effective width is not valid for simply supported T-beam when the slab begins to behave inelastically. For a composite beam-to-column joint, it is believed that the effective width of slab which acts compositely with steel beam does not vary significantly under increasing or decreasing load. A small portion of slab adjacent to column face, which is probably limited by the column flange width if there is no torsional member, remains effective in the inelastic range as well as in the elastic range.

The stress-strain relationship of concrete elements near the column face is modified as shown in Fig. 5 (curve C2) to increase the strength and the deformability. The concrete strength is assumed to be 1.3 f'_c and the corresponding strain is 1.9 ε_o, where ε_o is the strain corresponding to f'_c under monotonic loading. For steel, curve S2 in Fig. 5 is used and the von Mises yield criterion and isotropic and kinematic hardening rules are employed. Curve SC_2 is used to represent the shear stud behavior.

The concrete model implemented in ADINA employs several basic features to describe the material behavior; namely (i) a nonlinear stress-strain relation including strain-softening, (ii) a failure envelope that defines cracking in tension and crushing in compression, and (iii) a strategy to model the post-cracking and crushing behavior of the material. In the solution, the material is subjected to cyclic loading.

3. ANALYTICAL RESULTS AND COMPARISON WITH TESTS

The two-step finite element analysis approach described above has been applied to obtain analytical predictions for the response of the two test subassemblages. In the following the predicted responses of the composite beam of EJ-FC and the joint panel zones of EJ-FC and IJ-FC are briefly described and compared with the test results. More detailed descriptions of the work can be found elsewhere (Lee, 1987).

3.1 Composite Beam

Figure 6 shows the analytical and experimental load (P) vs. end rotation (Θ_b) curves of the composite beam of EJ-FC. Monotonic loading analysis were first performed to compare the skeleton curves under positive and negative loadings. For cyclic loading, the unloading and reloading analyses were initiated at $\Theta_b^+ = 0.0078$ and $\Theta_b^- = 0.0105$, while the calculated data from the monotonic analyses were saved to restart the analysis and the results are compared with the experimental curve of the 22nd cycle. Good correlation can be observed. The elastic stiffness and ultimate strength obtained from the analyses are in close agreement with the experiments, and the general shape of the hysteresis loops is similar. Some discrepancies, however, can be observed: (1) For cyclic loading, the Bauschinger effect of the steel beam reduces the tangent stiffness during reloading in the opposite direction and (2) The opening and closing of concrete cracks occurred gradually in the test specimens, but rather suddenly in the 2-D analysis.

3.2 Connection Panel Zone

The panel zone of the specimen EJ-FC shows different characteristics when the composite beam is subjected to positive or negative bending moment. Under negative moment, the panel zone behaves like steel beam-to-column panel zone, but under positive moment, the effect of composite slab was significant. Utilizing the enlarged panel zone assumption, the effective depth D_b was increased by 29.5% under positive moment (the test results showed 28.9% increase).

Figure 7 and 8 show the load (P or $P_1 + P_2$) vs. panel zone distortion (γ_p) relationships of specimens (EJ-FC and IJ-FC) The experimental and analytical results are in good agreement except that the general yielding from the analyses is higher by 13% to 20%. This is due to high residual stress in the panel zone because of welding of the web plate for the transverse beams. The post-yielding behavior between the experimental and the analytical curves exhibits discrepancies during the transition range, but eventually traces the skeleton curves after about 1.5% shear deformation. The calculated stiffnesses are lower by 17.0% to 24.4% than the experimental stiffnesses in this range.

The experimental reloading curve (the 20th cycle curve) is compared with the analytical curves based on two hardening rules: kinematic hardening and isotropic hardening. The experimental curve is between the two analytical curves. This should be subject for future research, which may begin by using an improved stress-strain relationship for high intensity cyclic shear (Wang, 1988).

4. SUMMARY AND CONCLUSIONS

A two-step analysis approach using finite element programs, ADINA and SAP IV, has been used to analyze the behavior of beams and panel zones in composite connection subassemblages under cyclic loading. The results have provided reasonable analytical predictions of the two full-scale connection specimens tested previously. The following conclusions may be drawn from the results presented:

1. The proposed method of analysis properly simulates the elastic and inelastic behavior of composite beams and composite connection panel zones when subjected to cyclic loadings.

2. The elastic stiffness and the strength of composite beams can be predicted accurately using the effective width concept.

3. The yield strengths of the panel zones from the analysis are higher by 13% to 20% than those from the experiments. Residual stresses due to welding are believed to be responsible for these differences.

4. Because of the higher predicted yield strength, the post-elastic stiffnesses of the composite panel zones from the analyses are lower by 17% to 24% than the experimental results.

5. The experimental and analytical hysteresis loops are very similar with the discrepancies due to difficulties in accurately simulating the three-dimensional concrete slab crack opening and closing and the Bauschinger effect.

6. The reloading curve of the composite beams based on the kinematic hardening rule is close to the experimental curve, but the loading surface of the panel zones after large inelastic shear deformation is between those prescribed by the kinematic and isotropic hardening assumptions.

Based on the results of this study, hysteretic models for inelastic dynamic analysis have been proposed for composite beams and connection panel zones in steel frame structures (Lu, Wang and Lee, 1989).

REFERENCES

Lee, S.J. 1987, "Seismic behavior of steel building structures with composite slabs," Ph.D. dissertation, Lehigh University.

Lee, S.J. and Lu, L.W., 1989, "Cyclic tests of full-scale composite joint subassemblages," Journal of Structural Engineering, ASCE, Vol. 115, No. 8, pp. 1977-1998.

Lu, L.W., Wang, S.J. and Lee, S.J., 1988, "Cyclic behavior of steel and composite joints with panel zone deformation," Proceedings 9th World Conference on Earthquake Engineering, Tokyo-Kyoto, Japan, Vol. IV, pp. 701-706.

Wang, S.J., 1988, "Seismic response of steel building frames with inelastic joint deformation", Ph.D. dissertation, Lehigh University.

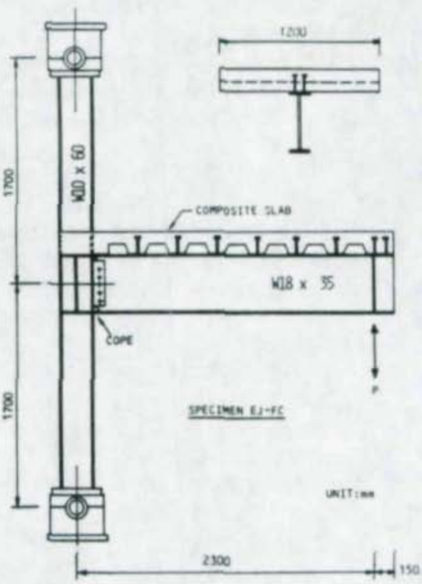

Fig. 1 Test Subassemblage EJ-FC

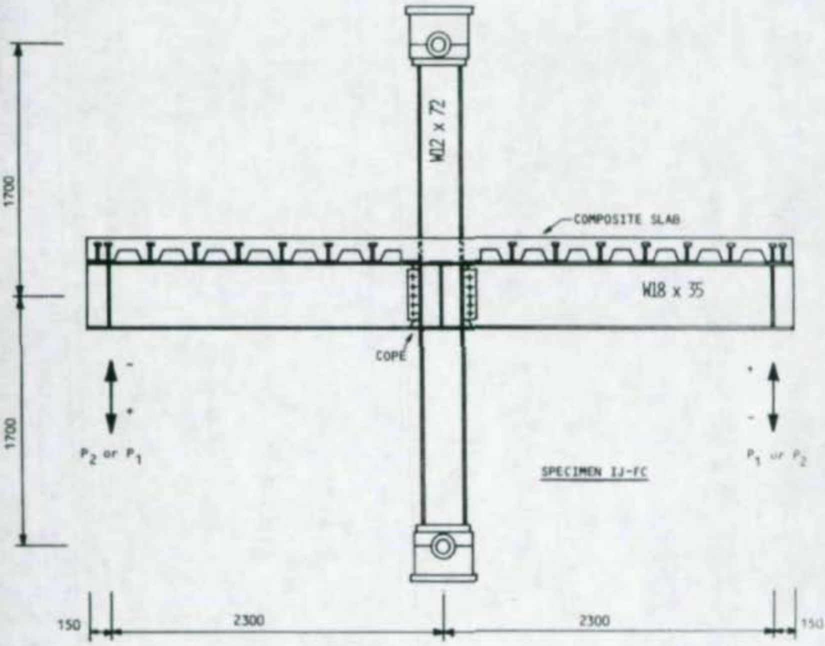

Fig. 2 Test Subassemblage IJ-FC

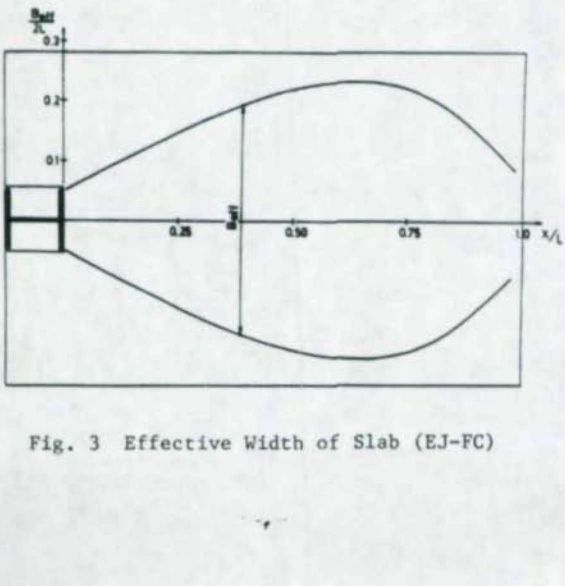

Fig. 3 Effective Width of Slab (EJ-FC)

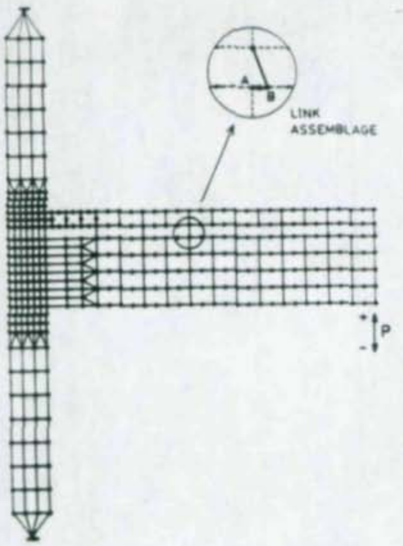

Fig. 4 2D Idealization of Subassemblage (EJ-FC)

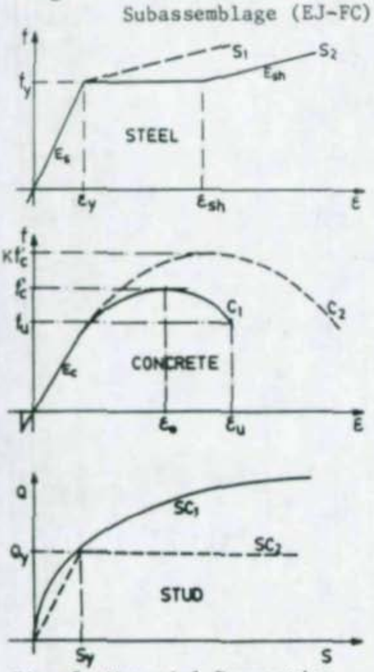

Fig. 5 Material Properties Assumed

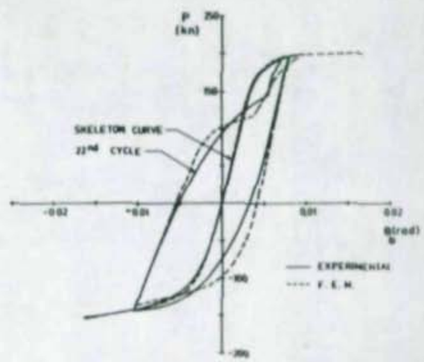

Fig. 6 Load vs Rotation Curves of Composite Beam in EJ-FC

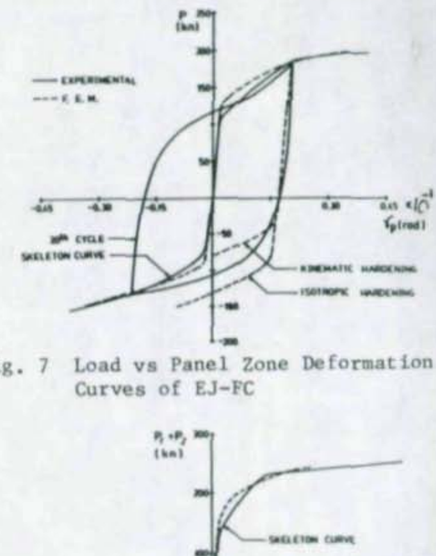

Fig. 7 Load vs Panel Zone Deformation Curves of EJ-FC

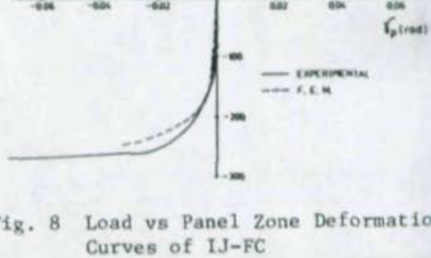

Fig. 8 Load vs Panel Zone Deformation Curves of IJ-FC

Technical Papers on

SEMI-RIGID CONNECTIONS

RELIABILITY OF ROTATIONAL BEHAVIOR OF FRAMING CONNECTIONS

Thomas R. Rauscher[1]

Kurt H. Gerstle[2]

Abstract

Fabrication differences and field conditions are likely to result in random variation in the stiffness and strength properties of field-bolted steel frame connections. A set of replicate specimens of one type of framing connection was tested in order to establish a statistical base, and the results were evaluated by probabilistic theory in order to determine the reliability with which strength and stiffness values can be expected. It was concluded that the determination of reliable strength and stiffness values for use in design requires probabilistic treatment of experimentally obtained statistical data bases.

1. INTRODUCTION

Design methods as outlined in the AISC Allowable Stress (1) and LRFD (2) Specifications authorize inclusion of connection effects under the heading of "Type 3" in the former, and "Partially Restrained" in the latter.

In both analysis and design including connection effects, connection behavior must be known. For typical beam-to-column connections of building frames, voluminous, if fragmentary, data are available (3,4,5). Attempts at rational prediction of connection behavior have been less than successful, but empirical expressions, based on test data, of the relation between the applied moment M and the resulting connection rotation θ are available. Among these, the most commonly used are those of Frye and Morris (6).

The deterministic moment-rotation curves of Ref. 6, and others similar, are often based on one single test, and do not account for the scatter which may inevitably be

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expected of connection behavior, specially if field-bolted. Little is available in the way of replicate tests which might provide a data base necessary for statistical prediction of connection behavior. Until such information about reliability of connection behavior is provided, its inclusion in design or analysis rests at best on a shaky basis.

This paper reports a study whose aim it is to provide a statistical data base for the purpose of establishing the degree of reliability of strength and stiffness for one connection type. To this end, nominally identical framing connection specimens from different sources were tested under identical conditions. The individual moment-rotation curves obtained from these tests form the data base for probabilistic determination of the reliability with which specified behavior of these connections can be expected.

2. TEST PROGRAM AND TEST RESULTS

Six fabricators provided double-web angle connection specimens fabricated according to the drawing and specifications shown in Fig.1. Two identical specimens were provided with untensioned bearing-type bolts (Series 1), and two with friction-type bolts (Series 2) tensioned according to shop practice of the individual fabricator, for a total of 12 specimens for each bolt type. Since each specimen contained two web-angle connections, we had in fact a sample of 24 of each connection. In addition, one fabricator supplied us with a set of six specimens with 3/8" thick web angles with F-bolts, attached to previously tested members (Series 3). This program gave us the opportunity to assess the following factors: Scatter of connection behavior, comparison of B-bolt versus F-bolt behavior, influence of angle thickness, effect of applied load history.

Because of length limitations, this paper will only describe and report results of Series 2. Complete results and evaluation can be found in Ref. 7. These double-web angle connections are commonly used as shear connections. Our discussion only concerns their rotational characteristics and therefore none of the conclusions should be interpreted as addressing their reliability in transmitting shear. We are here only concerned with the way in which they can be expected to rotate under applied moment.

The specimens were mounted as shown in Fig.2 in a 1000 kip MTS universal testing machine, and loaded either monotonically or cyclically, leading to the moment-rotation curves from 22 connections of 11 specimens of Series 2 which are shown in Fig. 3. A systematic random pattern is seen here for both stiffness and strength. Non-linearity is mainly due to yielding of the outstanding angle legs, and bolt slip occurs only under rotations well in excess of admissible values.

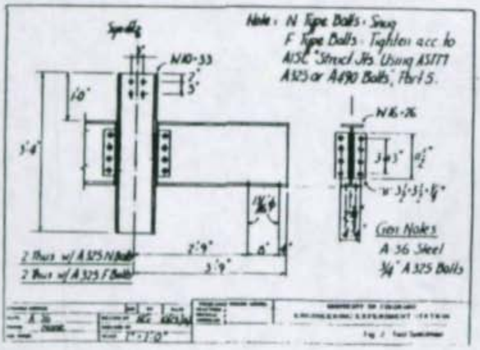

Fig. 1 Test Specimen

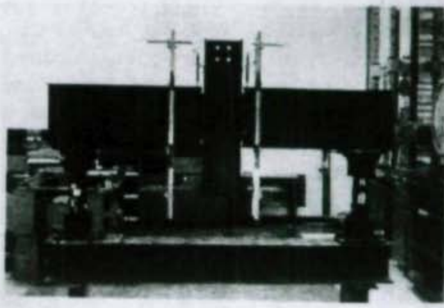

Fig. 2 Test Setup

3. STATISTICAL ANALYSIS

The parameters used to describe the connection response in the statistical analysis which follows were the secant modulus K_{sec}, the elastic limit moment M_{el}, and the moment under permissible rotation M_s, as shown in Fig.4.

The secant modulus K_{sec} was based on the moment corresponding to a rotation of .002 radians, well within the elastic range. M_{el} was obtained visually as the moment corresponding to the onset of flattening of the M-θ curve. M_s was the moment corresponding to the end rotation of a uniformly loaded simple beam under allowable midspan deflection L/360, computed as .009 radians.

The purpose of our study is to assess the reliability with which strength and stiffness of these web angle connections can be predicted. To this end, we will subject the strength parameters M_{el} and M_s and the stiffness parameter K_{sec}, defined in Fig.4, to statistical analysis with the aim of predicting their minimum values which may be expected with specified probability, or confidence level. In addition, we will try to extract information about systematic differences between products of different fabricators in order to obtain insight in problems of quality control.

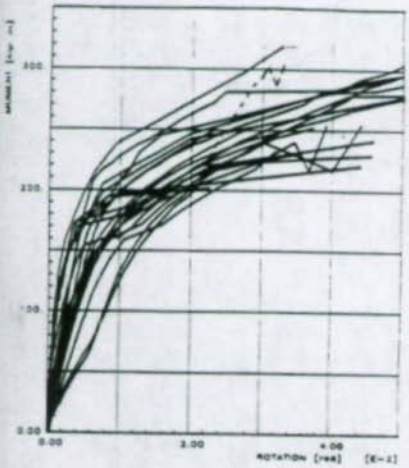

Fig. 3 Connection Response, Series 2

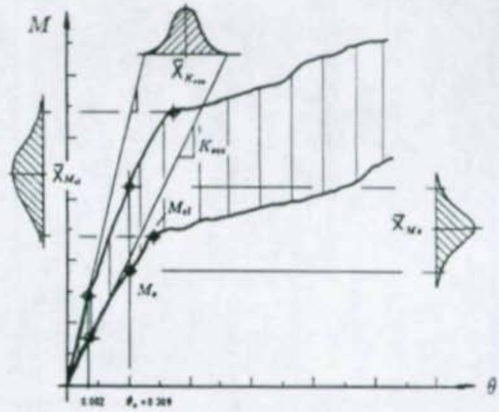

Fig. 4 Descriptive Parameters

Test values can be plotted in the form of a bell-shaped probability curve shown in dashed line in Fig. 5, representing a normal distribution. The shape of this curve is completely defined by the mean value X and the standard deviation S. The coefficient of variation S/X indicates the degree of scatter of results among nominally identical specimens. The probability P of exceeding any particular value of the parameter x is given by the area under the bell curve which is to the right of that value, and which can range from zero to unity. The probability P can be found for a distribution with given X and S for any value of x by integration, or from available tables (8). In this way, the minimum strength and stiffness can be determined which can be expected at a specified level of confidence - say, 95 times out of the next 100 specimens, as will be assumed in what follows.

The methods just described depend on the premise that all specimens belong to the same population. However, the techniques of different fabricators could be so different that their products might not belong to one population. Such a case is shown in Fig.5 , which plots stiffnesses K_{sec} for all specimens of Series 2. An analysis of variance (ANOVA) (8) indicates that these connections are better treated as belonging to two groups: Population A, consisting of Fabricators 1,4,5 and 6, whose products appear to have stiffnesses only about half of those of Population B, consisting of Fabricators 2 and 3. This will be discussed further below.

These techniques were applied to the test data in the following sequence: The strengths M_{el} and M_s, and the stiffness K_{sec} were first subjected to an analysis of variance to determine the likelihood of their belonging to one or more populations to within the 95 percent-level of confidence, using the F-Test described in Ref.8. For each population, the values X and S of the normal distribution were computed, and the minimum value of each parameter which might be expected within 95 percent confidence level was calculated.

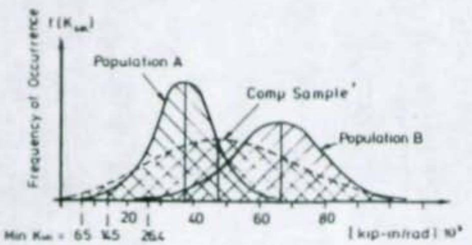

Fig. 5 Frequency Distribution for Ksec

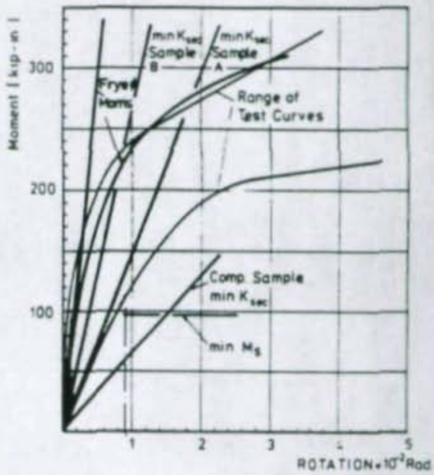

Fig. 6 Connection Properties, Series 2

The strengths M_{sl} and M_s, defined in Fig.4 , were subjected to the statistical treatment outlined. An ANOVA showed to within a 95 percent confidence level that the strength of all 22 specimens belonged to one population, whose characteristic values X and S are shown in Table 1, and that one might expect 95 out of the next 100 specimens to have strengths in excess of M_{sl} = 89 kip-inches and M_s = 99 kip-inches.

The observed stiffnesses K_{sec} showed a great deal of scatter, indicated by the dashed curve of Fig.5. The ANOVA showed two distinct populations: Population A, consisting of 14 specimens from Fabricators 1,4,5, and 6, and Population B, of 8 specimens from Fabricators 2 and 3. The statistical characteristics of each of these populations, as well as those of the composite sample of 22 specimens, are shown graphically in Fig.5. These results show that of the next 100 specimens from the first set of fabricators, 95 can be expected to have a stiffness K_{sec} in excess of 14500 kip-inches/rad., and of those from the second set of fabricators, 95 can be expected to have stiffnesses in excess of 26400 kip- inches/rad. If all 22 specimens are lumped together, then a minimum stiffness of only 6500 kip-inches/rad. can be assumed at the 95 percent confidence level, a value so low as to be negligible. The expected stiffness of specimens from Fabricators 2 and 3 is about twice that of specimens from Fabricators 1,4,5, or 6. One might look for obvious manufacturing differences among these fabricators. We found little clue as to the causes: Three different bolt tension control methods were used by the fabricators of Population A, among whom two used the same method as one of the fabricators of Population B. The reason for these seemingly systematic differences remains unknown.

4. DISCUSSION OF RESULTS

How will these results affect the designer who might wish to include connection restraint as provided by Type 3 Construction in the ASD, and PR Design in the LRFD Specifications? An example of this approach has been given by Lindsey (9) in an effort to optimize purlin size. In such a case, the engineer's likely recourse for the determination of connection stiffness and strength are analytical formulations such as that of Frye and Morris, which, as stated earlier, are deterministic and have in some cases (4) been found at variance with test data.

For the 1/4 inch web angle connections of Series 2, the curve predicted by Frye and Morris is shown in Fig.6, along with the range of the M-θ curves from our tests. The Frye and Morris curve is somewhat on the high side. Its initial stiffness is also shown, and the connection strength can readily be extrapolated.

If for safety's sake it is specified that these connection properties should be at the 95 percent level of confidence, then our statistical calculations would permit a serviceability moment and stiffnesses as also shown in Fig.6 , of values greatly below those given by deterministic formulation, or by any one of the test curves.

It is clear that in any case the choice of either a deterministic formulation such as that of Frye and Morris, or a single test case, may lead to connection strength and stiffness grossly on the unsafe side of reliable values in the actual structure.

5. *CONCLUSIONS*

Based on the test results and analyses which have been presented, as well as those of Series 1 and 3 (7), we can draw the following conclusions for rotational behavior of the web angle connections under consideration:

1. The bearing-bolt connections showed unpredictable behavior; they are not recommended for joints intended to offer rotational constraint.

2. The friction-bolt connections exhibited a systematic pattern of behavior, whose non-linearity was caused largely by yielding for thin web angles, and by bolt slip for thicker angles.

3. The scatter of stiffness is much less for the stronger than for the weaker connections; on this basis, it may be expected that the statistical variation of joints designed as moment- resistant may be more favorable than that of the web-angle connections.

4. The strength of the connections, while showing considerable scatter, varied

insignificantly among fabricators. Statistical minimum values can be determined with reasonable level of confidence.

5. Initial stiffness varied significantly among fabricators for the thin web-angle connections, although no physical reasons could be identified. It was not possible to assign meaningful statistical stiffness values for these specimens based on the totality of our test data. The thicker web angle connections, from one fabricator, showed much more consistent response.

6. Deterministic predictions of connection behavior, based on either empirical formulations or single test data, are likely to overestimate reliable values of strength and stiffness. Statistically designed replicate test series are needed to establish these characteristics.

6. REFERENCES

1. A.I.S.C., Allowable Stress Specifications, Sec.1.2, 1978.
2. A.I.S.C., L.R.F.D. Specifications, Sec. A2, 1986.
3. Goverdhan, A., "A Collection ...", M.S. Thesis, Vanderbilt Univ., Nashville, Tenn., 1983.
4. Kishi, N., and Chen, W.F., "Data Base ...", Report, School of Civil Engineering, Purdue Univ., West Lafayette, Ind., 1986.
5. Nethercot, D.A., "Steel Beam to Column Connections ...", CIRIA Project Record 338, Sept.1985.
6. Frye, M.J., and Morris, G.A., "Analysis of Flexibly-Connected Steel Frames", Can. Jnl. of C.E., Vol.2, 1975, P.280.
7. Rauscher, T.R., "Reliability of Double-Web Angle Connection Behavior", M.S. Thesis, C.E.A.E. Dept., Univ. of Colorado, Boulder, 1989.
8. Lipson, C., and Sheth, N.J., "Statistical Design and Analysis of Engineering Experiments", McGraw-Hill, N.Y., 1973.
9. Lindsey, S.D., Ioannides, S.A., and Goverdhan, A., "LRFD Analysis and Design of Beams", AISC Engineering Journal, Fourth Quarter 1985, P.157.

7. ACKNOWLEDGMENT

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PLASTIC CAPACITY OF END-PLATE AND FLANGE CLEATED CONNECTIONS-PREDICTION AND DESIGN RULES

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Abstract

A refined evaluation of the plastic capacity of the tensile zone in end-plate and flange cleated connections is presented. Compared to experimental results, it provides a better accuracy than existing methods.

1. INTRODUCTION

Beam-to-column joint using end-plate or cleated connections with no stiffening of the column web is very economical. Generally, such a joint has a semi-rigid behaviour and is partially resistant. The prediction of its non-linear response, in terms of beam end moment M_b - relative rotation ϕ, is especially of concern. Amongst the parameters governing the idealized M_b - ϕ response of the connection properly, the plastic capacity M_y plays a paramount role ; its determination is the subject of present paper. The space allocated to the paper prevents from developing in detail the theoretical aspects, for which the reader is begged to refer to (Jaspart, 1991). Only the basic ideas are presented and discussed herewith, as well as comparisons between theoretical predictions and experimental data.

A knowledge of the Annex J of EC3 (Eurocode 3, 1990) and its background would be very helpful to fully understand what follows.

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2. PLASTIC CAPACITY OF AN EXTENDED END-PLATE CONNECTION

2.1. General

The theoretical plastic capacity $M_{v,\,th}$ of a connection should basically be defined as the maximum bending moment developed by a connection made with a material which exhibits an elastic-perfectly plastic stress-strain diagram ; thus strain-hardening would be fully disregarded. To identify such a capacity on an experimentally recorded M_b - ϕ curve is neither obvious nor easy because of unavoidable strain-hardening effects. How to define the experimental plastic capacity is discussed elsewhere (Jaspart, 1991) ; within present paper, it is given as shown in figure 1.

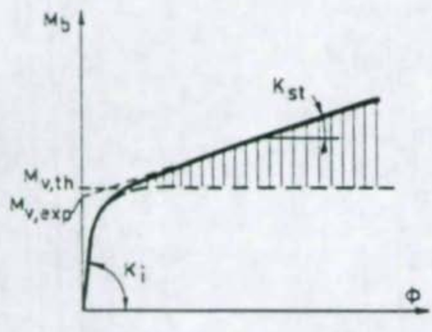

Figure 1

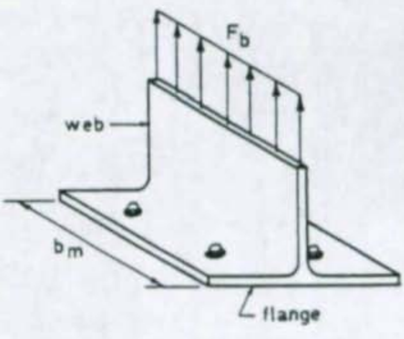

Figure 2

Usually, the bending moment at the beam end is substituted by a pair of statically equivalent tensile and compressive forces F_b in the connection. Thus the strength of a connection corresponds to the onset of a limiting design resistant force $F_{b.Rd}$ in the flanges. The resistance of a connection can be exhausted when any of all the possible local collapse modes takes place, either in the tensile or compression zone. Because it is deeply desired that the connection be prevented from a brittle mode of collapse, fracture of the fillet welds can never be governing. In addition, the resistance of the column web to load introduction, either in tension or in compression, is a specific problem which is solved elsewhere (Jaspart, 1991). Consequently, it remains to look at the resistance of an end-plate connection in the tensile zone which may be associated to the collapse either : i) of the bolts, or ii) of the end-plate, or iii) of the column flange. The two last collapse modes involve plate components subject to transverse loads : both can similarly be analysed suitably based on some idealization of the connection part. Several attempts have been made in this respect ; they are relative to one of the two general approaches termed respectively "plate model" and "T-stub model" ; both allow for predicting the aforementioned limiting force $F_{b.Rd}$ when either the column flange, or the end-plate is of concern.

Model	Column flange	End-plate	Both
Plate	Packer-Morris Zoetemeijer	Wittaker-Walpole Zanon Surtees-Mann Zoetemeijer	Zoetemeijer EC3 Packer-Morris
T-stub	Zoetemeijer EC3	Zoetemeijer Packer-Morris Kato-McGuire Agerskov EC3	

Table 1

Experimental results have been compared with the theoretical values computed according to the methods listed in Table 1 (see § 2.3). It has been found that EC3 method, based on the T-stub model, is quite valuable. The T-stub idealization consists in reducing the tensile part of the connection to T-stub sections of appropriate equivalent length b_m, connected by their flange onto a presumably infinitely rigid foundation (presumably by 4 bolts) and subject to a uniformly distributed force acting in the web plane (fig. 2).

Though based on rather simple expressions, EC 3 method is widely applicable. It is barely - but then very slightly - unconservative. Its accuracy, which can vary largely and look like too conservative, is dependent on the collapse mode of the T-stub. The latter can be due to (fig.3) :

a) bolt fracture with no prying forces, as a result of a very large stiffness of the T-stub flange, or

b) onset of a yield lines mechanism in the T-stub before the strength of the bolts be exhausted, or

c) mixed collapse involving yield lines at the toe of the fillets in the T-stub and exhaustion of the bolt strength.

The accuracy is found especially satisfactory when the plastic capacity is governed by collapse mode (a) or (c) ; EC3 is found much too safe when a plastic mechanism forms in the T-stub. Similar conclusions can be drawn from the tests on flush end-plate connections (Moore, 1988).

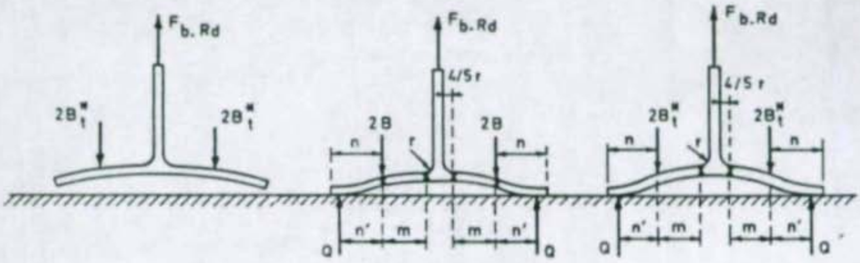

Figure 3

2.2. Amendments to the T-stub model of EC3.

The question raises whether refinements could be brought to the T-stub model of EC3 with the result that the amended model would provide a higher resistance for above collapse mode (b) without altering significantly the accuracy regarding both collapse modes (a) and (c).

An attempt in this respect has been made recently by the junior author (Jaspart, 1991). In all the existing methods, it shall be noticed that the forces in the bolts are always idealized as point loads. Thus, it is never explicitely accounted for the actual sizes of bolts and washers, on the one hand, and on the degree of bolt preloading, on the other hand. Care taken of the both aspects should influence first, the location of some of the yield lines forming the plastic mechanism, and second, the contribution of the external loads to the virtual work relative to this mechanism.

In the T-stub model, the plastic mechanism is composed of parallel straight yield lines which develop in the flange of the T section. Two of them are always located at the toe of the fillets. The two others are located in the vicinity of the bolt rows. Either they coincide with the axes of the bolt rows (Zoetemeijer, 1974; Eurocode 3, 1990) ; that means the bolt size is fully disregarded and the load is applied on the axis of the bolts (fig. 4.a). On the contrary, bolts and washers are assumed so stiff that the yield lines are forced to develop at the inner extremity of the bolt/washer diameter (Kishi et al, 1987), where the bolt load is also assumed to be applied (fig. 4.c). None of these models is in very fair agreement with experimental observations. Indeed the lines of maximum curvature are actually not straight but slightly curled and their pattern in the close vicinity of the bolts is found to depend on the stiffness of bolts and on the degree of bolt preloading (fig. 4.b). For practice purposes, one cannot imagine to account for such a complex actual pattern. It must be noticed that, for well proportioned connections, the yield lines are not far from complying with Zoetemeijer's assumption ; it is therefore justified to refer to the latter for what regards the location of the yield lines.

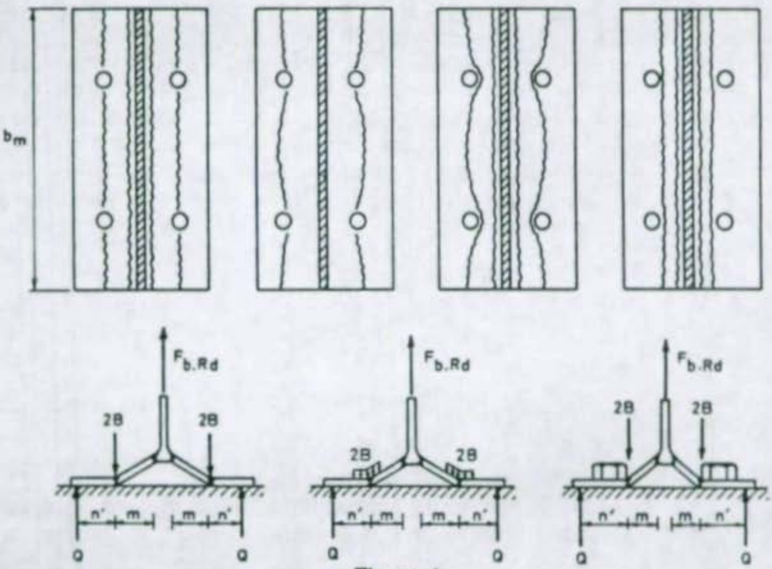

Figure 4

However account will be taken of the bolt size ; it will be assumed that the bolt load exerted onto the T-stub flange is uniformly distributed over a certain length D located symmetrically with respect to the bolt axis (fig. 4.b) ; D means the diameter of the bolt head/screw or washer. (Of course, the location of the yield lines does no more coincide necessarily with the section of maximum bending moment and results in a non-compliance with the fundamental theorems of plastic design ; the authors are of the opinion that the error remains sufficiently small to be acceptable). Accordingly half of the force in the bolts develops a negative external work when the plastic mechanism forms with the result of an expected higher connection capacity compared to EC3 model. For sake of simplicity, the resultant bolt loads 2B is substituted by two equal statically equivalent load components B acting at a distance $\pm$ e = 0.25 D from the bolt axis (fig. 5).

Applying the principle of virtual work to above plastic mechanism, on the one hand, and the equations of equilibrium, on the other hand, provides the limiting force $F_{b.Rd}$ associated to collapse by onset of a plastic mechanism :

$$F_{b.Rd} = (8n' - 2e)\, b_m\, m_p \,/\, [2mn' - e(m + n')] \qquad (1)$$

with :

$$n' = \mathrm{Min}\,[n\,;\,1{,}25\, m] \qquad (2)$$

$$m_p = 0.25\, f_y\, t^2 \qquad (3)$$

b_m in accordance with EC3

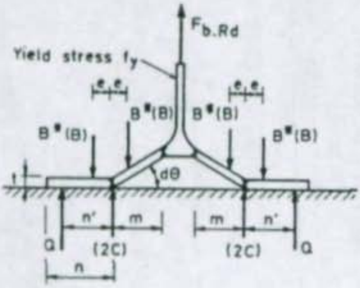

Figure 5

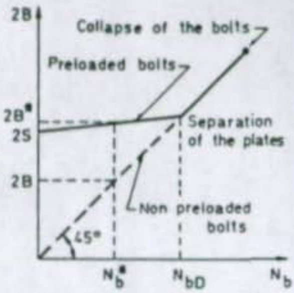

Figure 6

Of course, equ. (1) confines itself to Zoetemeijer's and EC3 formulae when distance e is vanishing.

What is said above is not explicitely influenced by the degree of bolt preloading. Actually the force in preloaded bolts of an elementary assemblage subject to an increasing external load N_b (parallel to the bolt axis) evolves according to figure 6. First, the bolt tension increase is a reduced proportion of the external load because of the compensating effect of the reduction in plate compression 2C. When the latter becomes equal and opposite to the initial contraction of the plate ($N_b = N_{bD}$), the plates start to separate and the system becomes statically determinate. At any higher load, the bolts experience the whole load N_b. Separation occurs at :

$$N_{bD} = 2S/K^* \tag{4}$$

where S is the preloading force per bolt and $K^* = 1/(1 + 1/\xi)$. The factor $\xi = A_t/A_b$ is the ratio between the axial stiffness of the effective plate compression area A_t and the resisting bolt cross-sectional area A_b ; it is taken as 5 as an average value (Agerskov, 1976). Proceeding as before with attention duly paid to the effect of bolt preloading, yields the amended expression of the limiting force $F^*_{b.Rd}$ (Jaspart, 1991) :

$$F^*_{b.Rd} = \{[8n'-2(1-K^*)e]b_m\, m_p + 4n'eS\}/[2mn' - e(1-K^*)(m+n')] \tag{5}$$

under the reservation that :

$$2B = (F^*_{b.Rd}\, n' + 2\, b_m\, m_p) / (2n' - e) \not> N_{bD} \tag{6}$$

because equ. (5) is valid in the range prior to plate separation. Should condition (6) not be fulfilled, then reference is made to (5) where $K^* = 0$, what results in $F^*_{b.Rd} = F_{b.Rd}$ with $F_{b.Rd}$ given by equ. (1).

The limiting force according to (5), subordinated to the additional check (6), constitutes a refinement of the relevant EC3 design rule. The capacity associated to the plastic mechanism can be assessed with a better accuracy than before (see § 2.3.). It would be easy to demonstrate (Jaspart, 1991) that above bolt effects do not at all alter the capacities associated respectively to the two other collapse modes of the T-stub.

The capacity of the T-stub is given as the lowest of the capacities relative to the three possible collapse modes.

Of course, the T-stub model is used several times when designing and extended end-plate connection : i) in the extended portion of the end-plate, ii) in the portion of the end-plate adjacent to the tensile flange of the beam, and iii) in the flange of the column. Above formulae must be written in an appropriate manner for each of these situations (Jaspart, 1991).

2.3. Comparison of the refined model test results.

Only few test specimens have been sufficiently instrumented to allow for a valuable comparison. It is referred to results obtained on isolated end-plates in Milano (Zanon and Zandonini, 1987) and on full connections in Liège (Jaspart, 1991) and Delft (Zoetemeijer, 1974).

Predictions according to the authors' proposal are superimposed in figure 7.a and b to a diagram established elsewhere (Damiani, 1986) for isolated end-plates ; those relative to full connections are listed in table 2.

It may be concluded to a very significative improvement of the present EC3 design rules and to a better accuracy. The refined model is still somewhat too conservative in a range where the bolts are not well proportioned to the stiffness of the plates (see especially the Italian tests with a end-plate thickness of 12 and 15 mm) ; then the conditions become close to Kishi's assumption with the result of an increase in the corresponding capacity. Such a range of not well proportioned connections will probably be met only exceptionnally because not economical at first sight ; anyway the approach provides conservative results in such cases.

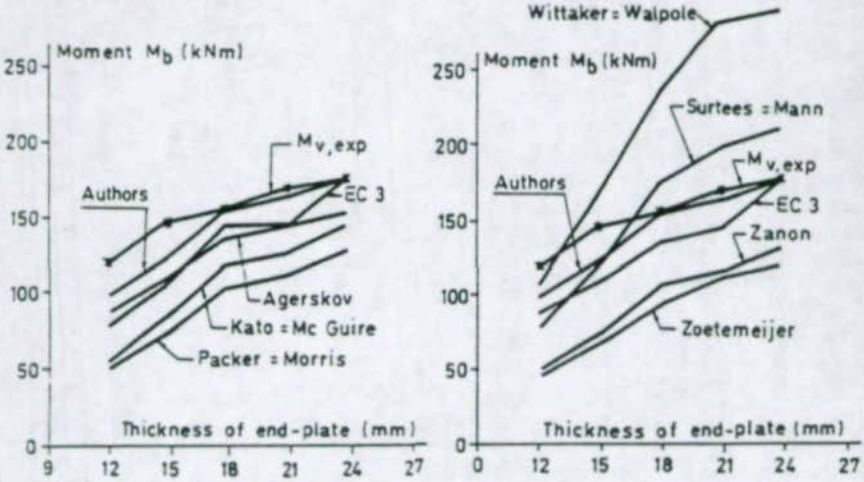

Figure 7

Test	Laboratory	$M_{v.theor}/M_{v.exp}$		
		Authors	EC3	Packer-Morris
01	Liège	0.98	0.98	1.10
04	Liège	1.04	1.04	1.15
07	Liège	0.98	0.98	1.08
014	Liège	1.01	1.00	0.91
T9	Delft	1.02	0.80	0.77
T20	Delft	1.08	0.91	0.90

Table 2

3. PLASTIC CAPACITY OF A FLANGE CLEATED CONNECTION

The critical part of a flange cleated connection is usually the tensile cleat and the adjacent zone (bolts in tension and column flange). Mainly two methods for assessing the pseudo-plastic capacity of the tensile zone are available (Hotz, 1983 ; Kishi et al, 1987). Both have been used for a comparison with fully instrumented test results performed in Trento (Bursi, 1990) on isolated cleats and in Liège (Jaspart, 1991), Sheffield (Davison et al, 1987) and Hamburg (Hotz, 1983) on cleated connections. It results that Kishi's method is largely unconservative and Hotz's one much too conservative. The large discrepancy is, to the authors' opinion, due to specific problems - linked to the cleated connection - which are not properly accounted for by both methods. It follows from experimental observations that :

- The collapse of a cleated connection by formation of a plastic mechanism involves a three-yield lines mechanism (two in the tensile cleat, one in the compression cleat) ;
- The initial clearance between the beam end and the column flange is likely to change the location of one yield line in the tensile cleat; the latter always develops at the toe the cleat fillet, at one time in the vertical leg, at one time in the horizontal leg.

In addition, the sole way to define the experimental plastic capacity of a cleated connection is not likely to correspond to the lowest strength of the connection components (Jaspart, 1991). Let us just mention that the degree of bolt preloading and the onset of an appreciable membrane action are the main reasons.

A fully original approach has been developed - on a basis similar to that used for end-plate connections - which accounts for the more accurate location of the yield lines, the sizes of the bolts/or washers, the bolt preloading and the plastic mechanism of the connection in its whole. As a result, a set of design formulae have been suggested, which are relative to all the possible collapse modes : bolts fracture, mixed plastic mechanism in the whole connection, and yield lines mechanism either in the cleats and in the column flange (Jaspart, 1991).

Test	Laboratory	$M_{v.theor} / M_{v.exp}$		
		Authors	Kishi-Chen	Hotz
03	Liège	1,01	(-)	0,52
06	Liège	1,01	(-)	0,55
012	Liège	1,02	(-)	0,78
JT08	Sheffield	1,05	(-)	0,75
TT1(*)	Trento	0,86	1,40	0,42
TT2(*)	Trento	0,89	1,46	0,44
A	Hamburg	0,89/1,04	1,28/1,64	0,51/0,65
B	"	0,83/0,97	1,16/1,51	0,47/0,60
E	"	0,89/1,04	1,21/1,57	0,49/0,63
F	"	1,01/1,15	(-)	0,47/0,60
G	"	0,80/0,94	1,15/1,47	0,45/0,58
H	"	0,92/1,06	1,25/1,62	0,51/0,65
I	"	1,00/1,12	(-)	0,48/0,61

Table 3

(*) not well proportioned connection (-) method not applicable
../.. extreme values because yield stress not measured

The theoretical results computed in accordance with this new approach have been compared with test data (table 3). It may be concluded that the method, that is founded on sufficiently simple formulae to allow for the daily practice, has a wide range of application and a very good accuracy. It could constitute an improvement to EC3, the Annex J of which is restricted to end-plate connections only.

4. CONCLUSIONS

Present paper is devoted to one aspect only of the design of beam-to-column joints : the plastic capacity of the tensile zone of the connection. It is shown that some amendments to EC 3 design rules for end-plate connections could be brought - without increasing the complexity of the design expressions - with a view to get a better and more homogeneous accuracy of the predictions. There is thus matter for thought when the Annex J of EC 3 will be reviewed by the relevant CEN Technical Committee. In addition, a set of design formulae for flange cleated connections, detailed elsewhere by the junior author, are found likely to be used as the background for a possible future additional Annex devoted to this type of connections. Further computations have shown (Jaspart, 1991) the non-significative influence of M-N-V interaction in the yield lines on the connection plastic capacity.

REFERENCES

1. Agerskov, H. : Analysis of bolted connections subject to prying. Jl of Struct. Div., ASCE, Vol. 103, 1977, pp. 2145 - 2163.
2. Bursi, O. : Tests and modelling of bolted double clip angles of steel building connections. Report Univ. of Trento, 1990.
3. Damiani, A. : Comportamento di giunti semi-rigidi in acciaio : analisi sperimentale e methodi di predizione. Ph. D. thesis, Univ. of Pavia, 1986.
4. Davison, J.B., Kirby, P.A. and Nethercot, D.A. : Rotational stiffness characteristics of steel beam-to-column connections. Jl of Constr. Steel Research, Vol. 8, 1987, pp. 17 - 54.
5. Eurocode 3 : Design of Steel Structures - Volumes 1 and 2 - Commission of European Communities, April 1990.
6. Hotz, R. : Traglastversuche für Stütze-Riegel-Verbindungen mit verbesserter Wirtschaftlichkeit. Der Stahlbau, 11, 1983, pp. 329-334.
7. Jaspart, J.P. : Etude de la semi-rigidité des noeuds poutre-colonne et son influence sur la résistance et la stabilité des ossatures en acier. Ph. D. thesis, Univ. of Liège, 1991.
8. Kishi, N., Chen, W.F., Matsuoka, K.G. and Momachi, S.G. : Moment-rotation relationship of top- and seat-angle with double angle connections. *Connections in Steel Structures.* Elsevier Appl. Sc. Publ., 1988, pp. 121-134.
9. Moore, D.B. : The design of end-plate connections. R.H. Wood Memorial Conference, Garston, Watford, 1988.

10. Zanon, P. and Zandonini, R. : Experimental analysis of end-plate connections. Connections in Steel Structures. Elsevier Appl. Sc. Publ. 1988, pp. 41-51.
11. Zoetemeijer, P. : A design method for the tension side of statically loaded bolted beam-to-column connections. Heron, Delft Univ., Vol. 20, N° 1, 1974.

ANALYSIS OF FLEXIBLY CONNECTED FRAMES UNDER NON-PROPORTIONAL LOADING

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Glenn Morris[2]

Abstract

This paper describes a procedure and a computer program for analyzing rectangular steel building frames with nonlinear beam-to- column connections, loaded by sequential, non-proportional loading systems. A Richard-Abbott function is adapted to express moment-rotation functions which simulate observed connection behavior under several cycles of load reversal. Examples illustrate the effects of connection deformation under non-proportional loading and the sensitivity of a simple structure to changes in connection properties.

1. INTRODUCTION

About 350 sets of currently useful moment-rotation (M - ϕ) data for monotonically loaded beam-to-column connections are available (Nethercott, 1990). There are only a few such data sets for common connection types under cyclic or reversed loading (Cook, 1983; Astaneh et al. 1985; Rauscher, 1989; Azizinamini, 1989).

Several types of function have been used to model the M - ϕ behavior of common connection types (Sommer, 1969; Frye and Morris, 1975; Ang and Morris, 1984; Richard and Abbott, 1975). The Richard-Abbott function is convenient and versatile for the purpose (Kishi and Chen, 1990; Attiogbe et al., 1989).

Attempts have been made to derive standardized moment-rotation functions for common connection types in terms of the geometric parameters for the connections, and to incorporate them into structural analysis computer programs (Frye and Morris, 1975; Ang and Morris, 1984). Because they incorporate M - ϕ functions for monotonically-loaded connections only, the programs accomodate only proportional loading. As structures usually carry sequential, non-proportional loading systems, analyses for proportional loading are of limited value. Accordingly, the objective of

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this investigation was to develop and program a procedure to account for the nonlinear moment-rotation behavior of the beam-to-column connections when analyzing rectangular steel frames under sequential, non-proportional loading systems.

2. MOMENT-ROTATION FUNCTIONS

In dimensionless form, the Richard-Abbott function is expressed as

$$M = S_o\phi\left[\frac{1 - \frac{S_p}{S_o}}{\left(1 + \left|\left(\frac{1}{\phi_o} - \frac{S_p}{\phi_o S_o}\right)\phi\right|^n\right)^{\frac{1}{n}}} + \frac{S_p}{S_o}\right] \tag{1}$$

It requires the four parameters M_o, ϕ_o, S_p and n to evaluate it. As illustrated in Fig. 1, it describes a family of curves that have an initial slope

$$S_o = \frac{M_o}{\phi_o} \tag{2}$$

For large values of rotation the curves are asymptotic to a line with a slope S_p, where S_p can be positive, negative or zero. The larger *the value of n in [1], the sharper the curve.*

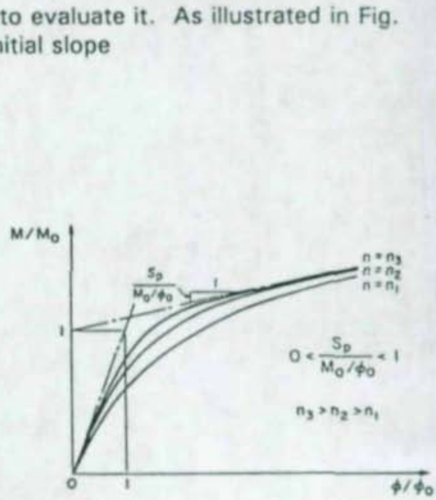
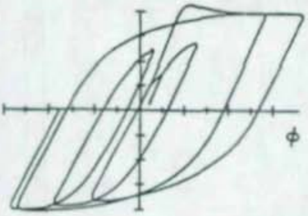


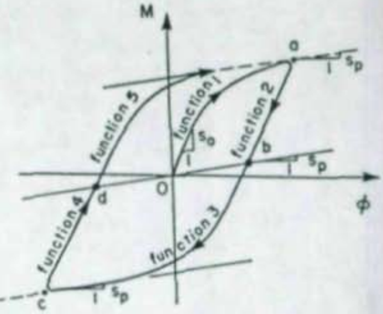
Fig. 1 - The Richard-Abbott Function

Tests of beam-to-column connections under several cycles of load reversal reveal moment-rotation behavior similar to that illustrated in Fig. 2. That behavior has been modelled by five separate functions, as illustrated in Fig. 3, all derived fom the Richard-Abbott function.

The initial loading curve is modelled by the Richard-Abbott function (function 1), as expressed by [1]. In the event of a moment reversal at point "a", the following linear relationship (function 2), with a slope of S_o, is invoked.

$$M = S_o\left(\phi - \phi_a + \frac{M_a}{S_o}\right) \tag{3}$$





Function 2 is assumed to be applicable until it intersects a line through point o with slope S_p. If the moment reversal continues, a double mirror image of the Richard-Abbott function (function 3) is applicable. It is expressed as

[4]
$$M = S_o(\phi-\phi_b)\left[\frac{1-\frac{S_p}{S_o}}{\left(1+\left|\left(\frac{1}{\phi_o}-\frac{S_p}{\phi_o S_o}\right)(\phi-\phi_b)\right|^n\right)^{\frac{1}{n}}}+\frac{S_p}{S_o}\right]+M_b$$

It is used until the moment again reverses, say at point c, after which the following linear function, (function 4) with a slope equal to the initial-loading slope, is applicable.

[5]
$$M = S_o\left(\phi-\phi_a+\phi_c+\frac{M_a}{S_o}+\frac{M_C}{s_o}\right)$$

Finally, if the moment continues to increase in the original direction, function 5, the Richard-Abbott function, displaced from inital point o to point d, is applicable. It is expressed as

[6]
$$M = S_o\phi\left[\frac{1-\frac{S_p}{S_o}}{\left(1+\left|\left(\frac{1}{\phi_o}-\frac{S_p}{\phi_o S_o}\right)\phi\right|^n\right)^{\frac{1}{n}}}+\frac{S_p}{S_o}\right]$$

3. STRUCTURAL ANALYSIS COMPUTER PROGRAM

3.1 Modeling of Structure

The analysis program treats three dimensional rectangular frames with rigid in-plane floor diaphragms. The structure is modelled as an assemblage of elastic beams, columns and bracing elements connected by nonlinear beam-to-column connections. Geometric nonlinearity of the structure and its members is accomodated.

The nonlinear moment-rotation behavior of the connections is represented by the five functions described above. Direct shearing and torsional deformations are ignored, as are deformation along the beam and rotational deformation about a vertical axis.

The connection moment-rotation behavior can be generated in either of two ways. For bolted-bolted double web angle connections, functions based on test data and expressed in terms of the connection geometric parameters (angle thickness and depth, bolt gages, etc.), have been incorporated. Similar functions for other connection types are being developed. Using them, the connection geometric parameters are input and the moment-rotation functions are generated automatically. Alternatively, experimental moment-rotation data for any connection can be input to an auxiliary program which generates the Richard-Abbott constants, M_o, ϕ_o, S_p and n. The constants are then input to the analysis program which uses them to generate the moment-rotation functions.

3.2 Iterative Analysis Procedure

Sequential, non-proportional load systems are accomodated. The premise of the iterative analysis procedure is that for a given load system, the correct deflection and internal force increments are obtained from a linear analysis provided appropriate connection stiffnesses are used. For a given load system, repeated cycles of linear analysis and connection stiffness modification are performed in order to arrive at those connection stiffnesses. The procedure is illustrated using the M - ϕ diagram for a typical connection, shown in Fig. 4.

The initial tangent stiffness, S_1, is first assumed for the connection, and all other connections are similarly linearized. A linear analysis is performed, predicting a connection moment M_1 and a rotational deformation ϕ_1. The nonlinear M-ϕ function for the connection is then used to compute ϕ'_1 and stiffness S_2, which gives a better approximation to the nonlinear M - ϕ relationship. Similar computations are performed for all other connections. Repeated cycles of linear analysis and stiffness modification are performed until a linear analysis predicts M and ϕ values at all connections that correspond approximately to points on their nonlinear moment-rotation curves.

Each time a new load system is applied, it is necessary, for each connection, to select the appropriate M - ϕ function from the five described above. The selection procedure is described using the M - ϕ diagram for a typical connection, shown in Fig. 3.

When load system 1 is applied, function 1 is selected. Repeated cycles of analysis and stiffness modification are performed until the moments and rotational deformations at all connections correspond to points on their M - ϕ curves. Assume that for the connection considered that point is a in the figure. The values M_a and ϕ_a are stored, with those for all other connections, as the cumulative M and ϕ values.

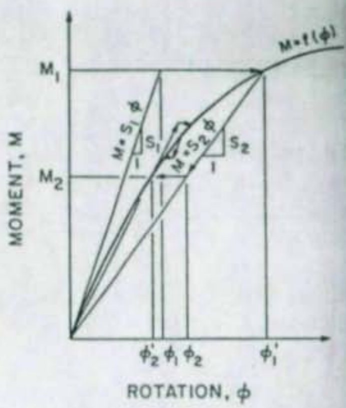


Fig. 4 - Iterative Analysis Procedure

With the application of the second load system, a linear analysis is performed using the secant stiffness for point a, and moment increments are computed at all connections. If the sign of the moment at the connection matches that of the M_a, function 1 is retained. Otherwise, function 2, with its linear stiffness S_a, is invoked. As loading 2 is applied, it is necessary to test whether or not the cumulative moment drops below M_b, in which case function 3 is invoked. Assuming that loading 2 produces moment M_c and rotation ϕ_c, corresponding to point c, they are stored as the cumulative values and the next load system is applied. The secant stiffness for point c is used in a new linear analysis and the sign of the moment increment is tested against that of M_c. Then either function 3 is retained or function 4 is invoked.

4. APPLICATIONS

For simplicity, examples involving only planar frames are presented.

Example 1

The frame in Fig. 5 (a) was analyzed under uniformly distributed gravity load, then with superimposed wind load and finally with the wind load negated by a load in the opposite direction. Although flexible for this application, bolted double web angle connections to the column flanges were assumed. They would be at the boundary between flexible and semi-rigid in the connection classification system proposed by Bjorhovde et al. (1990).

The beam moment diagrams and drifts at roof level following successive loadings are shown in Fig. 5(b). They demonstrate that the static span moment of 375 kN m was maintained. Following the application and removal of the horizontal load the moment diagram was symmetrical, although the "softening" of the right connection is evidenced by the residual drift of 10 mm to the right.

Shown also are the bending moment diagram and lateral drift for loadings 1 and 2 applied simultaneously. The right connection moment under sequential loading was 13 percent larger and the drift 33 percent smaller than those under simultaneous loading. Fig 5(c) illustrates that connection moments and rotational deformations at each stage corresponded to points on their moment-rotation diagrams.

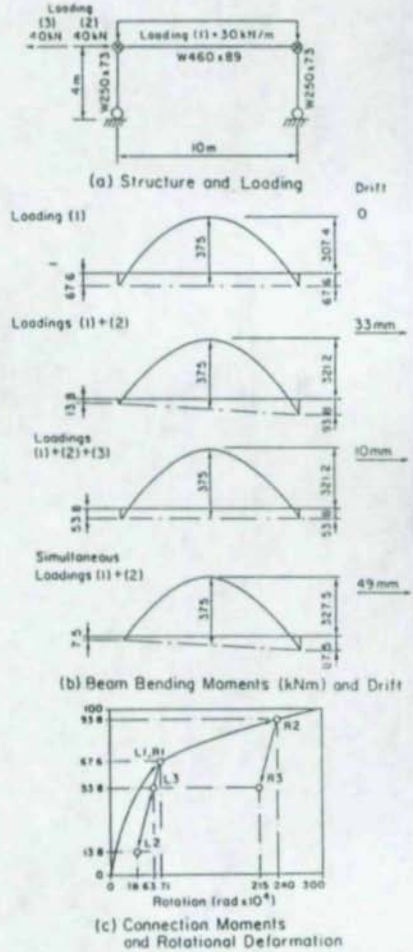


Fig. 5 - Example 1

Example 2

To test the sensitivity of a simple structure to variations in connection moment-rotation behavior, a structure similar to that in Example 1 was analyzed repeatedly. Connections represented by Richard-Abbott functions with several values each of: moment M_o, initial stiffness (M_o / ϕ_o), "strain hardening stiffness" S_p, and parameter n, were assumed. The gravity load was 45 kN/m and the wind loads were 50 kN.

Table 1 shows the connection parameters, the corresponding beam end and mid-span moments and the lateral drifts for the analyses where M_o and the initial stiffness were varied. For this structure, wide variations in S_p and n affected the analysis results by less than 10 percent.

As M_o, which defines the "knee" on the M - ϕ curve, was increased from 160 to 800 kN m, the structural behavior changed from nonlinear to linear. At the lower value, the midspan moment increased by 8 percent as a result of lateral loading, and the permanent drift was 22 mm. Both values decreased to zero when M_o was 800. The effect of variation in the initial stiffness was very significant. The beam end moments decreased by 40 percent, the drift under horizontal load increased by 50 percent and the permanent drift decreased from 12 to 2 mm as the initial stiffness decreased from that corresponding to the middle of the "rigid" range to that on the boundary between "semi-rigid" and "flexible".

The structure, with rigid beam-to-column connections, was analysed under gravity load, first with the correct column I values, then with one half of those I values. The 50 percent reduction in column bending stiffness caused a 28 percent reduction in the right beam end moment and a 74 percent increase in drift. Thus the effect of the connection properties on structural behavior was comparable to that of member properties.

S_o = 24, 000 kN m,		S_p = 1,000 kN m,			n = 4
Loading System(s)	M_o (kN m) Location	160	240	500	800
(1)	Left	136.6	140.8	141.6	141.6
	Centre	425.9	421.7	420.9	420.9
	Right	136.6	140.8	141.6	141.6
	Drift (mm)	0	0	0	0
(1)+(2)	Left	2.4	36.2	41.5	41.6
	Centre	460.1	426.3	421.0	420.9
	Right	202.4	236.2	241.5	241.6
	Drift (mm)	70	51	40	37
(1)+(2)+ (3)	Left	102.4	136.2	141.5	141.6
	Centre	460.1	426.3	421.0	420.9
	Right	102.4	136.2	141.5	141.6
	Drift (mm)	22	3	0	0

Mo = 240 kN m,			S_p = 1,000 kN m,		n = 4
Loading System(s)	$Ø_o$ (rad) Location	.0005	.001	.005	.010	0.020
(1)	Left	187.2	183.8	161.4	140.8	112.3
	Centre	375.3	378.7	401.9	421.7	450.2
	Right	187.2	183.8	161.4	140.8	112.3
	Drift (mm)	0	0	0	0	0
(1)+(2)	Left	62.1	60.6	51.6	36.2	10.0
	Centre	400.4	401.9	410.9	426.3	452.6
	Right	262.1	260.6	251.6	236.2	210.0
	Drift (mm)	44	45	46	51	67
(1)+(2)+(3)	Left	162.1	160.6	151.6	136.2	110.0
	Centre	400.4	401.9	410.9	426.3	452.6
	Right	162.1	160.6	151.6	136.2	110.0
	Drift (mm)	12	11	5	3	2

Table 1 - Beam End Moments and Frame Drifts, Example 2

6. REFERENCES

Astaneh, A., Nader, M. and Malik, L., 1989, "Cyclic Behavior of Double Web Angle Connections", Journal of Structural Engineering, Vol. 115, No. 5, pp. 1101-1118.

Ang, K.M. and Morris, G.A., 1984, "Analysis of Three-Dimensional Frames with Flexible Beam-Column Connections", Canadian Journal of Civil Engineering, Vol. 11, No. 2, pp. 245-254.

Attiogbe, Emmanuel, Morris, Glenn and Pinkney, Bruce, 1989, "Modelling Functions for Moment-Rotation Curves", Proceedings of the Twelfth Canadian Congress of Applied Mechanics, Carleton University, Ottawa, Canada.

Azizinamini, A., Bradburn, J.H. and Radziminski, J.B., 1985, "Static and Cyclic Behaviour of Semi-Rigid Steel Beam-Column Connections", Structural Research Studies, Department of Civil Engineering, University of South Carolina, Columbia, SC, 219 p.

Bjorhovde, R., Colson, A. and Brozetti, J., 1990, "Classification System for Beam-to-Column Connections", Journal of Struct. Eng., Vol. 116, No. 11, pp. 3059-3076.

Cook, N.E. Jr., 1983, "Strength and Stiffness of Type 2 Steel Frames", Report to AISC, Department of Civil, Environmental and Architectural Engineering, University of Colorado, Boulder, CO.

Frye, M.J. and Morris, G.A., 1975, "Analysis of Flexibly Connected Steel Frames", Canadian Journal of Civil Engineering, Vol. 2, No. 3, pp. 280-291.

Kishi, N., and Chen, Wai-Fah, 1990, "Moment-Rotation Relations of Semirigid Connections with Angles", Journal of Structural Engineering, Vol. 116, No. 7, pp. 1813-1834.

Nethercott, D.A., 1990, "The Influence of Semi-Rigid Joint Action on the Behaviour of Steel Frame Structures", Proceedings of the Forth Rail Bridge Conference, Edinburgh, pp. 696-721.

Rauscher, T.R., 1989, "Reliability of Double Web Angle Connection Behavior", Structural Research Series 8904, Department of Civil, Environmental and Architectural Engineering, University of Colorado, Boulder, CO

Richard, R.M. and Abbott, B.J., 1975, "Versatile Elastic-Plastic Stress-Strain Formula", Journal of the Engineering Mechanics Division, ASCE, Vol. 101, No. EM4, pp. 511-515.

Sommer, W.H., 1969, "Behaviour of Welded Header Plate Connections", Master of Applied Science thesis, University of Toronto, Toronto, Ontario

MOMENT-ROTATION CHARACTERISTICS OF BOLTED CONNECTIONS

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Abstract

This paper presents the main findings concerning design procedures for moment-rotation relationships for bolted connections of any structural form, taking into account column flexibility. A comparison of theoretical and experimental results for common bolted connections in the USSR is also given.

1. INTRODUCTION

The study presented in this paper was conducted at the steel construction research institute, Promstalconstructsiya. It included the following:

1. Development of design methods for the strength of bolted joints: end-plate, friction, bearing and friction-bearing.

2. Development of design methods for the moment-rotation (M-α) relationship of bolted joints. It is noted that a bolted joint is considered as an assembly that consists of the bolted connection and the adjacent part of the column.

3. Development of a program for the design of multistory frames, taking into account the actual rigidity of the bolted joints.

4. Experimental investigations of a full-size multistory steel frame.

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5. Development of a design specification for steel multistory frames, taking into account actual joint rigidity.

During the development of strength design methods for bolted connections (Kalyonov et al., 1989, 1990), it was found that the governing magnitude of external forces that act on a connection for a given amount of reliability is deformation controlled. In other words, the strength limit state is interconnected with the serviceability limit state. For example, for end-plate connections with pretension bolts, the criterion is the separation of the end-plate from the column in the bolt zone. For bearing and slip-resistant connections the limiting displacement is a shear-type, mainly consisting of three components: the difference between the hole and the bolt diameters, the mostly elastic displacement associated with shear and bending of the body of the bolt, and the elastic-plastic bearing displacement of the connected elements.

2. END-PLATE CONNECTIONS IN FRAMES

As part of the research effort, the computer program FLORA was developed for IBM PC compatible computers. It was used to analyze the strength and deformation characteristics of the connections, including the M-α-relationship of end-plate connections with and without pretensioned bolts.

The program considers the following influences:

(1) elastic behavior of end-plates, bolts, column flange, web and stiffeners

(2) contact interaction between the end-plate and the column flange

(3) geometrical non-linearity, caused by combined bending and compression in the column flange.

The computer program was based on a stiffness type finite element solution. Triangular and rectangular plane shell elements were used, recognizing plane stress in thin-walled structures. Thus, the column flange and end-plate represent two (2) independent regions of the connection, and these were discretized into finite elements. The bolts were modeled as internal links between the nodes of the finite elements in the column flange and the end-plate.

The column flange and its stiffeners were modeled as elastic links along the line connecting the column flange with the web and the stiffener. The stiffness of these elastic links was found by the computer program ATAS, described in the following. The external links of the finite element model could be set as perfectly rigid or flexible. This allowed for modeling of different boundary conditions for the end-plate and the column flange.

The FLORA program gives the following output:

(1) values of normal and tangent stresses, displacements along the X, Y and Z, and rotations with respect to X and Y for every node of the finite element mesh

(2) bolt forces

(3) moment-rotation relationship for the connection, in tabular and graphic form.

The program was developed to allow for the determination of the strength and deformation data for the entire joint, as well as for each of the individual elements (end-plate, bolts, flange, web and stiffeners of column).

Comparisons between the numerical results of FLORA and ATAS with numerous experimental data for end-plate connections have given satisfactory agreement. For example, the results for a connection between an IPE A 550 beam and an HE 300 A column are shown in Fig. 1, demonstrating good correlation through the ultimate limit state. Analysis of the displacements at the upper flange level of the beam showed that for connections with column web stiffeners and pretensioned bolts, the deformations of the end-plate are about 50% of the total, and the tension displacements of the column web are 2 to 3 times the bending displacements of the column flange. For connections without column stiffeners and pretensioned bolts, the end-plate displacements are about 30% of the total displacements, and the tension displacements of the column web are one third to one fourth of the bending displacements of the column flange.

For the same connections, but without pretensioned bolts, the end-plate displacements are about 85 to 90% of the total for a column with stiffeners, and about 55 to 60% for the case of no stiffeners. At the level of the bottom flange of the beam, the displacements of the end-plate and the column web essentially coincide during the initial stages of loading. In the following loading stages, the end-plate displacements are 1.5 to 1.8 times larger than those of the column web.

3. CONNECTIONS WITH WEB CONNECTED PLATES

This type of connection has been widely used in the USSR, and many other forms of connections are variants of this kind.

Displacements in such connections may be related to any of the following responses:

1. Bending of the beam web and the connected plates without mutual shear of the connected elements.

2. Mutual shear of the beam and the connected plates, including bearing

displacements of the beam web and plates, and shear and bending displacements of the bolts.

The common approach consists of separate evaluations of the connection behavior, with and without mutual shear. The deformations of the connection combines the displacements of various components.

The program ATAS (Berdichevsky et al., 1990) was used to examine the strength and deformation of the beam web and plates. This program can also be used to solve one and two-dimensional linear and nonlinear problems of structural mechanics and theory of elasticity.

The present version of ATAS provides solutions of the following:

(1) linear problems:

- (a) beam bending for any load, any number of supports, and any other physical characteristics
- (b) beam with bending and shear
- (c) tension member
- (d) in-plane problems of theory of elasticity
- (e) plate bending

(2) nonlinear problems:

- (a) in-plane problems of theory of plasticity for small deformations
- (b) mutual bending of two beams in one-sided contact
- (c) in-plane problems of theory of elasticity for large deformations

The output data of ATAS are displacements at the nodes of the finite element mesh, and also values of other key characteristics, such as normal, tangent and principal stresses, deformations, and forces.

The design scheme for a connection with web connected plates, in accordance with program ATAS, is shown in Fig. 2a. The stress-strain state of the elements is that of an in-plane problem of theory of plasticity with small deformations. The applied load can represent any combination of moment and lateral force.

As an example, the connection for which the M-α-relationship (without considering mutual shear of beam and plates) will be determined is illustrated in Fig. 3. Fig. 4 shows the deformed finite element mesh at the limit load of 1200 kN. Shear yielding has taken place in the cross-hatched elements, and the moment-rotation relationship is shown in Fig. 5.

In general, the development of an M-α-diagram for a multi-bolt connection, as shown schematically in Fig. 2b, reflects a statically indeterminate problem with nonlinear relationship between the shear forces and displacements of every bolt.

The following assumptions were used in the development of the solution:

(1) Displacements and rotations of adjacent parts of column, beam web and connected plates caused by bending are equal to 0

(2) The center of rotation of the connection coincides with the center of symmetry of the bolt group

(3) The forces acting on every bolt in connection and the corresponding displacements are oriented perpendicular to the line that connects the center of the bolt hole with the center of rotation

(4) The displacements at the points where the bolts are in contact with the hole material are directly proportional to the distance from center of gravity of the bolt group to the center of rotation.

Some representative values of connection rotations can be found on the basis of the fourth assumption. Thus, the displacements Δ will appear at the contact points of each bolt with the hole material. Consequently,

For bolt i:

$$\Delta_i = \alpha\, r_i \tag{1}$$

Using the calculated values of Δ_i for each bolt the individual bolt forces N_i can be found.

The contact point displacements consist of bearing displacements in the beam web and the connected plates, and shear and bending displacements of the bolts. A special feature of the program algorithm is that the N_i-Δ_i relationship for each deformed element is piecewise linear; it is constructed on the basis of numerous experimental data (Karmalin and Pavlov, 1989). Adding the displacements Δ for a constant force, N, gives the relationship between N and displacement at the contact point. The values are multiplied by the distance from the center of the bolt hole to the center of rotation. The moment developed by each bolt is then found as

$$M_i = N_i\, r_i \tag{2}$$

and the total moment is computed from

$$M_t = \Sigma M_i = f(\alpha) \qquad (3)$$

This procedure is then repeated for any number of given values of the rotation. The value of the moment is calculated for each of them, after which the M-α-curve can be constructed.

Figure 5 shows the moment-rotation relationship for the connection shown in Fig. 3. This takes into account mutual shear of the beam and the connected plates for the section through the center of gravity of the bolt group.

The M-α-curves for the cases without and with mutual shear of the elements need to be added, along α-axis, to find the final moment-rotation relationship. In this context, it is possible to regard the bending deformation of the connected plates and the beam webs to be an order of magnitude smaller than the deformations associated with mutual shear of beam and plates. It is therefore possible to approximate the connection behavior as the behavior of a rigid-plastic body.

The moment-rotation relationship and the experimental M-α-curve for the same specimen is shown in Fig. 5. The comparison demonstrates satisfactory correlation between the theoretical and the experimental results.

4. SUMMARY

Software has been developed and tested to construct moment-rotation-relationships *not only for the bolted connections presented here, but also for connections of any* form. It is hoped that numerous connections for multistory steel structures can be analyzed with the help of this program.

5. REFERENCES

1. Berdichevsky, S., Gambarov, A., and Kudriavtsev, L., "Universal FEM Algorithm for Local Stiffness Matrix by the Newton-Raphson Method", Moscow, USSR, dep. 11.11.90, 5655-B90.

2. Kalyonov, V., Glauberman, V., Martynchuk, V., and Soskin, A., "Guide to Design, Manufacture and Erection of Steel Structures with End-Plate Connections", Moscow, 1989.

3. Kalyonov, V., Karmalin, V., Pavlov, A., and Streletskij, N., "Guide to Design of Bolted Connections Subjected to Shear", Moscow, 1990.

4. Karmalin, V., and Pavlov, A., "Load Capacity and Deformability of Bearing and Friction-Bearing Connections", Proceedings, IABSE International Colloquium on Bolted and Special Structural Connections, Volume 1, Moscow, 1989.

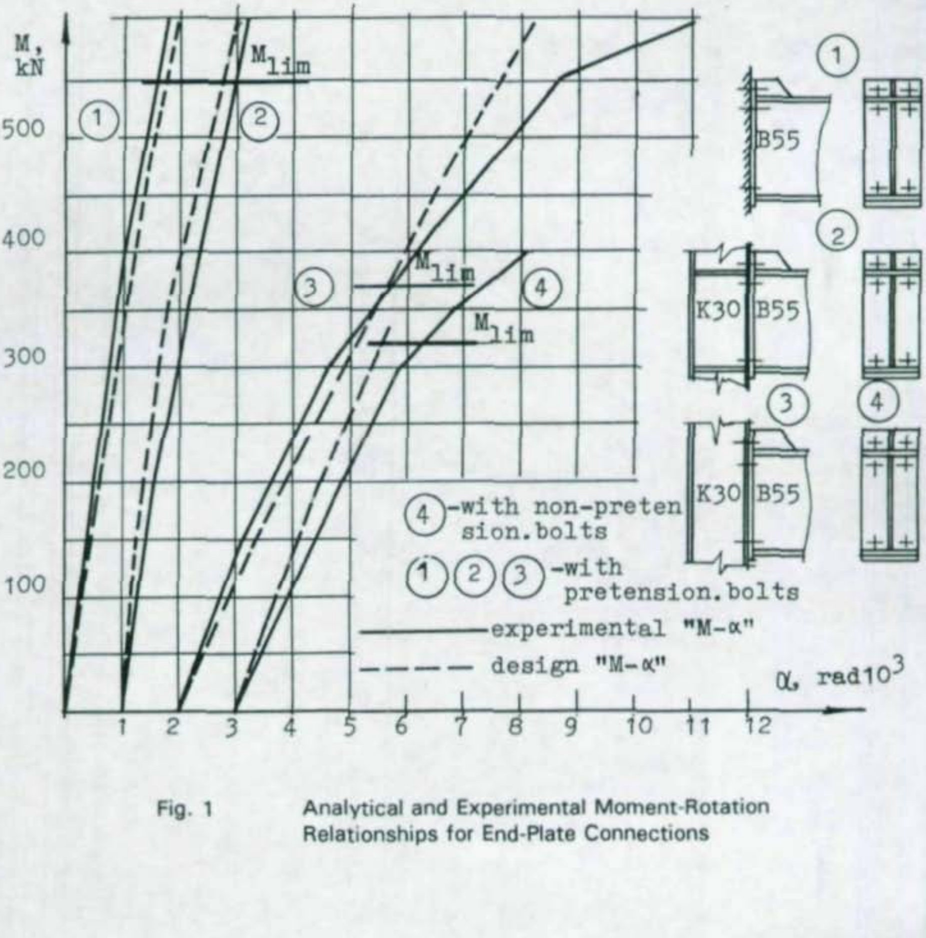

Fig. 1 Analytical and Experimental Moment-Rotation Relationships for End-Plate Connections

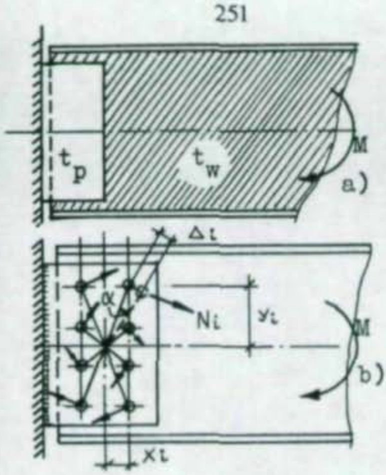

Fig. 2 Notation and Analytical Approach for Connections with Web-Connected Plate(s)

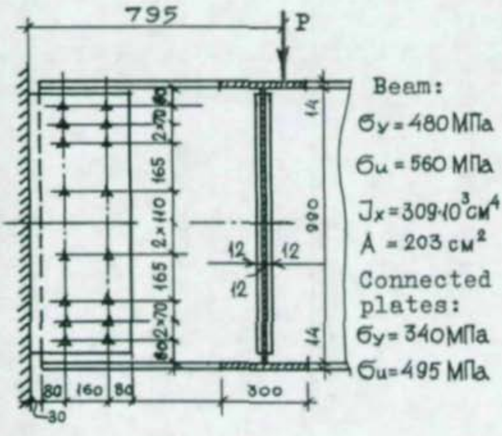

Fig. 3 Connection Example

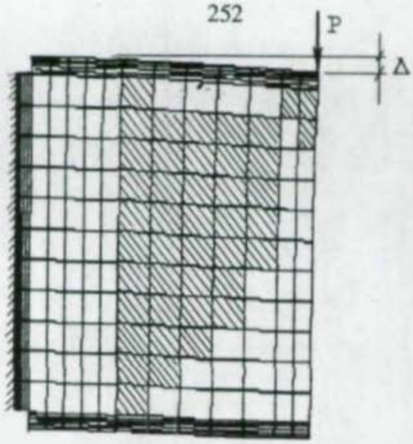

Fig. 4 Deformed Finite Element Mesh at the Limit Load of 1,200 kN

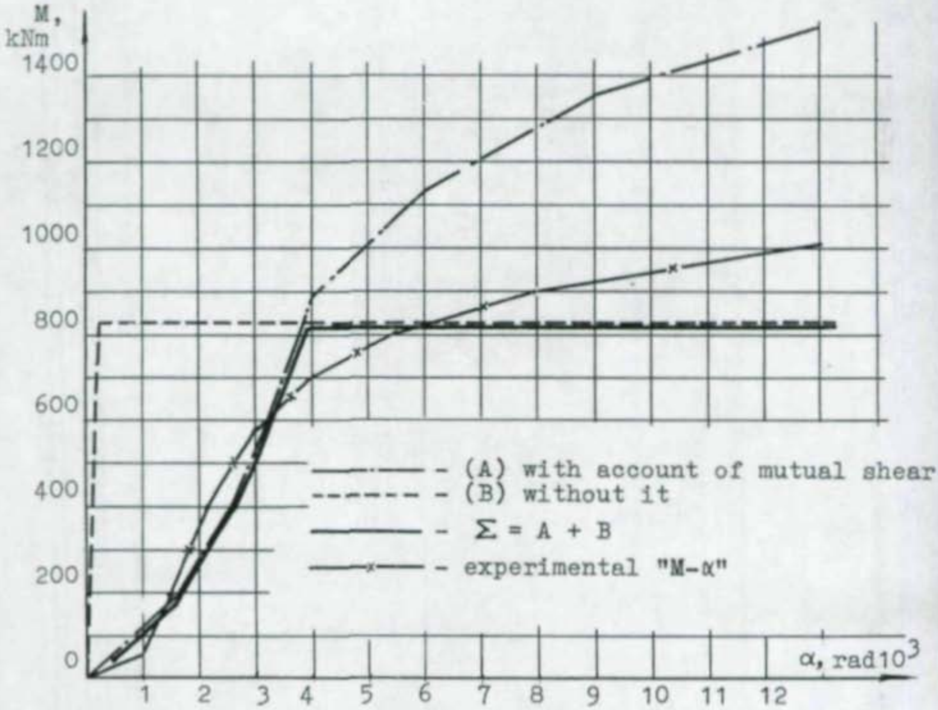

Fig. 5 Analytical and Experimental Moment-Rotation Relationships for Connections with Web-Connected Plates

Technical Papers on

AVAILABLE CONNECTION SOFTWARE

DESIGN OF CONNECTION IN THE EUREKA "CIMSTEEL" PROJECT

Jacques BROZZETTI*

Abstract

The EUREKA "CIMSTEEL" project is a collaborative venture of 37 Companies from six European countries (Denmark, Finland, France, Italy, Netherlands and United Kingdom) aiming at introducing Computer Integrated Manufacturing (CIM) techniques within the constructional steelwork industry. The integration covering the various aspects of the total product life cycle from conceptual design up to and including steelwork erection and maintenance. The first part of the paper presents briefly the project work achievement starting from the framework for steelwork enterprise modelling, then the Logical Product Model (LPM) and finally the Target System Architecture (TSA) of the CIM. In a second part, the Connection Design and detailing activity, using EUROCODE 3, is presented as part of a computer application module of the CIM system.

1. "CIMSTEEL" BACKGROUND

1.1 What is the EUREKA "CIMSTEEL" project ?

The EUREKA "CIMSTEEL" project is to advice and to develop the most suited methods and technologies for incorporating Computer Integrated Manufacturing (CIM) techniques and systems in relation with management, design and manufacturing activities within the European steelwork industry.

The "CIMSTEEL" project is not intended for developing a common European stand alone computer package unit dealing with the design activity fabrication and management process of a steelwork enterprise.

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The participating European countries involved in the project have, in the context of the market for steel across Europe defined the main objectives of the project as :

- to make recommendations on common methodology and models for the use of steelwork enterprise in their development, and on the implementation of computer aided information systems adapted to specialized area of application,
- to ensure that information or data can be interchanged between systems and companies most efficiently, unambiguously and with reliability,
- to develop computer software applications which have been identify as an urgent need, or filling the current gap, as for example, the development of Connection design and detailing software modules based on the recently edited 5th issue of the EUROCODE 3.

The scope of this project is very broad and necessarily bound by economical and time resources, and available technologies. Within these limitations, a great progress has being achieved since the project started in mid 1987.

Focussing at three main aspects, a brief overview of the methodology used and which outline the progress made will be presented in the following.

1.2 The CIM system as a result of a Product Modelling

The Modelling is the key element of the framework of the CIM system development and provides an understanding of the functional requirements of the different phases of a Construction project, and it also provides a logical structure of the application models needed during the different phases of production.

a) Framework for steelwork enterprise modelling

Computer Integrated Manufacturing (CIM) system requires that all functions, activities, data, resources are put in a proper relationship to each other. This is referred as the "Functional Models".

Functional models of the steelwork industry activities in the various partner countries have been elaborated, representing the industry to-day (model "AS-IS") and anticipating the industry of to-morrow (model "SHOULD-BE"). The second model was an extension of the "AS-IS" model, generally slightly modified and improved to take into account a more rational analysis of the organizational structure, or an anticipation of technical and technological changes.

The majority of the national models established concerned the steel buildings application industry, which represents the most challenging market sector in Europe.

All these "national industry models" were modelled using $IDEF_0$ or SADT (Structural Analysis and Design Technique) methodologies which were set as a prerequisite to any modelling representation in order that each partner is able to communicate unambiguously.

Tentative for a Generic European Functional Model was issued, still at an informative stage, since through Europe the enterprise views are different, and cover various steel construction aspects which are application dependent and make difficult the adoption of a common standard industry model.

The enterprise framework definition is an essential preliminary step from which an integrated computer processable model can be elaborated and developed taking into account the entire construction process.

b) Logical Product Model (LPM)

A Logical Product Model (or a data model) is a formal abstract representation of how a given set of information can be defined within an integrated computer system.

The benefit expected from such standard data model description is the potential possibility of transferring product information between CIMSTEEL systems in a comprehensive and consistent manner.

A third release of a complete Logical Product Model has been achieved by the working group experts in charge of the task. This LPM-3.1 has been defined using, as graphical modelling language, $IDEF1_x$* and then converted into a computer oriented definition language called EXPRESS. The LPM is in compliance with the general AEC reference model (which is a discipline oriented model for Architecture, Engineering and Construction).

The working group of experts in charge of the Logical Product Model definition makes its development in conformity with the ongoing elaboration of STEP specifications and requirements (STEP : Standard for the Exchange of Product Model Data) as worked out under the ISO/TC 184/SC4.

c) Target System Architecture (TSA)

A common agreement, among partners was made necessary in order to define a long term overall Architecture of the CIM system. This long term product definition eases the co-ordination and the development of computer application modules, and authorizes the plugging conformability into the system of any exchangeable application modules via Product Model File technology. However, it is not possible to move directly from current existing computer application system to the general Target System Architecture as it has been conceptually defined (fig. 1). In order to facilitate the progressive implementation of the Target System Architecture a step by step flexible implementation has been defined which encompasses a number of integration levels, from the stand alone system, the interfaced system through STEP translator technology or PM file, and to the local integration, of specific application system (fig.2).

* IDEF, ICAM DEFinitional Methods published under ICAM Program of the US Air Force

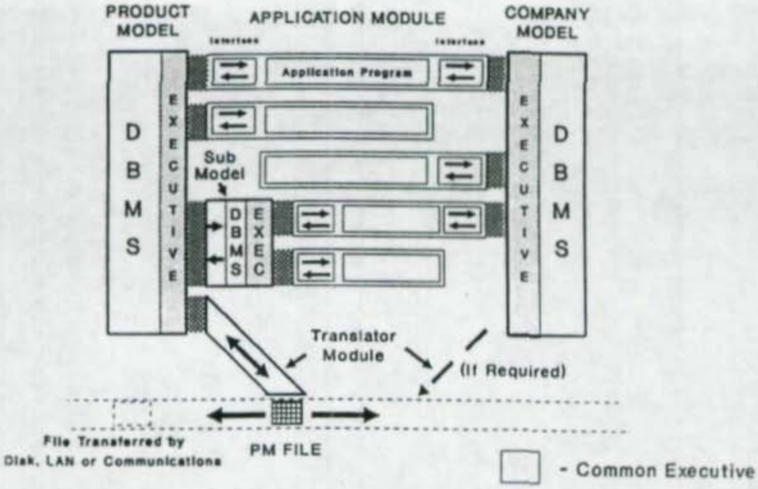


Figure 1 - Conceptual level of the Target System Architecture

The Target System Architecture is therefore the integrated CISMTEEL system which is configured by plugging in exchangeable Software Modules. The Translator Modules allow information to be imported and exported to other systems (local and remote) via STEP files and appropriate communications network.

Presently the work is focussed on the definition of functional blocks mainly concerned with the design and manufacturing activities.

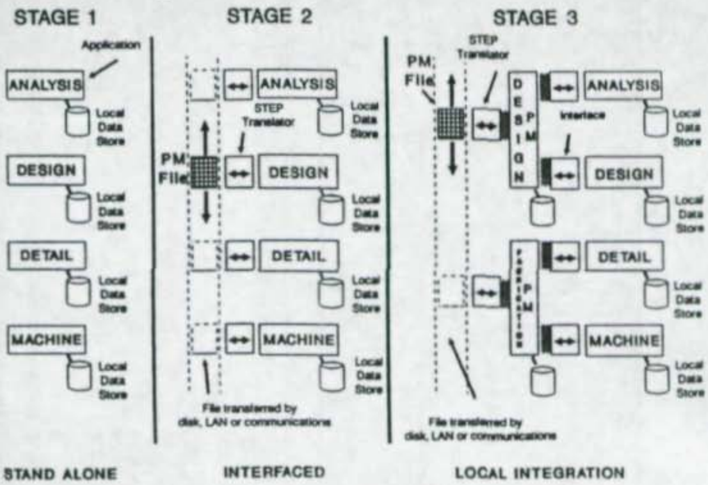


Figure 2 - Progressive implementation stages of CIMSTEEL

2. "CIMSTEEL" CONNECTIONS DESIGN AND DETAILING

Among the design and the detailing tasks, those concerning connections require much attention, and very often they are the key point for economy of the project. However, this application is faced with the emergence of The EUROCODE 3, for which it exists little experience, and no available software tools. These are the reasons the connections design and detailing CIMSTEEL application were considered, within the framework of the European collaboration as a first priority item. The development concerned with this item is presented in the following considering a modelling oriented view point and a structural oriented view point.

2.1 A logical application product model view point

Following the same modelling framework methodology as used for defining the organizational activities and information flows which are experienced during the conception, manufacturing and management phases of steel structures, throughout their entire life cycle, specific functional and data models were elaborated which relate to the design and detailing applications.

Two application classes have been analyzed, the Structural Design application and the Connections Design and Detailing. Figures 3 and 4 represent the general SADT diagrams of both specific applications and show the interrelation that exists between them. This interrelation being underlined by the analysis function, and by the structural model generation function.

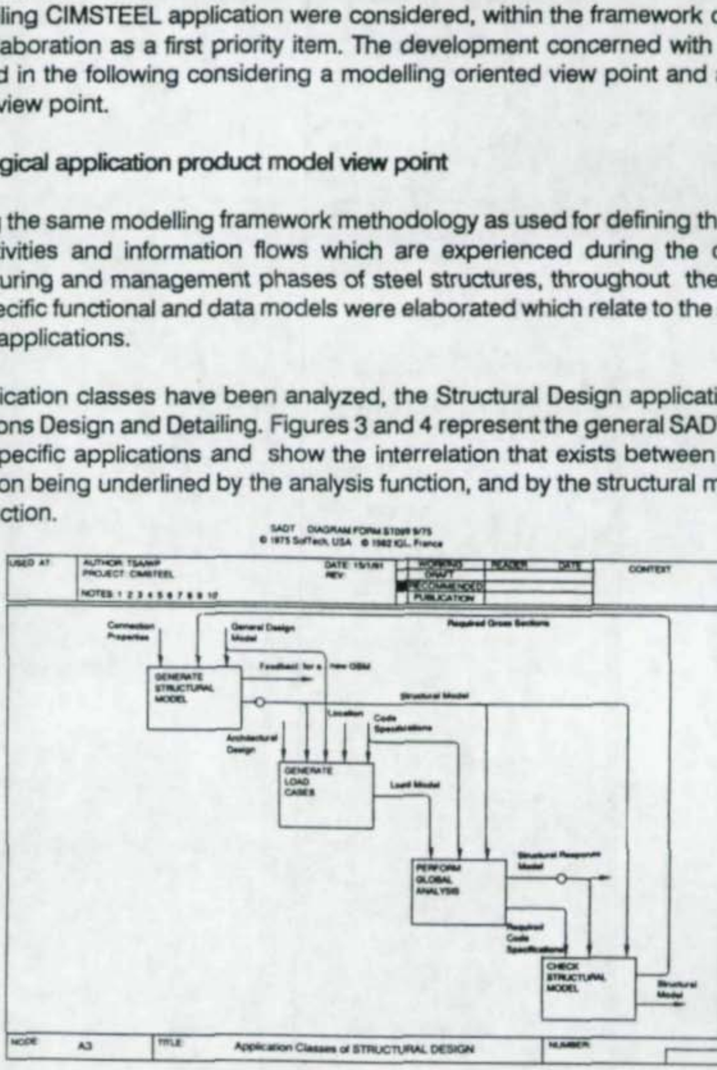


Figure 3 - SADT diagram of Structural Design

What is important to notice about these diagrams is that there are consistent with the overall schema of the CIMSTEEL model. Drawing the attention on figure 4, the three main functions are the design of connection, the checking of the connection according to code requirements (EUROCODE 3), the mapping to the general structural design model.

One has to understand that the main objective and originality of the presented functional models is the dimensionning of a connection "node" which involves the following sequential steps.

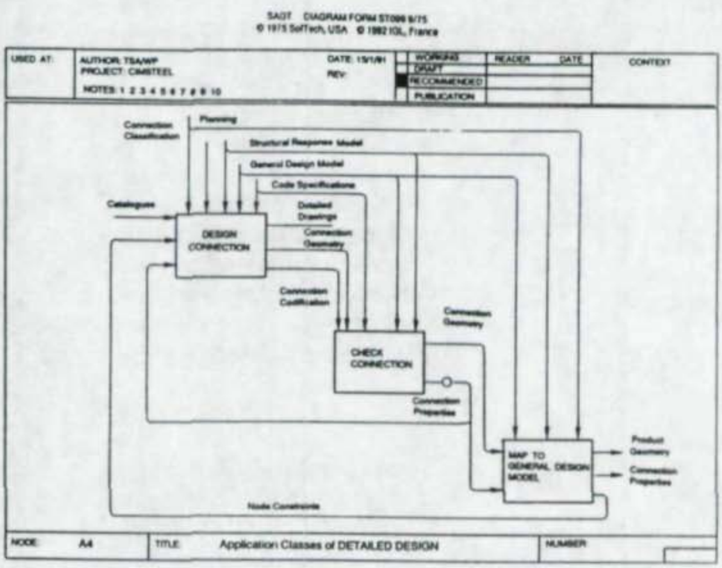
<u>Step 1</u> : perform the primary modelisation of the steel frame structure by a wire frame model

<u>Step 2</u> : choose the node, and affect to it the geometry and the spatial topology of the attached member

<u>Step 3</u> : select the type of connection to be designed and detailed through a codification system. One conception type of connection will be chosen, and their individual connecting elements (cleat, stiffener, fastener, weld . . .) will be automatically sized to fulfil strength, geometry, fabrication and erection rules or requirements

<u>Step 4</u> : validate the connection so dimensionned by undertaking a code checking procedure, i.e. a procedure by which all requirements according to EUROCODE 3 are shown to be satisfied

<u>Step 5</u> : transfer the data, to a CAD system, to issue production and manufacturing drawings.



<u>Figure 4</u> - SADT diagram of Connections Design

For the time being, the connection design and detailing model is thought in term of simple design methods referring to flexible and rigid connections (shear or moment type connection). In such base there is a complete independancy between the structural analysis phase and the design and detailing phase of the connection.

However, the allowance of semi-rigid partial strength type connection as authorized by EUROCODE3 introduces a dependancy between the determination of the moment, rotation characteristic of the connection and the evaluation of the frame structural response which adds a real complexity to the design. This leads to an iterative process between the evaluation of the load effect in the structure (which depends upon the load behaviour of the connection) and the determination of the semi-rigid connection properties (stiffness and strength) which in turn depends upon the forces resulting from the frame analysis. Concerning the design and detailing of semi-rigid connection, the complete schema of the design process is not yet fully clarified, and attention to this problem will be addressed in a later development stage of the project.

2.2 Structural oriented view point

Among the activities programme of the CIMSTEEL project, the production of software tools for the design and detailing of connections, according to EUROCODE 3, is one of the major task. The software packages will be formed by several independent partners. In order to co-ordinate the work, to ease the information transfer, and to assure a proper quality assurance of the software products, a common operational development methodology has been agreed upon, and a list of work items has been set out which includes the following points.

a) Codification system for connections

Within the CIMSTEEL functional model context the implementation of a logical codification system of connections was made necessary, in order to allocate for a selected node of a structure the proper family of connections which satisfies the desired functionalities. This codification system which is informatic oriented, is based on a coded definition of a number of parameters implemented in the Product Model (members profil, position of the connection, relative position of members, secondary elementary parts used in the connection, type of fasteners, type of loads to be transmitted, stiffness mode).

b) The inventory of connection

It is a survey of the most useful type of connections in use in steel structures in various European countries. The aim of this inventory work is to classify into the relevant families the inventoried connections, and to label them with the proper codification system derived. This also serves the purpose of distributing the task of elaborating the procedures guide among various partners.

c) Elaboration of the procedures guide

For each connection family a procedures guide is established which details step by step the design methodology, the data organization, the information flux needed in order to dimension automatically the connection for a given limited set of basic information (joint loads, geometry and topology of the attached members). This constitutes the knowledge base of the computer aided design and detailing of connections; This knowledge base is a set of various design or practice rules, functional requirements and facts, which are necessary to produce a conceptual basic solution satisfying not only geometry but also strength requirements. To resolve the dimensionning of a connection, two methods have been investigated, an informatic procedural approach founded on the derivation of the sequential set of rules, and an expert system approach.

d) Specification of product library files or product data base

Connections design and detailing applications would typically use a system dependent implementation of librairies. However the product data (profil, bolts . . .) has to be made available in an implementation-independent format e.g. a sequential ASCII files in order to be used by several stand alone softwares.

e) Development of connections design and detailing software modules

This is the ultimate step of the work which is, for the moment, at a preliminary stage of development.

3. CONCLUSIONS

CIMSTEEL as to be understood as an open methodology, using the current specifications of the ISO-STEP standards, to achieve an integration of heterogeneous systems (softwares and hardwares) rather than developing a stand alone integrated system.

The structure of data exchanges between applications, the incremental implementation of these exchanges and the step by step progressive implementation of the integration process have been carefully studied.

The feasibility of the CIMSTEEL concept has been proved to be successful by the realization of two demonstration operations carried out on existing heterogeneous computer softwares from various origins performing a specific task as for examples wireframe modelling, analysis, design and drawings.

4. BIBLIOGRAPHY

[1] Rapports Phase I, EU 130 CIMSTEEL (fascicules 1, 2, 5) EU ISO/FRCC, CTICM/LGCH, juin 1989

[2] EUREKA PROJECT 130
Computer Integrated Manufacturing for Constructional Steelwork including Expert Systems
Phase I Report, The International Task Group, November 1988

[3] A.F. DECELLE
"ESTIME" un système de CAO destiné à l'évaluation de structures industrielles à ossature métallique
Thèse de Doctorat, Université de Savoie, février 1990

[4] F. BOERO
EUREKA EU 130, CIMSTEEL Project - Target System Architecture Proposal ITALSIEL, September 1990

[5] EUREKA EU 130, CIMSTEEL
Logical Product Model for Structural Steelwork, Version 3.1
LPM Working Group, December 1990

KNOWLEDGE-BASED SYSTEM FOR CONNECTION DESIGN

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Abstract

A prototype expert system called CONXPRT (for CONnection eXPeRT) was developed using the C programming language in conjunction with state-of-the-art artificial intelligence techniques. At its current state, CONXPRT contains adequate knowledge to perform the analysis and design of three types of simple framing connections, but it can easily be expanded to cover other types of connections.

1. INTRODUCTION

To study the advantages of knowledge-based expert systems (KBES) techniques for steel building connection design, a prototype system called CONXPRT was developed. CONXPRT contains information and knowledge for the design of three simple (Type II) framing connections: framing angles , end plate shear connections, and single plate (also called shear plate, shear tab, or shear bar) framing connections. These connections were selected because they are very commonly used in steel framed buildings; they are conceptually different; their design approaches are different; and they have been studied by many researchers, which means that the expertise necessary for developing an expert system is available. Each of these three connections can be used to attach a beam to the flange or web of a column or to the web of a girder.

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Two prominent phases can be distinguished in the development process of CONXPRT. The first phase involved the development of an expert system shell which contains tools for adding knowledge to the system, and facilities for applying that knowledge. This shell is associated with many characteristics most of which are essential for building an adequate knowledge-based expert system for connection design. These characteristics are not offered, however, by any of the commercially available expert system building tools. Some of the features of this shell are:

- it is specifically designed to deal with connection design problems, which means that all the necessary design tools are available and that all the redundancies that are usually associated with general purpose expert system building shells are eliminated.
- it is capable of processing complex mathematical formulations at a reasonable speed.
- it can access external databases to retrieve information pertaining to structural shapes and materials.
- it allows an easy access to the knowledge base for possible updating and modification.
- it is very user friendly and has a very effectively designed explanation facility.
- it is expandable and it can be easily adapted for other design applications in structural engineering.

The second phase consisted of adding the adequate connection design expertise and knowledge to the initially empty system. This operation consisted of defining a connection design problem (such as an end plate shear connection), then adding design rules until the system can solve the given problem on its own. These rules contain information about the conditions and order of applicability of many connection design computational routines that were implemented along with the expert system shell.

2. DESCRIPTION OF CONXPRT

2.1 General

Most expert systems use an architecture similar to the one shown in Figure 1..Because of the complexity of the knowledge involved in connection design, a more sophisticated architecture had to be used for the implementation of CONXPRT. This architecture is composed of 1) a user interface, 2) a system driver that controls the operation of the expert system, 3) a module for the graphical representation of connections, 4) a design module, and 5) a database manager for dealing with information pertaining to structural shapes and materials. A schematic view of this architecture is shown in Figure 2, and detailed descriptions of the various modules are given in the following subsections.

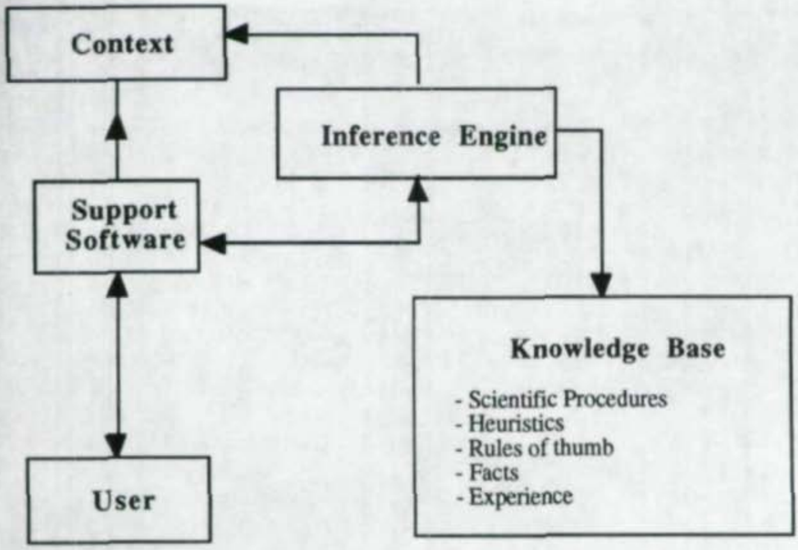

Figure 1 Typical Expert System Architecture

2.2 The User Interface

This module consists of functions and routines that are used by the system to perform input and output operations and other miscellaneous tasks. This interface is written in the C programming language which offers many advantages including speed of execution and flexibility. The input routine allows the user 1) to enter design loads, 2) to specify desired sizes and dimensions of certain components of the connection under design, and 3) to associate each member or connector (fastener or weld) with an appropriate structural material. The interface provides pop-up and pull-down menus for performing these operations along with good explanation facilities. The output routines can be used 1) to display information on the computer screen, 2) to print design reports, and 3) to save data on the computer storage media. These routines include a full screen editor/viewer that is used to help the user examine the recommendations of the system and its line of reasoning. Other tools for handling different kinds of data are also available in the interface. These tools consist of functions and routines that can be used to read and write data from and to storage media, and to prepare data for the menu driver and the editor/viewer.

2.3 The System Driver

The role of the system driver is to prepare the system's working environment at start-up, and to activate the appropriate modules as they are needed (Figure 3). This is the only module that remains continuously active from the moment the expert system is started until it is shut down. The first actions taken by this driver consist of loading the knowledge-base, the structural shapes and materials databases, and the program

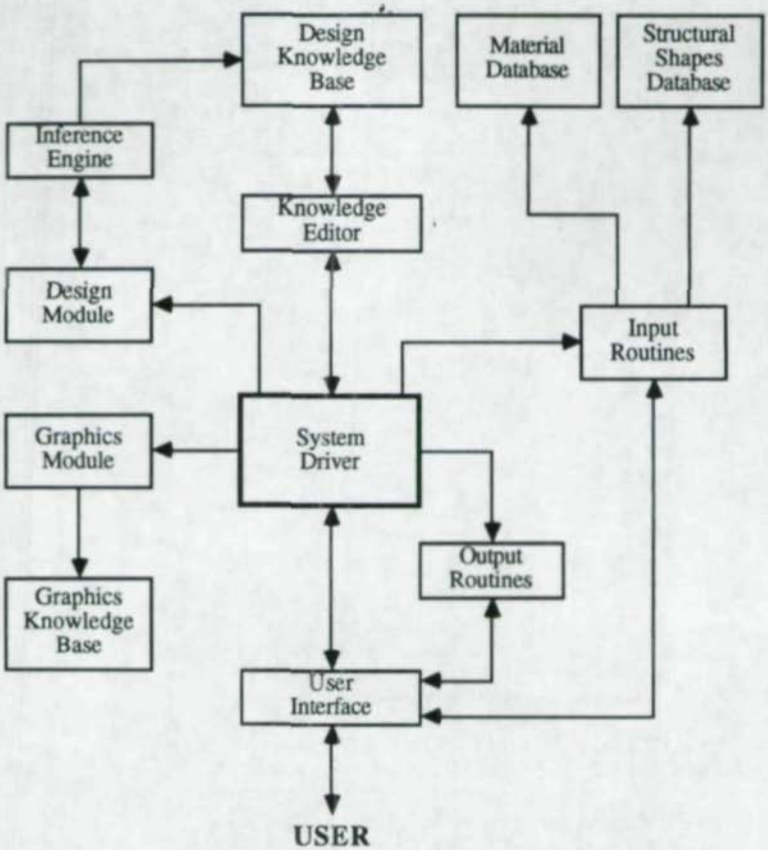

Figure 2 Architecture of CONXPRT

menu system into the computers main memory. Then, a continuous communication process with the user is started, using the various input/output routines that are available in the user interface. At first, control is given to the user who can either ask the system to take an action (such as creating a new connection problem or modifying an existing one), or simply request information such as expert advice. The system driver first executes the user's request, then it tries to update the graphical display and the design of the connection. The graphics and the design modules are called respectively to perform these tasks.

2.4 The Graphics Module

This module is used by CONXPRT to provide the user with a graphical representation of the connection being considered for design. The graphical representations are

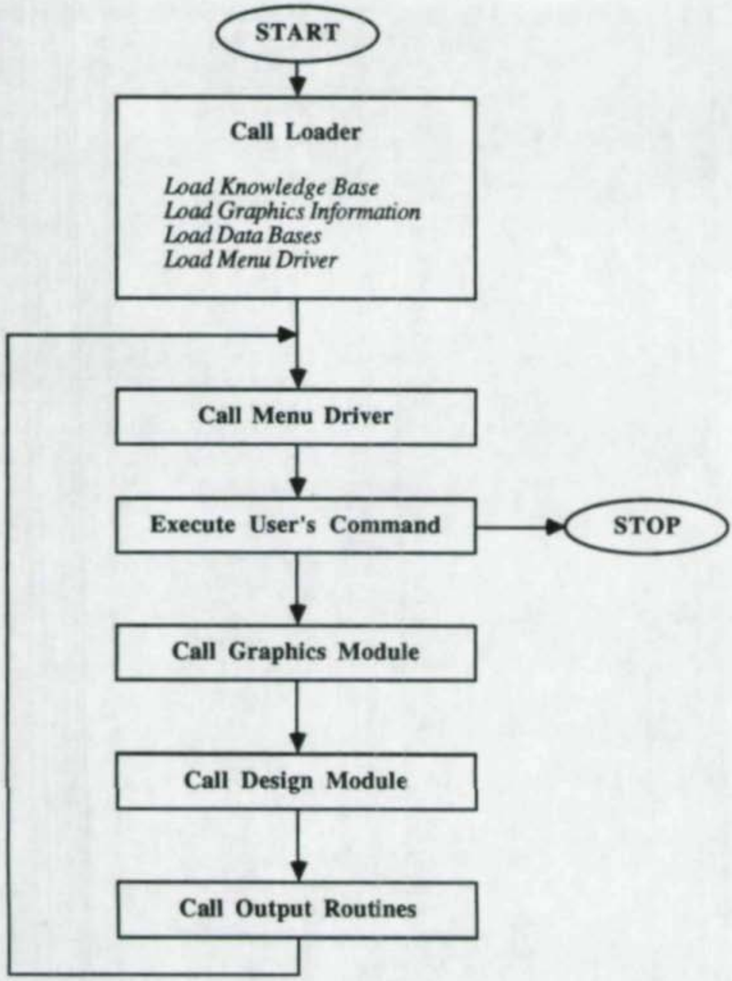

Figure 3 Operation Sequence of The System Driver

prepared in advance for all possible configurations that are required for the three connections, and the graphics character set of the computer is used to perform this task in order to eliminate the need for special graphics hardware. These representations are stored in a special database and rules are generated to permit the system to access this database. The graphics module uses a backward chaining inference mechanism to select the appropriate representation for the connection under consideration. The database contains 72 representations for different connection configurations including possible beam cope locations. These representations are not meant to be actual detail drawings, but they are sufficiently descriptive for the purpose of connection design.

2.5 The Design Module

The design module is called continuously by the system driver, but is only executed if a change in the connection description or in the design load is detected. This module can be thought of as a design team composed of an inexperienced designer and an expert. The inexperienced designer may be only able to perform simple design checks and calculations --such as checking the shear capacity of a bolt group or modifying the value of a given design parameter--, and his role is to execute the orders of the expert (the inference engine) who makes all the decisions based on the information contained in the knowledge-base. The module is composed of a design controller, an instruction queue, an inference engine, a knowledge base, and many computational routines as shown in Figure 4. The computational routines can be classified into two different categories. The first category includes functions that can be used to modify the design parameters, and the second category includes routines that can perform design checks. The execution of a design check routine can result in a success or in a failure. Failure occurs when a design parameter is found to be inadequate and a failure code is returned to the controller.

The controller starts by informing the inference engine that a design session is about to be started. The inference engine then responds by searching in the knowledge-base for a design procedure for the connection under consideration, and then passes this procedure to the controller as a list of instructions that are added to the initially empty instruction queue. The design process consists of executing the given instructions until the queue becomes empty. The controller processes each instruction as a call to one of the computational routines. When a call results in a success, the next instruction in the queue is executed, but when a call results in a failure, the controller stops the design process and informs the inference engine about the problem. The latter then searches for a solution in the knowledge-base using a backward chainer that was adapted for the specially-designed production rule system that is used to represent design knowledge. When a solution is found, its consequent actions are added to the front of the instruction queue, and the controller resumes the execution process. This process is terminated when all the design check routines are executed successfully, or when the system detects a problem with the design for which the user has to take part in making the final decision.

One of the most important features of this module is its ability to provide the user with the line of reasoning it used to reach any particular decision. It does this by creating a text file that contains a list of all the rules that were used to perform the design, the reasons for which those rules were selected, and the actions taken by the system after any particular rule is selected. This text file can be examined by the user at the end of the design process.

2.6 Structural Shapes and Materials Databases

Information pertaining to the geometric properties of structural shapes is stored in adatabase that is loaded in the computer's memory when the system is activated. This information includes the designations, dimensions, and properties of all the W, M, S, HP, C, and MC-shapes and also those of structural angles and tubes.

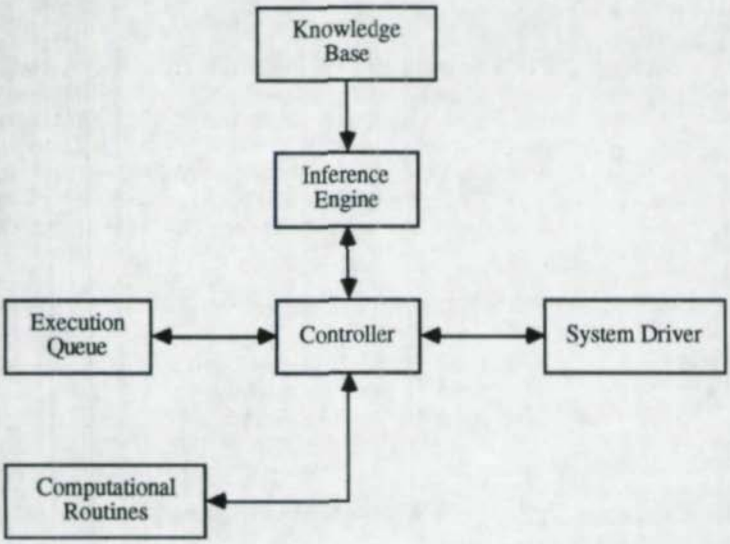

Figure 4 Architecture of the Design Module

The material properties of structural steels, connectors, and welds are kept in an external database for easy modification and update. The information contained in this database is loaded into the computer's memory when the system is activated. The yield and ultimate tensile stresses are given for structural steels; the shear and tensile capacities are given for bolts; and the ultimate shear stress and the unit shear capacity per 1/16-in. thickness are given for welds.

2.7 The Knowledge Editor

One of the most important features of expert systems is the fact that the knowledge they contain is independent from the methods of applying it. Because of this separation, knowledge can easily be modified without having to alter the control part of the system. The expert system shell that was used to implement CONXPRT provides a knowledge editing module that permits the modification, addition, and deletion of design rules. This module can only be accessed by the knowledge engineer and a password protection is used to enforce this limitation. The knowledge editor contains tools for retrieving, editing, printing, and saving rules in the knowledge-base. Pop-up menus are used to select the operations that are involved in editing the knowledge-base, and a built-in file editor is used to edit the rules. This rule editor has a unique feature that allows the knowledge engineer to enter the conditions and the consequent text of his rule in plain English.

3. PRACTICAL ASPECTS

CONXPRT is menu driven and has instructions on the screen with each menu. Help screens are designed for easy use. A quick input key facilitates specification of the connection. An entire connection design is simplified and the final design is shown on one screen. A report listing all connection design checks can be reviewed on the screen before printing. A complete connection design can be copied with a single key stroke and then reconfigured. This feature makes it easy to design, for instance, a beam-to-girder connection on one end and a beam-to-column flange connection on the other end of a beam.

All strength limit state checks, including block shear and coped beam strength, are made. ASD modules use rules and procedures from the 9th Edition Manual of Steel Construction. LRFD modules follow the AISC 1st Edition Load and Resistance Factor Design Manual. The latest available references are used to supplement AISC's procedures. Expert advice from long-time fabricator engineers was used to develop the design rules. If a design cannot be accomplished, CONXPRT provides an explanation, including necessary references. Expert advice on how to improve the connection is also given.

The software has provisions to set default values for a particular project or general shop needs. For instance, bolt diameters can be restricted to a specific range or even a specific diameter. Any detailing dimension can be similarly restricted. The user can also specify a preferred set of angle sizes. Databases of standard shapes, structural steels, weld metals, and bolt materials are included. The software will generate required cope sizes, allows the use of bolt stagger and permits different bolt diameters for shop and field use. Both tightening clearance requirements are automatically checked. When necessary, the user is warned if a specified connection is not recommended (for example, single plate-to-column webs) or if clearances are less than may be desirable. For simple connections, CONXPRT will accept a specified shear or will design the connections using the maximum number of bolts that can be used.

ACKNOWLEDGEMENTS

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A CAD SYSTEM FOR SEMI-RIGID JOINTS IN NON SWAY STEEL FRAMES

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Abstract

A CAD programme based on a method previously developed by the Authors for limit state analysis and design of beams in partially restrained non sway frames is presented. The approach defines suitable design limit state domains of the joint-beam system and makes use of the joint moment-rotation relationship. The method allows the determination of the lowest values of joint stiffness, strength and ductility, which are compatible with the limit state conditions assumed in the design analysis. The programme also features procedures for the preliminary sizing of the joint components.

1.0 INTRODUCTION

Steel braced frames have been traditionally designed as simple frames. Beam-to-column joints were modelled as ideal hinges, and designed on the basis of the beam end reaction. Recent investigations of the response of typical forms of simple connections (Zandonini and Zanon, 1988; Nethercot et al.,1988, Jaspart, 1991) point out that the degree of partial continuity they provide may be enough to achieve a more economical design. The benefit stems above all from non negligible weight savings for the beams with no or little increase in the cost of the connections (Van Douwen, 1981). A method for limit state analysis of beams with rotational end restraints has been set up by the Authors (Zandonini and Zanon, 1988), which can be considered a generalization of the approaches based on the "beam line" concept (Kennedy, 1969; Nethercot, 1985). This method is a powerful design tool. An approach to designing of

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joint/beam systems was then developed (Bursi et al., 1989; Zandonini and Zanon, 1990), which, with reference to different design procedures, makes it possible to size joints on the basis of the minimum required values of initial stiffness, ultimate moment resistance and rotation capacity.

One of the main advantages of the method is the associated graphical representation of both the beam limit state conditions and the response of the joint beam system with reference to the primary parameters of design interest, i.e. applied load, moment at the joint, its rotation, and beam midspan deflection. This enables users to immediately check and control the response of the joint-beam system being affected by the joint response. This characteristic makes the method very suitable for interactive computer aided design. A programme is being developed for the design and check of joints and beams in partially restrained steel frames, the key characteristics of which are briefly illustrated in this paper.

2. ANALYSIS METHOD

The response of the beam of figure 1a with rotational nonlinear end restraints simulating joint behaviour and subject to a uniformly distributed loads can be suitably defined by the end moment M, the end rotation Φ and the midspan deflection δ (fig. 1b).
If the plastic hinge model is assumed, the beam limit state conditions shown in figure 2 occur. Besides the formation of a plastic mechanism, the condition of excessive deformation (δ_u) is also considered; this conventional ultimate limit state is important for rather flexible joints.
In the figure the loads at which the different limit states are attained are also indicated; they are to be compared with the corresponding design loads: the unfactored total load $W_{unf} = D + L$ (serviceability), and the factored load $W_{f,u} = \gamma_{U,D}\ D + \gamma_{U,L}\ L$ (ultimate resistance).
The mathematical expressions defining the beam limit state conditions identify (Zandonini and Zanon, 1988) the multidomain shown in figure 3 in suitably normalized coordinates: line B'C' is the upper limit of the serviceability limit state domain, whereas lines OA, AB, BC and CO bound the elastic ultimate domain and lines BB", B"C" and C"C the inelastic ultimate domain.
Due to compatibility and equilibrium of the joint-beam system, the constitutive law of the joint denotes as well the response of the joint-beam system in the coordinates M and Φ. The response curves in the other quadrants (Fig. 3) are easily obtained through the relations between the characteristic parameters.
At the final design stage, when the beam section and the joint M-Φ curve are known, the intersections of the joint-beam response curves with the beam multidomains (serviceability, elastic ultimate, inelastic ultimate) give the values of M, Φ, δ and W associated to each limit state condition. Comparison of these values with the corresponding design values allows to find out whether the required level of reliability is achieved.

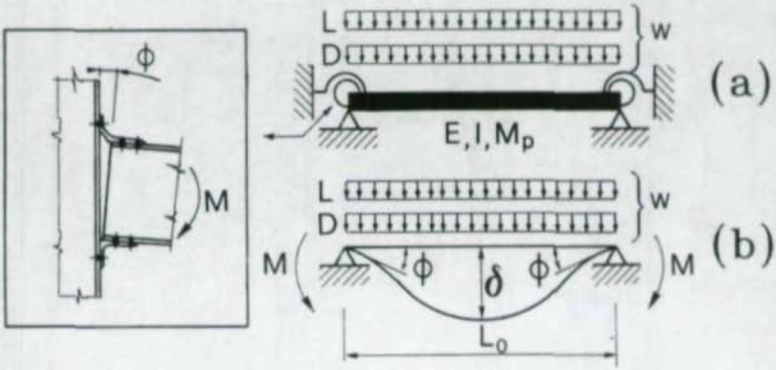


Figure 1

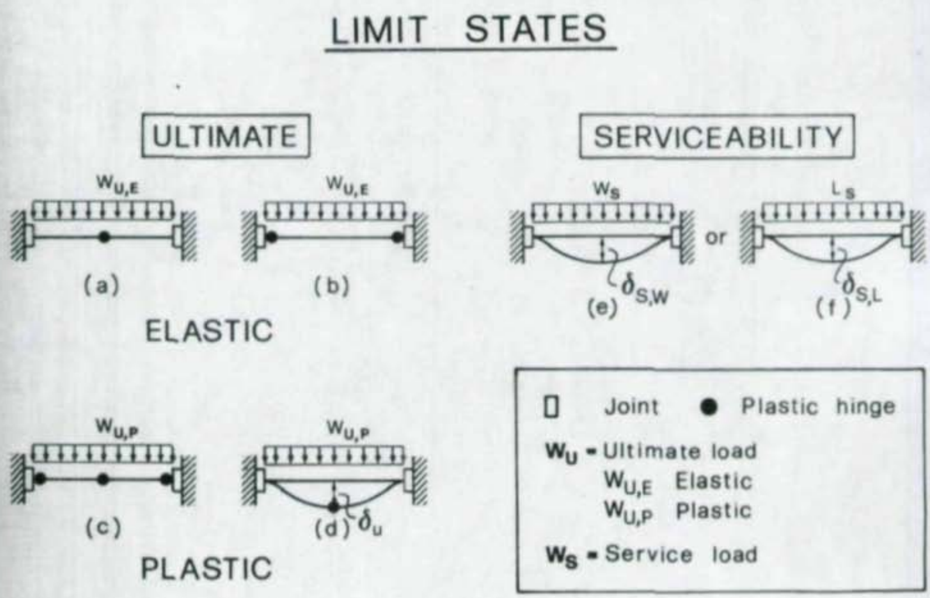


Figure 2

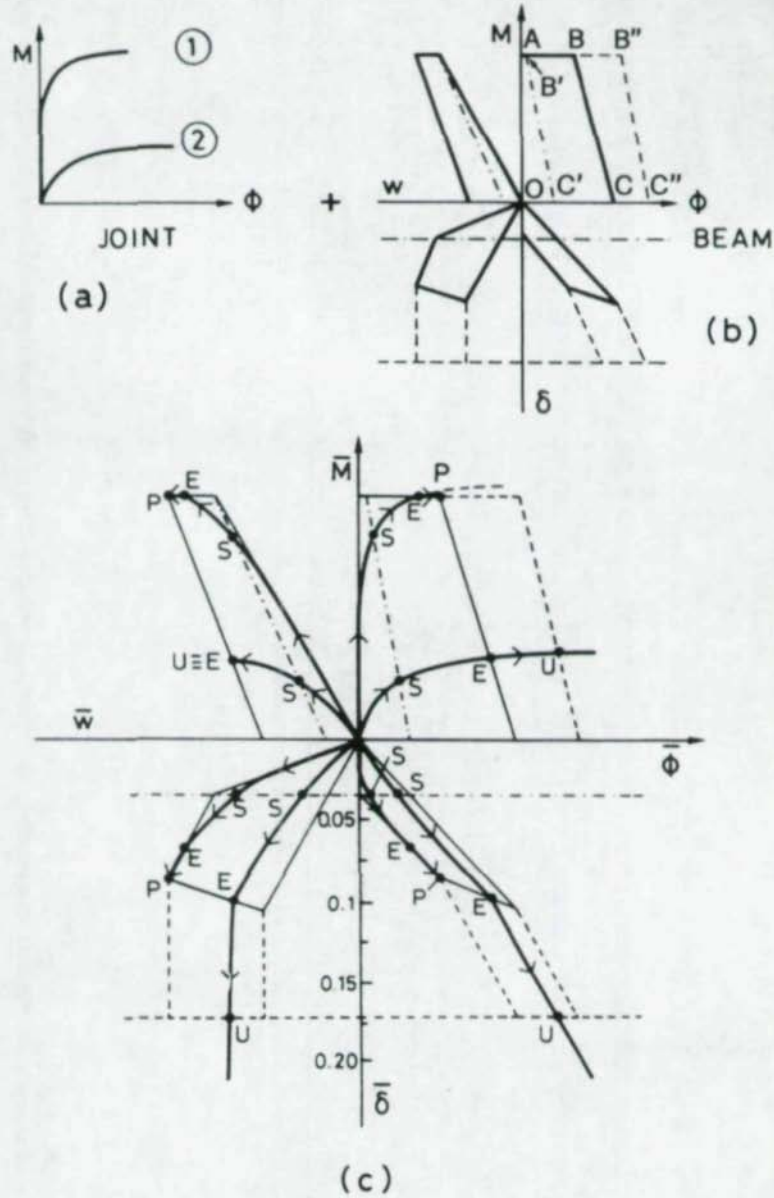


Figure 3

3. DESIGN APPROACH

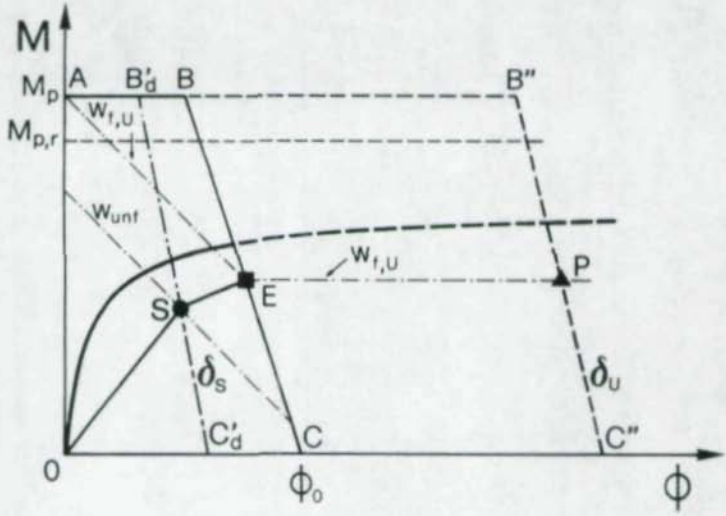
Let us assume that a model is available, which is capable of approximating the joint moment-rotation response, as is affected by the key geometrical and mechanical parameters. A comprehensive set of analyses of a joint-beam system (which maintains the same beam section) would permit to identify the most flexible and/or weakest joint that fullfils all design requirements.
In general, the lower design boundary of the joint response can be defined (Zandonini and Zanon, 1990) for either the elastic (Fig.4a) or plastic analysis (Fig. 4b). The moment-rotation curve must lie above the OSE and OSP lines, respectively.
In order to achieve effective design solutions for the joint-beam system, with reference also to the cost/performance ratio, different design procedures can be envisaged; in particular, elastic and plastic design may be performed as follows:

(1) <u>Elastic Design</u>

- The beam is sized so that it would possess, with partial restraints, the same elastic strength as in fully restrained conditions: this is obtained if the beam plastic moment meets the following requirement
$$\Phi_b \, M_p = W_{f,u} \, L_o^2 \, / \, 12$$
with Φ_b the resistance factor, which may be assumed as 0.9, according to the AISC-LRFD specification;
- The beam limit state domains are determined and plotted;
- Points S and E are obtained as intersections with the relevant domain and the load lines associated to the unfactored and factored load respectively. The lower bound for the joint response is then defined (line OSE in figure 4a);
- The moment-rotation curve of the joint is computed through a suitable prediction model. The calculation is iterated until the joint characteristics are selected which make the joint response to lie above, and as close as possible to, the line OSE. If relevant, a further check should be performed on point E, which must be lower than the horizontal line identified by the value $M_{p,r}$ of the beam plastic resistance reduced by the presence of bolt holes.

(2) <u>Plastic Design</u>

- The partially restrained beam is required to possess the same plastic strength as in fully restrained conditions, by imposing that
$$\Phi_b \, M_p = W_{f,u} \, L_o^2 \, / \, 16 \; ;$$
- The next steps are the same as in elastic design; the joint M-Φ curve must in this case lie above (and as close as possible to) the line OSP (Fig. 4b).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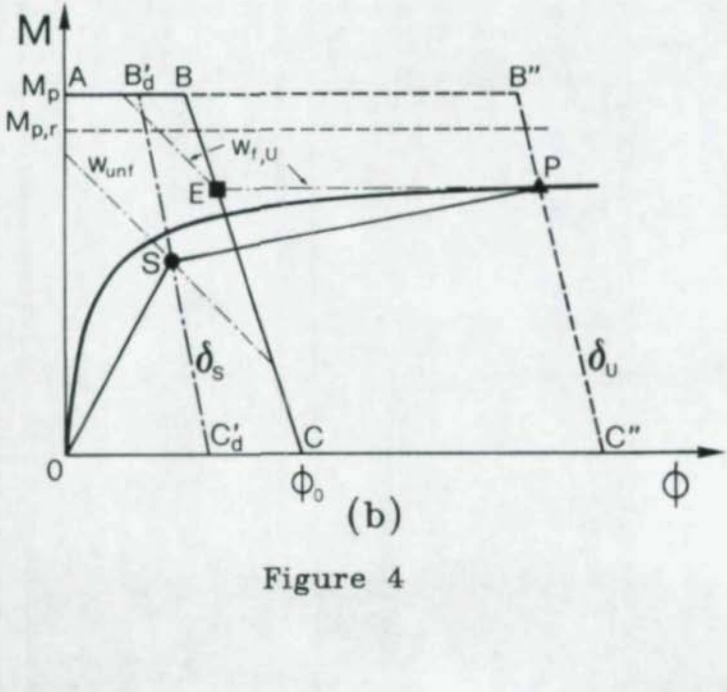
(a)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(b)

Figure 4

4. PROGRAM MAIN FEATURES

The computer aided analysis and design system utilizes an interactive approach based on menu-driven graphics, and consists of four main parts: (A) Input of design data and selection of relevant Code of practice; (B) Selection of the beam shape and of the joint form, and preliminary sizing of the key components of the joint; (C) Analysis of the joint-beam system; (D) Checks of all the elements and fasteners under the design conditions.
Iteration is possible at any time of the first three parts in order to obtain the most effective solution, compatible with the design restraints.
The main features of each part are the following:

(A) Input of design data

Input data are requested in terms of beam span, loads (Dead and Live), code of practice (Italian CNR standards, Eurocode 3 or AISC-LRFD), deflection limits for serviceability checks (and, if relevant, for the inelastic excessive deformation limit state), type of analysis (elastic or plastic), and steel grade. Analysis may use either the SI or the United States system of units.

Output of this part consists in the calculation of the unfactored and factored total load W, on the basis of the partial safety factors specified by the Code adopted, and in the definition of the plastic moment to be required of the beam (see section 3).

(B) Selection of beam shape and joint components

- Beam shape: a first selection relates to the section type (European, American and Japanese wide flange rolled sections are implemented); this preliminary sizing is assisted *providing information on the minimum required plastic moment* and the span to depth ratio.
  Output consists in the determination of the beam limit state domains, and in the definition of the required design moment to be sustained by the joints (Fig. 5).
- Subsequently the joint form is selected on the basis of menu-driven graphics. Presently, only the web and flange angle connection of figure 6 is fully implemented; in the following reference will be hence made specifically to this connection form.
- The top and bottom angles are sized first: indications for first trial dimensions are provided based on the design force F_d computed as in figure 5.
  The web angles suggested dimensions are defined so that the full shear action due to the factored design load can be sustained by these elements.
- Interactive graphics allows for a proper selection and distribution of the bolts in all the angle components. Geometrical checks specified in the relevant Code as well as related to the erection feasibility are automatically performed. A first trial solution is also provided to the user.

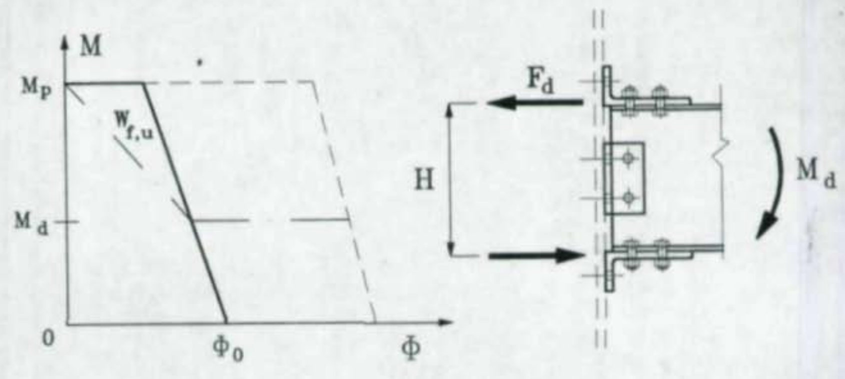

Figure 5

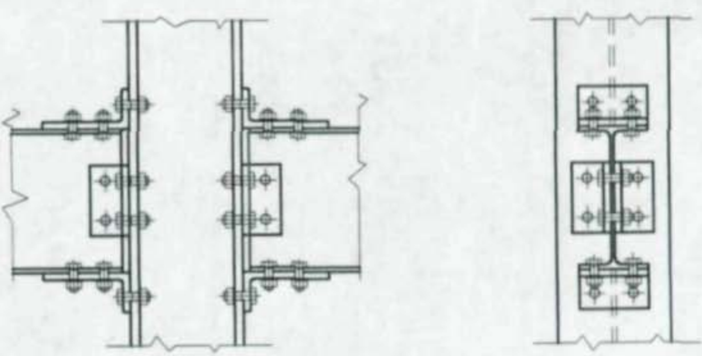

Figure 6

(C) Analysis of the joint-beam system
- A suitable design model is selected for the approximation of the joint rotational response. Presently, the sole method developed by Kishi et al. (1988) is implemented.
- The lower bound of the acceptable joint M - Φ curve is determined (line OSE for elastic analysis and line OSP for plastic analysis, as in figure 4).
- Design checks are then made in a graphical, and numerical, form.
- Any change of the general design parameters (e.g. type of analysis, steel grade, deflection limits) and/or of joint parameters (e.g. angle size and material, bolts diameter, pattern and material) may be done at this stage. Reanalysis, and graphical comparison of the results permits immediate further action until all requirements are met.

(D) Checks of elements and fasteners
Overall behavioural models of the joint do not imply that all local phenomena are automatically checked. In this case, a final "validation" of the joint previously designed is required, and all checks specified in the relevant Code should be performed.

5. ACKNOWLEDGEMENTS

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6. REFERENCES

Zandonini R. and Zanon P. (1988), "Analysis of Beams in Partially Restrained Non Sway Frames", Report No. 3, Department of Engineering, University of Trento, Italy.

Nethercot D.A., Davison J.B., and Kirby P.A. (1988), "Connection Flexibility and Beam Design in Non-Sway Frames", AISC, Engineering Journal, vol. 25, No. 3, pp. 99-108.

Jaspart J.P. (1991), "Joint Semi-Rigid Action and its Influence on the Performance of Steel Frames" (in French), Ph D. Thesis, University of Liege, Belgium.

Van Douwen A.A. (1981),"Design for Economy in Bolted and Welded Connections", Joints in Structural Steelwork, Proc. Int. Conference,Middlesbrough, England, April, pp. 5.18-5.35.

Kennedy D.J.L. (1969), "Moment Rotation Characteristic of Shear Connection", AISC, Engineering Journal, vol. 6, No. 4, October, pp. 105-115.

Nethercot D.A. (1985), "Joint Action and the Design of Steel Frames", The Structural Engineer, Vol. 63A, No. 12, London, December, pp. 371-379.

Bursi O., Zandonini R., and Zanon P. (1989), "Design of Beams in Partially Restrained Non Sway Frames", Int. Coll. Bolted and Special Structural Connections, Moscow, May, pp. 135-142.
Zandonini R. and Zanon P. (1990), "Beam Design in PR Braced Steel Frames ", AISC National Engineering Conference, Kansas City, March, pp. 26-1 - 26-26 (to be published in the AISC Engineering Journal).
EUROCODE 3 (1991), "Common Unified Code of Practice for Steel Structures", Commission of the European Communities, Brussels.
AISC (1986), Load and Resistance Factor Design Specification for Structural Steel Buildings.
CNR-UNI 10011 (1985), Steel Structures - Instructions for Design, *Construction, Testing and Maintenance (in Italian)*.
Kishi N., Chen W.F., Matsuoka K.G., and Nomachi S.G. (1988), "Moment-Rotation Relation of Top- and Seat-Angle with Double Web-Angle Connections", Connections in Steel Structures: Behaviour Strength and Design, Eds. Bjorhovde R., Brozzetti J. and Colson A., Elsevier, London, pp. 121-133.

Technical Papers on

GLOBAL BEHAVIOR OF SEMI-RIGID CONNECTIONS

CONNECTION MOMENT-ROTATION CURVES FOR SEMI-RIGID FRAME DESIGN

André Colson[1]

Reidar Bjorhovde[2]

Abstract

The paper presents the results of a study of semi-rigid frame analysis and design, using various options for representation of the connection properties. In particular, the use of actual versus simplified moment-rotation curves has been examined in detail, with a view towards design specification requirements. It is shown that it may be unconservative to use highly simplified, usually piecewise linear, M-ϕ-curves, especially if these are based on ultimate moment capacities less than those of the actual connections. Further, it is important to maintain accurate connection stiffness properties for the full range of response, to ensure that frame stability is adequate. Computer representation of non-linear moment-rotation curves is not difficult, negating the need for the simplified relationships.

1. ULTIMATE LIMIT STATES CONSIDERATIONS

In general, current limit states design specifications (EC, 1990; AISC, 1986; CSA, 1990) are formulated on the basis of ultimate strength considerations. The key concept requires that the internal stress resultants in a structural element for the most severe loading case are smaller than the ultimate capacities of the element. This is expressed by the basic limit states criteria, some typical representations of which are given in Eqs. (1) and (2).

In LRFD Format (AISC, 1986):

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$$\phi R_n \geq \Sigma \gamma_i Q_i \tag{1}$$

where R_n = nominal resistance, ϕ = resistance factor for the applicable limit state, Q_i = nominal effect of load i, and γ_i = load factor for load i. The lefthand side of Eq. (1) is the design resistance; the righthand side is the required resistance.

In Eurocode 3 Format (EC, 1990):

$$R_d \geq S_d \tag{2}$$

where R_d = design resistance, incorporating reliability, and S_d = design value of the internal stress resultant for the appropriate factored load combination.

In all cases the requisite reliability has been established through probabilistic means, assuming that the quantities in Eqs. (1) and (2) are statistically independent random variables. Basically and logically, the load factors are greater than 1 and the resistance factors are smaller than 1. (It is noted that Eurocode 3 uses sets of partial safety factors. These are used to develop the lefthand side of Eq. (2), and are therefore also larger than 1, since their product will eventually be utilized as divisors to arrive at the design resistance R_d.)

For statically determinate structures, the two sides of Eqs. (1) and (2) are effectively independent, since the internal stress resultants are not functions of the rigidity of the members. For statically indeterminate structures, however, where it is necessary to use the material and geometrical properties of the members to find the stress resultants, it is critical to know whether additional reliability has to be built in, to account for the variability of the material characteristics. Thus, it may occur that load *factors smaller than 1 (or resistance factors larger than 1) would be used in* conjunction with Eqs. (1) and (2). This is particularly important for structures such as frames with semi-rigid connections, where the moment-rotation curves are heavily influenced by the properties of the fasteners and the fastening elements.

It is the purpose of this paper to discuss the M-ϕ-curves that have to be used in the analysis of semi-rigid frames. Emphasis will be placed on beam-to-column type connections, but the procedures may be generalized for application to other forms.

2. ACTUAL VERSUS IDEALIZED CURVES FOR CONNECTIONS

Due to the complexity and multitude of connections, and the fact that no general analytical technique has been devised that can cover all relevant factors in all cases, testing continues to be the most reliable method to obtain a connection moment-rotation curve. Thus, the cruciform specimen simulates symmetric loading, and the cantilever test is used to represent the unsymmetric case. Effects such as the panel zone deformation, which are essential to the overall connection response, will be obtained in the unsymmetric test.

Several deterministic data bases for experimental results have been developed (Goverdhan, 1983; Kishi and Chen, 1986), but it is clear that all possible types and sizes of connections and their elements have neither been tested nor evaluated analytically. Further, even when test data are available, the number of results for identical joints is not sufficient to give a reliable basis for probabilistic developments.

Reliability evaluations of semi-rigid connections is therefore still in its infancy (Rauscher and Gerstle, 1991), and any frame analysis must be deterministic, based on some form of a mean or other representative curve. Studies too numerous to be cited here have been devoted to develop numerical and mathematical M-ϕ-curves, usually fitted to test data; any number of these may suitable for frame analyses.

Finally, recent studies and design code developments have attempted to devise means of allowing the designer somewhat easier access to the realm of semi-rigid design (Bjorhovde et al., 1990; Eurocode 3, 1990). Bjorhovde et al. have developed a classification system into which connections fit according to initial stiffness, ultimate moment capacity, and ductility. This appears to hold a promise for a more rational approach to dealing with the many types of connections, since it is impractical at best to have designers devise M-ϕ-curves for each and every connection. The proposal is a first step, however, and it is anticipated that modifications may be made.

The Eurocode proposal makes use of a tri-linear M-ϕ-curve that is comparable to the actual curve in initial stiffness, but it deviates significantly from the real curve in that a plateau, identified as the design moment, M_d, is set at a level well below the ultimate moment. This response of the design model is therefore always below the real connection.

As an illustration, Fig. 1 shows a bolted end-plate connection that has been analyzed using a variety of approaches. Originally utilized by Bijlaard et al. in a study of semi-rigid frames (Bijlaard et al., 1989; ECCS, 1990), the connection has an IPE 220 beam and an HEB 140 column, and uses a total of 6 16 mm high strength bolts of the European grade 8-8, which is comparable to ASTM A325.

For the case with horizontal and diagonal stiffeners, Table 1 shows the connection design properties determined on the basis of the Eurocode 3 (EC 3) criteria, as well as the results using the authors' approach (1990) and that of Yee and Melchers (1986). In column 5, the actual ultimate moment capacity was determined from a plastic hinge mechanism identical to that of EC 3, but with the yield plateau moment computed on the basis of the ultimate strength (F_u) of the end plate and the bolts. A number of experimental studies have given good correlation with models using F_u in lieu of F_y (Colson and Bjorhovde, 1991).

Figure 2 shows the resulting M-ϕ-curves for a connection with pre-tensioned bolts, with the "design curve" representing the EC 3 result, and the "actual curve" indicating the calculations using the authors' approach. The actual curve correlates closely to experimental results. The boundary line separating rigid and semi-rigid behavior is also included. In addition to having the same initial stiffness, it is seen that the design M-ϕ-curve reflects the behavior of a rigid-partial strength connection, whereas the actual curve shows that the connection is of the rigid-full strength type (Bjorhovde et al., 1990).

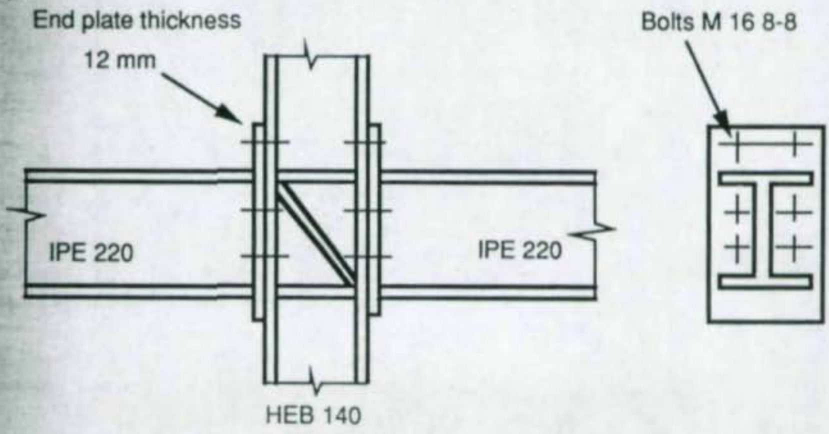

Figure 1 Extended End-Plate Connection

Table 1 Example Connection Characteristics

	Snug-tightened bolts (EC 3)	Pre-tensioned bolts (EC 3) **Case A**	Pre-tensioned bolts (Yee)	Estimated actual moment (authors) **Case B**
Ultimate moment capacity (kNm)	M_d = 47.68	M_d = 47.68	54.00	M_u = 69.67
Initial stiffness (kNm/rad)	C = 14,000	C ≈ ∞ (60,000 used for numerical purposes)	C = 24,800	C ≈ ∞ (60,000 used for numerical purposes)

Equation (3) gives the power model representation that was used to formulate the actual moment-rotation curve in Fig. 2. In this equation, M is the moment at the given ϕ-value, C is the initial stiffness, and M_u is the ultimate moment capacity. n is the shape parameter; it was chosen as 3 for this connection, but it ranges from 1.5

to 4.5 for typical joints in steel structures (Goverdhan, 1983; Kishi and Chen, 1986).

$$\phi = \frac{M}{C} \frac{1}{[1 - (M/M_u)^n]} \qquad (3)$$

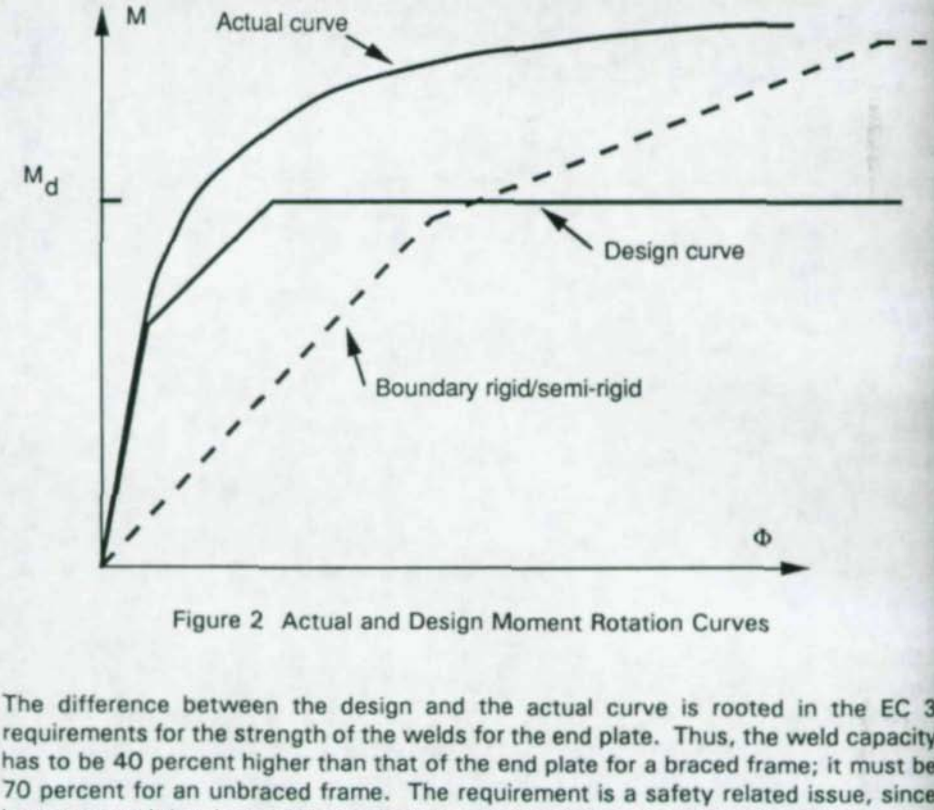

Figure 2 Actual and Design Moment Rotation Curves

The difference between the design and the actual curve is rooted in the EC 3 requirements for the strength of the welds for the end plate. Thus, the weld capacity has to be 40 percent higher than that of the end plate for a braced frame; it must be 70 percent for an unbraced frame. The requirement is a safety related issue, since it was argued that it would be critical to force the joint failure away from the welds.

The American LRFD criteria contain no such specific clauses, although the safety philosophy is comparable. Further, since the specification does permit the use of partially restrained (PR), i.e. semi-rigid, connections, but gives no specific rules along the lines of the EC 3 approach, for example, a direct comparison is not possible. The designers who make use of PR concepts define their own M-ϕ-curves, including multi-linear forms, but the ultimate moment capacity is generally close to the actual value (Lindsey, 1988). The difference between the actual and design cases is therefore not an issue in the United States.

Similar differences between actual and simplified models of connection response are numerous. An important case is that of the connection that has been designed for simple behavior, but where a subsequent concrete slab provides for continuity and therefore end restraint. Unless appropriately analyzed and detailed, the actual connection will have response characteristics that are significantly different from the assumed values. Overstrength effects in particular will prevail; it will be shown here that these influences may be undesirable from structural as well as economic standpoints.

3. FRAME ANALYSIS EXAMPLE

In order to demonstrate the influence of overstrength effects on the response of a complete frame, the two story, two bay unbraced frame of Fig. 3 was designed using the design and actual curves of cases A and B of Table 1. The in-plane beam-to-column connections for the exterior columns are pinned; the interior ones are semi-rigid. The column bases are all pinned. All out-of-plane connections are assumed to be pinned. Beam and column sizes, spans and lengths are given in Fig. 3; the loading case that was used is illustrated in Fig. 4. The steel grade is A36, with F_y = 36 ksi or 248 MPa.

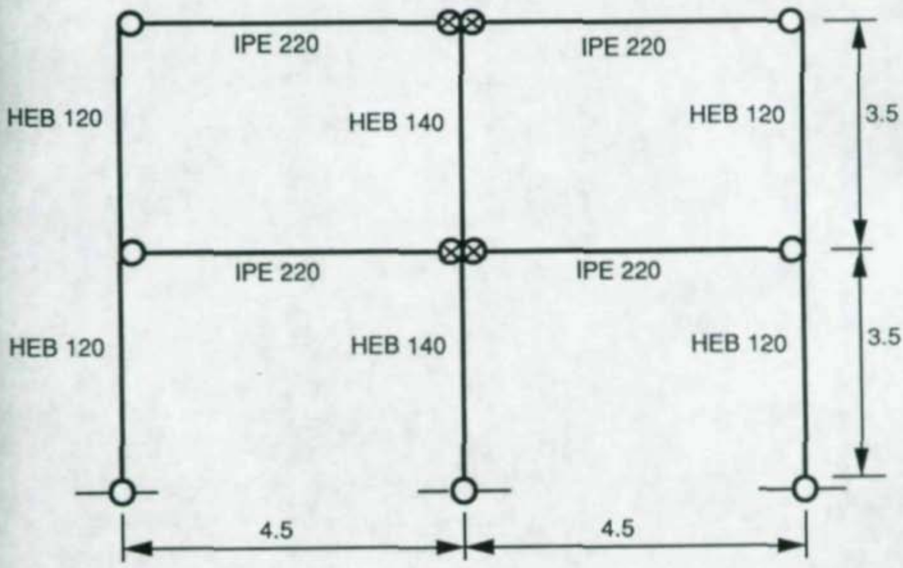

Figure 3 Structure with Semi-Rigid Connections

The analysis and design were carried out with the aid of the computer program PEP-Micro (Galea and Bureau, 1990). This program takes into account inelastic member behavior, structural second order effects, and non-linear connection response. The program allows for the connection properties either to be input in the form of an algebraic equation or in the form of digitized test data. Table 2 gives the results of the frame analysis.

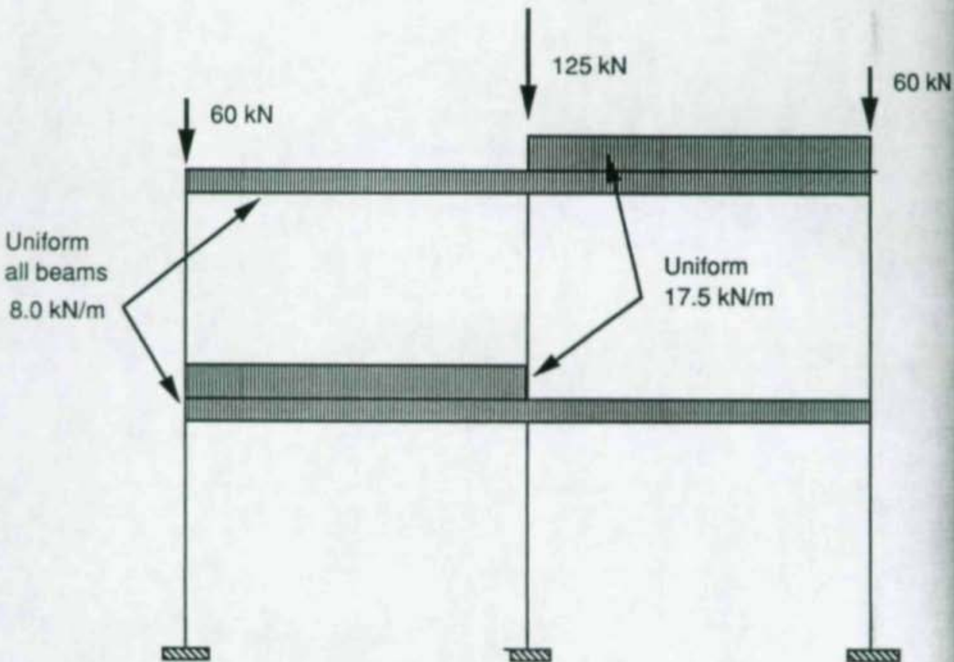

Figure 4 Governing Load Case for Frame Design Example

The following observations can be made on the basis of the results:

(1) The frame collapse mode is the same for Cases A and B. However, for Case B a plastic hinge has formed in the beam, near the beam-to-column connection, since the plastic moment of the beam is less than M_u for the connection.

(2) The collapse load factor is significantly higher for Case B. This is due to the higher moment capacity of the connection (M_u for Case B vs. M_s for Case A).

(3) Using a load factor of λ = 1.391 for both cases (note that Case A fails for a

value of 1.392), the moment distribution in the interior column is slightly less favorable for Case B than for Case A.

Table 2 Frame Analysis Results

Result	Case A	Case B
Collapse Mode	Plastic hinges in beam span. Large connection rotations.	Plastic hinges in beam span and near beam-to-column connection.
Collapse Load Factor	$\lambda = 1.392$	$\lambda = 1.518$
Stress Resultants in Interior Column at Load Factor $\lambda = 1.391$		
<u>First Story of Column</u>:		
Bending moment at top (B) (kNm)	8.26	12.23
Axial force (kN)	421.2	429.5
<u>Second Story</u>:		
Bending moment at B (kNm)	3.64	6.25
Bending moment at top (kNm)	9.32	12.15
Axial force (kN)	297.8	301.7

4. ADDITIONAL OBSERVATIONS ON FRAME ANALYSIS RESULTS

The most important results for the study that is the subject of this paper are the magnitudes of the moments for the interior column, since this member is framed to the beams of the structure with semi-rigid connections. With the load factor $\lambda = 1.391$, this can be considered the governing design value for the structure, for the ultimate limit state of reaching the plastic collapse mechanism of Case A. This limit state is reached for a λ-value of 1.392.

Checking the interior column as a beam-column, using the criteria of EC 3 and the AISC LRFD Specification, the interaction equation (IE) sums for the two cases have been determined. For a design according to EC 3, the checks for A and B give:

For Case A: IE Sum = 0.983

For Case B: IE Sum = 1.079

Obviously, Case B is in violation of Eurocode 3, although it may be argued that a sum of 1.079 might be acceptable under certain circumstances. However, it is essential to recognize that this has occurred not as a result of an actual structural condition, but rather due to a connection representation concept. The actual strength and stiffness have been artificially lowered, and in consequence the stability of the structure is less affected. That is, the smaller stiffness and strength of the connection draws less of a moment into the column, and it is therefore able to satisfy the interaction equation.

The above checks pertain to the in-plane condition of the interior column. The size of the member is actually governed by out-of-plane (weak axis) buckling, for which the connection restraint plays no role. On the other hand, stability about both axes has to be satisfied, and the Case B solution strictly is not acceptable.

As noted earlier, since the LRFD Specification does not use a simplified moment approach, Case A has no practical meaning. It would be the designer's choice to model the connection and its interaction with the structure. In most cases it appears that the moment-rotation curve is closely similar to the test result, although a multi-linear form is often the choice (Lindsey, 1988). Case B is therefore representative of the American approach.

Checking the in-plane condition of the column by Section H of the LRFD Specification gives an interaction equation sum of 0.93. The HEB 140 is therefore satisfactory by LRFD rules, even when the moment effect of the semi-rigid connection is taken into account. The main reasons are that the axial load is by far the overriding influence, and the low-rise frame produces very small moments M_{nt} and M_{lt} for use with the interaction equation.

As a final comment regarding Case B and the EC 3 interaction equation, it is realistic to speculate that collapse will most likely not occur, even with an IE Sum of 1.079. The reason is that plastic redistribution will take place. A limited localized overstress is therefore of little actual import.

5. INFLUENCE OF CONNECTION INITIAL STIFFNESS

To examine the influence of the initial stiffness, C (= R_0 in EC 3 terminology), of the connection, a range of frame analyses were performed, using the same ultimate moment capacities as before for Case A (= 47.68 kNm) and Case B (= 69.67 kNm). However, the values of C were varied from 3,000 to 115,000 kNm/rad. For the purposes of this paper, the beam-to-column classification scheme of EC 3 was used, which gives the boundary between rigid and semi-rigid as 10,348 kNm/rad for the connection in question.

The initial stiffness effects have been evaluated in terms of the maximum bending moment in the first story of the interior column. Using the load factor value of 1.391, the results are shown in Table 3.

As would be expected, it is clear that the higher the initial stiffness, the larger is the overstrength effect, as the term has been used in this paper. Thus, in the case of the bolted end-plate connection with horizontal and diagonal stiffeners and pretensioned bolts, the overstrength value will be as high as approximately 50 percent. It applies for C equal to infinity, which is a possible choice of initial stiffness for the connection. This finding emphasizes that it is very important to have accurate data on the initial stiffness, particularly for very stiff connections, such as the end-plate type and similar joints.

A key result of this analysis is that the overstrength effect vanishes, for all practical purposes, when the initial stiffness places the connection within the semi-rigid range of behavior. This means that there is equal distribution and re-distribution of the structural rigidity between the framing members and the connections.

Although the above finding needs to be examined in detail for a range of structures, where the stiffness distribution between columns, beams and connections is broader than that of the example frame, it clearly holds unique promise for making use of semi-rigid concepts in frame design. As long as frame drift and deflection needs are satisfied, and semi-rigid connections appear to be able to provide adequate performance, the less costly semi-rigid joints will promote savings. Finally, the semi-rigid structure is more forgiving, meaning that the redistribution characteristics are better than those of a rigid frame. It is less influenced by member and connection modeling methodology than the rigid structures, and offers redundancy that is not found in simple structures.

6. SUMMARY AND CONCLUSIONS

The study has focused on the application of rational concepts of semi-rigid connection characteristics to practical design approaches. The conclusions that have been arrived at are based on a detailed study of one application of semi-rigid framing principles and analyses. Most importantly, it has been found that it may be unconservative to use simplified representations of the connection moment-rotation relationships. This applies especially to response forms that downgrade the moment capacity to achieve improved deformation characteristics. This is an artifice that may have a negative effect on the perceived behavior of the structural frame.

It is recommended that any moment-rotation curve that is used in analysis and design should be as close to its experimental data base as possible. An experimental curve is probably the best tool; in lieu of such a reliable curve, an algebraic solution that matches the test data is also acceptable.

It has also been shown that incorporating the principles of semi-rigid analysis into the frame design will lead to more uniform frame stiffness and reliability. The reliability aspect is in need of a major research effort, to be able to define the safety of the complete structure in the proper limit states design format.

Table 3 Evaluation of Overstrength Effects: Bending Moment in Interior Column

------	-------	Rigid x 10^3	Rigid x 10^3	Rigid x 10^3	Rigid x 10^3	EC 3 Limit x 10^3	Semi-Rigid x 10^3	Semi-Rigid x 10^3
------ ------	C M_u	115	60	25	15	10.35	6	3
Part. Str. **Case A**	47.68	8.64	8.76	8.97	9.51	11.43	13.64	16.67
Full Str. **Case B**	69.67	12.48	12.23	11.57	11.30	11.16	13.40	16.01
Load Fac.	------	1.391	1.391	1.391	1.391	1.391	1.391	1.391
Ovr-Str. Eff. (%)	------	+ 55	+ 48	+ 28	+ 1.8	- 0.02	- 0.01	- 0.003

7. REFERENCES

1. American Institute of Steel Construction (AISC) (1986). "Specification for the Load and Resistance Factor Design, Fabrication and Erection of Steel Structures for Buildings", AISC, Chicago, Illinois, U S A.

2. Bijlaard, F. S. K., Nethercot, D. A., Stark, J. W. B., Tschemmernegg, F., and Zoetemeijer, P. (1989). "Structural Properties of Semi-Rigid Joints in Steel Frames", IABSE Periodica, No. 2, Survey Paper No. S-42/89.

3. Bjorhovde, R., Colson, A., and Brozzetti, J. (1990). "Classification System for Beam-to-Column Connections", Journal of Structural Engineering, ASCE, Vol. 116, No. 11, November (pp. 3063-3080).

4. Canadian Standard Association (CSA) (1990). "Steel Structures for Buildings - Limit States Design", CSA Standard No. CAN3-S16.1-M90, CSA, Rexdale, Ontario, Canada.

5. Colson, A., and Bjorhovde, R. (1991). "Discussion of 'Moment-Rotation Relations of Semirigid Connections with Angles', by N. Kishi and W. F. Chen", Journal of Structural Engineering, ASCE, Vol. 117, No. 9, September (pp. 2809-2811).

6. Commission of the European Communities (CEC) (1990). "Eurocode 3: Design of Steel Structures", CEC, Brussels, Belgium, November.

7. European Convention for Constructional Steelwork (ECCS) (1990). "Analysis and Design of Steel Frames with Semi-Rigid Joints", Document No. TWG 8.1/8.2, ECCS, Brussels, Belgium.

8. Galea, Y., and Bureau, A. (1990). "PEP-Micro: Programme d'Analyse Non-Lineaire des Structures a Barres", CTICM, St. Remy-les-Chevreuse, France.

9. Goverdhan, A. (1983). "A Collection of Test Results for Beam-to-Column Connections", M.S. Thesis, Vanderbilt University, Nashville, Tennessee, U S A.

10. Kishi, N., and Chen, W. F. (1986). "Date Base of Experimental Results for Steel Beam-to-Column Connections", Research Report No. CE-STR-86-34 (in 2 volumes), School of Civil Engineering, Purdue University, West Lafayette, Indiana, U S A.

11. Lindsey, S. D. (1988). "Practical Considerations in the Design of Frames with PR Connections", in "Connections in Steel Structures: Behavior, Strength and Design", edited by R. Bjorhovde, J. Brozzetti and A. Colson, Elsevier Applied Science Publishers, Ltd., London, England.

12. Rauscher, T. R., and Gerstle, K. H. (1991). "Reliability of Rotational Behavior of Framing Connections", In "Connections in Steel Structures II: Behavior, Strength and Design", Proceedings of the Second International Workshop, Pittsburgh, Pennsylvania, U S A. April 10-12.

13. Yee, Y. L., and Melchers, R. E. (1986). "Moment-Rotation Curves for Bolted Connections", Journal of Structural Engineering, ASCE, Vol. 112, No. 3, March (pp. 615-635).

CONNECTION RESPONSE AND STABILITY OF STEEL FRAMES

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Abstract

The main results of a research project on the influence of joint flexibility on the stability of steel sway frames are presented. The attention is mainly focussed on design aspects, with reference to simplified frame analysis approaches and to joint classification as proposed by Eurocode 3.

1. INTRODUCTION

Joint response and joint action in frames have been extensively investigated in the last decade (Anderson et al., 1987; Bijlaard et al., 1989; Nethercot and Zandonini, 1988; Cosenza et al, 1988). The current knowledge allowed to develop procedures for the design of partially restrained frames, as indicated by recent European (*Eurocode 3, 1991) as well as American codes of practice (AISC-*LRFD, 1986), which explicitly permit frame design based on realistic models of joint behaviour. However, theoretical and design knowledge of semi-rigid frames is far from being exhaustive, particularly if sway frames are considered. Interest in semicontinuous sway frames has been somewhat reduced as a results of the present drift requirements under service loads: the increase of lateral deflection due to joint flexibility makes fairly difficult to meet these requirements, which are often very strict also for "rigid" frames. Joint behaviour studies pointed out that many connections traditionally classified as rigid are actually semi-rigid. In many instances, then, structural stiffness was accepted in the past on the basis of rigid frame analysis, whereas the same frames would be rejected if analysed via a more realistic semi-rigid analysis.

This paper intends to illustrate the main results of the first phase of a research project aimed at studying the effect of joint

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action on the stability of sway steel frames. The investigation considered full strength joints, as defined by Eurocode 3 (1991), and assumed (1) the joint initial stiffness (2) the frame configuration and (3) the ratio between of lateral to vertical loads as the main variables.
Besides providing a better understanding of the key behavioural characteristics, the results made it possible to investifgate some simplified methods proposed by different authors for the approximation of critical elastic and ultimate load factor. The design aspect is here emphasized, and an approach to the determination of the ultimate loading capacity, which has been developed by the authors, is also presented.

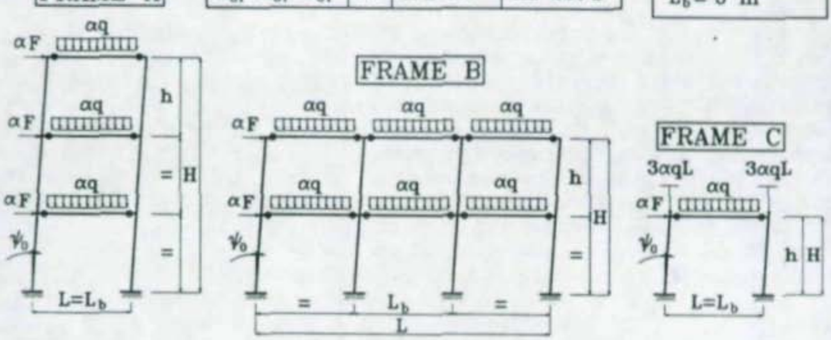
2. THE PARAMETRIC STUDY

The three frame configurations shown in Figure 1 were investigated:

FRAME	R	beams	columns
A_{WK} B_{WK} C_{WK}	1.2	IPE 200 A	HE 140 A
A_{ST} B_{ST} C_{ST}	1.2	IPE 330	HE 220 B

$q = 40\,kN/m$
$h = 4$ m
$L_b = 5$ m

FRAME A



$$\psi_0 = \frac{1}{150} r_1 r_2 \begin{cases} r_1 = \sqrt{\frac{5}{H}}, H \text{ in metres} \\ r_1 = 1 \text{ if } h \le 5 \text{ m} \\ r_2 = \frac{1}{2}\left(1+\frac{1}{n}\right) \\ n = \text{number of columns in one storey} \end{cases}$$

$$\beta = \frac{F}{q\,L}$$

$$0. \le \beta \le 0.7$$

$$R = \frac{EI_b}{L_b}\,\frac{h}{EI_c}$$

while frame A and B differ only in geometry, frame C has also a different loading condition with vertical forces applied to the columns simulating the gravity load of six additional upper storeys. Structural imperfections are simulated through out-of-plumb of amplitude ψ_0 defined as in figure. For each frame configu-

ration two main sets of column and beam sections were selected, identifying a weaker and stronger solution (labelled as WK and ST respectively). The beam to column stiffness ratio $R = (EI_b h/EI_c L_b)$ is however very similar in both solutions and close to 1.2. In order to investigate the influence of R, an analysis of the strong frames was then performed also for values R=0.8 and R=1.6, obtained by changing the column size (HE240B and HE200B respectively).
Beam-to-column joints had an ultimate moment capacity equal to the plastic moment of the beam, hence they are full strength joints. Their moment rotation response was represented via the five branch piecewise linear relation shown in figure 2. The stiffness deterioration (i.e. the ratio of the joint stiffness in the various branches to the initial value) was kept constant for all joints.
The main parameters investigated were (1) the value of the joint initial stiffness K_o (and then of the ratio ζ of the joint and the beam stiffness EI_b/L_b) and the ratio β between the horizontal forces F and the storey vertical load qL. The parameter ζ ranged from 0.5 to 25.0, covering all the semi-rigid range defined by Eurocode 3. The case of rigid joints was also analysed for comparison. The latter parameter, β, ranged from 0. (gravity loads only) to 0.7.
A numerical approach was used (Poggi, 1988) allowing for second order plastic zone analysis of partially restrained frames. Joints are modelled as inelastic springs; the beam-joint element is shown in figure 3.
Loads were increased proportionally up the the frame collapse, through a common multiplier α.

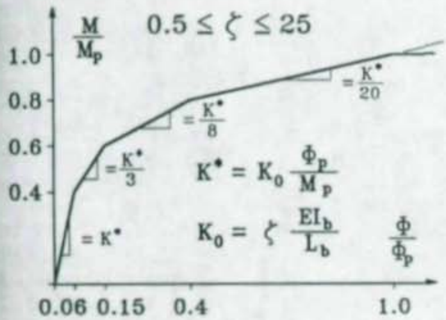


Figure 2

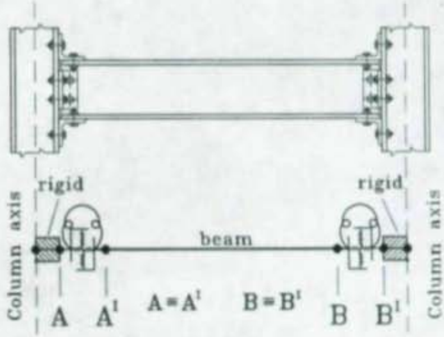


Figure 3

3. MAIN RESULTS

3.1 Elastic Critical Load

The elastic critical load was the first considered important index of the influence of joint flexibility on the overall behaviour of the frame. The relationship between the critical load multiplier

α_{cr} and the joint stiffness parameter ζ is presented in figure 4 for frames A_{ST} and B_{ST}. Joints are considered linear elastic, and fully characterised by the initial stiffness K_o. The value of $\alpha_{cr,\zeta}$ is normalised with reference to the rigid frame ($\alpha_{cr,\infty}$). The critical load is very sensitive to ζ in the lower zone of the semi-rigid range, whereas its sensitivity rapidly decreases with increasing joint stiffness. Convergence to the rigid frame multiplier $\alpha_{cr,\infty}$ is faster for the less slender frames (B and C), and for frames with higher value of R; however, at the upper bound of the semi-rigid range (ζ = 25) $\alpha_{cr,\iota}$ was in all cases greater than 90% of $\alpha_{cr,\infty}$. The results seem then to indicate that Eurocode 3 criterion satisfactorily identifies the "rigid" joint range for critical elastic analyses.

3.2 Frame Response and Ultimate Load Carrying Capacity

First and second order inelastic analyses allowed to determine the relations among joint flexibility (i.e. ζ), loading condition (i.e. β) and frame performance. The influence of joint flexibility on frame behaviour and collapse resistance proved to be very similar to that revealed by the critical elastic analysis, with a range of rather low values of ζ with a characteristically high structural sensitivity, and a range of greater ζs' in which sensitivity is rather low. The rate of convergence towards the rigid frame performance is lower for the lateral drift under service loads (computed from the ultimate load carrying capacity $\alpha_{u,II}$ assuming an "average" common γ factor equal to 1.43) than for the collapse load. The curves $\alpha_{u,II}$ - ζ are shown in figure 5 for frame C_{WK} and different values of β; in figure 6 the ultimate carrying capacity of frame A_{ST} is plotted as a function of ratio β in order to illustrate the influence of the relative importance of horizontal forces. The effect of joint flexibility on the ultimate strength reaches its maximum value in the range of β at the upper bound zone of practical interest, i.e. $0.05 \leq \beta \leq 0.20$. An assessment of the load carrying capacities suggests that the joint classification of Eurocode 3 is quite severe: values of ζ just above 5 are enough to get the difference in ultimate load within 10%, which represents an usually accepted "tolerance" for design approximation.
A comparison between first and second order ultimate strengths points out the importance of the P - Δ effect. The results indicate (see Fig. 7) that the maximum decrease in the frame resistance is, in general, still associated with values of β between 0.1 and 0.2. Then the importance of geometrical effects decreases considerably, as lateral loading increases further.

4. SIMPLIFIED DESIGN ANALYSIS

4.1 Elastic Critical Load

Methods for the design approximation of the elastic critical load of sway frames were proposed by several researchers. The so-called

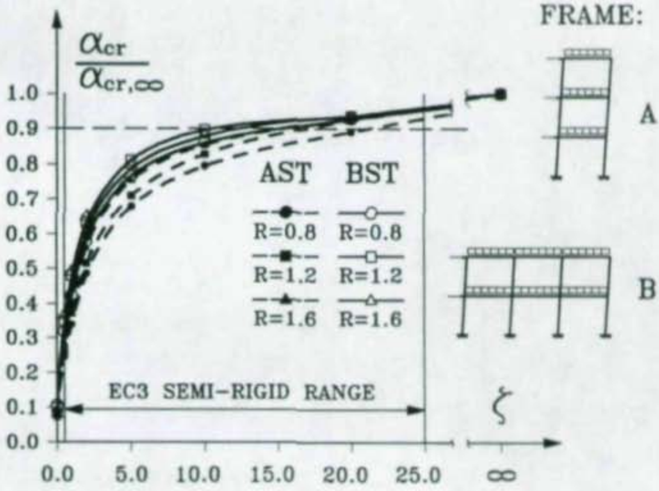


Figure 4

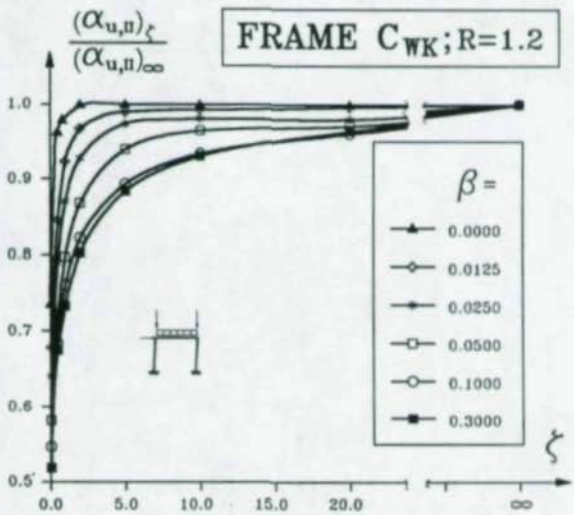


Figure 5

linear method, developed by Cosenza et al. (1988a), and the Horne method (1975) were considered in the study. Critical loads determined through these methods were compared with the numerical results.
The Horne method estimates the critical load from the maximum storey sway index computed via a linear elastic analysis under sole horizontal forces. This approach, meant for rigid frames, moderately underestimates (up to 20%) the critical load of semi-rigid frames with rather low values of ζ, and tends to become rather accurate when the joint stiffness lies in the upper zone of the Eurocode semi-rigid range (for ζ = 25 the error is always lower than 11%). In a few cases only (R = 1.6) $\alpha_{cr,\zeta}$ is overestimated (which is on the unsafe side) for high values of ζ, the error, however was never higher than 13%.
The linear method computes $\alpha_{cr,\zeta}$ through an interpolation of the values for the two limit conditions of $\zeta = 0$ (pinned joints) and $\zeta = \infty$ (rigid joints):

$$\alpha_{cr,\zeta} = \frac{A\zeta\alpha_{cr,\infty} + \alpha_{cr,o}}{A\zeta + 1} \tag{1}$$

with A being a constant, which Cosenza et al. propose to compute as $A = (2 + 3R)/12$. By using this relation a very satisfactory estimate was obtained for single bay frames (A and C) for which maximum error was 5%. The relation proved less accurate for multibay frame B, for which underestimation of α_{cr} was as high as 25%. Greater accuracy can be obtained if reference is made to the equivalent Grinter frame, and A is computed on the basis of a modified value od R, i.e.

$$R_{eq} = \frac{EI_{b,eq}\,h}{EI_{c,eq}\,L_b}$$

with

$$I_{b,eq} = \sum_{1}^{n-1} I_b \qquad \text{and} \qquad I_{c,eq} = \sum_{1}^{n} I_c$$

Maximum error is then reduced to 13%.

4.2 Ultimate carrying capacity

Although second order analysis represents the only fully adequate design tool for sway frames, simplified approaches are certainly of interest, at least, in the preliminary design stages.
Jaspart (1988) already investigated the applicability of the Merchant-Rankine approach to partially-restrained sway frames. Due

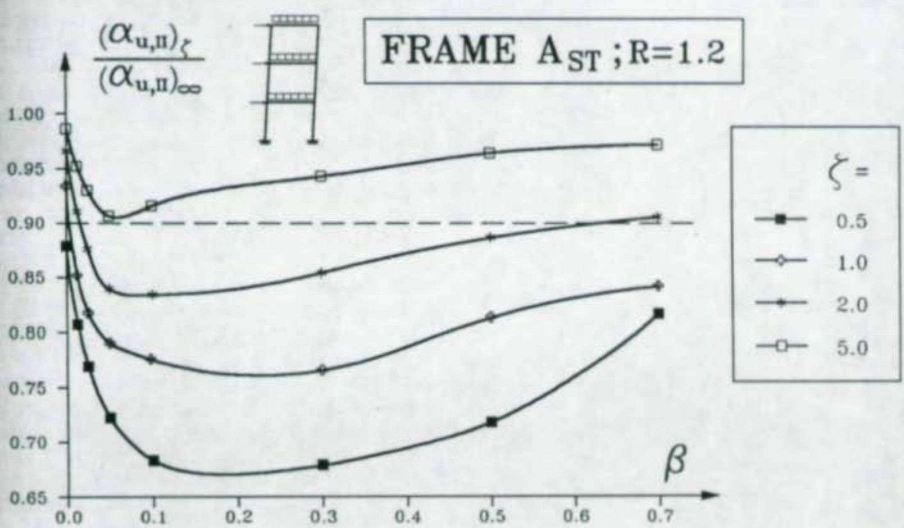


Figure 6

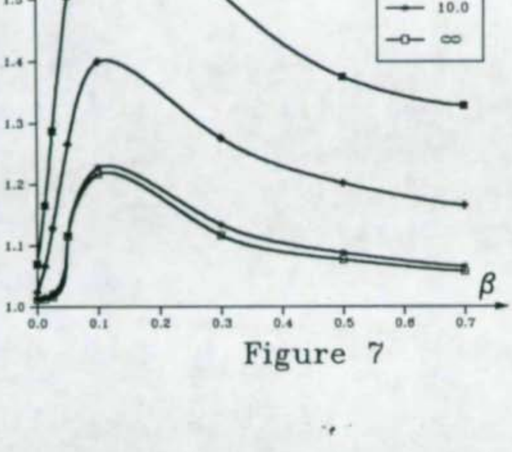


Figure 7

to the wider range of the parameters considered, a more general assessment of this approach was undertaken in the framework of this study. The critical elastic loads $\alpha_{cr,\zeta}$ computed on the assumption of linear joints (section 4.1) were used in the Merchant-Rankine formula.
A comparison with the "actual" values of $\alpha_{u,II}$ reveals that the approach is conservative for rather low values of the ratio ß between the horizontal and vertical loads, and becomes unconservative as ß increases (e.g. see figure 8 where the results of frame C_{WK} are shown). Error ranges from 38% on the conservative side to 44% on the unsafe side, with the lower values associated to frame B. As expected, agreement was satisfactory only for frames, whose deformation at collapse is similar to the critical deflected shape: i.e. for values of ß between 0.0125 and 0.05. For lower ßs' the frames failed generally for beam mechanism (Fig. 9a), whereas the panel mechanism (Fig. 9c) was associated with the collapse of frames with greater horizontal forces. This was confirmed by a Merchant-Rankine prediction based on a value of the "critical" load, obtained through a second order elastic analysis under solely vertical loads, but accounting for joint nonlinearity. The use of this elastic limit load significantly improved the approximation in cases with high horizontal forces; however, underestimation of the ultimate capacity for low values of ß (which is the most important situation for practice) becomes still more remarkable.
On the other hand, joint behaviour is quite different from the ideal elastic-plastic model; the inelastic response of partially restrained frames is consequently different from the traditional assumptions of plastic analysis, to which the Merchant-Rankine approach refers. Jaspart (1988) suggested to modify the method by introducing a pseudo-plastic moment of the joint in the calculation of the first order collapse multiplier. This approach leads, in effect to a moderate improvement of the degree of accuracy, mainly *for frames, the mechanism of collapse of which involves* joint plastification, i.e. for rather high ßs' and ζs' (Fig. 10). The approximation obtained is again satisfactory for moderate to medium ßs', whilst the method tends to be rather conservative for frames subject predominantly to vertical loads and quite unconservative for frames subject to high lateral forces.

Evaluation of the curves $(\alpha_{u,II}/\alpha_{u,I})_\zeta$ versus ß permitted a tentative development of an alternative approach relating the ultimate resistance of a partially restrained frames to the first order plastic multiplier of the rigid frame $(\alpha_{u,I})_\infty$, the number n of storeys, the ratio ß, and the critical multipliers $\alpha_{cr,\zeta}$ and $\alpha_{cr,\infty}$ through the following expression

$$(\alpha_{u,II})_\zeta = (\alpha_{u,I})_\infty \frac{1}{C_\beta} \sqrt[3]{\frac{\alpha_{cr,\zeta}}{\alpha_{cr,\infty}}} \qquad (2)$$

where C_β is a factor depending on the number of storeys and on the parameter ß:

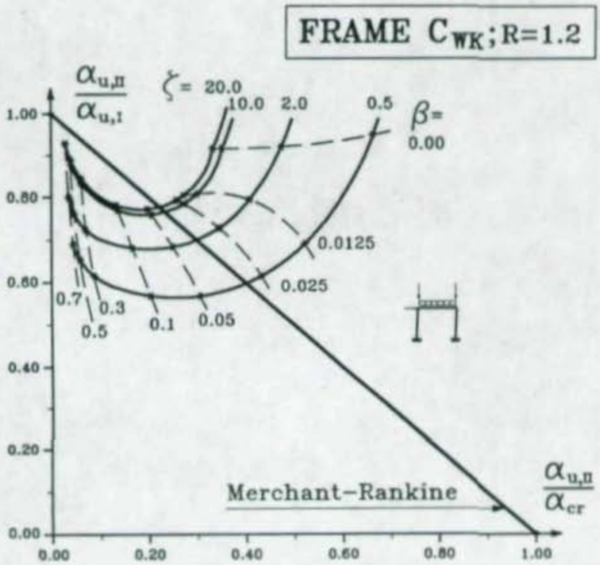


Figure 8

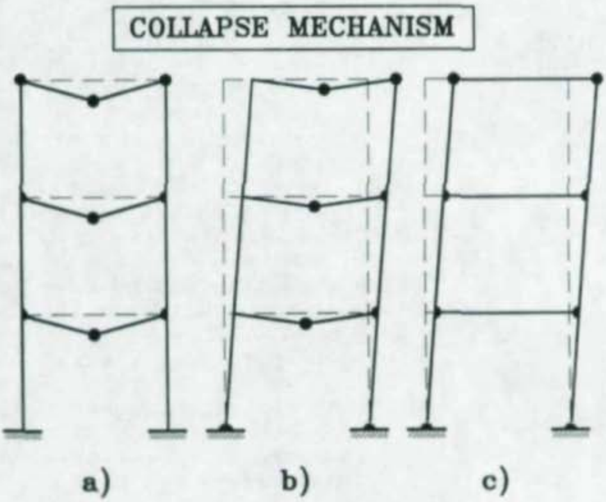


Figure 9

$$C_\beta = \sqrt{n}\beta + a \qquad \text{for } \beta \le 0.10$$

$$C_\beta = -\frac{1}{9}\ [(\sqrt{n} + 0.5)\beta - \sqrt{n} - 10a + 1) \qquad \text{for } \beta \ge 0.10$$

with a equal to the value of the ratio $(\alpha_{u,I}/\alpha_{u,II})_\infty$ for $\beta = 0$.

Assuming for a the average value $a = 1.05$, equation 5 gives generally a conservative estimate of $\alpha_{u,II}$ also for high horizontal loads. Underestimation of the collapse load is of less than 20% in average for frames with rather flexible joints, and tends to become lower than 10% for frames with $\zeta \ge 10$. Only for frame C_{WK} does the formula significantly overestimate (up to 15%) the ultimate load. If the value of $\alpha_{cr,\zeta}$ is estimated through the linear method the proposed formula permits approximation of the ultimate resistance of a partially restrained frame via analysis based on the traditional ideal models of pinned and rigid joint. The accuracy is however somewhat reduced.

4. SUMMARY AND CONCLUSIONS

The results of an extensive numerical study of the joint flexibility influence on the performance of steel sway frames have been summarily reported, and discussed as part of an assessment of design criteria and simplified approaches. The main indications provided by the analyses of the frames in figure 1 are the following:

(1) Sensitivity of frame response to joint flexibility is high in the range of rather flexible joints, and tends to decrease quite rapidly with increasing joint stiffness. The rate at which the performance approaches rigid frame behaviour depends on the parameter considered. If an approximation of 10% is accepted in design, the lower bound of the rigid joint range is fairly accurate as far as the determination of the elastic critical load, and rather conservative with reference to the ultimate load capacity.

(2) Serviceability checks should indeed be critically reviewed so that a better consistency is achieved with respect to the use of different analysis models. This would probably enlarge the practical scope of use of partially restrained sway frames.

(3) The critical load multiplier can be computed with satisfactory accuracy by means of the so-called linear method. The approximation may be further improved for multibay frames, if reference is made to the equivalent Grinter frame.

(4) The Merchant-Rankine approach is suitable for determining the ultimate resistance when lateral loads are relatively small with respect to vertical loads ($\beta \le 0.05$). For greater values the method substantially overestimates $\alpha_{u,II}$.

(5) An approach seems possible which obtains the ultimate load

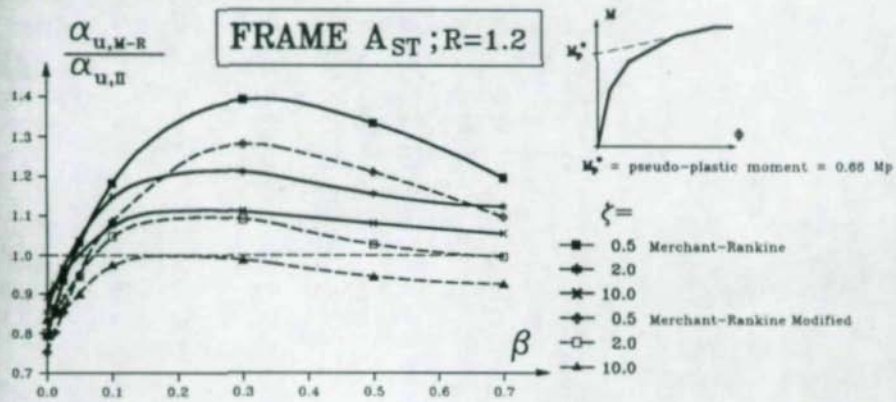


Figure 10

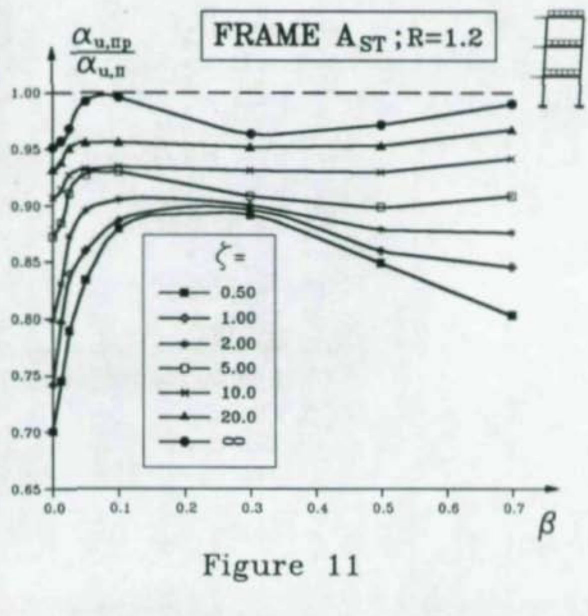


Figure 11

factor of a partially restrained frame by suitably reducing the plastic collapse load of a rigid frame (see equation (2)).

5. ACKNOWLEDGEMENTS

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6. REFERENCES

-Anderson D., Bijlaard F., Nethercot D.A. and Zandonini R., (1987)," Analysis and Design of Steel Frames with Semi-Rigid Connections", International Association for Bridge and Structural Engineering (IABSE), Survey S-39/87, April.

-Bijlaard F., Nethercot D.A., Stark J. and Tschemmernegg F. (1989), "Structural Properties of Semi-Rigid Joints in Steel Frames", International Association for Bridge and Structural Engineering (IABSE), Survey S-42/89, May.

-Nethercot D.A. and Zandonini R. (1988), " Methods of Prediction of Joint Behaviour: Beam-to-Column Connections", Stability and Strength series, Vol. 8 'Structural Connections', Ed. Narayanan R., Elsevier Applied Science Publ., London.

-Cosenza E., De Luca A. and Faella C. (1988a), "Elastic Buckling of Semi-Rigid Sway Frames", Stability and Strength series, Vol. 8 'Structural Connections', Ed. Narayanan R., Elsevier Applied Science Publ., London.

-Cosenza E., De Luca A. and Faella C. (1988b), "Inelastic Buckling of Semi-Rigid Sway Frames", Stability and Strength series, Vol. 8 'Structural Connections', Ed. Narayanan R., Elsevier Applied Science Publ., London.

-Commission of the European Communities (1991), "Eurocode 3 - Design of Steel Structures - Part 1 General Rules and Rules for Buildings".

-American Institute of Steel Constuction (1986), "Load & Resistance Factor Design - Manual of Steel Construction ", First Edition, USA, 1986.

-Poggi C. (1988), "A Finite Element Model for the Analysis of Flexibly Connected Steel Frames", International Journal for Numerical methods in Engineering, Vol. 26.

-Horne M.R. (1975), "An approximate method for calculating the elastic critical loads of multi-storey plane frames", The Structural Engineer, Vol. 53, No. 6, June.

-Jaspart J.P. (1988), "Extending of the Merchant-Rankine Formula for the Assessment of the Ultimate Load of Frames with Semi-rigid Joints", Journal of Constructional Steel Research, No. 11.

PREDICTION OF THE INFLUENCE OF CONNECTION BEHAVIOUR ON THE STRENGTH, DEFORMATIONS AND STABILITY OF FRAMES, BY CLASSIFICATION OF CONNECTIONS.

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Abstract

In engineering practice, often the question arises what the influence is of the mechanical behaviour of the connections between the beams and the columns within a structural framework on the distribution of forces and moments, and the stability, deformations and displacements of such a framework. This paper presents the Eurocode-3 classification (Eurocode, 1990), which is a design aid for determination of connection behaviour with regard to the general mechanical behaviour of the structure, and an alternative method. These design aids can be used to predict whether the flexibility of a connection can be neglected or not with respect to the general mechanical behaviour of the structure, and may be of great help in the common structural designers practice.

1. INTRODUCTION

In Eurocode 3, criteria are given for nominally pinned, semi-rigid and rigid beam-to-column connections, when the distribution of forces and moments in the structure is determined with either the elastic or the plastic theory. The structural properties of beam-to-column connections, such as stiffness, strength and rotation capacity, should be in accordance with the assumptions made in the design of the structure. In figure 1 the structural properties of a beam-to-column connection is indicated qualitatively.

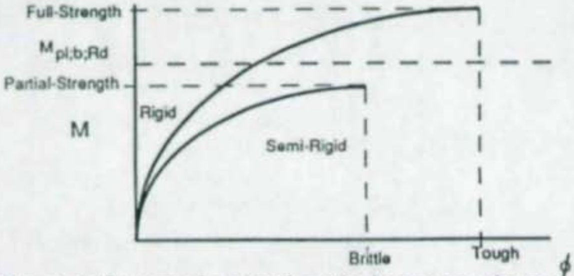


Figure 1: Structural properties of a beam-to-column connection presented in the moment-rotation diagram.

The classification of beam-to-column connections provides the designer a quick answer to the question how a certain beam-to-column connection, of which the properties are characterized in the moment- rotation diagram, will behave in the structure. This behaviour can either be rigid or semi-rigid with respect to stiffness, full-strength or partial-strength with respect to moment capacity and plastic or brittle with respect to rotation capacity. In the following sections the boundaries between the areas of behaviour of beam-to-column connections are treated. The elaboration of the classification as presented in Eurocode 3 together with a classification for standardization of beam-to-column connections are given.

2. BOUNDARIES

The distribution of forces and moments in a structure is influenced by the flexibility of the connections in that structure. This also holds for the stability, the deformations and displacements.

The question arises when the flexibility of beam-to-column connections may be neglected. In other words in what cases can the connection be assumed to be rigid and when as a hinge. This depends on the stiffness ratio between the beam-to-column connection and the connected beams and columns. The structural behaviour will be analyzed by showing the relationship between the parameters $\bar{c}$ and ρ. The parameter $\bar{c}$ is the relative rotation stiffness

$$\bar{c} = c \cdot \frac{l_l}{EI_l}, \tag{1}$$

in which c is the rotation stiffness of the beam-to-column connection and

$$\frac{EI_l}{l_l} \tag{2}$$

is the flexural stiffness of the beam. The parameter ρ is the ratio between the flexural stiffnesses of the beam and the column

$$\rho = \frac{EI_l \cdot l_k}{EI_k \cdot l_l} \tag{3}$$

in which

$$\frac{EI_l}{l_l} \tag{4}$$

is the flexural stiffness of the beam and

$$\frac{EI_k}{l_k} \tag{5}$$

is the flexural stiffness of the column. On the hand of a one bay one storey unbraced and similar braced frame the relationship between $\bar{c}$ and ρ can be determined. In figure 2 this frame is shown.

This can be carried out at a constant ratio between the Euler buckling load of the frame with semi-rigid connections end the Euler buckling load of the same frame but now with rigid connections. For this ratio the value 0.95 is chosen.

In figure 2 the relationship between $\bar{c}$ and ρ is shown for the unbraced as well for the braced frame. The mathematical background of these relationships is given in (Meijer, 1990).

On the basis of the Merchant-Rankine formula

$$F_{cr} = \left(\frac{1}{F_{pl}} + \frac{1}{F_E}\right)^{-1} \qquad (6)$$

for the determination of the carrying capacity of the unbraced frame it can be shown that the carrying capacity F_{cr} will drop by not more then 5% in case

$$F_{E(\bar{c})} = 0{,}95 \,.\, F_{E(\bar{c} = \infty)} \qquad (7)$$

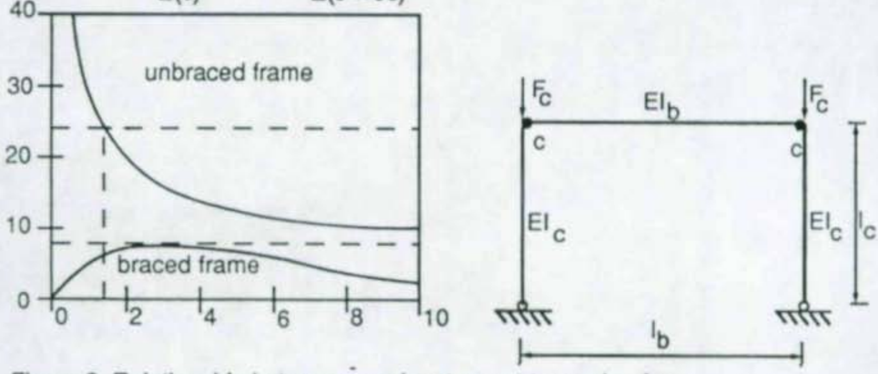


Figure 2: Relationship between $\bar{c}$ and ρ at a constant value for

$$\frac{F_{E(\bar{c})}}{F_{E(\bar{c}=\infty)}} . 100\% = 95\% \qquad (8)$$

Figure 2 gives the relationship between the geometry of the frame and the ratio of flexural stiffnesses between the connection and the beam for those rotation stiffnesses of the connection which can be assumed to be perfectly rigid, because the flexibility of the connection causes a drop of the carrying capacity of the frame of not more than 5%.

3. CLASSIFICATION ACCORDING EUROCODE 3

If the design of the frame must be verified against the requirements, all the geometrical data are known. These are the sections used as columns and beams together with the lay-out of the beam-to-column connections. From the lay-out of the connections the moment-rotation relationship can be determined on the basis of Annex J of Eurocode 3. With that data the parameters $\bar{c}$ and ρ can be determined and via figure 2 it can be shown how much influence the connection stiffness has on the distribution of forces and moments and on the stability of the frame.

If the design has to be made yet not all the data is known and it is difficult to estimate before hand what influence the connections will have on the behaviour of the frame. In Eurocode a further simplification is given for this classification to estimate the connection influence.

By choosing a constant boundary value for the parameter factor $\bar{c}$ this parameter becomes independent of the parameter ρ. For braced frames the boundary value is $\bar{c} = 8$ and for unbraced frames this boundary value is $\bar{c} = 25$.
The lines of $\bar{c} = 8$ and $\bar{c} = 25$ are drawn in the graph of figure 2. In that figure it can be seen that the boundary value $\bar{c} = 8$ covers the ρ-$\bar{c}$- relationship for braced frames completely. The boundary value $\bar{c} = 25$ for unbraced frames covers the ρ-$\bar{c}$-relation only if $\rho \geq 1.4$. For $\rho < 1.4$ the boundary value $\bar{c} = 25$ is in principle unsafe. This is not really a problem because in (Bijlaard, 1991) has been shown that at the value of $\rho = 0.1$ the Euler buckling load based on $\bar{c} = 25$ would not more than 85% of the Euler buckling load if the value for $\bar{c}$ would be $\bar{c} = \infty$. For relative slender frames it holds that the carrying capacity of the frame based on the Merchant-Rankine formula drops as follows:

$$F_{E(\bar{c} = 25)} = F_{pl} \Rightarrow 8\% \text{ and} \tag{9}$$

$$F_{E(\bar{c} = 25)} = 2.F_{pl} \Rightarrow 5{,}6\% \,. \tag{10}$$

Frames for which holds that $\rho < 0.1$ are not very realistic, so the value $\rho = 0.1$ can be used as a boundary. It can be concluded that $\bar{c} = 25$ is a sufficiently safe boundary value for the rotation stiffness of beam-to-column connections in unbraced frames in order to consider them as rigid, provided that holds $F_E/F_{pl} \geq 1$. Now, the boundaries for the stiffness are determined.

As far as the moment capacity of beam-to-column connections is concerned the question of classification is more simple. If the moment capacity of the beam-to-column connection is equal to the plastic capacity of the connected beam, the connection is considered as full- strength. If not, the connection is considered as partial-strength.

On the basis of the boundary for the rotation stiffness and the boundary for the moment capacity a bi-linear moment-rotation characteristic is achieved. This bi-linear boundary is rather severe for classifying beam-to-column connections, when it is compared with the moment-rotation characteristic of a beam section. If the moment acting on a beam section exceeds the elastic moment capacity

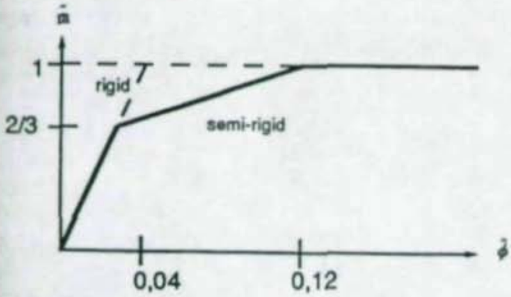
$$M_e = 0.85\ M_{pl} \qquad (11)$$

for an I-section, then plastification will occur and the stiffness will decrease. If residual stresses are taken into account plastification will start already at

$$M = 0.7\ M_{pl}. \qquad (12)$$

So it is reasonable to cut off the bi-linear characteristic with a third branch. From tests it appeared that beam-to-column connections with end-plates have an elastic behaviour up to at least 2/3 of the moment capacity of the connection. Based on that, for full-strength connections it is required that the elastic behaviour is present up to at least 2/3 $M_{pl,beam}$.

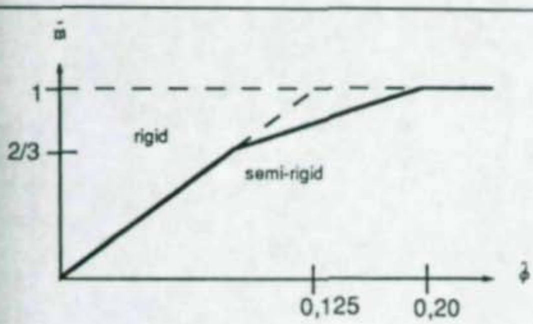
Theoretically the plastic capacity of a beam section is reached at an infinitively large rotation of the plastic hinge. In practice already a large percentage of the plastic moment capacity is reached at a relative small rotation. In figure 3 the boundaries are shown for braced and unbraced frames as given in Eurocode 3.



(a) Unbraced frames

if $\bar{m} \leq 2/3$: $\bar{m} = 25\,\bar{\phi}$

if $2/3 < \bar{m} \leq 1.0$: $\bar{m} = (25\,\bar{\phi} + 4)/7$



(b) Braced frames

if $\bar{m} \leq 2/3$: $\bar{m} = 8\,\bar{\phi}$

if $2/3 < \bar{m} \leq 1.0$: $\bar{m} = (20\,\bar{\phi} + 3)/7$

$$\bar{m} = \frac{M}{M_{pl;\ beam}} : \bar{\phi} = \frac{\phi\ E\,I_b}{l_b\,M_{pl\ ;beam}}$$

Figure 3: Boundaries for the classification of beam-to-column connections according to Eurocode 3.

In figure 3 both axes of the moment-rotation characteristics are normalized by dividing the moment by the plastic moment capacity of the beam, so:

$$\bar{m} = M / M_{pl;beam} \tag{13}$$

and by dividing the rotation by a reference-rotation, so:

$$\bar{\phi} = \phi/(\frac{M_{pl;beam} \cdot l_{beam}}{EI_{beam}}) \tag{14}$$

A classification with respect to rotation capacity is not yet possible in case of unbraced frames. Only in cases where the moment capacity of the beam-to-column connection is larger than

$$1.2\, M_{pl;beam}, \tag{15}$$

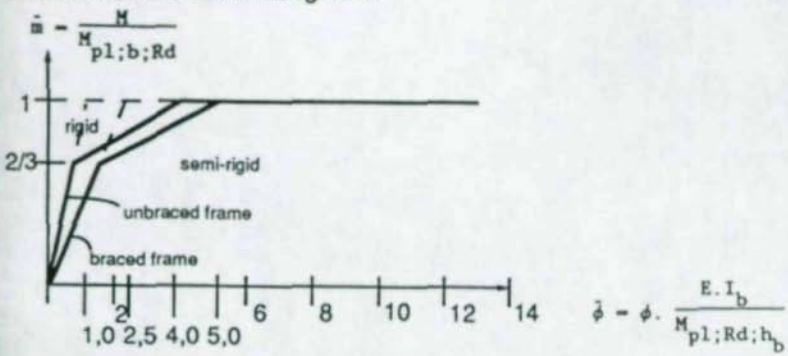
the rotation capacity need not be checked. The plastic hinge will always form in the beam section adjacent to the connection. In other cases the rotation capacity need be checked if redistribution of moments has been taken into account. For unbraced frames the required rotation capacity has to be calculated and checked against the rotation capacity which is available in the connection. For braced frames the required rotation capacity can be determined by looking at beam-mechanisms which will form.

4. CLASSIFICATION FOR THE PURPOSE OF STANDARDIZATION

An objection to the classification indicated in Eurocode is that judging the influence of the connection behaviour on the frame behaviour can only take place when the beam length is known. If only the lay-out of the connection is known, it can be desirable to classify the connection. This is the case with standardization of connections, where the the beam section and the column section together with the lay-out of the connection itself is known, but not the structural surrounding of it. In establishing a classification fit for this purpose, use can be made of the knowledge that for rolled beam sections the span is approximately 20 times to 50 times the section height. In normalizing the rotation-axis of the moment-rotation characteristic the rotation is divided by another reference rotation, namely:

$$\bar{\phi} = \phi / \left(\frac{M_{pl;beam} \cdot h_{beam}}{E\ I_{beam}}\right) \tag{16}$$

In case of an unbraced frame a connection is classified as rigid if $\bar{c} > 25$ and in case of an umbraced frame a connection is classified as rigid if $\bar{c} > 8$. Assuming a beam length equal to 20 (braced) or 25 (unbraced) times the beam height, this leads to a classification like shown in figure 4:



If $\bar{m} > 1.2$ than the rotation capacity need not to be checked
If $\bar{m} < 1.2$ than the rotation capacity need to be checked

Figure 4: Boundaries for the classification with standardized beam-to- column connections

5. INTERPRETATION FOR UNBRACED MULTI-STOREY FRAMES AND MULTI-BAY FRAMES

In practice, multi-bay and multi-storey frames appear often. The question arises how to use the classificationmethods in the case of these frames.

Figure 5 shows a two storey unbraced frame. In order to make a Euler buckling load calculation, a kinematic model of the frame is given. It is assumed that the columns of the frame will have a point of contraflexure somewhere between the first floor and the roof on distance: g.l (where 0 < g < 1) and that this point of contraflexure will not move if the connection stiffness is taken into acount (Meijer, 1990). Further it is assumed that the force F_2 will be also a fraction of the force F, so F_2= f.F (where 0 < f < 1). The stiffness of both the beam on the first floor and the connections is represented with a rotational stiffness C, and the column is assumed to have stiffness EI.

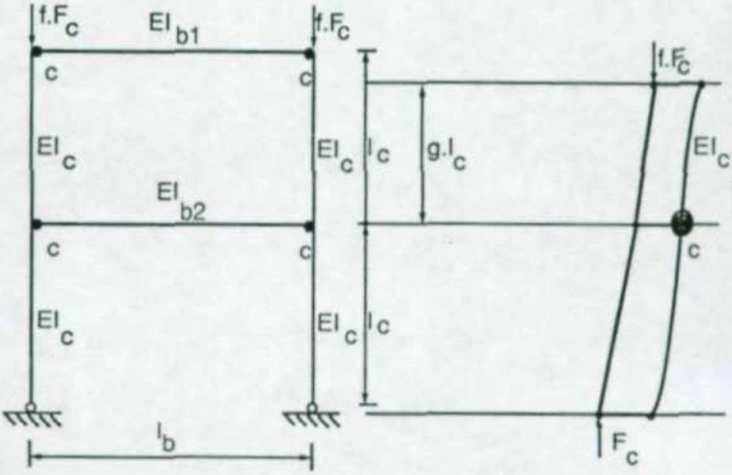


Figure 5: Two storey unbraced frame and it's kinematic model

Numerical experiments to this kinematic model proved (Steenhuis, 1991) that the error due to the use of the classification based on the single-storey theory in this case causes a maximum drop in in the Euler buckling load for extreme values of f=g=1 like shown in figure 6.

For lower values of f and g it appears that the error mostly is 0% percent. Following the reasoning that the drop of the Euler buckling load is of moderate influence on the second order elasto-plastic failure load, it can be concluded that the classifications based on research at one-storey frames are also applicable for multi-storey frames.

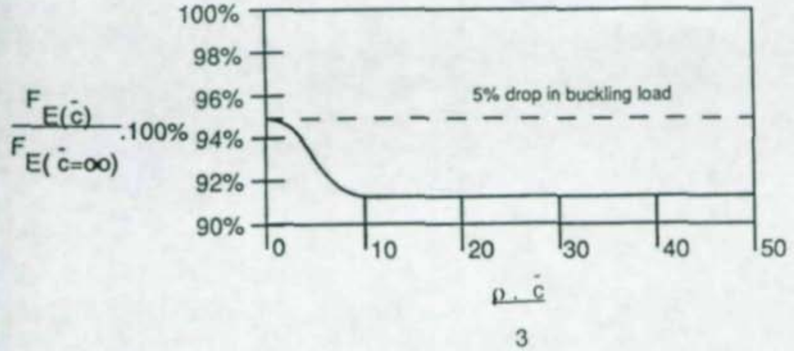


Figure 6: Results of error-analyses at a two-storey frame if f=g=1 for values of $\frac{\rho \cdot \bar{c}}{3}$

For braced frames, the classification based on research at single-storey frames can also be used for multi-storey frames.

(Meijer, 1990) proposed and tested in some practical situations a distribution of the connection stiffness equal to the relative stiffness of the joined beams. From his work, it can be concluded, that the classification-methods based upon single-bay reseach can be used for multi-bay frames as well.

6. OTHER PROPERTIES

The use of the proposed classification methods can lead toward connection design with a rigidity which cause a drop in second-order buckling load less then 5 percent of the second order bucking load with infinity rigid connections. Based upon this classification, (Meijer, 1990) calculated some practical examples of frameworks and noticed, that the deformations of the frames (especially horizontal deformations) may be underestimated by more than 5 percent locally. However, it was found that this was no problem, because the deformations were smaller than the maximum allowable deformations.

7. CONCLUSIONS

In order to classify connection in terms of both strength (full-strength, partial-strength and nominally pinned) and stiffness (rigid, semi-rigid and nominally pinned), the Eurocode provides a classification method, which is regardless to the certain properties like the length of columns and the number of storeys and bays. In addition to this method, an alternative classification which is also regardless to the length of beams is presented.

References

- Bijlaard F.S.K. et.al. - SG course about new codes EC-3 and TGB-Staal (in Dutch) - TNO Building and Construction research - March 1991.
- Eurocode 3 Editorial Group - Eurocode 3, Design of Steel Structures - Edited draft issue 53 - November 1990
- Meijer H.S. - Influence of the rotational stiffness of column-beam connections on the behaviour of braced and unbraced frames (in Dutch) - Technical University Eindhoven - August 1990
- NNI - NEN 6772:1992 TGB 1992 Steel structures Connections (in Dutch) - March 1991
- Steenhuis C.M. - Numerical calculations on a two storey frame - BI-91-050 - TNO Building and Construction research - March 1991.

SEMI-RIGID CONNECTIONS IN LATTICE GIRDERS FORMED WITH HOLLOW STRUCTURAL SECTIONS.

Gwynne Davies[1].

Abstract

The semi-rigid nature of various types of structural hollow section joints used in lattice girders are described, and the effect of the joint parameters on strength and stiffness indicated. Modelling is seen to be more complicated than for beam and column connections between two members. Some comparisons are made between gap and lap joints in trusses.

1. INTRODUCTION

Steel Structural Hollow Sections (SHS) are widely used in construction and are accepted as part of the family of structural steel sections. They are popular in construction because of their structural efficiency, particularly when loaded in compression, tension or torsion, and their clean lines which give architects and engineers freedom to feature the structure as part of the overall building design. The predominant use of welding for making the connections between SHS retains the structural efficiency and clean lines by maintaining member effectiveness, whilst dispensing with the need for gussets and other unsightly connection devices. The size of SHS used in buildings normally excludes the use of internal stiffening. Rectangular Hollow Sections (RHS) were introduced to avoid the expensive end profiling required when using Circular Hollow Sections (CHS), making easier welding and also providing greater resistance to bending by suitable choice and orientation of the RHS. These are largely used for on-shore work or for topside fabrication in off-shore structures. The use of welding however in no way guarantees that the members are effectively riigidly connected. Indeed the joint usually behaves in a more flexible manner having semi-rigid properties for both rotational as well as axial behaviour.

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Both the strength and stiffness of these joints depend on the various geometric parameters, and the type of joint. Lattice girders function best when the forces are predominantly axial, and the bending moment effect is minimal - they then nearly conform to the ideal pinned truss. Some secondary moments will however be set up due to the semi-rigid nature of these joints. The normal approach to the design of lattice frames is to ensure that member centre lines are concurrent (noding) and then to calculate member axial forces, assuming pinned connections at all joints. However since the chords are themselves formed from one or two rolled lengths there is justification for assuming that chord interconnections are rigid but that the branch connections to the chords (or to each other) will vary from pinned to rigid. In hollow section construction centreline noding may give rise to design and fabrication problems, leading to less than optimum efficiency and fabrication economy, and there are times when it is more important to ensure fabrication efficiency than to maintain centreline noding. Clearly there would be considerable analytical advantage in ensuring joints which were rigid axially, and either be rigid or have low rotational stiffness, so that standard computer programs could be used for analysis.

2. PARTIALLY RIGID SHS CONNECTIONS

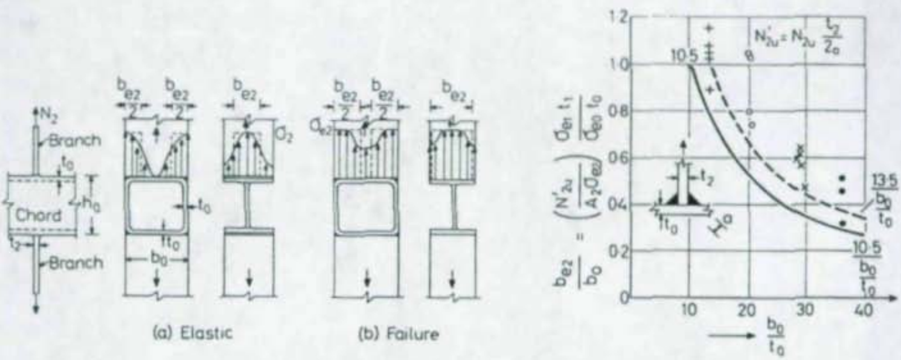

Fig.1. Effective width of cross - plate welded to RHS

The difference in the stress distributions in the two plate connections shown in Fig.1 for a full width joint, is the basis for understanding the difference in behaviour between SHS and I type connections, whether for columns in building frames or chords in lattice girders. Whereas the stiffpoint is at the centre for the I section connection it is on the outside and adjacent to both the webs for the RHS. The configuration shown represents the position of greatest connection stiffness which can be achieved. It is important to note that a reduction in the relative width of the branch does not produce a significant change in stiffness of the I connection, but can bring about a major reduction in strength and stiffness in the case of the RHS since the connection is now made entirely through the relatively flexible connecting chord wall. As the width ratio $\beta = b_1/b_0$ decreases, so also do both the strength and the stiffness of the connection.

This is illustrated in Fig.2, where the yield line method of analysis has been used to examine the variation of the branch axial yield load with the various parameters for a 90° joint according to Eq.1.

$$N_y = \frac{8m_{po}}{(1-\beta)} \left(\frac{h_1}{b_1} \cdot \beta + 2\sqrt{1-\beta} \right) \qquad \text{..........(1)}$$

where $m_{po} = \sigma_{eo} t_o^{\,2}/4$

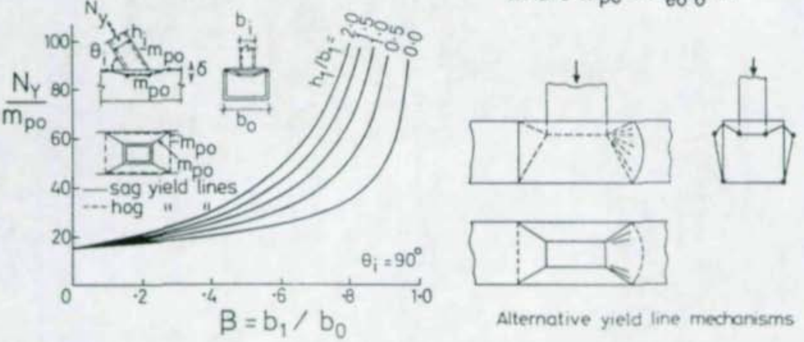

Fig.2. Yield line models for T and X - RHS joints.

In the graph shown only the connecting wall yield mechanism is used, as it has been found that the alternative collapse mechanisms shown give higher values of yield load. It has also been shown that the local deformation of the joint is affected by the same joint parameters, and that the axial and rotational stiffnesses for a T joint as derived by a finite difference approach (Korol and Mansur,1979) is shown in Fig.3.

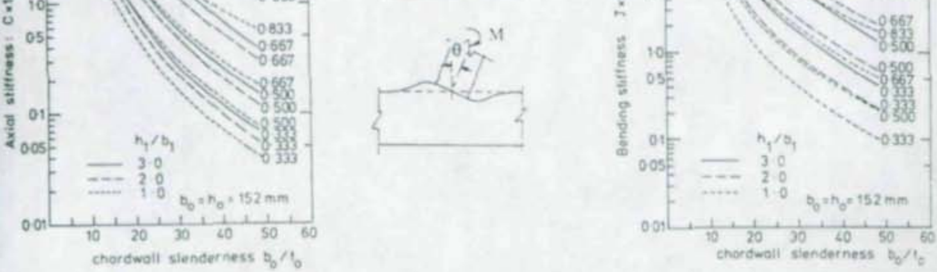

Fig.3. Axial and rotational stiffness for RHS Tee joint. (Korol and Mansur,1979)

Yield line analysis for moment connections can also be carried out , and typical results are shown in Fig.4, for top face deformation only. For near fullwidth joints a different model is needed to account for chordwall slenderness. The stiffness of the joint is also evaluated in terms of the three parameters

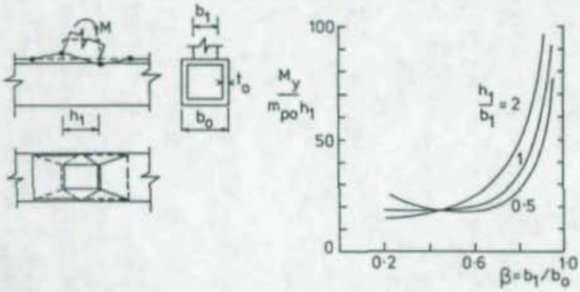

Fig.4. Yield line model for T joint moment

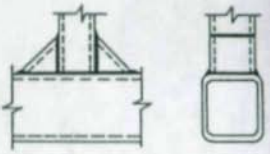

Fig.5. Near full-full width T joint

b_0/t_0, h_1/b_1, and width ratio b_1/b_0 which are all seen to be important. As the width ratio increases so the stiffness increases significantly - Fig.4, particularly as the full width joint is approached. When used for Vierendeel girders, there is considerable merit in ensuring near rigid joints by welding on the corner stiffeners shown in Fig.5.

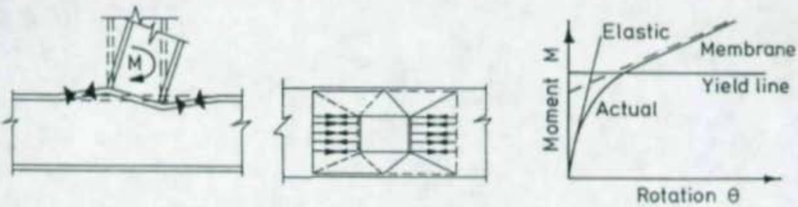

Fig.6. Joints with membrane action.

In some of the more slender walled chords, the connection has a reserve of strength due to membrane action in the connecting wall associated with large local deflection, especially for joints with smaller width ratios, an example being shown in Fig.6. This has also been observed for branch axial loading, particularly in tension. For modest values of β this effect can readily be calculated from yield line analysis, and rigid plastic membrane action. For joints in axial compression, buckling or deformation of the chord side walls will ensure that the enhanced strength will be less than that in tension. Although the T and X joints are not frequently used in lattice girder connections they do illustrate the essential elements of semi-rigid behaviour of joints in hollow sections. The design expression used for the strength of the K gap joints in RHS shown in Fig.7 has been derived from the empirical results of a significant programme of tests on isolated joints. Because of the increased options for geometrical layout offered by the rectangular section, particularly when member orientation is allowed for, the design recommendations have become rather more complicated, compared to those needed for circular or square sections. A very small gap can also introduce a very stiff point in the crotch, resulting in sudden rupture of the chord or tie wall, while an excessively large gap will allow large local deflections with corresponding increased bending moments in the members, even if the centre lines

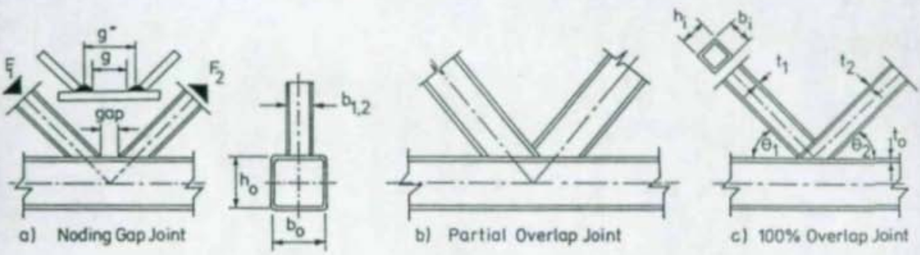

Fig. 7 Typical RHS joints

are noding. The bending moments set up are truly secondary moments and are not due to the absolute rigidity of the joint, but rather associated with the various components of axial and flexural stiffnesses in what is really a multi semi-rigid joint. Although a whole range of partially overlapping joints could be arranged to ensure centre line noding - as shown in Fig.7, they involve double cutting of at least one of each pair of branch members, thus introducing extra fabrication costs. It has frequently been recognised that there is merit in encouraging the use of fully overlapped joints, to take advantage of the greater axial stiffness of the connection resulting from the very small local deformations which have been observed in tests. There is of course the ensuing penalty of larger chord bending moments associated with joint eccentricity, but these have to be balanced against improvements in performance, and simpler joint design procedures.

3. JOINTS IN TRUSSES.

3.1 Elastic behaviour.

Lattice girders are commonly analysed in practice assuming rigid joints, most commercial stiffness packages also incorporating the opportunity to provide pinned or semi rigid connections at the ends of the members. The nature of the connection of members in gap joints however is not merely a matter of providing semi-rigid end connections, since there is interaction between more than two members. In order to understand this problem the finite element method has been used (Coutie and Saidani, 1990) to derive the stiffness matrix of a typical RHS connection -shown in *Fig.8, in terms of the forces and deformations at each joint extreme centreline and loading point. By imposing individual unit deformations at these points in turn a 12X12* (or 15X15 if loading point included) *joint stiffness matrix was obtained, which included* the effect of size and form of the welds and local flexibilities involved. The usual prismatic beam elements were then interconnected through the above nodes. This has

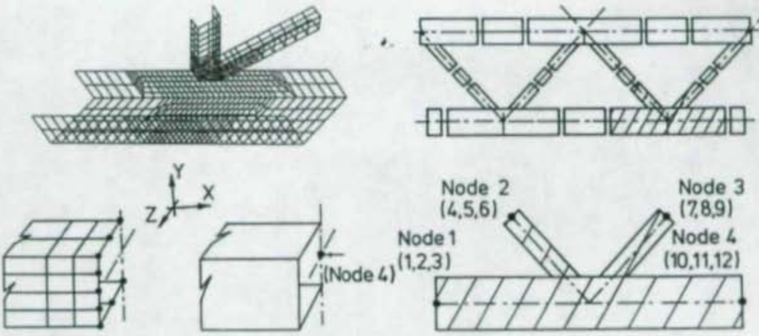

Fig.8. Elastic FE analysis of K joint in lattice girder (Coutie and Saidani, 1990)

enabled the structure stiffness matrix for the whole truss to be derived and analysed for any of the common systems of loading. In this way the essentially semi-rigid nature of the joints have been automatically incorporated into the elastic truss analysis.

3.2 Non linear behaviour.

Comparisons between the behaviour of experimental tests on isolated RHS K gap joints with the non linear results obtained from FE analysis incorporating the effects of material and geometric nonlinearity have been made (Strommen, 1982) .Other investigators (e.g Davies, 1990) have examined RHS cross joints with inclined branches and carried out parametric studies. Three dimensional CHS cross joints under different loading conditions have also been examined (Paul, 1990).

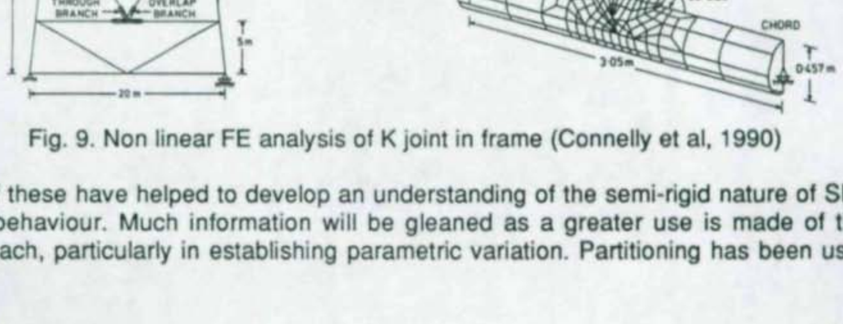

Fig. 9. Non linear FE analysis of K joint in frame (Connelly et al, 1990)

All of these have helped to develop an understanding of the semi-rigid nature of SHS joint behaviour. Much information will be gleaned as a greater use is made of this approach, particularly in establishing parametric variation. Partitioning has been used to analyse the non- linear behaviour of a CHS gap joint within the context of an oil rig frame, but where the remainder of the structure remained elastic - see Fig.9. No serious attempt has been made to carry out an FE analysis of a structure where several of the joints and members interact nonlinearly, in view of the size of the problem.

3.3 Other modelling techniques.

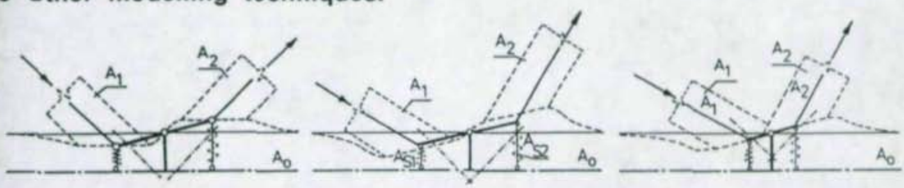

Fig. 10. Micro-bar joint models (Czechowski, 1984)

Various joint assumptions have been considered such as, pinned, pinned bracing to continuous chords, rigid inter branch connections but pinned to chordface (as for fully overlapped joints) or completely rigid. It is of course necessarry to include axial as well as rotational stiffness assumptions. Some investigators (e.g Czechowski, 1984) have used micro-bar joint models to incorporate various stiffnesses, as indicated in Fig.10, for a gap joint, the basic stiffnesses being obtained from isolated joint test results. CHS joints have also been examined on a similar basis (Ogowa et al, 1987)

3.4. Testing of trusses

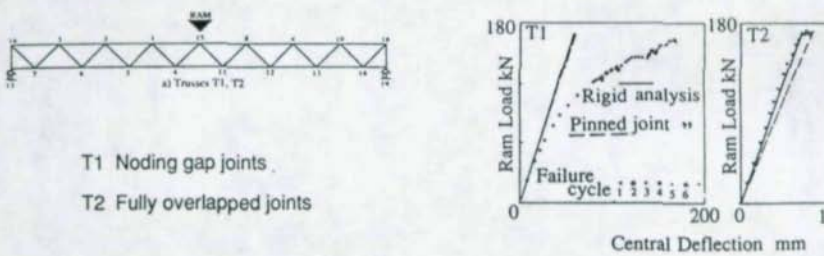

Fig.11. Comparison of lattice girder deflections (Coutie, Davies et al, 1990)

A series of RHS lattice girders using both gap and lap joints have been tested to failure. Some, but not all have found that the results indicate that joint strengths in trusses are well represented by the isolated joint tests, but that girder deflections can in some cases exceed those calculated on the basis of the usual pinjointed assumptions. It can be shown that the method of measuring or calculating member forces in lattice girders can have an important bearing on the conclusions drawn. In carefully controlled tests it has been shown (Coutie et al, 1990) that girders with noding joints having low β's and large gaps, can be both weaker and much more flexible than those fabricated with fully overlapped joints - even though the latter have large eccentricities - Fig.11.

4. CONCLUSIONS

The paper has indicated the particular nature of the semi-rigid joints used in lattice construction. Joint strengths have been largely assessed on the basis of isolated tests, supported by yield line, FE, and other methods of analysis. Axial and flexural stiffnesses from tests have likewise been compared with analytical methods. Both the axial and rotational semi-rigid nature of lattice girder joints involving several members point to a more complex situation than that pertaining to beams and columns in traditional frames. Although good progress has been made it is not surprising that that there are still some divergencies between the joint modelling assumptions needed to get good agreement with tests, and those convenient for the design situation. Various aspects of analysis and design at serviceability and ultimate states will continue to be tackled in a piecemeal fashion with heavy reliance on empirical justification, until reliable, and parametrically based semi-rigid joint characteristics covering the whole range are produced.

5. REFERENCES

Connelly, L.M and Zettlemoyer, N. Frame behaviour effects on tubular joint capacity. Tubular Structures, Ed. Niemi and Makelainen, Elsevier Applied Science 1990 pp81-89.

Coutie, M.G., Davies, G., Philiastides, A., Roodbaraky, K and Yoemans, N. Fully overlapped RHS joints - isolated joints and trusses. Elsevier (as above) pp1-10.

Coutie, M.G. and Saidani, M., The use of FE techniques for the analysis of RHS structures with flexible joints. Elsevier (as above) pp224-231.

Czechowski, A., Investigation into the statics and strength of RHS lattice girders. Welding of Tubular Structures, Pergamon, 1984 pp129-136.

Davies,G. and Roodbaraky, K., Use of FE as a tool in evaluating the strength of joints in RHS. Elsevier (as above) pp214-223.

Korol, R.M. and Mansour, M.H., Theoretical analysis of haunch reinforced T joints in square hollow sections. Canadian Journal of Civil Engineering Vol 6 No.4 1979 pp601-609.

Ogowa, K., Yamanari, M., Makino,Y., Yamashita, M. and Sakamoto, S. Buckling and post buckling behaviour of complete tubular trusses under cyclic loading. Proceedings Offshore Technology Conference, OTC 5439, May 1979, pp161-169.

Paul, J.C., Van der Valk,C.A.C. and Wardenier,J., The strength of circular multiplanar X joints. Elsevier (as above) pp73-80.

Strommen, E.N., Ultimate strength of welded K joints of rectangular hollow sections. Division of Steel Structures, Trondheim University, 1982.

CYCLIC BEHAVIOR OF FRAMES WITH SEMI-RIGID CONNECTIONS

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Marwan N. Nader[2]

Abstract

The main objective of the research project was to study realistic seismic behavior of steel rigid, semi-rigid and flexible (simple) frames. A one story, one bay steel structure was constructed such that the connections can be changed. The structures with rigid, semi-rigid and flexible connections was subjected to various intensities of records obtained from the El Centro, 1940, Taft, 1952 and Mexico City 1985 earthquakes. The studies indicated that the semi-rigidity of the steel frames does not necessarily result in larger lateral drift and more damage than rigid frames.

1. INTRODUCTION

1.1 Dynamic Characteristics of Semi-rigid Frames

Steel building structures are divided into two major categories of fully restrained (FR) or rigid and partially restrained (PR) or semi-rigid. In rigid structures, the dynamic response of the structure to base excitations is governed almost entirely by the strength, stiffness, ductility and energy dissipation capability of the members, whereas, in semi-rigid structures, dynamic response is significantly affected by not only the members but also by the strength, stiffness and ductility of the connections. In addition, soil-structure interaction for rigid and semi-rigid steel structures can be different with different effects on reducing or increasing the seismic response.

The parameters that affect dynamic response of the semi-rigid steel structures are natural periods of vibration, mode shapes, damping, and dynamic characteristics of

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the ground motion. To investigate the significance of these parameters and to extract the effects of other parameters, a compafative experimental study was planned and undertaken (Nader and Astaneh, 1989). The following sections summarize the research project and its findings.

2. RESEARCH PROGRAM

2.1 Objectives and Scope

The main objectives of the research were:

1. To conduct comparative studies of dynamic response of rigid, semi-rigid and flexible steel structures in terms of lateral drift and base shear response,
2. To investigate hysteresis response of connections while they are part of the structure, and;
3. To investigate the effects of connection non-linearities on the dynamic behavior of rigid, semi-rigid and flexible steel structures.

2.2 Research Program

The study consisted of subjecting a one story one bay steel structure with rigid, semi-rigid and flexible connections to a variety of base excitations using shaking table of the Earthquake engineering research Center of the University of California at Berkeley and studying the response of the structure.

The structure that was used in experiments and three types of connections are shown in Figure 1. The structure consists of two similar and parallel frames in the North-South direction of the laboratory. The frames were connected by two spandrell beams in the East-West direction. In addition, to reduce torsional effects diagonal braces were added in the East-West direction. The connections used in the frames in the N-S direction were either rigid, semi-rigid or flexible connections. The bases of the columns in all tests were fixed to the shaking table.

The material of the structure was hot-rolled specified as A36 steel with a nominal yield stress of 36 ksi (2.48 Mpa). Concrete blocks were attached to the structure at roof level to simulate the weight and to generate stresses in the beams and columns close to the stresses that would exist in an actual structure. The total weight of the structure was 27,423 pounds (122 kN).

The test structure and connections were instrumented to measure accelerations, surface strains, absolute and relative displacements, rotations and forces at critical locations. Using the recorded data, moment-rotation, axial load-axial-displacement and shear force-shear displacement response of connections were calculated. Also, shear, axial load and bending moments in columns were established.

3. BEHAVIOR OF CONNECTIONS

3.1 Moment-Rotation Response of Connections

Typical hysteresis moment-rotation response of connections in rigid and semi-rigid test structures are shown in Figure 2(a). The plots are for Taft earthquake record with maximum peak acceleration of 0.35g. The M-θ response of rigid connection was almost elastic and maximum rotations did not exceed 0.003 radians. However, maximum moment in rigid connection reached 251 kip-in (28.3 kN-m) The semi-rigid connections experienced maximum rotation of about 0.008 radian while moment was about 200 kip-in (22.5 kN-m). In general, comparison of M-θ response of connections indicated that semi-rigid connections developed less moment and more rotation than rigid connections but were quite ductile throughout the response.

Under larger accelerations, semi-rigid connections still showed very high ductility without sudden loss of strength or stiffness. Figure 2(b) shows response of the semi-rigid structure to Taft and Mexico City earthquake records both scaled to maximum peak acceleration of 0.5g. The relatively large and sudden increase in strength at larger rotations is attributed to kinematic hardening and development of catenary forces in angle legs in the connection.

After completion of all tests of semi-rigid structures, no sign of brittle failure or local buckling was observed. Yielding primarily was in the legs of connection angles and was well distributed over the legs of angles. This is a desirable behavior since due to relatively uniform distribution of yielding and lack of strain concentration, low cycle fatigue fracture is delayed and connection continues to yield, deform and dissipate energy without fracturing.

3.2 Axial Force-Axial Displacement Response of Connections

Typical axial force-axial displacement response of connections in rigid and semi-rigid test structures are shown in Figure 3 for Taft record scaled to 0.35g maximum peak acceleration. The hysteresis loops for rigid connections were symmetric with minor energy dissipation characteristics. However, the hysteresis loops for axial force-axial displacement of semi-rigid connections were unsymmetric. The connection was flexible when pulled and relatively rigid when pushed against the column. The phenomenon is very similar to the observed response of full scale connection tests in the laboratory (Ho and Astaneh, 1990). In these laboratory tests, it was observed that the axial cyclic loading can deteriorate the connection causing its failure under shear. The relatively low stiffness of semi-rigid connections in axial direction when pulled away from the support need more attention and study. It may result in reduction of the force response and some increase in displacement response of the structure. More research is needed before the total effect of the axial flexibility of semi-rigid connections on global dynamic response of the structure can be fully established.

3.3 Shear Force-Shear Displacement Response of Connections

Typical shear force-shear displacement hysteresis responses of connections in rigid and semi-rigid test structures were similar. However, in semi-rigid connections, due to hole enlargements during cyclic loading and cyclic friction slips, the stiffness of connection in shear, i.e. slope of the hysteresis loops was not as stable as for the rigid connections.

4. DYNAMIC BEHAVIOR OF RIGID AND SEMI-RIGID STRUCTURES

4.1 Introduction

Dynamic response of rigid, semi-rigid and flexible test structures to base excitations was measured in terms of periods of vibration, critical damping ratios, roof drift ratio and base shear developed in the frames during each test. Following is a summary of the experimental results.

4.2 Period of Vibration and Damping Ratios

The fundamental mode of vibration for test structures was lateral displacement of roof in N-S direction. The second mode was torsional mode of vibration resulting in roof rotating about a vertical axis. Table 1 shows values of critical damping ratios and fundamental period of vibration for three types of structures tested. The values are calculated using the free vibration tests that were conducted prior to each shaking table tests. To conduct a free vibration or "tie-back" test, the roof beams were tightly secured to the floor of the laboratory using a steel cable. By abruptly cutting the cable and releasing elastic energy stored in the system, the free vibration tests were conducted. As table indicates, the period of vibration of semi-rigid and rigid structures was very close and was about 0.31 second.

However, when the structures were subjected to base accelerations exceeding 0.35g, causing some inelastic activities, the period of vibration of semi-rigid structure was about 7/17 second while period of vibration of rigid structure was about 7/21 seconds indicating that period of vibration of semi-rigid structure was elongating more than that of rigid structure during inelastic phase of response.

4.3 Base Shear and Lateral Drift Response

Maximum values of base shear and lateral drifts that were recorded during the tests are given in Table 2 for flexible, semi-rigid and rigid test structures for various levels of earthquakes. Figure 4 shows the hysteresis loops of base shear and lateral drift for response to Taft earthquake with peak acceleration scaled to 0.35g. As Table 2 and Figure 4 indicate, base shear developed in semi-rigid structure was less than that of rigid structure but lateral drift for semi-rigid structure was not greater than the drift of rigid structure but lateral drift for semi-rigid structure was not greater than the drift of rigid structure.

5. CONCLUSIONS

The following conclusions could be reached by studying experimental data obtained from the tests and partially reported in previous sections.

1. Semi-rigid steel structures may have the potential of performing satisfactorily during the medium and strong earthquakes. If connections are ductile and optimum rigidity is present, low cycle fatigue fracture of the connection welds can be avoided by the use of bolted semi-rigid connections.

2. Axial force-axial displacement cyclic response of semi-rigid connections should be incorporated in seismic design and analysis of steel semi-rigid structures.

3. Semi-rigid steel structures are more flexible than the rigid structures. However, the extra flexibility of the semi-rigid frames does not necessarily result in larger drifts in the semi-rigid structures during earthquakes.

6. REFERENCES

1. Ho, I. and Astaneh-Asl A. (1991), "Behavior and Modeling of Steel Double Angle Connections Subjected to Cyclic Axial Force", Report No. UCB/EERC-../91, Earthquake Engineering Research Center, University of California Berkeley.

2. Nader M. and Astaneh-Asl A. (1989), "Experimental Studies of a Single Story Steel Structure with Fixed, Semi-rigid and Flexible Connections", Report No. UCB/EERC-89/15, Earthquake Engineering Research Center, University of California Berkeley.

ACKNOWLEDGMENTS

The project was sponsored jointly by the U.S. National Science Foundation and the University of California at Berkeley. The tests reported here were conducted using the shaking table at the Earthquake Engineering Research Center of the University of California at Berkeley.

FREE VIBRATION			
Structure	Damping (%)	Period of Vibration (sec)	
		FFT	Cycles/time
Flexible	1.87	0.44	0.44
Semi-rigid	0.50	0.33	0.31
Fixed	0.67	0.30	0.31

Table 1. Damping and Fundamental Period of Vibration

MAXIMUM VALUES OF BASE SHEAR AND LATERAL DRIFT						
Earthquake Signal and Intensity	Flexible Structure		Semi-rigid Structure		Fixed Structure	
	Shear (kips)	Drift (in.)	Shear (kips)	Drift (in.)	Shear (kips)	Drift (in.)
El-Centro 0.15g	4.14	0.42	4.62	0.20	5.40	0.23
El-Centro 0.25g	8.10	1.09	11.76	0.56	14.95	0.61
El-Centro 0.35g	10.00	2.08	18.81	1.15	18.12	0.82
Taft 0.15g	5.35	0.61	9.14	0.55	16.82	0.60
Taft 0.35g	9.52	1.57	20.00	1.41	25.88	1.22
Taft 0.50g	11.49	2.00	24.28	2.35	N.C	N.C
Mexico 0.35g	9.22	1.45	21.2	2.00	N.C	N.C
Mexico 0.50g	12.26	2.05	22.26	2.30	N.C	N.C

N.C = test was not conducted

Table 2. Maximum Base Shear and Drift Values

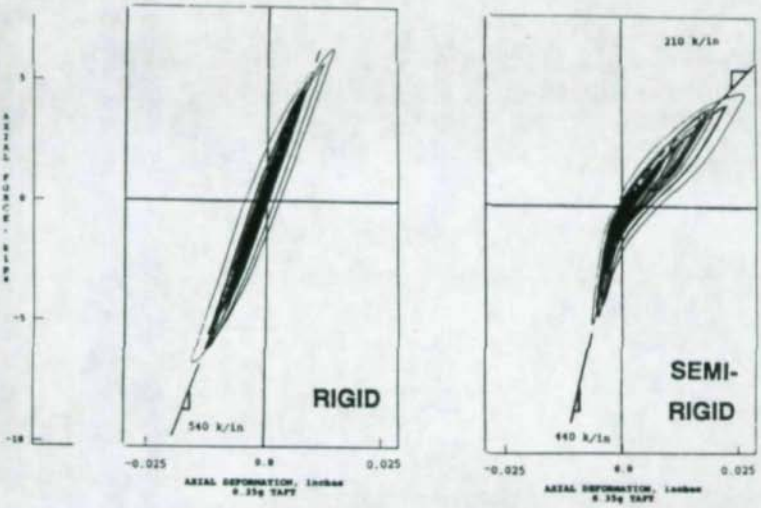

Figure 3. Axial Force-Axial Displacement Response of Connections

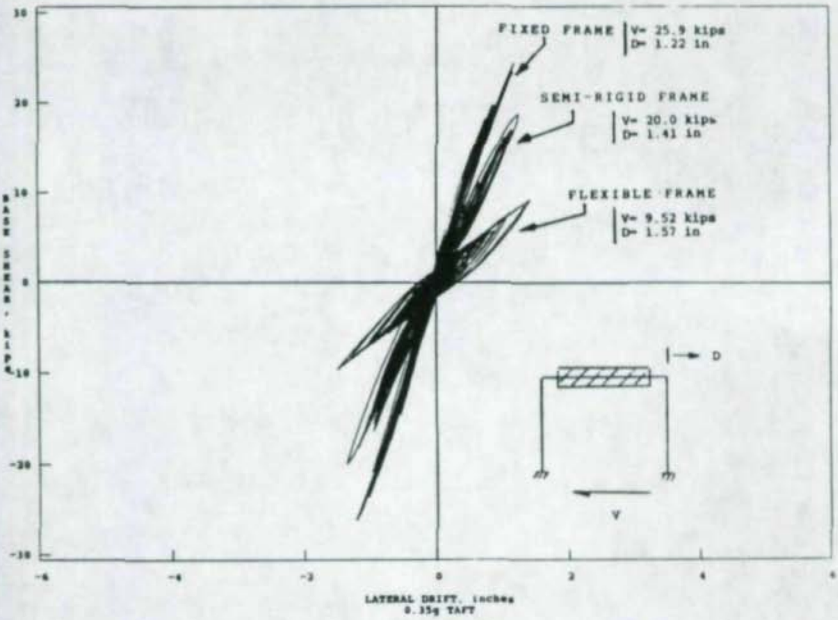

Figure 4. Base Shear-Lateral Drift Response of Structures

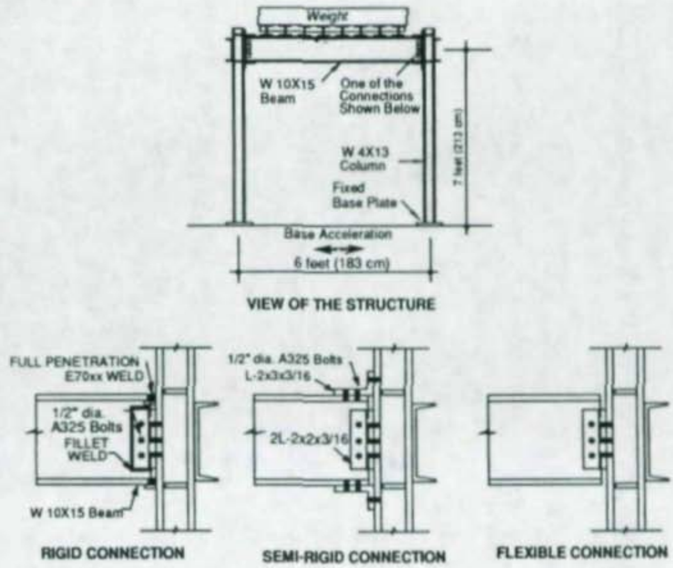

Figure 1. Test Structure and Three Types of Connections

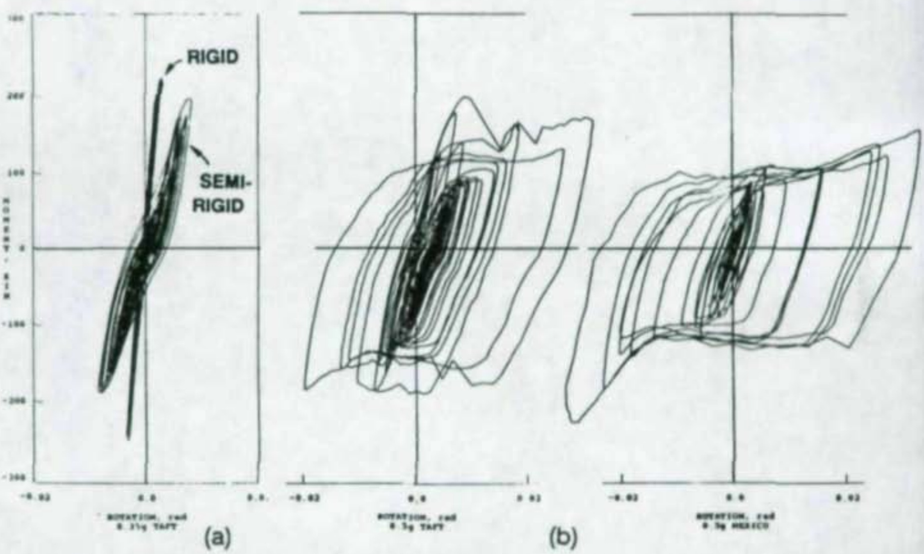

Figure 2. (a) M-θ Response of Rigid and Semi-Rigid Connections to 0.35g Taft
(b) M-θ Response of Semi-rigid Connection to 0.5g Taft and Mexico City Earthquakes

DYNAMIC TESTS OF SEMI-RIGID CONNECTIONS

Richard Kohoutek[1]

Abstract

Fabricated standard connections (80), welded and bolted, will be tested. This reports on the initial test of (26) connections, about 200 kg each, tested at Vipac - Melbourne, where they were subjected to several dynamic tests including a dynamic force produced by a shaker. The results of the experiment will form a basic data base of a performance of standard connections not presently available. With the theory already developed, the prediction of static forces, and the dynamic response of framed structures will be possible for an operating range of frequencies.

Preamble

In this workshop [2], the difference between a connection and a joint will be made. The *joint comprises the detailing of an assembly joining typically a beam and column.* A connection includes some parts of a beam and column in addition to the joint. It is the connetion which is the subject of this research. Our interest is in the performance of the connection and the structure, rather than in the sizing of a bolt or a weld. This is not to deny the effect of joint details, which may exert a large influence on the performance of the connection. This joint influence will, of course, be included in the behaviour of the connection, together with the adjoining parts of the beam and column which are subjected to local peaks of moments and shear forces.

Both the static and dynamic analyses recognise some variation from the complete rigidity, commonly expressed by a relationship between a moment M and a rotation of connection $\xi = \delta + \zeta$, where ζ is the rotation maintaining moment through a connection to other members (Kohoutek, 1985). The rotation δ is a free flexing or rotational slack which reduces moment stiffness of the connection. The relationship is diagrammatically shown in Figure 1 together with some limits and examples. The horizontal axis represent the behaviour of an ideal hinge and the vertical axis a completely fixed connection. The real connections fall somewhere between those extreme values as shown in the Figure 1.

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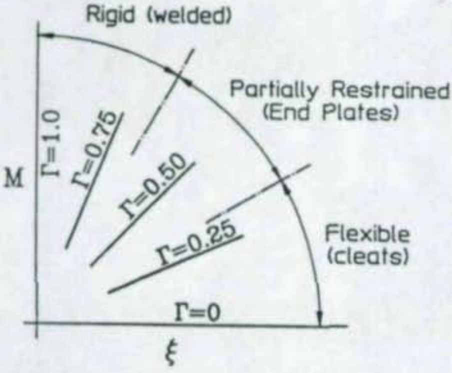
[2] AISC terminology is reversed (Australian Institute of Steel Construction)



The connection behaviour has a major influence on the performance of a bar; be it a column, loaded predominantly by an axial force, or a beam, subjected mainly to bending moment and shear force. There is an intrinsic relationship between the static and dynamic behaviour of a bar, which allows findings of the dynamic behaviour to be utilized also in the static analysis; the relationship is utilized, but not discussed here, because of space limitations. The connection's performance influences natural frequencies, the dynamic as well as static moment distribution in frames, static stability through effective length of bars and the critical loads, as shown in other paper in this volume.

FIGURE 1. *Relationship between moment and rotation.*

1. INTRODUCTION

In this paper, the dynamic behaviour of semi-rigid connections is investigated in the elastic range, based on the assumption of small deformations. The dynamic rigidity exhibited by a connection is related to that of a connection under static load where the stiffness may be approximated by the slope of the moment-rotation relationship.

2. SELECTION OF CONNECTIONS

Since the establishment of a data base of common connections is among the aims of this research project, it was necessary to select the connections which were commonly used in the design and fabrication of steel framed structures. A survey of Engineering Consultants (Kohoutek, 1988) was conducted in which the respondents were asked to select from forty different structural joints the most frequently used. This formed the basis for the selection of the twenty two bolted and four welded joints which were tested.

The test connections were fabricated according to AISC Standardised Structural Connections using 250 UC 72.9 members and M20 bolts 8.8/TF. The fabricated standard connections are T-shaped with the horizontal member, the column in a structure, being 1.5 m long and the cantilever stub having a length of 1.2 m. The selected bolted connections include strong axis joints such as moment end plate, web side plate, angle cleat, angle seat, and flexible end plate joints with structural bolts and two end plate joints using prestressed bolts. Weak axis joints such as web side plate and angle cleat joints to coped and non- coped beams were also tested. These latter connections were the subject of a further series of tests after modifications were made to the connections by welding two stiffener plates into the column section in line with the beam flanges, as may occur with an exterior column connection.

3. TEST RIG

The test bed, made for this project, consists of the plate 1.5 m square and 40 mm thick cast into a cube of concrete. The test pieces were supported at either end between a semi-circular platen and a 50 mm diameter bar, and were restrained by using two 100 mm channel sections joined by spacers and two holding-down bolts. The restraining was to stop the test pieces from walking off the supports when they were excited.

4. TESTING OF SEMI-RIGID CONNECTIONS

A simple beam of 250 UC 72.9 section was used as calibration to determine boundary conditions created by clamping cross beams described above. The aim of the testing of the connection pieces was to evaluate the natural frequencies of vibration of the specimens, especially the frequency of the first bending mode. A range of different tests were conducted to determine the most reliable method. A Hewlett Packard 5423A two-channel FFT Spectrum Analyzer was used to measure the response time functions of the specimen. The sustained excitation was produced by a large and small shakers and the impulse load at various points on the test pieces was induced by impacting with a rubber mallet.

Testing of the connections by forced vibration was carried out using a large electrodynamic shaker connected to the top of the cantilever by an extension bar and load cell as in Figure 2. The shaker was programmed to provide a random signal with the band width limited to a range which included the expected first cantilever bending mode.

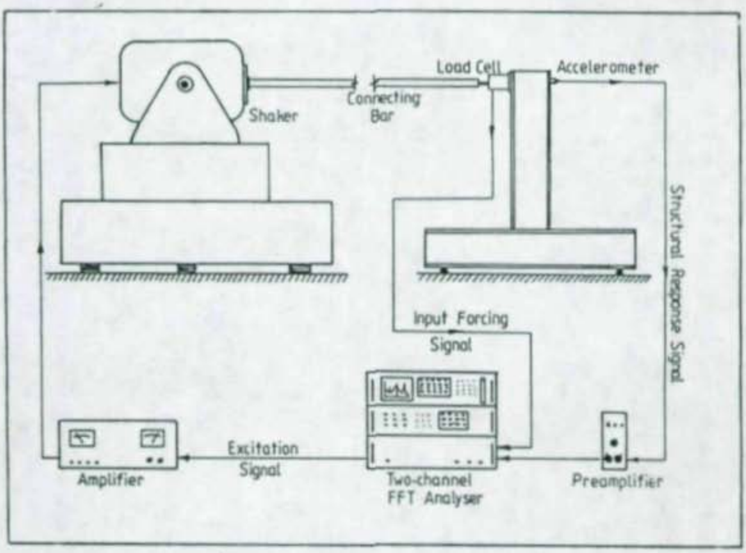
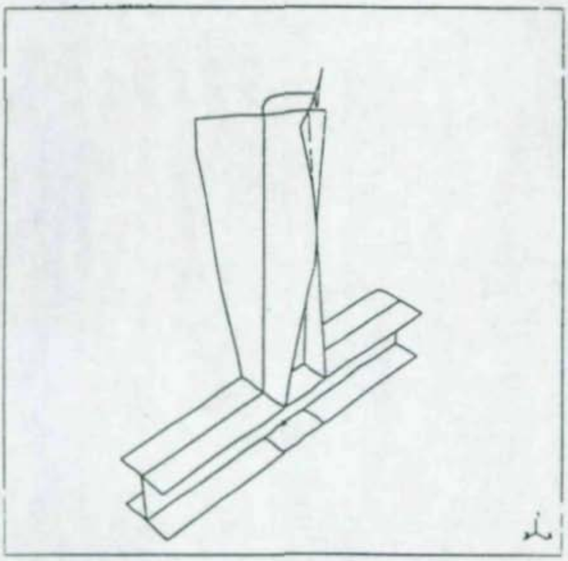


FIGURE 2 *Schematic diagram of test setup for forced vibration.*

5. FINITE ELEMENT MODEL OF CONNECTION

Testing of the connections under free vibration due to impact loading initially gave frequency spectra which were cluttered with many undiscernable modes of vibration of the test piece.



A finite element formulation was undertaken to establish the approximate frequencies of the major vibrational modes of the test pieces so that these could be considered more closely rather than investigating various modes such as flange modes. The dynamic finite element analysis software package used was I-DEAS v 3.8. The formulation of the model was limited with respect to the joint details that could be modelled and the boundary conditions that could be imposed. Boundary conditions imposed on the beam ends were pinned restraints contrary to the semi-rigid connections that existed.

FIGURE 3 *Results of dynamic finite element formulation. First cantilever torsional mode 165.69 Hz*

The finite element model enabled the identification of three major modes of vibration, one shown in Figure 3. The other modes as calculated by the dynamic finite element analysis proved inconclusive and we could not use them in comparison with the initial test results.

These results were used in the testing of the connections and enabled the positive identification of the out-of-plane vibrations of the cantilever, which varied between 3 - 20 Hz depending on the joint type, and the torsional vibrations of the cantilever, measured at approximately 155 Hz for the majority of connections. An earth loop was also consistently measured, with a frequency peak existing at 50 Hz due to the instrumentation picking up the frequency of an electrical current.

6. EVALUATION OF BOUNDARY CONDITIONS

In any analysis the boundary conditions will affect the behaviour of the structure and in this case the rigidity of the restraining joints has affected the natural frequency of the test piece. Therefore, it was necessary to evaluate the support rigidity before the rigidity of the connections could be tested.

The test procedure outlined the measurement of the natural frequency for the first *bending mode of the beam in the test piece and thus with a known natural frequency a* mathematical model can be used to evaluate the rigidity of the restraint.

6.1 Modal Analysis

The modal analysis of a 250 UC 72.9 calibrating beam of length 1.5 m was used to determine the first natural frequency and derive the boundary conditions. An electrodynamic vibration exciter was used to provide an excitation force to the centre of the beam as illustrated in Figure 4. Two accelerometers were used to measure the input force signal and the system response, being attached to the top of the shaker and to the beam respectively. The modal analysis required the second accelerometer to be moved to different positions on the beam at which the response time histories are measured. A Hewlett Packard 5423A two-channel Spectrum Analyzer was used to measure the input and response time functions to give the natural frequency and the mode shape of the vibrating beam.

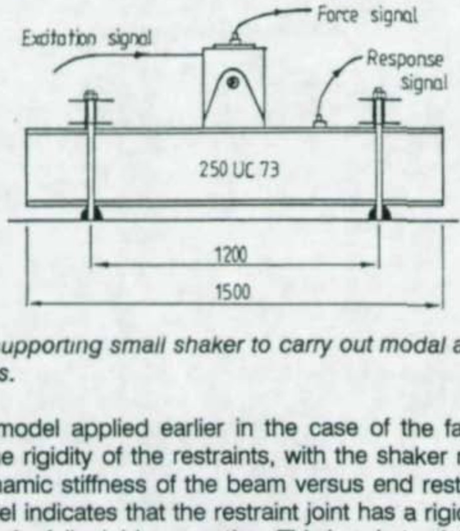


FIGURE 4 *Beam supporting small shaker to carry out modal analysis for evaluation of boundary conditions.*

The mathematical model applied earlier in the case of the fabricated connections is used to evaluate the rigidity of the restraints, with the shaker mass being applied mid span. A plot of dynamic stiffness of the beam versus end restraint rigidity is shown in Figure 5. The model indicates that the restraint joint has a rigidity of 0.42, that is it has 42 % of the rigidity of a fully rigid connection. This is substantially greater than a pinned joint. The torque applied in tightening the holding bolts was shown to influence the measured frequencies to a minimal extent and in order to maintain consistency the applied torque was kept constant at *100 Nm* on each bolt.

The evaluation of the boundary conditions as above was further verified by a modal analysis carried out on a connection set up in the rig. The analysis evaluated the second bending mode of the beam and the natural frequency. The model gives a restraint rigidity which corresponds quite closely to the value obtained.

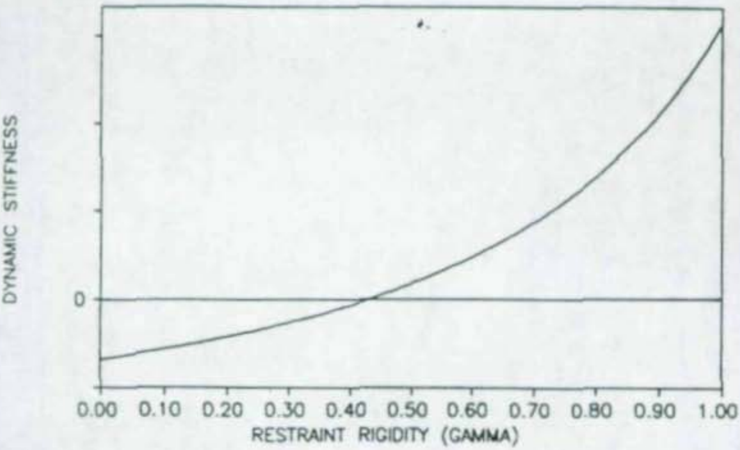
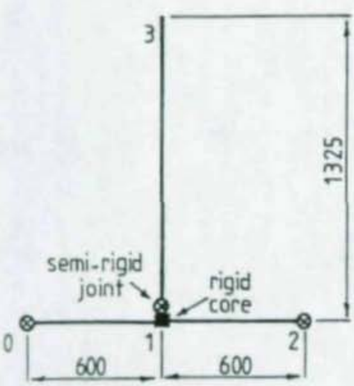


FIGURE 5 *Plot of dynamic stiffness versus end rigidity for a beam vibrating at its first bending mode frequency.*

7. MODEL FOR TEST SPECIMENS



The measured natural frequency of vibration of the cantilever for each connection and the boundary conditions were measured. A model for the the semi-rigidity of the beam-cantilever connection is relatively simple to formulate. The test pieces were analysed as shown in the following Figure 6. The members contributing to the test piece are represented by their centrelines and a rigid connection is employed to denote the continuity between members (0,1) and (1,2) because of a continuity of beam (0,2). Semi-rigid connections are used at the remaining ends to model the dynamic behaviour of the test piece.

FIGURE 6 *Adopted analytical model of test pieces*

8. RESULTS OF TESTS

The three connections with moment end plate, web side plate and angle seat joints respectively were tested with the results given below.

Dynamic connection rigidity was evaluated using the mathematical model of the connection, and were based on the boundary conditions of the restraining end connections having a dynamic rigidity of $\Gamma = 0.42$.

TABLE 1 *Summary of measured frequencies and calculated dynamic rigidities.*

Connection Designation	Natural Frequency (Hz)	Rigidity Γ	Type of connection
B4/1	98.83	0.59	Extended end plate
B8/1	72.27	0.37	Web side plate
B13/1	54.69	0.23	Web and seat angle

The frequency response of the test specimens did not change with the variation in amplitude of forced vibration. For the range of amplitudes applied the dynamic response of the connections tested was independent of the type of excitation, namely whether it undergoes free or forced vibration for the range of applied loads.

9. INFLUENCES OF CONNECTION RIGIDITY

With the experimental evaluation of connection rigidity, variations in the frequency response of the test pieces were observed as listed in Table 1. As an example consider the difference between the extended moment end plate connection, B4/1, and the combined web and seat angle connection, B13/1. A difference of 45 percent in the first natural frequency of the cantilever exists, and in terms of the connection semi-rigidity co-efficient the two joint types vary to a greater extent.

We could also find changes in response frequency with the use of more bolts in identical joints, suggesting that an increase in dynamic rigidity may be achievable with the use of a greater number of bolts for some joint types. It is postulated that an increase in the number of bolts for a particular joint would serve to bring a greater area of the member in contact with the joint components and thereby increase the friction forces which exist. Furthermore, the increased space required would result in a redistribution of bolt locations such that forces transferred from the beam to the column would also be redistributed over a larger portion of either member. For instance, the web side plate connection discussed herein used six bolts and was found to have a substantially greater dynamic rigidity than an identical connection with three bolts and a shorter end plate.

Another factor which increased a connection rigidity was the use of stiffeners. It is suggested that a judicious use of stiffeners in a bolted connection could achieve the dynamic rigidity of a connection with a more detailed or costly joint type. This is an area which would require further investigation before clear relationships can be established.

10. CONCLUSIONS

A procedure for the experimental evaluation of dynamic rigidity in structural connections has been described in this paper. The understanding of some of the problems associated with this experiment makes possible to improve the procedure so that it will be applied to structural steel connections of any type or size.

The full scale testing of structural connections to evaluate dynamic rigidity was vindicated by the substantial variations observed in connection response. Major variations in dynamic stiffness were shown to exist, as demonstrated by the three connections included herein. The experimental results for other connections tested indicated that the use of stiffener plates and the number of bolts used in the joint detail also significantly influence the dynamic behaviour of a connection, and could be used as a design parameter to achieve a desired coefficient of rigidity.

Those design variations lead to the possibility of modifying connection behaviour to some chosen performance by an alteration in joint details. This may be fruitfull for a structure which has connections with a low co-efficient of rigidity and suffers serviceability problems from vibrations induced by periodic loading, such as that due to mass imbalance in reciprocating motion of machinery. The dynamic stiffness of such a connection could be increased with the addition of stiffeners or the use of more bolts in the joint so that structure response is moved from the vicinity of the resonance frequencies. The prediction of the type of alterations required, for example stiffener thicknesses or the number of bolts, is an area open to further investigation although the existence of a substantial database of connections will allow modifications of connections without extensive testing.

The experimental analysis of the test connections was mainly based on the connection behaviour subject to free vibrations. It was found that the behaviour of the connections was not affected by the magnitude of applied load under forced vibrations.

This investigation demonstrated that the variation in the dynamic rigidity of bolted joints is significant and dependent on the joint detailing which can be successfully measured.

11. ACKNOWLEDGEMENTS

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12. REFERENCES

1. Behrens, M.P. (1989). "Dynamic Performance of Bolted Semi-Rigid Joints," B.E. (Hons) Thesis, University of Wollongong.
2. Bendat, J.S. and Piersol, A.G. (1980). "Engineering Applications of Correlation and Spectral Analysis," 302pp, Wiley, New York.
3. Bjorhovde, R. et al. (1988). "Connections in Steel Structures," Elsevier Science Publishers.

4. Chen, W.F. (1987). "Connection Flexibility in Steel Frames," Elsevier Science Publishers.
5. Ewins, D.J. (1984). "Modal Testing : Theory and Practice," 269pp, Research Studies Press, Wiley, New York.
6. Goverdhan, A.V. (1984). "A Collection of Experimental Moment Rotation Curves Evaluation of Predicting Equations for Semi-Rigid Connections," Ph.D. Thesis, Vanderbilt University, Nashville, Tennessee, U.S.A..
7. Howlett, J.H. et al. (1981). "Joints in Structural Steelwork," Proceedings of the International Conference held at Teesside Polytechnic, Cleveland 6-9 April 1981. Pentech Press.
8. Jones, S.W., Kirby, P.A., Nethercot, D.A. (1983). "The Analysis of Frames with Semi-Rigid Connections - A State-of-the-Art Report," Journal of Constructional Steel Research, Vol. 3, No. 2.
9. Kohoutek, R. (1985). "Dynamic Analyses of Frames with Semi-Rigid Connections," I.E. Aust. Proceedings of Metal Structures Conference, held at Monash University, Melbourne, 23-24 May, L. Pham, ed., pp 7-10.
10. Kohoutek, R. (1985). "Analysis of Beams and Frames," Chapter 4 in "Analysis and Design of Foundations for Vibrations," pp 99-156; P. Moore, ed., 512pp, 1985, published by A.A.Balkema.
11. Kohoutek, R. (1986). "Classical and Parametric Resonance of a Frame - Case Study," Proceedings of The Tenth Australasian Conference on Mechanics of Structures and Materials, held at Adelaide University, 20-22 August, G. Sved, ed., pp 229-232.
12. Kohoutek, R. (1988). "Serviceability Performance of Steel Joints," Aust.ISC, Journal of Steel Construction, Vol. 22, No. 3.
13. Kolousek, V. (1973). "Dynamics in Engineering Structures," Praha-Academia, London-Butterworth.
14. Pham, L., Mansell, D. S.(1985). "Survey of Research Needs for Metal Structures," I.E. Aust. Proceedings of Metal Structures Conference, held at Monash University, Melbourne, 23-24 May, L. Pham, ed., pp 172-177.
15. Marino, J.J. (1988). "Dynamic Performance of Welded Semi-Rigid Joints," B.E. (Hons) Thesis, University of Wollongong.
16. Nethercot, D.A. (1985). "Utilisation of Experimentally Obtained Connection Data in Assessing the Performance of Steel Frames," in Connection Flexibility and Steel Frames, pp13-37; W.F. Chen, ed., 1985, ASCE, New York.
17. Popov, E.P. (1985). "Flexibility in Steel Seismic Moment Connections," in Connection Flexibility and Steel Frames, W.F. Chen, ed., 1985, ASCE, New York, pp101-119.
18. Segal, A. and Barnuch, M. (1980). "A Nondestructive Dynamic Method for the Determination of the Critical Load of Elastic Columns," Experimental Mechanics, pp 285-288.
19. Sweet, A.L., Genin, J. and Mlakar, P.F. (1977). "Determination of Column-buckling Criteria Using Vibratory Data," Experimental Mechanics, pp 385-391.
20. Wakabayashi, M. and Nakamura, T. (1986). "Hysteretic Restoring-Force Characteristics of Steel Components and Systems," in Steel Structures - Recent Research Advances and their Application to Design, M.M.Pavlovic, ed., Elsevier Applied Science, London, pp511-546.

INFLUENCE OF PANEL CONNECTING SYSTEM ON THE DYNAMIC RESPONSE OF STRUCTURES COMPOSED BY FRAMES AND COLLABORATING CLADDINGS

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Abstract

The behaviour of trapezoidal sheet panels under monotonic and cyclic loads is herein interpreted on the base of available experimental results.

It was observed that the types of connecting system, used in modern steel technology, play a fundamental role on the strength and ductility of corrugated sheet panels. Simplified analytical models for interpreting the cyclic behaviour of such panels under shear loading conditions are proposed. They are referred to two connecting systems which use rivets and welds for assembling single sheets together.

The importance of the connections on the seismic response of pin-jointed steel structures with bracing panels is also pointed out by means of dynamic analysies.

1. INTRODUCTION

The design method, so-called «stressed skin design», was proposed for steel structures during the seventies in the field of activity of ECCS. The main principle consisted in considering the capacity of claddings made of light gauge steel panels to contribute by means of a diaphragm action to the behaviour of the structure as a whole.

The degree of collaboration between cladding panels and the main structure can be assumed as a base to identify structural behaviours as follows (Mazzolani and Sylos Labini, 1984):

a) the main structure is designed to resist vertical and seismic loads, while the panels cooperation is taken into account in the serviceability limit state only, when checking maximum sways and story drifts;

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b) the whole structure is designed so that panels and frames together have to resist vertical and seismic loads;

c) the frame has the task to resist vertical loads only while horizontal forces due to earthquake or wind are supported by cladding panels.

As a consequence, the choice of connections should be done according to the given design criteria. Referring to beam-to-column joints, case a) requires rigid connections and case b) can accept semi-rigid connections. As a bracing effect can be guaranteed by claddings, also pin-ended connections can be used in case c), which provides the maximum economy in reducing both structural weight and manufacturing and erecting cost. In spite of these advantages, examples of structures like c) type are not yet available in seismic zones, probably due to lack of specific knowledge and experience.

As a first contribution to this development, the problem of the seismic behaviour of pin-jointed structures with bracing panels has been examined as an economical structural solution for building in zones of low seismicity. The important influence of the connecting system has been also pointed out for this application (Mazzolani and Piluso, 1990), due to the fact that the transfer of shear forces from the structure to the panels is guaranteed by means of different technological systems which use bolts, rivets, screws or welds.

From the results of the available literature (Sanpaolesi, 1984) it was observed that two limit behaviours are given by riveted and welded connections, while the use of other systems (bolts, screws) provides intermediate behaviours. For this reason the analytical models, which are developed here, are referred to these limit cases. The influence of such limit models is evaluated in term of seismic response of a pin-jointed structure with bracing panels.

2. EXPERIMENTAL DATA

2.1 Riveted and screwed panels

The cladding panel is usually assembled by means of mechanical fasteners (bolts, rivets or screws).

The available test results for these types of panels are mainly under monotonic loading conditions. The knowledge of the cyclic behaviour is required in order to take into account the stiffening effect of panels which is always present and influences the dynamic behaviour of structures.

From the technical literature, the available data on the cyclic behaviour of riveted and screwed panels are given by the experimental program performed at the University of Pisa (Sanpaolesi, 1984). These data are used here as a base for the calibration of the analytical model.

A single layer panel composed by four elementary sheets, which are assembled by means of rivets (MODEL 1a), is shown in fig.1. The complete panel is screwed to the upper and lower chords. The behaviour under monotonic shear loads is clearly non-linear as it is shown by the shear force versus shear displacement curve given in fig.2a. The discontinuities after the maximum load are due to the collapse of some rivets which produces the disconnection of elementary sheets and therefore leads to the loss of the load carrying capacity of the panel. From the cyclical point of view the hysteresis loops are characterized by large slips of the connections. A reduction of the energy dissipation capacity during the cycles following the first one is evident in fig.2b.

An improvement of the ductility can be obtained by means of the substitution of the riveted connections with screwed connections (fig.1, MODEL 1b). The corresponding experimental monotonic behaviour is shown in fig.3a, while in fig.3b the behaviour under cyclic loads is given. In this case an improvement of ductility is observed together with an increase of the energy dissipation capacity.

The comparison between these two cases (rivets and screws) confirms that the first case (rivets) leads to the worst behaviour and, therefore, can be assumed as a lower bound.

2.2 Welded panels

An important increase of strength and ductility with a strong reduction of slips can be obtained by using spot welds for the connections between elementary sheets and by inserting the complete panel into a perimetral frame (fig.1, MODEL 2). A further improvement of strength and ductility could be obtained adopting continuous instead of spot welds.

In case of panels composed by sheets connected by means of spot welds and inserted into a perimetral frame, the monotonic and cyclic behaviours have been investigated in the already mentioned experimental program (Sanpaolesi, 1984). The shear force versus shear displacement curve under monotonic loading is given in fig.4a. The softening branch is due to the local buckling of the panel which produces the collapse. The hysteresis loops under loads comparable with the collapse static load are given in fig.4b; a very appreciable improvement of the energy dissipation capacity can be observed in case of welding with respect to the previous cases of mechanical fasteners.

3. ANALYTICAL MODELS

3.1 Riveted and screwed panels

The above experimental results have pointed out that the most important feature of the cyclic behaviour of riveted and screwed panels is represented, in the shear forces versus shear displacement loops, by the presence of large slips due to the collapse of rivets or screws.

The simplified model proposed in fig.5 (Mazzolani and Piluso, 1990) has been developed in order to interpret the worst behavioural case, in which the energy dissipated during the reloading phase can be partially neglected, as it can be obtained in a safe interpretation of the testing behaviour of model 1a (fig.2b)

The increasing branch has been described by means of a curve of the Ramberg-Osgood type:

$$v = \frac{F}{K_o} + \left(\frac{F}{B}\right)^n \tag{1}$$

where:

v is the shear displacement;

F is the shear force;

K_o is the initial shear stiffness provided directly by the experimental curve.

The parameters n and B characterizing the non-linearity of the loading phase can be obtained by imposing the passage of the theoretical curve through two given points (v_1, F_1) and (v_2, F_2) of the experimental one. It results:

$$n = \frac{\ln\left(\frac{v_1 - \frac{F_1}{K_o}}{v_2 - \frac{F_2}{K_o}}\right)}{\ln\left(\frac{F_1}{F_2}\right)} \tag{2}$$

$$B = \frac{F_1}{\left(v_1 - \frac{F_1}{K_o}\right)^{\frac{1}{n}}} \tag{3}$$

The unloading branch is linear and its slope is given by ρK_o, where also the coefficient ρ can be directly obtained from the experimental curve. The unloading phase is followed by a slip branch corresponding to the release of the total accumulated deformation until the reloading in opposite sense is reached.

It is clear that the proposed model overestimates, in loading phase, the energy dissipated by the cycles following the first one. This approximation is compensated after the unloading phase by the passage through the slip branch up to the reloading phase, where the energy dissipation is neglected.

We introduce (fig.5):

v_1^+ and v_2^+ the values defining the current position of the unloading branch in the positive range;

v_1^- and v_2^- the analogous values for the unloading branch in the negative range.

Both unloading branches can assume different positions as far as the number of cycles increases and therefore they can be defined as moving branches, whereas the Ramberg-Osgood curve represents a fixed branch.

Referring to the case in which v is greater than zero, it can be observed that:

- if $v > v_1^+$, the point P(v,F) belongs to the fixed branch (loading); therefore equation (1) provides the current value of F and the position of the moving branch has to be updated by means of the values:

$$v_1^+ = v \tag{4}$$

$$v_2^+ = v - \frac{F}{\rho K_o} \tag{5}$$

- if $v_2^+ \le v \le v_1^+$, the point P(v,F) belongs to the moving branch (unloading) therefore:

$$F = \rho K_o (v - v_2^+) \tag{6}$$

and the values v_1^+ and v_2^+ must not be updated;

- finally if $v \le v_2^+$ the point P(v,F) lies on the slip branch and therefore F=0, while the values of v_1^+ and v_2^+ must not be updated.

In an analogous way the case v<0 can be studied.

An important feature of the proposed analytical model consists on its easy codification. This is desiderable in view of its implementation in a computer program for the dynamic inelastic analysis of structures braced with trapezoidal sheet panels connected to the main structure.

3.2 Welded panels

From the cyclic point of view, the shear force versus shear displacement constitutive relationship can be considered stable, provided that it exhibits the same behaviour as in monotonic test even if the number of cycles increases. On the other hand, the behaviour can be unstable when its stiffness decreases as the number of cycles increases.

With reference to the panels in which elementary sheets are assembled by means of spot welds, the simplified analytical model here presented (fig.6) neglects the slight slip phase of the loading branches of cycles following the first one, while the unloading branch can be either linear or non-linear. In the specific case the use of a linear unloading branch is preferable because the corresponding approximation provides a reduction of energy dissipation compensating the overestimate corresponding to the loading branch.

The loading curve can be represented by means of the following equations, which are still of Ramberg-Osgood type:

$$v = v_o^- + \frac{F}{K_o} + \left(\frac{F}{B}\right)^n \tag{7}$$

for F>0 and:

$$v = v_o^+ + \frac{F}{K_o} - \left|\frac{F}{B}\right|^n \tag{8}$$

for F<0.

The numerical coefficients of equations (7) and (8) and of the one describing the unloading branch can be obtained by minimizing the difference between the energy dissipation corresponding to the analytical model and the one corresponding to the actual hysteresis loops, as given in fig.4.

Being B(F^+,v^+) the point at the highest shear displacement experienced in the loading range, the unloading curve in the positive range can be provided by the relation:

$$v = \frac{F - F^+}{\rho K_o} + v^+ \tag{9}$$

while the one corresponding to the negative range is given by:

$$v = v^- + \frac{F - F^-}{\rho K_o} \tag{10}$$

It is important to point out that the proposed hysteretic model represents a simplification and an adaptation to this problem of a more general model which has been originally proposed for the simulation of beam-to-column joint behaviour under cyclic loads (De Martino et al., 1984) and presented at the workshop in Cachan (Mazzolani, 1987).

The model in its original form allows to take into account either the slippage phenomena in the loading branch or the non-linearity of the unloading branch. Furthermore, the original model also gives the possibility to introduce a stiffness degradation as the number of cycles increases.

In this case the assumed simplifications are justified in view of the application of the model in a computer program for the dynamic inelastic analysis of structures.

4. APPLICATION OF THE PROPOSED MODELS

4.1 Analyzed structure

As a first application of the above models, a pin-jointed steel structure braced by means of trapezoidal sheet panels has been analyzed (fig.7). Its trasversal behaviour under seismic loads has been examined considering two cases: riveted panels and welded panels. The story weight is 1120 kN at each story.

The given structure has been studied by means of a dynamic inelastic analysis which has been performed using a ground motion generated starting from the elastic design response spectrum given in the new proposal of the italian seismic code (CNR-GNDT) for soil type S_1, with a PGA (peak ground acceleration) equal to 0.15 g, 0.25 g and 0.35 g for low, medium and high seismicity zones, respectively.

This structural typology can be economically advantageous in particular in low seismicity zones, because it provides the maximum economy in cost. The convenience to use this typology in low seismicity zones has been analysed by considering riveted or screwed panels. These results have been compared with the most common structural type: the pin-jointed X-braced scheme (Mazzolani and Piluso, 1990).

In the following section the main differences in the seismic response of pin-jointed structures braced by means of riveted panels and welded panels are given.

4.2 Results of dynamic inelastic analysis

Dynamic inelastic analysies have been performed for the two cases of riveted and screwed panels by using the analytical models described in 3.1 and 3.2 respectively. The computations have been carried out using a scope-oriented computer program (Mazzolani and Piluso, 1990) in which bracing panels are introduced by means of an equivalent couple of diagonals. It has been assumed that these diagonals have a cyclic behaviour corresponding to the one provided by the proposed analytical models.

The program allows to evaluate all time histories, such as the ones of nodal displacements, stress and strain of members and panels.

For the three considered zones:

zone 1, low seismicity

zone 2, medium seismicity

zone 3, high seismicity

the maximum displacements (fig.8) and the maximum interstory drifts (fig.9) are compared as significant behavioural parameters.

Two cases are considered for the structure braced by panels (fig.7):

a. the panels are riveted

b. the panels are welded

The results of fig.9 are very important in view of the control of the ductility demands for the bracing panels.

In fact, the required interstory drifts must be compared with the available ductility of panels, which is given by the experimental data (fig. 2, 3 and 4). From this comparison, we can conclude that riveted and screwed panels could be used in low seismicity zones only, while welded panels are strictly necessary in zones of higher seismicity. The above result confirms how the connecting system plays a very important role in view of the possibility to use pin-jointed structures in seismic zones, provided that cladding panels made of corrugated sheets should guarantee sufficient ductility as requested from their bracing function.

4. REFERENCES

De Martino A., Faella C., Mazzolani F.M.: «Simulation of beam-to-column joint behaviour under cyclic loads», Costruzioni Metalliche, N.6, 1984.

Mazzolani F.M.: «Mathematical Model for Semi-Rigid Joints under Cyclic Loads», published in «Connections in Steel Structures, Behaviour Strength and Design», Elsevier Applied Science, 1987

Mazzolani F.M., Piluso V.: «Skin effect in pin-jointed steel structures», Ingegneria Sismica, N.3, 1990

Mazzolani F.M., Sylos Labini F.: «Skin-frame interaction in seismic resistant steel structures», Costruzioni Metalliche, N.4, 1984.

Sanpaolesi L.: «Indagine sperimentale sulla resistenza e duttilità di pannelli-parete in lamiera grecata», Italsider, Quaderno Tecnico N.7, 1984.

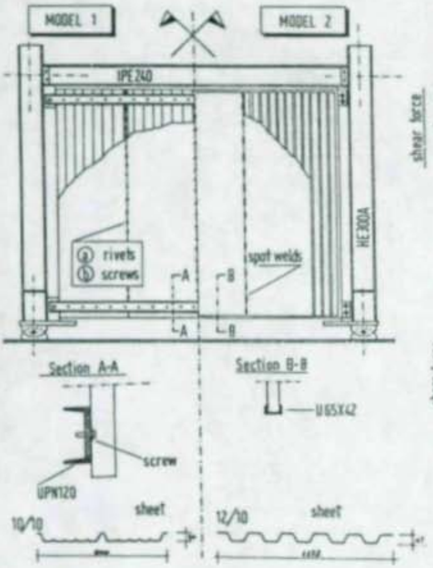

FIG.1

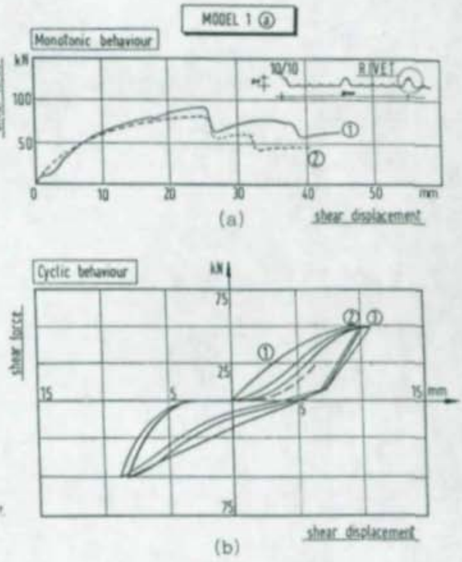

FIG.2

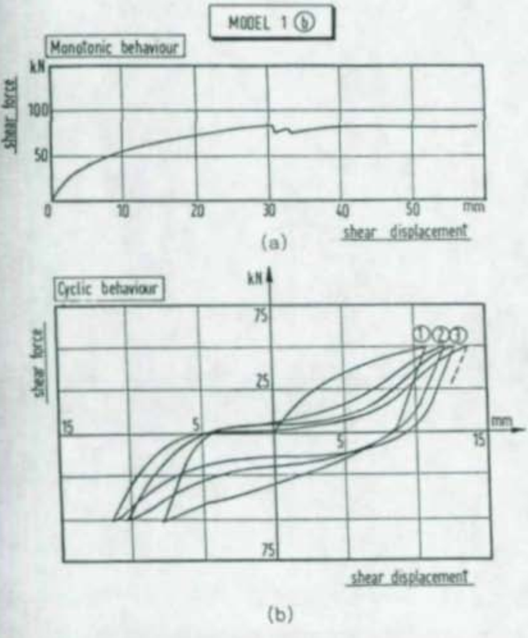

FIG.3

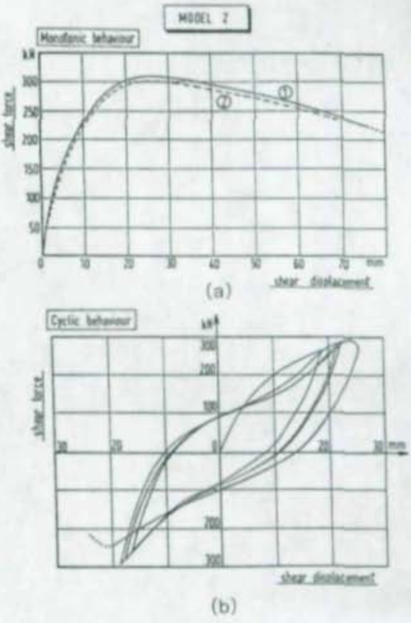

FIG.4

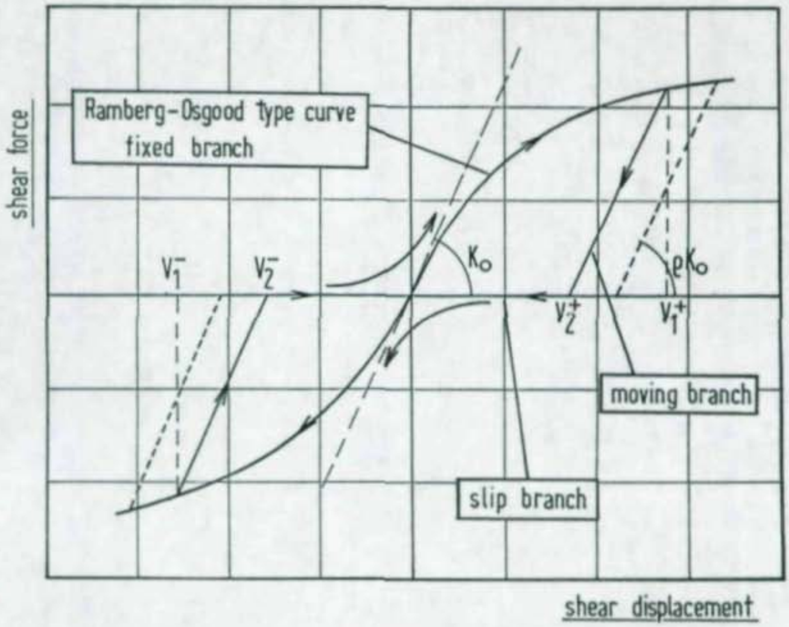
Ramberg-Osgood type curve
fixed branch
shear force
K_o
V_1^-
V_2^-
V_2^+
V_1^+
ρK_o
moving branch
slip branch
shear displacement

FIG.5

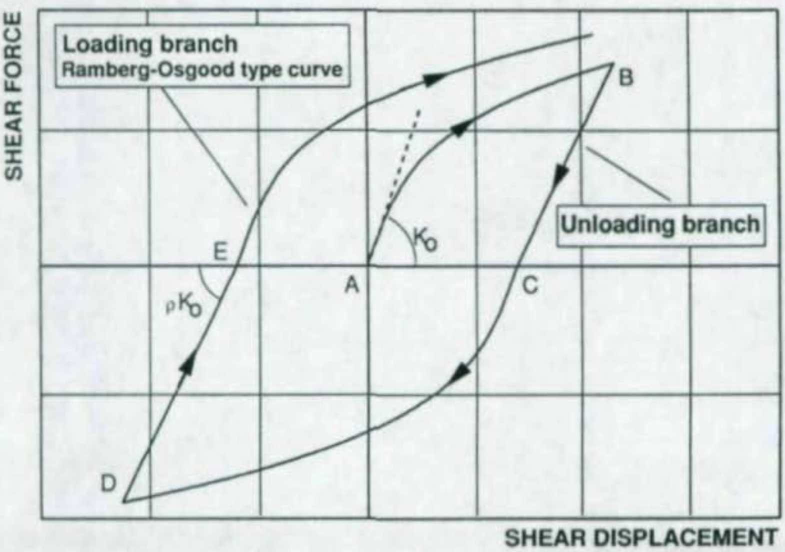
Loading branch
Ramberg-Osgood type curve
SHEAR FORCE
B
Unloading branch
K_o
E
A
C
ρK_o
D
SHEAR DISPLACEMENT

FIG.6

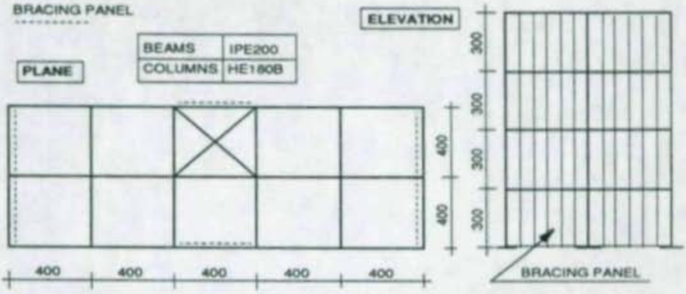

Fig.7

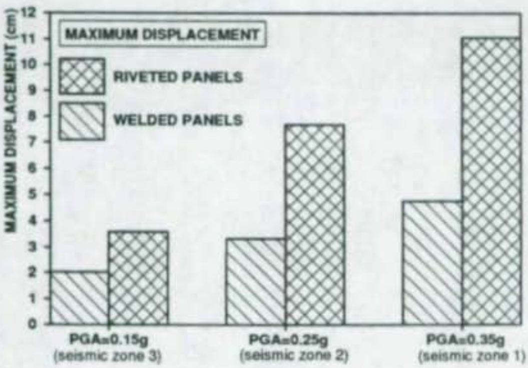

Fig.8

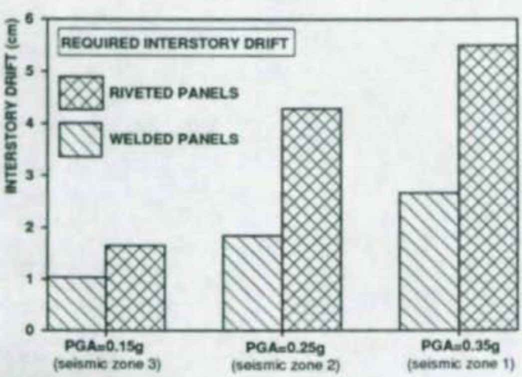

Fig.9

ANALYTICAL MODELING OF CYCLIC BEHAVIOR OF BOLTED SEMI-RIGID CONNECTIONS

Laszlo Dunai[1]

Le-Wu Lu[2]

Abstract

An analytical model is introduced to study the cyclic behavior of bolted semi-rigid connections. The study concentrates on the deformation of the connecting elements and assemblages of the connections by applying a 2D finite element model. The cyclic plasticity material behavior is modeled by the extended Mroz model. Results of illustrative cyclic loading analysis of top-and-seat-angle connection is presented.

1. INTRODUCTION

The influence of connection flexibility on the monotonic loading and cyclic behavior of steel framed structures has been under research in recent years. The purpose of studies on monotonic behavior is to obtain the moment-rotation relationship of the semi-rigid joints, which is the basic aspect of the design procedures. Studies on cyclic behavior investigate the hysteretic moment-rotation behavior and the capability of the semi-rigid joint for energy absorption. Most of the research on cyclic behavior are experimental investigation applying load reversal on the specimens (e.g. Astaneh et al., 1989; Azizinamini et al.,1985; Ballio et al., 1989; Chasten et al. 1989; Dicorso et al. 1989). Some analytical models have been derived for the prediction of cyclic characteristics of semi-rigid joints based on the results of laboratory tests (e.g. Dicorso et al. 1982; Moncarz and Gerstle 1981; Mazzolani 1988).The authors' aim is to model the cyclic behavior of bolted semi-rigid connections based on the cyclic plasticity characteristics of material of joint assemblages. Such analytical model can predict the main features of hysteretic behavior and the effects of different structural

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solutions on it. This paper shows the main characteristics of the proposed model and an application for the cyclic loading analysis of top-and-seat-angle connection.

2. MATHEMATICAL MODEL AND COMPUTATIONAL METHOD

2.1 General Assumptions of the Model

The moment-rotation relationship of a structural steel joint - in general - influenced by (1) deformation of joint due to load introduction, (2) shear deformation of the web of column (3) deformation of the connection (connecting elements). In this study only the third effect is investigated.

It is assumed to have a so called "primary element" of the connection which can represent its load-deformation behavior. This element (e.g. T-stub for end-plate connection, bolted angle for top-and-seat-angle connection) is assumed to carry the load mainly by plane deformation.

2.2 Features of Finite Element Model

A 2D model is introduced for a segment of the primary element of the connection and for the bolt in it (Dunai and Lu, 1990; Krisnamurthy 1980). Bolt pretensioning, bolt bending and bolt head deformation effects are taken into consideration in the bolt model. Isoparametric plane stress elements are used for both angle/end-plate segments and for bolts in the FEM representation.

For the handling of contact problem due to load reversal (separation/recontact) a simple and efficient method is introduced (Dunai, Lu 1990).

The material model of Mosaddad and Powell (1982) is adopted in this study. This model extends the basic Mroz model to consider cyclic behavior of material, with combined isotropic and kinematic hardening. The actual material properties are calculated from the virgin state (first half cycle of loading) and fully cycled state of the material. This transition is controlled by a weighting function, based on accumulated plastic strain.

The effect of changes in geometry due to deformation is also taken into consideration in the FEM model by a simplified approach (geometric nonlinearity).

An incremental loading process is applied for the solution of the highly nonlinear problem (boundary, material and geometric nonlinearity) arised from the mathematical model.

3. CYCLIC LOADING ANALYSIS

Cyclic loading analysis was performed for an angle segment, and the results are used to predict the cyclic moment-rotation relationship of a top-and-seat-angle connection.Angle segment was experimentally investigated for monotonic loading (Lewitt et al., 1966). Figure 1 shows the original and deformed configuration of the angle segment obtained from comparative numerical study.

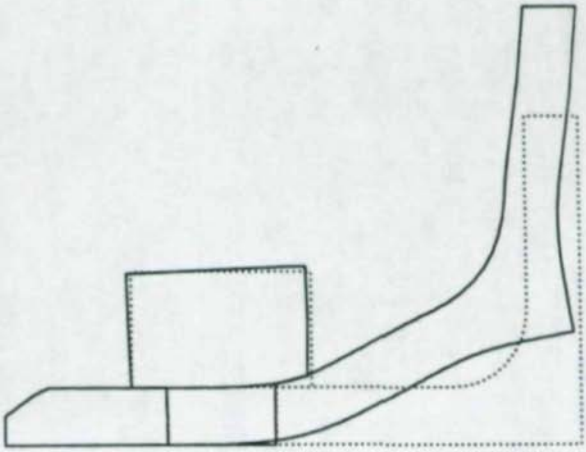

Figure 1

Cyclic loading analysis was performed applying load reversal for the previous angle segment. Load-deformation relationship for two and a half hysteretic loops can be seen in Fig. 2.

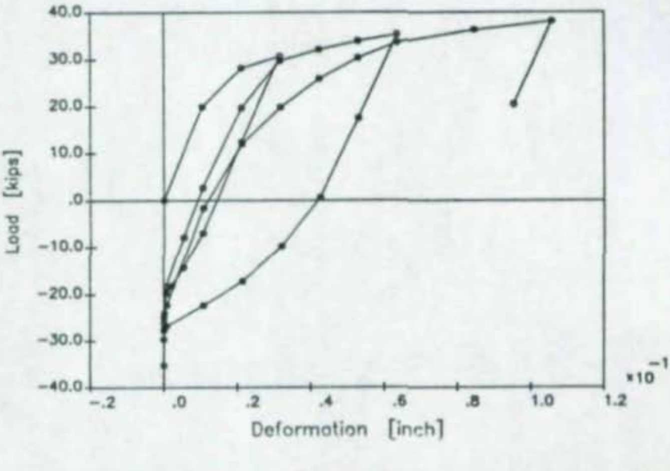

Figure 2

An illustrative cyclic moment-rotation curve is derived from the results of cyclic loading angle segment analysis (see Fig. 3). It can be seen in the figure the typical characteristics of the moment-rotational performance (local softening/hardening of rotational stiffness).

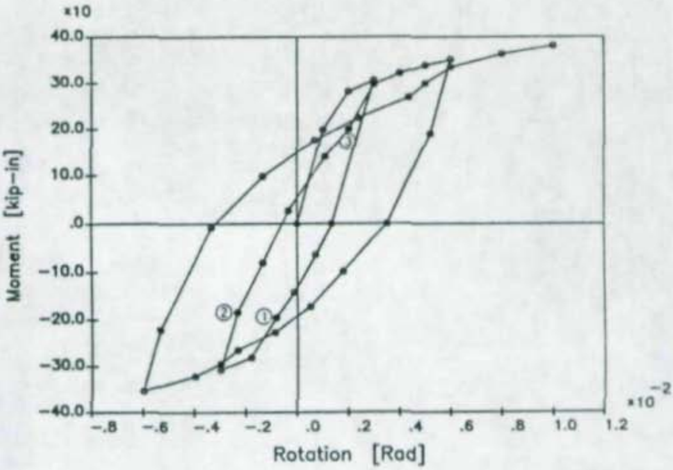

Figure 3

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REFERENCES

ASTANEH A., NADER M. N., MALIK L. (1989). "Cyclic behavior of double angle connections," *Journal of Structural Engineering, Vol. 115, No.5, pp. 1101-1118*

AZIZINAMINI A., BRADBURN J. H., RADZIMININSKI J. B. (1985). "Static and cyclic behavior of semi-rigid steel beam-to column connections," Report of Department of Civil Engineering, University of South Carolina, Columbia, SC

BALLIO G., DE MARTINO A., MAZZOLANI F. M. (1989). "The semi-rigid behavior of beam-to-column bolted joints," International Colloquium on Bolted and Special Structural Connections, Moscow, Proceedings, Vol. 3, pp. 16-24

CHASTEN C. P., FLEISCHMAN R. B., DRISCOLL G. C., LU L. W. (1989). "Top-and-seat-angle connections and end-plate connections: behavior and strength under monotonic and cyclic loading," Proceedings, 1989 National Steel Construction Conference, AISC, Nashville, TN, pp. 6-1 to 6 32

DICORSO P., HARPER W., REINHORN A., DICKERSON J. (1989). "Structures with semi-rigid connections," Steel structures Proceedings, Structures Congress, ASCE, San Francisco, CA, pp. 529-538

DUNAI L., LU L.W. (1991). "Analytical studies of monotonic and cyclic behavior of flexible connections with bolted angles," ATLSS Report, Lehigh University, Bethlehem, PA (to be published)

KRISNAMURTHY N. (1980). "Modelling and prediction of steel bolted connection behavior," Computers and Structures, Vol. 11, pp. 75-82

LEWITT C. W., CHESSON E., MUNSE W. H. (1966). "Restraint characteristics of flexible riveted and bolted beam-to-column connections," Structural Research Series NO. 296, Department of Civil Engineering, University of Illinois, Urbana, IL

MAZZOLANI F. M. (1988). "Mathematical model for semi-rigid joints under cyclic loads," Connections in Steel Structures, Behavior, Strength and Design, Elsevier Publishers Ltd., New York, NY, pp. 112-120

MONCARZ D. P., GERSTLE K. H. (1981). "Steel frames with nonlinear connections," Journal of Structural Division, ASCE, Vol. 107, No. ST8, Aug., pp. 1427-1441

MOSADDAD B., POWELL G. H. (1982). "Computational models for cyclic plasticity, rate dependence, and creep in finite elemnt analysis," Report NO. UCB/EERC-82/26, University of California at Berkeley, CA

Technical Papers on

EXAMPLES OF FRAME DESIGN

BEHAVIOUR OF SEMI-RIGID CONNECTIONS AND IMPLEMENTATION IN FRAME DESIGN

D A Nethercot[1]

Abstract

The results of recent research into the behaviour of non-sway steel frames provided with semi-rigid/partial strength joints is synthesised into a set of design proposals. These cover behaviour about the major and minor axes and recognise the possibility of biaxial behaviour of the columns. The results are presented as a set of relatively simple design steps.

INTRODUCTION

The design of steel frame structures has always entailed the making of certain assumptions concerning suitable approximations to their actual behaviour. The most important of these have traditionally been (Baker et al, 1956):

1. Sway frame or non-sway frame
2. Pin joints or rigid joints

Once these choices have been made, the design process can follow a series of logical steps.

Recently the second of these choices concerning the behaviour of the joints has been expanded through consideration of a third, intermediate form of joint - semi-rigid. Thus modern design codes such as EC3 and the AISC LRFD now include rules covering both the principles and (to a lesser extent) the application of semi-rigid design. These are the result of studies from a wide variety of sources that have contributed greatly towards the development of a better understanding of the processes of end-restraint and moment transfer. However, conversion of this

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information into comprehensive, yet readily understood, design rules has been less successful (Nethercot, 1989), close examination of the relevant code clauses suggesting that the "ordinary engineer" would find considerable difficulty in their actual implementation. This is understandable given the complexity of the subject; translation of the basic research information into straightforward design procedures is arguably a more challenging task than the production of the original results.

This paper will present the basis for a complete design treatment, citing the background supporting evidence wherever possible, for a limited class of structure. By restricting the application, it is possible to give adequate attention to all the detailed aspects of the subject, thereby avoiding the overwhelming complexity inherent in trying to apply the principles to all forms of construction.

Connection Behaviour

The basis for any treatment of semi-rigid joint action is the moment-rotation (M-ϕ) characteristic of the connections. This has been well researched from the point of physical testing, numerical analysis and simple behavioural/predictive methods (Nethercot and Zandonini,1989). Inevitably some gaps in appreciation of the behaviour of particular joint types remain but sufficient data are now available for the general principles to have been firmly established. Thus it is clear that all normal forms of steelwork joint have some degree of rotational stiffness (semi-rigid) and moment capacity (partial strength). Moreover, the relationship between moment and rotation is generally non-linear, depending in a complex fashion on the exact joint details and with reductions in slope (stiffness) as the maximum capacity is approached. From all the available data the key performance indicators may be established as:

i	Moment capacity	- M_c	Controls the form of the bending moment distribution at collapse
ii	Rotational stiffness	- k_i	Influences deflections at working load.
iii	Rotation capacity	- ϕ_u	Controls the scope for moment redistribution and thus for employing a mechanism type approach to determining collapse loads.

Applying these concepts to the simple illustrative example of a beam attached at either end to rigid columns by similar semi-rigid connections of Figure 1 permits the principal effects on member behaviour listed above to be identified.

Each quantity is discussed in the draft EC3 with simplified quantitative procedures being given for certain arrangements. Alternatively, reference may be made to more specialist literature (Nethercot and Zandonini, 1989).

Frame Behaviour

For non-sway frames two basic design premises are possible:

i Strong column, weak beam
ii Weak column, strong beam

In the former failure is assumed to be triggered by collapse of the beams, the columns being capable of supplying some degree of restraint; in the latter the stronger beams are assumed to be capable of supplying restraint to the columns as they initiate failure by buckling. Figure 2 illustrates for a simple portal frame the basic ideas. When considering the collapse of columns, it is necessary, if the full three-dimensional aspect of the problem is to be recognized, to consider behaviour about the a) strong axis and b) weak axis.

Combining these two factors suggests that the most logical approach for frames consisting of conventional I and H sections is to link i to a and ii to b. Thus in the strong-axis direction the columns are assumed to assist the beams but for behaviour about the weak axis end-restraint from the beams is assumed to be available to help resist column failure. This idea goes back to the original work on plastic design for rigid jointed frames (Baker et al, 1956), for which the parallel would be the so-called $P_x E_y$ approach, signifying plastic design of the major axis beams and elastic design of the minor axis beams. Such an approach forms the basis for the Joint Committee method (Institution of Structural Engineers and Welding Institute, 1971) and for its subsequent development into the variable stiffness technique (Wood, 1974). If semi-rigid joints are employed, the same basic philosophy is present i.e. major axis beams which are assumed to transfer most of the moment caused by the floor loading are designed as economically as possible, whilst minor axis beams are designed somewhat less economically but, more importantly, provide significant restraint as a means of enhancing column strength. Such an approach is likely to lead to the best balance in terms of resisting the applied loading in the most economical way.

Design of Major Axis Beams

Assuming that the end connections possess sufficient ductility, these beams may be designed for strength using a quasi-plastic approach as illustrated in Figure 2, with the net moment at mid-span M_B given by

$$M_B = wL^2/8 - M_J \tag{1}$$

or since
$$M_J = x\frac{wL^2}{12} \tag{2}$$

in which x = fraction of full (elastic) fixity

Thus
$$M_B = \frac{wL^2}{24}(3 - 2x) \tag{3}$$

in practice the designer is free to select a suitable figure for x based on the preferred type of connection and thus the anticipated level of moment capacity. Since the joints are assumed to reach M_J before a plastic hinge forms at the mid-span cross-section of the beam, this member will not need to be restricted to those cross-sections capable of participating in plastic-hinge action i.e. its geometrical proportions need only meet the limits necessary for the attainment of the plastic moment capacity of the cross-section.

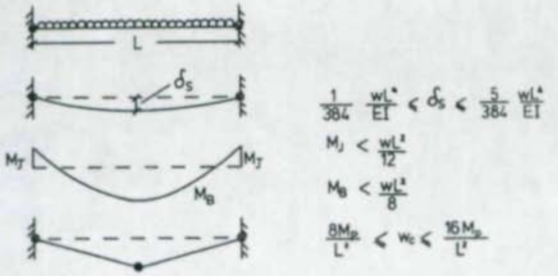

Figure 1 Behaviour of Beam with Semi-rigid/Partial Strength Connections

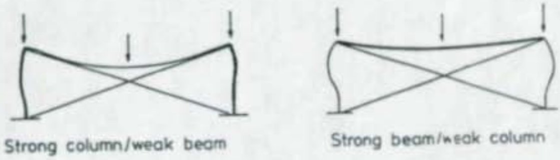

Figure 2 Types of Non-Sway Frame

At serviceability it is assumed that the joints are still functioning on the initial, relatively steep part of the M-ϕ characteristic. This assumes that this initial stiffness (or the designer's estimate of it) will be available for approximately 70% of the M-ϕ characteristic (assuming a load factor on collapse of the order of 1.5). The effect of the end restraint may be likened to that of a spring restraint and, strictly speaking, deflection calculations should be made on this basis. However, a safe (and sufficiently accurate) estimate of the mid-span deflection may be obtained by assuming the ratio of joint moment to mid-span moment to be the same at serviceability as it is at collapse

This gives -

$$\delta_s = \frac{5}{384}\frac{wL^4}{EI} - \frac{M_sL^2}{8EI} \qquad (4)$$

in which $$M_s = \frac{wL^2}{12}\left[\frac{M_J}{wL^2/8 - M_J}\right] \qquad (5)$$

or $$\delta_s = \frac{wL^4}{EI}\left[\frac{125-100x}{9600}\right] \qquad (6)$$

The second term in Eq. 4 is the deflection due to the joint moment at the serviceability condition M_s, expressed as the ratio of joint moment at collapse M_J to mid-span moment at collapse M_B times the elastic support moment assuming fixed ends of $wL^2/12$.

It should be borne in mind that design at the serviceability condition is essentially a requirement to ensure that deflections do not exceed a given limit. Great precision is thus not required; the simplest calculation that produces an acceptable deflection is all that is necessary. However, an improved estimate of deflections, making better allowance for the combined restraining effects of the end connection and the column i.e. allowing for column flexibility, has recently been devised (Gibbons, 1991). This has been worked out in a very general way and consists essentially of modifying the beam deflections calculated on the basis of rigid joints so as to allow for the additional component due to joint flexibility. The principle will readily be appreciated from the basic expresssion:

$$\delta_{sr} = \delta_{rigid} + (1-\mu)(d_{pin} - \delta_{rigid}) \qquad (7)$$

in which δ_{sr}, δ_{rigid} and δ_{pin} are the deflections of the beam in the semi-rigid frame, fully rigid frame and assuming simple supports respectively and μ defines the degree of full fixity under elastic conditions provided by the actual joint arrangement; it may adopt values between zero for pinned ends and 1.0 for fixed joints. δ_{pin} and δ_{rigid} may be calculated by any convenient method.

Column Design

Steel column design under a combination of compression and applied end moments is normally undertaken by satisfying *interaction formulae. All steel design* codes contain suitable equations. Thus a first approach to the inclusion of semi-rigid joint effects in column design would be to attempt to assess the end moments for which the columns should be designed. Figure 3 illustrates the major axis moments acting at the point of failure on the major axis beams. Since the beam to column connections are assumed to have developed their full moment capacity, the end-moment will remain fixed at the value M_J irrespective of the degree of end rotation that has taken place. This is, of course, a consequence of assuming that the joint M-ϕ characteristic has a sufficiently long plateau (rotation capacity) for the beam collapse mechanism to form. The end moment transferred to the column, assuming columns of equal stiffness above and below the joint, becomes:

$$M_C = 0.5\,(M_J + wLd/4) \text{ for an external column} \tag{8}$$

$$M_C = 0.5\,(M_{JL} + M_{JR}) + 0.5\,(w_1L_1d/4 - w_2L_2d/4) \text{ for an internal column with an unbalanced arrangement of beams/loads} \tag{9}$$

in which M_{JR}, M_{JL} are the joint moments on either side of the external column w_1,w_2 and L_1, L_2 are the load and span of the beams on either side of the joint.

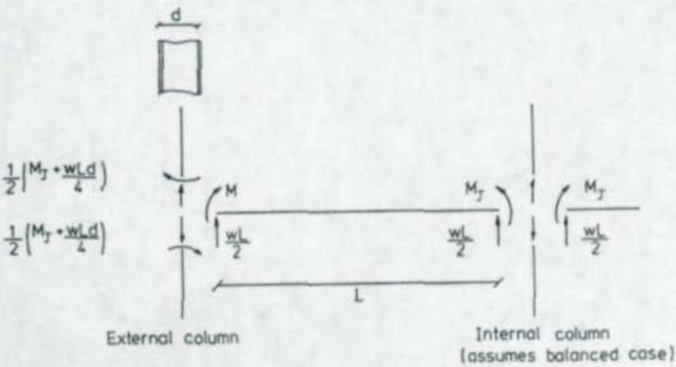

Figure 3 *Structural Actions for Column Design*

Since the connection has reached the plateau on its M-ϕ curve, its effective stiffness will be zero and thus no restraint about the major axis of the column will be possible from the beam/connection combination; the appropriate effective length for strong-axis buckling is therefore the full clear span.

In practice open section columns are much more prone to failure by buckling about the weak axis, possibly combined with some degree of torsion. Indeed, use of the conventional interaction formula approach with known end moments shows that it is normally necessary to have more restrictive support conditions about the minor axis if failure under either major axis bending or biaxial bending is to be anything other than in the weak axis direction. For restrained columns the situation is more complex as the beam/joint combination both provides some degree of restraint and controls the transfer of moments. As indicated in the previous paragraph, if the connection reaches its moment capacity, then its behaviour is analagous to that of a plastic hinge with both the moment transfer and restraining effects adopting a known value. However, it has already been proposed that both the beams and the connections to the minor axis of the column be designed only up to a load level at which significant restraint is still present. Considerable work is still being undertaken on this aspect of the problem (Gibbons, 1991); as a result of material produced to date, the following procedure would appear to be reasonable.

Sophisticated numerical analyses for a variety of cases, covering much of the practical range, have indicated that the beneficial effects of end restraint – even with relatively flexible, low moment capacity connections such as double web cleats – will almost always outweigh the deleterious effects of applied minor axis moments (Gibbons et al, 1991). This suggests a particularly simple approach in which for minor axis buckling the columns are designed as restrained but subjected to axial load only. Thus in this particular case neither the major axis nor the minor axis moments need be considered in the design; indeed no attempt need be made to try to calculate the minor axis moments. This proposal is, of course, directly analagous to the variable stiffness method of column design as proposed (Wood, 1974) for columns in rigid jointed frames in which the minor axis beams are assumed to remain elastic up to collapse.

Indeed some attempts (Shea, 1989) at adapting Wood's approach have been made but further work is necessary to refine its ability to reflect certain key factors in order to make it sufficiently sensitive to the additional effects introduced by the presence of partial strength/semi-rigid connections. Thus for the present the simple approach of decoupling the 3-dimensional problem into its components and:

1. designing for major axis buckling using the axial load and end moments of Fig. 3 with $L_{EX} = L$

2. designing for minor axis buckling under axial load only with L_{EY} calculated as indicated below

is recommended. This separation of behaviour about the two axes and an (implied) neglect of twisting follows directly from observations of the absence of twisting as a cause (rather than as a consequence) of failure in both full-scale tests

(Gibbons et al, 1990) and in numerical analyses (Wang and Nethercot, 1988) of subassemblage behaviour.

Determination of L_{EY} requires that the stiffness of the minor axis beam/connection combination be estimated using the suggestion (Galambos, 1982):

$$K = \left[\frac{1}{1+K_B/K_J} \right] KB \qquad (10)$$

in which K_B is the stiffness of the beam (I/L)

Since minor-axis joints are assumed to remain on the initial, steep part of the M-ϕ curve, all connections may be assumed to contribute i.e. it is not necessary to restrict consideration of the restraining effect to the unloading connections (Bjorhovde, 1984). Using the chart form of presentation of EC3 Annex E given as Fig. 4 requires restraint factors η_1 and η_2 for either end to be calculated from:

$$\eta_1 = \frac{K_c + K_1}{K_c + K_{B1} + K_{11} + K_{12}} \qquad (11)$$

$$\eta_2 = \frac{K_c + K_2}{K_c + K_2 + K_{21} + K_{22}} \qquad (12)$$

and $K_c, K_1, K_2, K_{11}, K_{12}, K_{21}, K_{22}$ are defined in Figure 5.

In practice since beam stiffnesses K_{12} etc will normally be much greater than column stiffness K_c, the effect of the K_J modification of Eq. 10 to K_B on the eventual L_E value will often be very small.

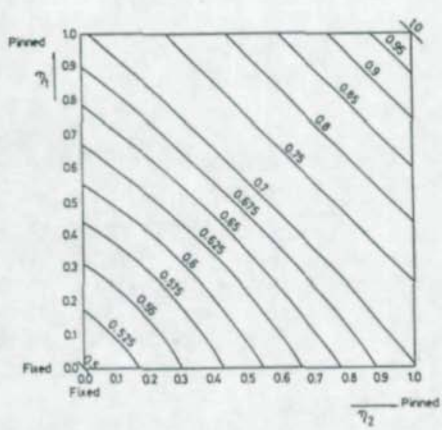

Figure 4 Effective Column Length Ratio L_E/L - Non-sway Case

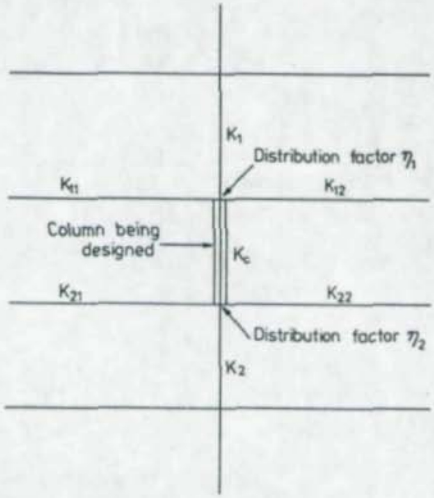

Figure 5 Member Stiffnesses used to Determine Column Effective Length

CONCLUSIONS

The essential features have been given of a method for the design of non-sway frames that recognises the presence of semi-rigid/partial strength joints. It is based on considerations of the true 3-dimensional behaviour of the main components, simplified as a result of careful appraisal of a large body of research data. Although certain details remain to be more fully worked out, the approach is believed to be competetive with traditional methods in terms of design effort, to properly recognise the essential features of the real behaviour, to provide the designer with considerable freedom of choice and to lead to economies in material use.

ACKNOWLEDGEMENTS

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REFERENCES

Baker, J.F., Horne, M.R. and Heyman, J., "The Steel Skeleton", Vol. 2, Cambridge University Press, 1956.

Bjorhovde, R., "Effect of End Restraint on Column Strength - Practical Applications", Engineering Journal, AISC, Vol. 21, No. 1, 1984, pp 1-13.

Galambos, T.V., Discussion on "Small End Effects on Strength of H-columns", Journal of the Structural Division, ASCE, No. 514, April 1982, pp. 1067-1077.

Gibbons, C., "Strength of Beam-Columns in Steel Frames having Semi-rigid Connections", Ph.D thesis, University of Sheffield, 1991.

Gibbons, C., Kirby, P.A., and Nethercot, D.A., "Experimental Behaviour of 3-D Column Subassemblages with Semi-rigid Joints", Third International Symposium on Stability of Steel Structures, Budapest, 1990, Vol. 1, pp. 225-233.

Gibbons, C., Wang, Y. C., and Nethercot, D.A., "The Influence of Steelwork Connections on the Behaviour of Simple 3-dimensional Frames", Journal of the Singapore Steelwork Society, 1991, (in press).

Institution of Structural Engineers and Welding Institute, "Joint Committee Report on Fully-rigid Multi-storey Welded Steel Frames", 2nd report, 1971.

Nethercot, D.A., "Frame Analysis and the Link Between Connection Behaviour and Frame Performance", Frame and Slab Structures, ed. G S T Armer and D B Moore, Butterworths, 1989, pp. 57-74.

Nethercot, D.A., and Zandonini, R. Z., "Methods of Prediction of Joint Behaviour", Structural Connections - Stability and Strength, ed. R. Narayanan, Elsevier Applied Science Publishers, 1989, pp. 23-62.

Shea, T.C., "Variable Stiffness Design of Steel Columns", M.Sc. dissertation, University of Sheffield, 1989.

Wang, Y.C., and Nethercot, D.A., "Ultimate Strength Analysis of Three Dimensional Column Subassemblages with Flexible Connections", Journal of Constructional Steel Research, Vol. 9, No. 4, 1988, pp. 235-264.

Wood, R.H., "A New Approach to Column Design", HMSO, 1974.

DESIGN ANALYSIS OF SEMI-RIGID FRAMES WITH LRFD

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Abstract

The approach is based primarily on the moment magnification factor method (B_1, B_2 factors in the AISC-LRFD Specification). While the B_1 and B_2 factors are applicable only to frames with rigid joints, modifications are made here to extend these factors to accommodate the effect of flexible joints. The modifications include the use of two design connection stiffnesses and the use of alignment charts for determining the effective length factor of the elastically-restrained columns.

The design moment of the member is then determined by the usual LRFD approach and the beam-column interaction equation is used for the design of the members. The implementation of this simplified method on personal computer is demonstrated. Numerical examples are also given.

1. INTRODUCTION

A simplified procedure for the analysis of unbraced flexible frames was introduced recently by Barakat and Chen (1990). The procedure is largely based on the philosophy of the so-called B_1 and B_2 amplification factor method recommended by the AISC/LRFD Specification (1986). While the amplification factor method is developed for elastic rigid frames, the simplified analysis procedure suggests a number of modifications to accommodate the presence of flexible connections. These include: two linearized moment-rotation relationships (expressed by R_{k0} and R_{kb}) for modeling the connection behavior and a modified relative stiffness factor (G') for determining the

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effective length of elastically-restrained columns. The modified initial stiffness (R_{k0}) is used for implementation in the first-order analysis of the nonsway frame configuration which produces M_{nt} moments. The secant stiffness R_{kb} is used for implementation in the first-order analysis of the sway frame which yields M_{lt} moments. The design moment of a member is then determined by the equation

$$M_u = B_1 M_{nt} + B_2 M_{lt} \tag{1}$$

Herein, the highlights of this analysis procedure is presented. The implementation of the simplified analysis procedure on personal computers is demonstrated. Numerical examples are also discussed.

2. MOMENT-ROTATION CURVES AND CONNECTION PARAMETERS

The idealized connection models are developed on the basis of connection M-θ_r curves which, in turn, are formulated on either experimental or analytical basis (Kishi and Chen, 1986, 1990, Chen and Kishi, 1989). For practical implementations, the power model by Kishi and Chen (1990) is used here for the development of the simplified linear connection models (R_{k0} and R_{kb}). As a good representative of the semi-rigid type of connections, the top- and seat-angle with double web-angle connection is used in the numerical examples.

2.1 Determination of R_{k0}

Two connection parameters are required for the determination of R_{k0}. They are the initial connection stiffness R_{ki} and the ultimate moment capacity M_u of the connection. If an experimental connection moment-rotation curve is available, R_{ki} and M_u can be obtained graphically. In lieu of an experimental M–θ_r curve, R_{ki} and M_u for some selected types of connections can be evaluated analytically using the procedure discussed by Kishi and Chen (1990).

With R_{ki}, M_u and a connection model, R_{k0} is determined graphically as depicted in Fig. 1. Note that R_{k0} is the secant stiffness corresponding to a rotation of θ_0. θ_0 is obtained as the intersection of the initial stiffness R_{ki} and the ultimate connection moment M_u. R_{k0} is recommended instead of R_{ki} as a representative connection stiffness for calculating M_{nt} because it was felt that R_{ki} was too high a value to be used for analysis recognizing that the connection stiffness degrades as the moment in the connection increases.

The secant stiffness R_{k0} determined by the above procedure is used in a first-order frame analysis to obtain M_{nt}.

2.2 Determination of R_{kb}

At advanced stages of loading, the connection sustains increasing rotations and consequently exhibits declining stiffness values. For sway frames, the connection is presumed to undergo noticeable deformation when the effect of lateral loads is added to that of gravity loads. In regard to the AISC moment magnification (B_1, B_2) method, this situation may be viewed as the phase in which M_{lt} is determined. The design connection stiffness to be used in this phase should therefore be less than that used for determining M_{nt}. Barakat and Chen (1990) proposed a stiffness value of R_{kb} for the sway analysis. The determination of R_{kb} is shown schematically in Fig. 2. In the figure, curve 1 represents the deformation due to column rotation, curve 2 represents the deformation due to connection flexibility, and curve 3 is the so-called beam line.

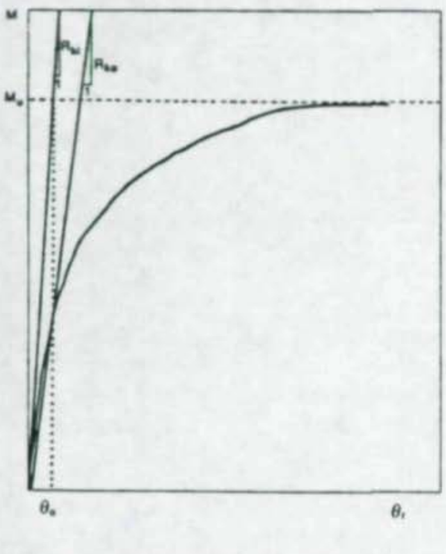

Fig. 1. Determination of R_{k0}

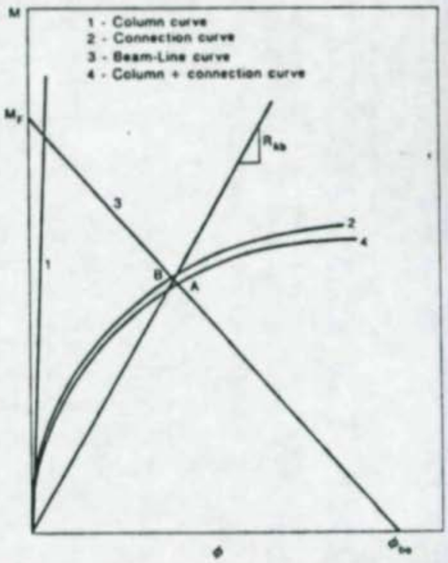

Fig. 2. Determination of R_{kb}

Curve 4 in Fig. 2 represents the combined effect of column rotation and connection deformation. Compatibility of rotational deformation at a joint will be satisfied at the intersection of the beam line (curve 3) and curve 4 (point A in the figure). However, for design purposes, it is reasonable to assume that the effect of column rotation is negligible compared to that of connection deformation. Consequently, curve 2 rather than curve 4 is recommended for determining R_{kb}. From Fig. 2 it can be seen that R_{kb} is obtained as the secant stiffness corresponding to a rotation defined by the intersection of curve 2 and curve 3 (point B in the figure).

3. DETERMINATION OF K FACTOR FOR COLUMN DESIGN

The column effective length factor K is obtained from the alignment charts in the usual manner using the modifications for elastically-restrained beam ends as described in the thesis by Barakat (1988).

The design procedure is summarized as follows.

1. Determine R_{k0} and R_{kb} according to Sections 2.1 and 2.2, respectively.
2. Perform a first-order analysis incorporating the connection stiffness R_{k0} for M_{nt}.
3. Perform another first-order analysis incorporating the connection stiffness R_{kb} for M_{lt}.
4. Determine the G factors.
5. Modify the G factors determined in Step 4 using the procedure described by Barakat (1988) to account for the effect of connection flexibility.
6. Determine K using the alignment charts (AISC nomographs).
7. Evaluate B_1 and B_2 according to AISC/LRFD equations.
8. Obtain the design moment using Eq. (1).
9. Use the beam-column interaction equation to design the member.

4. NUMERIC EVALUATIONS

The same frame configurations and connections described by Goto and Chen (1987) were used here in a numerical study aimed at illustrating the proposed method of analysis and rationalizing its approach using the four semi-rigid connections shown in Fig. 3. In this study, exact second-order and linear first-order analysis were conducted using the computer program FLFRM (Goto and Chen, 1986). Exact analysis which accounts for second-order effects was conducted with the actual nonlinear M-θ_r curve of the connection. As mentioned earlier, the first-order analysis was conducted according to the loading arrangement associated with B_1 and B_2 method and implementing connection stiffness R_{k0} or R_{kb} as applicable. A number of combinations of connections and frames are utilized to illustrate the versatility of the proposed method. The results of analysis are presented in a tabulated form. It is worth mentioning that the related tables include not only the maximum column moment of each floor, but all moments that control the design of all columns in those floors where different member sizes are used.

Table 1 shows moment values of column members (M_{exact}) for two of the five frames determined by exact second-order analysis using connection III-16, as well as moment values (M_u) determined by the simplified analysis procedure. It can be seen that the simplified procedure offers very good predictions for the design column moments in these frames. Table 2 contains the normalized (by exact solution, i.e. M_{exact}) column moments in Frame FR-5. As can be seen, all four connections (III-11,III-14,III-16 and III-17) were used in the analysis. It is evident from these tables that the predictions of the design column moments by the proposed method of analysis are very good and generally conservative for most members. The unconservative moment values (all of which happened to occur in top floor columns) are considered to be within the allowable tolerance in engineering practice (less than 5%). In addition, these values represent, in most cases, better estimates than those obtained by the conventional rigid frame analysis as will be shown later.

Table 1. Moment Values Determined by Proposed (M_u) and Exact (M_{exact}) Analyses, (Connection III-16 is Used, Fig. 3).

Frame Code*		M_u	M_{exact}	M_u/M_{exact}
FR-3	Col. 2	-308	-302	1.03
	Col. 5	-420	-369	1.137
4-Bay				
2-Story	Col. 7	-128	-133	0.968
	Col. 10	-341	-337	1.009
FR-5	Col. 2	-501	-484	1.035
	Col. 3	-603	-505	1.196
2-Bay				
2-Story	Col. 5	-225	-210	1.071
	Col. 6	-339	-354	0.959

*Column numbering sequence: from first floor to top floor, and from left to right in each floor.

The selection of a different, and at the same time, softer connection stiffness (i.e. R_{k0}) as opposed to the initial stiffness (R_{ki}) was based on parametric calculations which showed that the performance of R_{k0} is more adequate for the simplified method of analysis. A sample of data supporting this suggestion is presented in Table 3 which was generated with the implementation of connection III-17. The table shows the results of the exact (second-order with real connection curve) and simplified methods of analysis. The simplified analysis was conducted with two different combinations of the idealized connection stiffnesses: The first combination used R_{ki} and R_{kb} in a first-order analysis to determining M_{nt} and M_{lt}, respectively. The second combination used R_{k0} and R_{kb} in a similar analysis.

Table 2 Design Moments M_u in FR-5 Normalized by Exact Solution M_{exact} (Two-Bay-Two-Story Frame)				
Connection Used (Fig. 3)	Normalized Design Moment			
	Col. 2	Col. 3	Col. 5	Col. 6
Connection III-11	0.993	1.209	1.083	0.950
Connection III-14	1.107	1.187	1.073	0.952
Connection III-16	1.035	1.196	1.071	0.959
Connection III-17	1.084	1.053	1.112	0.966

It can be seen that the design moment M_u determined by using R_{k0} for the calculation of M_{nt} are closer to the exact solution than those obtained by using R_{ki}. This can be explained by examining the values of M_{nt} which are computed by implementing these stiffness values. As compared to R_{k0}, the higher stiffness R_{ki} causes larger moments to be allocated at beam ends which are, consequently, transferred to column ends. It is evident that the modified initial stiffness R_{k0} is a more adequate and reasonable choice for the simplified analysis as opposed to the initial stiffness R_{ki}.

Table 3. Analysis Results for Moments M_{nt} and M_{lt} Using Combinations of R_{ki} and R_{k0} with R_{kb} for Linear Models of Connection III-17 (Fig. 3).

Frame Code	Column No.	Analysis with R_{ki} & R_{kb}			Analysis with R_{k0} & R_{kb}			Exact Analysis
		M_{nt}	M_{lt}	M_u	M_{nt}	M_{lt}	M_u	
FR-3	2	42	251	317	29	251	307	284
	5	239	177	434	236	177	432	424
4-Bay 2-Story	7	56	92	150	42	92	137	133
	10	311	53	365	306	53	361	377
FR-5	2	00	424	462	00	424	468	431
	3	268	311	606	271	311	613	583
2-Bay 2-Story	5	00	201	208	00	201	210	188
	6	231	112	348	236	112	353	366

5. FRAME ANALYSIS ON PERSONAL COMPUTERS

One of the main objectives of the simplified analysis procedure was to establish a simple and practical analysis procedure for the design of steel frames with semi-rigid connections. Herein, attempt is made to provide for even more efficient implementations of flexible frame analysis. The use of personal computers for the analysis and design process is sought. For the problem at hand (B_1 and B_2 concept of analysis), the number of variables, quantities and operations involved is quite large and their determination is a time-consuming process. To this end, the above procedure has been implemented in a spreadsheet program to facilitate the design. Details of this development are given elsewhere (Barakat, 1988).

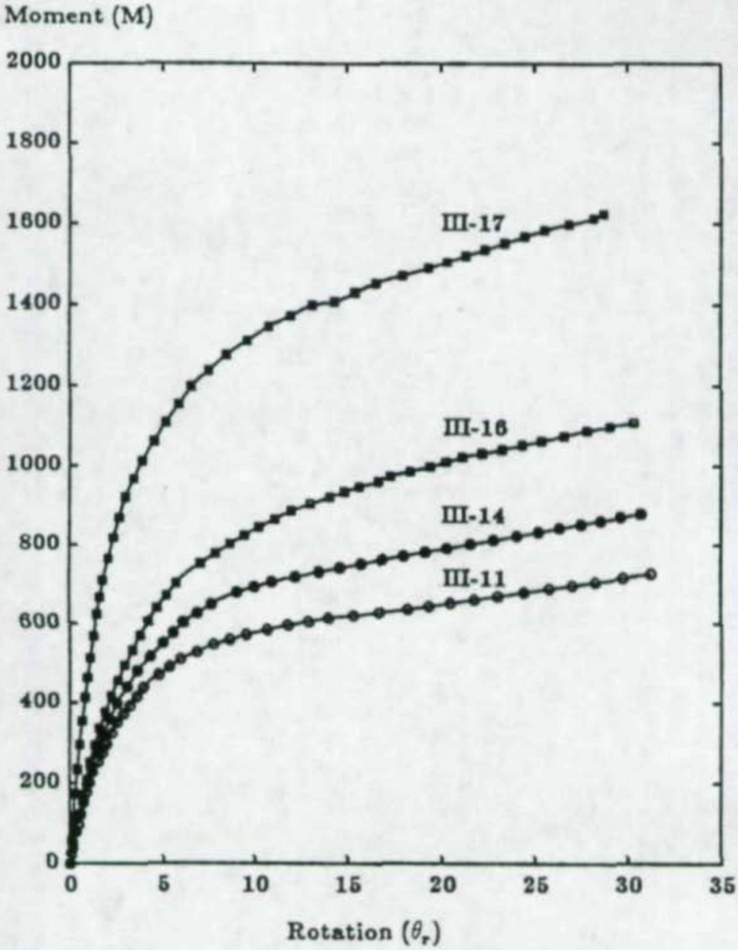

Fig. 3. Experimental Connection Curves (Kishi and Chen, 1986).

6. DRIFT OF SEMI-RIGID FRAMES

As a result of the flexibility of the connections, semi-rigid frames tend to deflect quite significantly (Gerstle and Ackroyd, 1990) under the action of lateral loads. Studies have shown that most flexibly-connected frames designed on the basis of strength violate the drift requirement for serviceability. Consequently, it is paramount that calculations for drift be performed to ensure that excessive lateral deflection will not occur. If the amount of drift exceeds the intended tolerance limit, the lateral stiffness of the frame must be increased.

A spreadsheet application for approximate drift calculations was proposed by Ackroyd (1990) for semi-rigid frames. In lieu of a computer program, the following prediction formulas (Cronembold and Ackroyd, 1986; Gerstle and Ackroyd, 1990) for drift can be used for a preliminary analysis.

For top and seat angles

$$\frac{\Delta}{H} = \frac{W}{90 + 160(B/H)} \tag{2}$$

For flange plates

$$\frac{\Delta}{H} = \frac{W}{130 + 160(B/H)} \tag{3}$$

In the above formulas, Δ is the lateral deflection at the top story of the semi-rigid frame, H is the overall height of the frame, B is the overall width of the frame, and W is the lateral load intensity (in kips/ft of vertical height).

7. SUMMARY AND CONCLUSIONS

The design of semi-rigid frames is intrinsically more complex than the design of rigid frames because of the difficulty and uncertainty in predicting the moment-rotation behavior of the connections. Fortunately, studies by Goto and Chen (1987), Wu (1988) and others have demonstrated that some errors in predicting the response of the connections will not noticeably affect the overall behavior of the semi-rigid frame. Furthermore, studies reported by Gerstle and Ackroyd (1990) have demonstrated that, for practical purposes, connections can be regarded as rigid and the frame can be designed as a rigid framer if the limit

$$\frac{EI_g}{R_k L} < 0.05 \tag{4}$$

is satisfied. In Eq. (4), EI_g is the flexural rigidity of the girder, L is the girder length and R_k is the connection stiffness.

If a second-order analysis program is available which can take into consideration connection flexibility, it should be used for design. In lieu of such a program, simplified design methods which are based on simplified behavioral models for connections (Wu and Chen, 1990) and for semi-rigid frame action can be used. Several of these simplified methods have been proposed in recent years. Herein, a simplified method based on the 1986 AISC/LRFD procedure for rigid frames has been extended to the design of semi-rigid frames.

The proposed simplified analysis procedure for unbraced flexible frames uses a number of assumptions in its formulation. In this paper, numerical examples are used to illustrate the applicability of the analysis procedure as a whole. It has been shown that the performance of the idealized connection models (R_{k0} and R_{kb}) as utilized in the analysis procedure is very good. It can be concluded that the analysis of flexible frames by B_1 and B_2 concept is feasible if proper modifications are undertaken.

The implementation of frame analysis on personal computers can best be made in a spreadsheet format. Based on our personal experience, the spreadsheet format has been found very efficient for such type of design problems.

It is important to note that the LRFD method, as all other methods, is strength based, which means that the design is based on strength, not serviceability. Since excessive frame drift is often a problem for semi-rigid frames, due consideration must be given to check the resulting design for serviceability requirements and proper measures must be taken to limit the drift of semi-rigid frames.

REFERENCES

Ackroyd, M.H. (1990) Electronic spreadsheet tools for semi-rigid frames, *Engineering Journal, AISC*, 24, 4; 141-146.

AISC (1986) *Load and Resistance Factor Design Specification for Structural Steel Buildings*, First Edition, Chicago, IL.

Barakat, M.A. (1988) *Simplified Design Analysis of Frames with Semi-Rigid Connections*, Ph.D. Thesis, School of Civil Engineering, Purdue University, West Lafayette, IN.

Barakat, M.A. and Chen, W.F. (1990) Practical analysis of semi-rigid frames, *Engineering Journal, AISC*, 27, 2; 54-68.

Chen, W.F. and Lui, E.M. (1991) Stability design of steel frames, *CRC Press*, Boca Raton, Florida, 373 pp.

Chen, W.F. and Kishi, N. (1989) Semi-rigid steel beam-to column connections: data base and modeling, *Journal of Structural Engineering, ASCE*, 115, 1; 105-119.

Cronembold, J.R. (1986) *Evaluation and Design of Type 2 Steel Building Frames*, M.S. Thesis, Department of Civil Engineering, Rensselaer Polytechnic Institute, Troy, New York.

Cronembold, J.R. and Ackroyd, M.H. (1986) Economy and safety of semi-rigid frame design: case studies, *Stability of Tall Steel Buildings, Workshop Proceedings, Third International Conference on Tall Buildings*, pp. 113-171.

Gerstle, K.H. and Ackroyd, M.H. (1990) Behavior and design of flexibly-connected building frames, *Engineering Journal, AISC*, 27, 1; 22-29.

Goto, Y. and Chen, W.F. (1987) On the computer-based design analysis for the flexibly jointed frames, *Journal of Constructional Steel Research, Special Issue on Joint Flexibility in Steel Frames* (W.F. Chen, Ed.), 8; 203-231.

Goto, Y. and Chen, W.F. (1986) Documentation of program FLFRM for design analysis of flexibly jointed frames, CE-STR-86-34, School of Civil Engineering, Purdue University, West Lafayette, IN.

Kishi, N. and Chen, W.F. (1986) *Data base of steel beam-to-column connections*, Structural Engineering Report No. CE-STR-86-26, School of Civil Engineering, Purdue University, West Lafayette, IN, 2 volumes, 653 pp.

Kishi, N. and Chen, W.F. (1990) Moment-rotation relations of semirigid connections with angles, *Journal of Structural Engineering, ASCE*, 116, 7; 1813-1834.

Wu, F.H. (1988) *Semi-rigid connection in steel frames*, Ph.D. Thesis, School of Civil Engineering, Purdue University, West Lafayette, IN.

Wu, F.H. and Chen, W.F. (1990) A design model for semi-rigid connections, *Engineering Structures*, 12; 88-97.

PRACTICAL DESIGN ALLOWING FOR SEMI-RIGID CONNECTIONS

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Abstract

The convenience of assuming 'pinned' or 'rigid' connections has blunted interest in their real behaviour because of mathematically complex solutions. However cheap computing power, new codes and more information have awakened fresh interest. The effects of joints on real structures is discussed. Using fixity factors and modified moment distribution the analysis need not be difficult. Even without full connection data, valuable qualitative conclusions of practical value are obtained.

1. INTRODUCTION

Semi-rigid connections can have benefits in steelwork design. It appears that sufficient information will soon be available to allow practical designers to use this method (Nethercot, 1985; Bijlaard et al, 1988).

The apparent difficulties in analysis are reduced by using the concept of fixity factors. They allow the designer to examine behaviour in fundamental terms and provide a 'feel' for the practical effects of varying key parameters.

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2. CONNECTION CHARACTERISTICS

2.1 Basic behaviour

Characteristics of a beam-to-column connection can be obtained by means of testing (figure 1). For a given moment a corresponding rotation is obtained for the beam plus the connection. By subtracting the elastic rotation of the beam, the applied moment can be plotted against the rotation of the connection. This is referred to as the M-ϕ curve.

Figure 2 indicates typical M-ϕ curves for various connections. Behaviour is generally non-linear. An initial stiff phase is followed by a phase of much reduced stiffness. In ductile connections, such as flush endplates or unstiffened extended endplates, a plateau is reached with large rotations at virtually constant moment.

The extended endplate is the stiffest. The flush endplate, although less stiff than the extended endplate, is 'well-behaved', with a very smooth and long plastic plateau. Flange cleats and header plate connections, though semi-rigid to some extent, are best regarded as nominally pinned, as moment capacity is limited.

2.2 Representation for elastic analysis

Many proposals for analysis have been presented (Jones et al., 1983) which call for close approximation of the M-ϕ curve. However it will be argued later that all that is required for practical design is a simplified linear representation. Figure 3 indicates a number of different linear representations. The first is a simple linear approximation which is in effect an average of the behaviour up to yield. The second is a closer approximation in that two stiffness ranges are used up to this yield value. This will give a more accurate representation of the serviceability behaviour, but the ultimate limit state analysis will be little changed. Thus unless deflections are likely to be critical, the simple linear solution is the one to use. Three or more straight lines can be used, as shown in the third solution, but this is generally found to be unnecessary.

3. FRAME ANALYSIS

3.1 The fixity factor

The fixity factor is the most important concept to be discussed in this paper. It is a powerful indicator of connection behaviour (Cunningham, 1990).

The fixity factor α defines the stiffness of a connection relative to the attached beam. The stiffness of the connection is represented by a spring of stiffness S_j. The fixity factor is the ratio of the rotation of the end of the beam due to a unit end moment, divided by the rotation of the beam plus the connection for the same moment (figure 4). With a true pin connection the value of α is 0. For a fully fixed connection there is no rotation of the spring and α becomes 1. This range of values from 0 to 1 is important in giving the practical designer a feel for the degree of fixity of a connection. The value of α indicates how the behaviour of the structure will be affected by the connection, better than the absolute value of S_j. The device of fixity factors allows us to examine frameworks in concept without the immediate need for detailed connection design data.

Figure 5 shows a typical variation of S_j against α. This diagram demonstates that when S_j is large, significant changes in S_j produce only minor changes in α. It also indicates that a rigid connection would require infinite stiffness, so by definition no connection is truly rigid. Nevertheless quite large reductions in stiffness from full fixity can be seen to have little effect on the structure, because only a minor change in α results from a large change in S_j. Conversely, at the lower end, even small increases in S_j produce appreciable increases in α. As even a 'nominally pinned' connection will always have some stiffness, this can lead to restraint moments of significant benefit to a structure.

It is because α is relatively insensitive to S_j that only an approximate M-ϕ curve is required, as the analysis is directly affected by α. This partly explains why structures behave well in practice, due to the enhancing effect of simple connections and the fact that 'rigid' connections do not have to be fully rigid.

3.2 Unbraced frames

The effect of sway on structures with semi-rigid connections is fundamentally important (Souroknikov, 1949; Disque, 1964). The basic effect of sway is to reduce the moment at a connection from the value initially reached under gravity load, due to the connection 'stretching' as it yields on the leeward side under the action of wind. This reduction in end restraint moment is referred to as wind shake-down. In an unbraced frame the full restraint moments developed from the semi-rigid gravity analysis cannot be used in the design of the beam.

3.3 Moment distribution

Moment distribution, suitably modified to take account of semi-rigid behaviour, is a powerful tool in the analysis of semi-rigid structures. Tables 1, 2 and 3 summarise results obtained during development of the method (Cunningham, 1987). The use of the fixity factor within moment distribution leads to great simplification of the analysis. The modified moment distribution method, in which the connections are treated as part of a modified beam, directly parallels that for fully rigid connections, see figure 6. This method can be used without modification for structures of any degree of fixity. Even plastic analysis is easily incorporated as the relevant factors are merely modified to suit a connection with zero stiffness for the relevant load phase.

3.4 Other methods of analysis

Other standard methods of analysis can similarly be modified to take account of α. Moment distribution was selected for illustration because it is a hand method. A general solution for non-uniform members with semi-rigid connections, using the stiffness method, has been provided (Cunningham, 1987).

4. EFFECTS ON STRUCTURES

4.1 Sleeved purlins

The most elementary semi-rigid structure is a sleeved purlin, which is in fact a semi-continuous beam. Figure 7 shows typical M-ϕ curves for a sleeve connection, based on tests for a purlin manufacturer (Cunningham, 1983). The sleeve behaviour is non-linear but can be well modelled using a bi-linear representation of the test results. Full scale test results correlated well with predictions obtained using this model.

4.2 Single storey portal frames

With this form of construction it is normally assumed that the base is pinned and the beam-to-column connection at the knee is fully rigid. In practice the base will have some stiffness, but the knee connection will not be fully rigid.

In practice the knee fixity factor is unlikely to exceed 0.8, because of the shear deformation of the connection and the bending of the end plate and of the column flange. The practical fixity factor of a base may be 0.1 to 0.3, but the moment due to gravity loading attracted to such a connection is quite small.

However the main significance of semi-rigid connections at the base is the reduction of eaves deflections produced by even a nominal 'pinned' base with four bolts and a reasonable thickness of baseplate.

4.3 Braced multi-storey frames

In a braced multi-storey frame of simple construction, with nominally pinned connections, the floor beam connections will actually have some restraint. A close examination of the actual connections may show that the structure can resist higher loads, or enable the beam sizes to be reduced. It may also be found that apparently excessive beam deflections may not actually arise when the end restraint is considered.

However the effects on external columns should also be considered. Because the beam moment is applied on one side only, there may be larger moments in these columns than anticipated.

4.4 Unbraced low-rise multi-storey frames

Low-rise unbraced frames can be designed for gravity loads on the basis of simple construction with portalisation to resist wind sway effects. This is referred to as the 'Wind Connection Method' (Anderson et al, 1991). The connections are loaded to only a limited extent by the wind moments, for the reasons given in 3.2, except in high-rise buildings where this method would not be appropriate. Thus the connections are not greatly increased over those for a simple framework without wind moments.

5. CONCLUSIONS

5.1 Analysis

Anomalies can arise from design approaches in which structural analysis is based on joints which are assumed to be either fully rigid or truly pinned. For greater consistency they should be designed using stiffness criteria as well as strength. Unfortunately, this information is difficult to obtain.

However gross assumptions on the above basis have led to structures which appear to be reliable, perhaps for the wrong reasons.

A better approach might be to take a nominal connection fixity factor for a 'pin' between 0.1 and 0.3 and for a 'rigid' connection between 0.7 and 0.9 and to design for the resulting moments. These gross assumptions are undoubtedly closer to real behaviour than assuming pinned and fixed connections.

5.2 Unbraced frames

The effects of wind shake-down on the connections should be considered in frames subject to sway.

5.3 Modified moment distribution

Moment distribution has been developed to a point where semi-rigid structures can be readily analysed by hand (see figure 6) or by computer. The necessary information is summarised in tables 1, 2 and 3.

5.4 Fixity factors

The concept of the fixity factor α is important in simplifying the analysis and giving the designer a 'feel' for the behaviour of semi-rigid structures.

5.5 Moment rotation curve

There is no need for extreme accuracy when modelling the stiffness curve, as the fixity factor is relatively insensitive to stiffness variations.

The simplest model is a single linear representation and in many situations this will be quite adequate. However, there are some situations where a bi-linear approach will be required and this will mean an analysis of the structure in stages.

5.6 Detail design

A designer can influence the behaviour of a steel structure by simply paying attention to the detail design of connections. Designers should always consider whether connections need to be stiffened or should be given a condition closer to a pin. Some knowledge of the semi-rigid behaviour of the framework can assist in making these decisions even if only approximate stiffness values are available.

REFERENCES

Anderson, D. et al: 'Wind-moment design for unbraced frames' The Steel Construction Institute, 1991.

Bijlaard, F.S.K. et al: 'Structural properties of semi-rigid joints in steel frames', unpublished IABSE paper, May 1988.

Cunningham, R: 'Report of sleeved purlin test results as carried out at Strathclyde University,' private report, Butler Building Products Limited, August 1983

Cunningham, R: 'Effects of joint continuity on tapered member steel portal framed structures', PhD Thesis, Heriot Watt University, April 1987.

Cunningham, R: 'Some aspects of semi-rigid connections in structural steelwork', The Structural Engineer, Vol. 68, No 5, March 1990.

Disque, R. O: 'Wind connections with simple framing', AISC Engineering Journal, July 1964.

Jones, S. W. et al: 'The analysis of frames with semi-rigid connections - a state of the art report' Journal of Constructional Steel Research Vol. 3, No 2, 1983.

Nethercot, D. A: 'Steel beam to column connections - a review of test data', CIRIA (Project 338), London, 1985.

Sourochnikoff, B: 'Wind stresses in semi-rigid connections of steel frameworks', Transactions ASCE, Paper no. 2402, Vol 115, 1949.

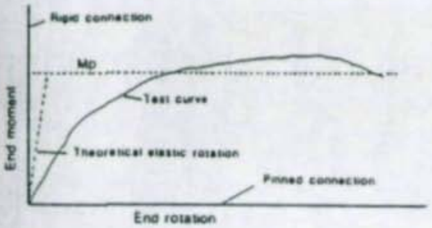


Figure 1 M-ϕ curve for typical extended end plate connection

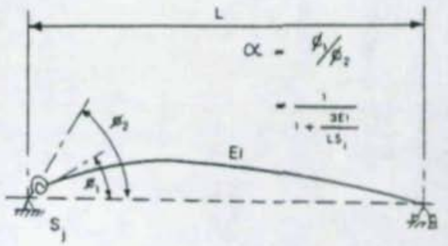


Figure 4 Fixity factor

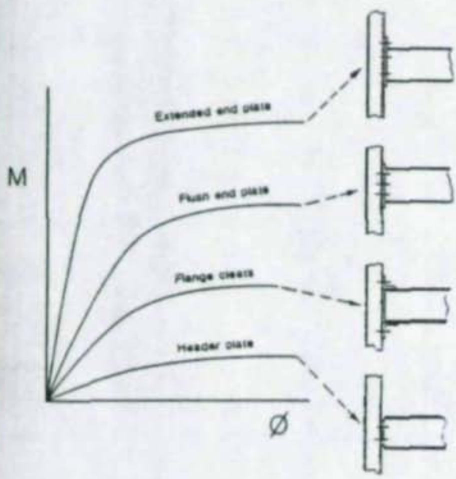


Figure 2 M-Ø comparisons for various connection types

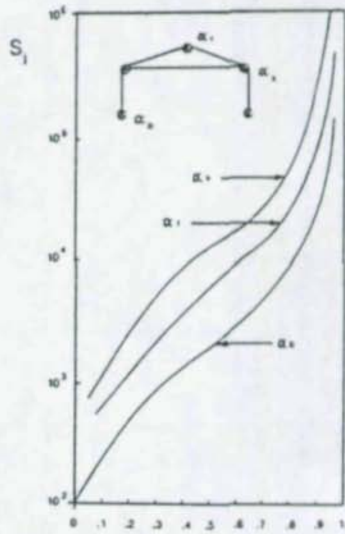


Figure 5. Variation of S_1 and α for a typical portal frame

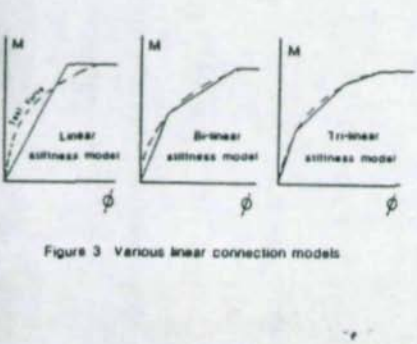


Figure 3 Various linear connection models

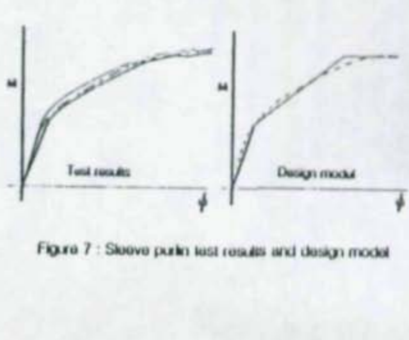
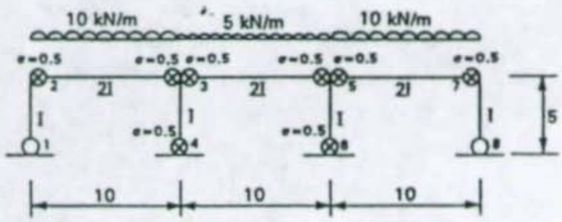


Figure 7 : Sleeve purlin test results and design model



Due to symmetry, consider only half of structure: Jr 35 mod. = Jr 35 (1 - α/2)

Fixed end moments $\quad M23 = -M32 = \dfrac{-10 \times 10^2}{12} \quad \dfrac{3 \times 0.5}{(2 + .5)} = -50$

$$M35 = -M53 = -50 \times 5/10 = -25$$

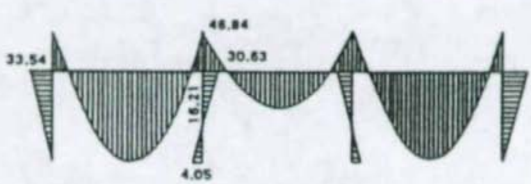
Carry-over factors = α/2 = 0.5/2 = 0.25

Distribution factors

Joint	Term	Relative stiffness	ΣJr	Distribution factor
2	Jr21	$\frac{1}{5} \times \frac{3}{4} = 0.15$	0.23	.15/.23 = 0.65
	Jr23	$\frac{2}{10} \times \frac{3 \times .5}{(4 - .5^2)} = 0.08$		.08/.23 = 0.35
3	Jr32	= Jr23 = 0.08	0.311	.08/.311 = 0.26
	Jr34	$\frac{1}{5} \times \frac{3 \times 1}{(4 - 0.5)} = 0.171$		.171/.311 = 0.55
	Jr35	0.08 × (1 - 0.5/2) = 0.06		.06/.311 = 0.19

Moment distribution

M21	M23	M32	M34	M35	M43	
0.65	0.35 0.25→	0.26 ←0.25	0.55 0.25	0.19		d/f C/O
 +32.5	-50 +17.5	+50 -6.4	 -13.8	-25 -4.8		FEM Bal
 +1.04	-1.6 +0.56	+4.38 -1.14	 -2.41	 -0.83	-3.45	C.O Bal
					-0.6	C.O
+33.54	-33.54	46.84	-16.21	-30.63	-4.05	Sum



Bending moment diagram

Figure 6 Example Worksheet

Table 1 - Joint stiffness factors

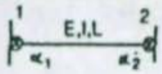
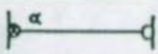
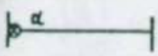
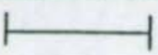
Case	Joint stiffness factor		Relative stiffness factor	
	J12	J21	Jr12	Jr21
	$\frac{4EI}{L}\frac{3\alpha_1}{(4-\alpha_1\alpha_2)}$	$\frac{4EI}{L}\frac{3\alpha_2}{(4-\alpha_1\alpha_2)}$	$\frac{3\alpha_1}{(4-\alpha_1\alpha_2)}\frac{I}{L}$	$\frac{3\alpha_2}{(4-\alpha_1\alpha_2)}\frac{I}{L}$
	$\frac{3EI}{L}\alpha$	0	$\frac{3}{4}\frac{I}{L}\alpha$	0
	$\frac{4EI}{L}\frac{3\alpha}{(4-\alpha)}$	$\frac{4EI}{L}\frac{3}{(4-\alpha)}$	$\frac{3\alpha}{(4-\alpha)}\frac{I}{L}$	$\frac{3}{(4-\alpha)}\frac{I}{L}$
	$\frac{3EI}{L}$	0	$\frac{3}{4}\frac{I}{L}$	0
	$\frac{4EI}{L}$	$\frac{4EI}{L}$	$\frac{I}{L}$	$\frac{I}{L}$
	$\frac{6EI}{L}\cdot\frac{\alpha}{2-\alpha}$	$\frac{6EI}{L}\cdot\frac{\alpha}{2-\alpha}$	$\frac{3I}{2L}\cdot\frac{\alpha}{2-\alpha}$	$\frac{3I}{2L}\cdot\frac{\alpha}{2-\alpha}$

Table 2 - Fixed end moments

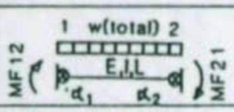
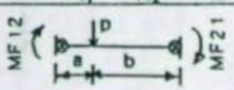
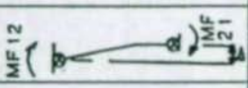
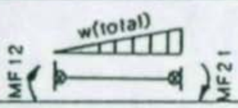
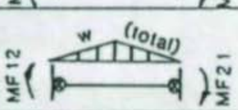
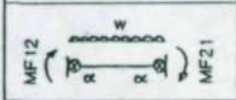
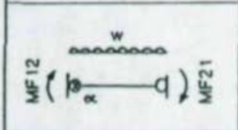
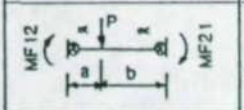
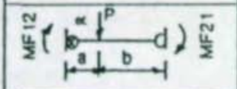
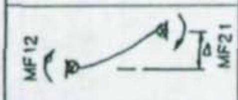
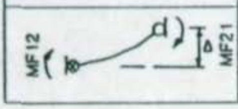
Condition	MF12	MF21
	$-\frac{WL}{12}\cdot\frac{3(2-\alpha_2)\alpha_1}{(4-\alpha_1\alpha_2)}$	$\frac{WL}{12}\cdot\frac{3(2-\alpha_1)\alpha_2}{(4-\alpha_1\alpha_2)}$
	$-\frac{Pab^2}{L^2}\cdot\alpha 1\left[\frac{2(2-a/L)-\alpha_2(1+a/L)}{(4-\alpha_1\alpha_2)b/L}\right]$	$\frac{Pa^2b}{L^2}\cdot\alpha 2\left[\frac{2(1+a/L)-\alpha_1(2-a/L)}{(4-\alpha_1a_2)a/L}\right]$
	$\frac{6EI}{L}\cdot\Delta\frac{(\alpha_2+2)\alpha 1}{(4-\alpha_1\alpha_2)}$	$\frac{6EI}{L}\cdot\Delta\frac{(\alpha_1+2)\alpha_2}{(4-\alpha_1\alpha_2)}$
	$-\frac{WL}{15}\cdot\frac{\alpha_1(7-4\alpha_2)}{(4-\alpha_1\alpha_2)}$	$\frac{WL}{10}\frac{\alpha_2(16-7\alpha_1)}{3(4-\alpha_1\alpha_2)}$
	$-\frac{5WL}{48}\cdot\frac{3(2-\alpha_2)\alpha_1}{(4-\alpha_1\alpha_2)}$	$\frac{5WL}{48}\frac{3(2-\alpha_1)\alpha_2}{(4-\alpha_1\alpha_2)}$

Table 3 - Fixed end moments specific cases

Condition	MF 12	MF 21
	$-\frac{WL}{12} \cdot \frac{3\alpha}{(2+\alpha)}$	$\frac{WL}{12} \cdot \frac{3\alpha}{(2+\alpha)}$
	$-\frac{WL}{8} \cdot \alpha$	0
	$-\frac{Pab^2}{L^2}\alpha\left[\frac{\left(2\left(2-\frac{a}{L}\right)-\alpha\left(1+\frac{a}{L}\right)\right)}{(4-\alpha^2)b/L}\right]$	$\frac{Pa^2b}{L^2}\alpha\left[\frac{\left(2\left(1+\frac{a}{L}\right)-\alpha\left(2-\frac{a}{L}\right)\right)}{(4-\alpha^2)a/L}\right]$
	$-\frac{Pab^2}{L^2} \cdot \frac{\alpha\left(2-\frac{a}{L}\right)}{2b/L}$	0
	$\frac{6EI}{L} \cdot \Delta \frac{\alpha}{(2-\alpha)}$	$\frac{6EI}{L} \cdot \Delta \frac{\alpha}{(2-\alpha)}$
	$\frac{3EI}{L} \cdot \Delta \cdot \alpha$	0

NOTE:

$\alpha = 1 / \left(1 + \frac{3EI}{LS_j}\right)$

S_j = *spring stiffness of connection*

L, E, I = relative to connected beam (or column)

THE EFFECT OF CONNECTION FLEXIBILITY ON PORTAL FRAME BEHAVIOUR

Timothy C H Liu[1]
Linden J Morris[2]

Abstract

A recent research project investigated the design and behaviour of portal (gable) frames. The project involved the testing of full-scale frames and the development of a sophisticated finite element program. This program was able to simulate the premature failure that had occurred in one of the test frames This ability to predict the structural response of a portal frame, including modes of failure, has allowed a number of supplementary studies to be made. One of these studies examined the effect of different stiffening arrangements in the column web on the eaves connection flexibility and the overall performance of portal frames. It is demonstrated by computer simulations that the application of current design guidance would have prevented premature failure.

1. INTRODUCTION

The only major application of simple plastic method to the design of steel structures has been to portal frame construction. To the practising engineer, the advantages of plastic design over elastic methods was that it provided a direct method for proportioning the member sizes and there was generally an immediate saving in material cost. However, in order to minimise transportation costs, particularly with respect to shipping volume for overseas projects, the site connections had to be deliberately located at member intersections (at the eaves and apex), which coincided with the positions of maximum moment. Also, to eliminate any connector plates from projecting beyond the member, end-plate connections were used. Initially, in order for the end-plate connections to achieve their design moment capacities, short haunches were introduced. These virtually doubled the depth of the connections, thereby increasing the lever arms for the tension bolts and hence improving the moment capacity of the connection. The portal frame fabricators soon realised that, if the length of the eaves haunches was increased, this resulted in the rafter sizes being reduced at the expense of larger column sections and achieved a more cost-effective structure. This meant that the

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eaves connections had to resist even larger moments than would have been the case of a frame with a uniform member size.

Basically, the connections at both the eaves and apex positions are required to effect continuity between the connected members, i.e. column/haunched rafter, or rafter/rafter, whilst transferring large moments and relatively small vertical shear loads. This continuity between members is essential, if the portal frame is to develop the plastic collapse mechanism, for which the frame would have been designed, i.e. the portal is designed as a rigid frame, with the plastic hinges assumed to form away from the connections. Therefore, though those parts of the members local to the connections are basically elastic, some plasticity will occur in the connected members because of local high stresses, but not sufficient to cause undue distress if current design guidance is employed. A design method for end-plate connections was proposed by the senior author over a decade ago (Horne and Morris 1981) and is now used extensively throughout the construction industry, particularly for portal frame connections (Morris and Plum 1988). It has to be emphasised that this limit state method allows the engineer to proportion the connection details, so that the required moment capacity can be achieved without distress. And as such, the method should not be regarded as a realistic *behavioural* model - that is not its function.

In portal frame construction, the strength and stiffness of the frame is not enhanced by the effect of concrete floors or walls present in multi-storey construction, though the thin gauge cladding affords some stressed-skin action against sway deflections. That is, this construction is one of a few examples of a pure structure. Consequentally, if there is any weakness in the design of a portal frame, then premature failure will occur, due to this inherent lack of reserve of strength/stiffness. It is essential that the practising engineer is aware of the influence of design decisions can have on the performance of the portal frame, e.g. effect of ignoring well established design guidance.

Apart from member stability, a major consideration is the design of moment connections. The practising engineer has never been concerned about the effect of connection flexibility on the performance of the portal frame, and the question that needs to be answered - should he? Therefore, this paper will concentrate on the effect of stiffening of an external column member on the flexibility of the eaves connection and then its global significance on the overall frame behaviour. Also, as virtually all portal frames in the United Kingdom are designed assuming a base fixity of zero, then actual moment capacity of these nominal pinned bases will also examined in terms of frame behaviour.

2. REALISTIC MODELLING OF STRUCTURAL BEHAVIOUR

2.1 General

There is need to examine the significance of connection flexibility both in local and global contexts. In the past, most research has been undertaken on the localised behaviour of connections, though more recently (under the guise of semi-rigid design) the effect of connection flexibility on the structural response of a structure is now earnestly being investigated. There are two main ways of modelling structural behaviour, either by testing (physical modelling) or by simulation using mathematical methods (theoretical modelling). First, the recent testing of a portal frame afforded an opportunity of comparing the experimental evidence of failure within an eaves connection with finite element modelling. Second, a study was made of the structural effect of different stiffening arrangements for the columm member on the flexibility of eaves connections and overall behaviour of a typical frame, as used in the UK construction industry.

2.2 Physical Modelling

The connection and frame details used in this particular study are based on those of the test frame described by Davies et al, 1990, and are reproduced in Fig 1(a); note the use of backing plates.

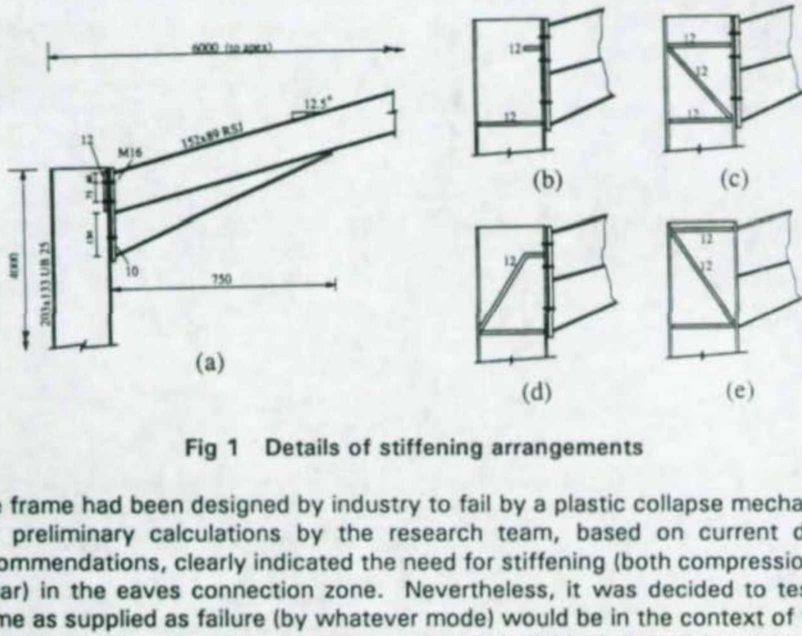

Fig 1 Details of stiffening arrangements

The frame had been designed by industry to fail by a plastic collapse mechanism, but preliminary calculations by the research team, based on current design recommendations, clearly indicated the need for stiffening (both compression and shear) in the eaves connection zone. Nevertheless, it was decided to test the frame as supplied as failure (by whatever mode) would be in the context of frame behaviour, which is certainly not the case with individual connection tests. The frame was loaded in a three-dimensional test assembly, and the predictable premature failure occurred in the column head region. That is, the column web started to yield in the compression zone, with the web and flange eventually buckling, whilst the central portion of the column web yielded under the action of the shearing action induced by the large applied moment, thereby allowing a shear hinge to develop. A secondary mode of failure occurred, i.e. lateral-torsional buckling of the column member, as a result of the primary failure modes. Some of the resulting test evidence is reproduced in Fig 2(a) (photograph of failed column head) and the applied load-vertical apex deflection of the test frame in Fig 3 (curve test); the actual rotation capacity for the eaves connection zone was not measured for this test frame.

2.3 *Theoretical Modelling*

The theoretical simulations detailed in this paper have been produced by finite element modelling techniques. This became possible, due to the successful development at Manchester University of a powerful, sophisticated finite element program, (Liu, 1988), partly in conjunction with a major research project investigating the design and behaviour of portal frames (Davies et al, 1990), and partly through subsequent improvements. This program allows the elastic and inelastic structural responses of a portal frame to be predicted up to failure and beyond, and has the capability of simulating any mode of structural failure. The simulations are undertaken in the context of modelling a portal frame (including the bolted connections) as part of a building, with the gravity loading being applied via the purlins and not in-plane of the portal frame, unlike other research work on pane frame behaviour. The initial verification of the model was established by comparison of the predicted theoretical behaviour with published experimental evidence; differences of only a few percent were obtained. Confidence in the theoretical model was further justified by the excellent simulations of the behaviour of three full-scale frames which had been tested to failure, one of which is described in detail in reference (Davies et al, 1990).

Each frame was discretised using iso-parametric elements, and in those regions where potential modes of failure were deemed to occur, then the mesh was refined locally. The physical dimensions, the measured geometric and material imperfections of the test frames were modelled, including the "nominal" pinned bases and holding down bolts. The iterative analysis program was continued until a stage had been reached, at which it was clear that the frame had started to unload. Examination of the computer results for the first frame (12m span) showed that the predicted modes of failure were identical to those experienced by the test frames, see section 2.2.

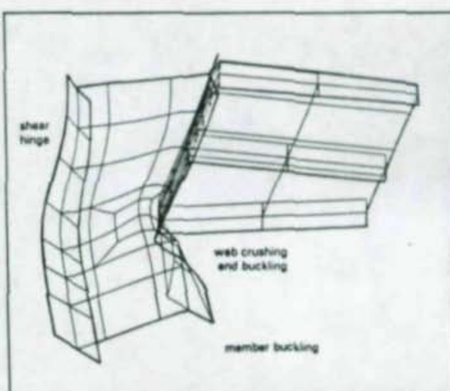

Fig 2 Experimental and simulated modes of failure of test frame

The simulated distortions of the column head (including the eaves connection) are reproduced in Fig 2(b) and clearly demonstrate the remarkable similarity between the failure modes of the physical (Fig 2(a)) and simulated models. The theoretical characteristic for the vertical apex deflection is plotted in Fig 3 (curve **A**) and shows good correlation with the measured test evidence; note that the *same curve* **A** *is reproduced in Fig 5.* The theoretical load-rotation curve is given in Fig 4 (curve **A**). As the result of the good correlation between the overall physical and simulated behaviour for this connection, this rotation curve (A) has been taken as being representative of the non-existent test data, and is used as the datum for comparisons.

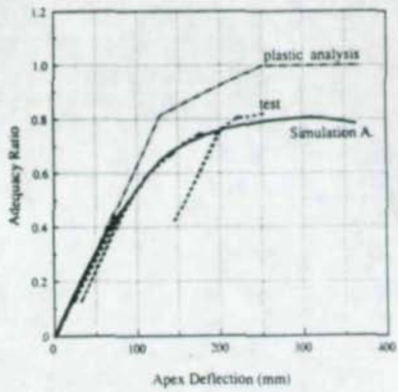

Fig 3 Test frame deflections

3. DIFFERENT COLUMN WEB STIFFENING

3.1 Details of stiffening arrangements

The ability to reproduce the physical behaviour of a portal frame and predict its mode of failure by means of computer modelling, allowed the authors to assess the influence of stiffening on the behaviour of the column member and the corresponding flexibility of the eaves connection, in the context of overall performance of the portal frame. Therefore, the details of the test connection zone, as given in Fig 1(a), were modified to effect different degrees of column web stiffening; the details of the stiffening arrangements are,

B. as test, plus compression and tension stiffeners (horizontal), Fig 1(b);
C. as test, plus compression and diagonal shear stiffeners, Fig 1(c);
D. as test, plus Morris shear stiffeners, Fig 1(d);
E. like Fig 1(c), haunched rafter welded direct to column member, Fig 1(e).

3.2 Theoretical simulation

Computer analyses were undertaken in order to compare different types of column web stiffening (B - E). *The resulting simulated structural responses of each* stiffened eaves connection are shown in Fig 4 (frame capacity-connection rotation) and Fig 5 (frame capacity-apex deflection). Both sets of curves allow direct comparison of the different arrangements, from which the influence of an individual stiffening system on the connection flexibility and frame performance can be deduced. Also, from the analyses, the areas of plasticity that had occurred within the connection at maximum load level, can readily be assessed; Fig 6 shows the theoretical yielded zones for the connections **A** and **D**.

The proportions of the eaves connections and the range of universal beam sections employed in portal frame construction, usually means that horizontal web stiffeners are necessary for the column, opposite to the inclined compression flange of the haunch. The addition of these web stiffeners to the original model, see Fig 1, prevented web crushing and flange distortion in the compression zone. However, as would be expected, these stiffeners had no significant influence on the shear yielding in the column web, with connection **B** failing from shear web buckling.

With reference to the stiffening used in connection **C**, (Fig 1(c)), some designers advocate that the diagonal shear stiffener should be in compression (as shown) rather than tension (cf Fig 1(d), but as the simulation for **C** revealed, the diagonal stiffener eventually failed by buckling along its free outstand edge, even though the stiffener plate was thicker than would have normally been used in practice. This meant for this particular portal frame that the design capacity of the frame was not achieved. The effect of including the Morris stiffeners **D** was to enhance the moment capacity of the connection and delayed the onset of significant shear deformation. Also, the column flange deformation in the tension zone was reduced due to the restraint from the horizontal projection of the Morris stiffeners. It was also decided to compare the rigidity of a fully fillet welded connection (end-plate not included) with that of the equivalent bolted connection (**C**). There was no marked difference, the former also failed by the buckling of the diagonal stiffener.

4. CONNECTION FLEXIBILITY AND FRAME BEHAVIOUR

Fig 4 shows the effect of the different stiffening systems on the connection flexibility, as indicated by the amount of rotation that takes place in the connection zone. The change in rotation for each connection has been defined as that which occurred between the horizontal plane corresponding to the position of the horizontal web stiffeners and a vertical plane located D/2 from, and parallel to, the end-plate. These theoretical rotations include, therefore, the individual components of rotation arising from shear action, column flange and end-plate distortions, bolt extension and yielding in haunched rafter web.

The simulated model of the test frame (curve **A**) shows that limited connection rotation occurred, due to the lack of adequate stiffening in the column head, caused by the multi-mode failure of the connection. There was a relative rapid unloading, which resulted in a loss of about 20% in load carrying capacity of the frame. The simple addition of the compression stiffeners improved the rotation characteristic of the connection **B**, partly due to the utilisation of the post-shear yield strength of the column web. This allowed the frame to achieve a higher load capacity, though still

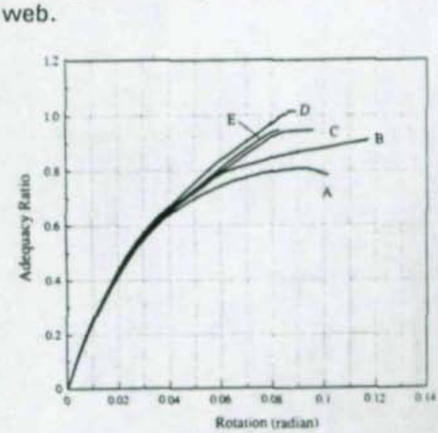

Fig 4 Simulated connection rotations

falling short of its design capacity by about 10%. The stiffening arrangements **C**, **D** and **E** provided the necessary shear yield resistance, thereby delaying the formation of a shear hinge. However, it can be seen from Fig 4, that the frames with the stiffening arrangements **C** and **E** failed to achieve the design adequacy level by some 5%. As already stated, examination of these simulated connections revealed signs of buckling in the diagonal compression stiffeners. Also, it is interesting to note that there is virtually no difference in behaviour between these two arrangements, despite that the connection (**C**) is bolted using M16, grade 8.8 bolts and the connection (**E**) is welded. Only the stiffening arrangement of connection **D** allowed the frame to achieve its design capacity following the development of a plastic collapse mechanism. Also, the fabrication cost of this connection is lower compared with that of either connection **C** or **E**.

The vertical apex deflection is sensitive to the frame stiffness and the formation of plastic hinges, and is a useful indicator of the frame's performance when subject to increasing load. Therefore, the plots of theoretical load-apex deflection curves for all the simulated connections (A-E), given in Fig 5, are indicative of the effect of the connection flexibility on frame behaviour. In addition, the results of a simple elastic-plastic analysis of the portal frame are shown, which indicates the history of plastic hinge formation, leading to the collapse mechanism. This analysis takes into account the partial fixity (15%) of the "nominal" pinned bases, as measured during the frame test, and also predicted by the finite element model for the frame. The latter simulation revealed that the magnitude of the base fixity at the steel/concrete connection was sensitive to the baseplate thickness, i.e. an increase of 1mm in thickness resulted in almost 60% increase in fixity, with the consequental effect on the frame stiffness and behaviour. This fixity was also shown to be sensitive to the size of the holding bolts.

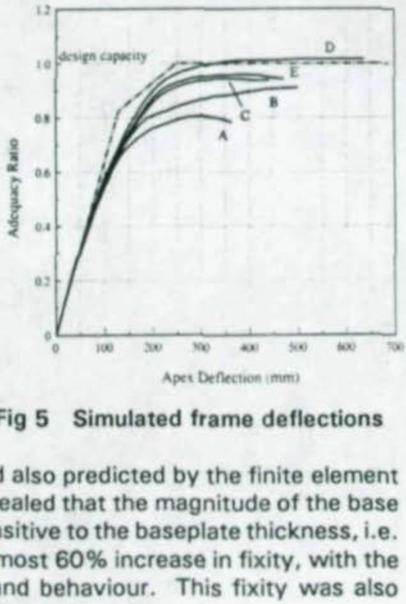

Fig 5 Simulated frame deflections

The load-deflection characteristics in Fig 5 show a similar pattern of behaviour to that of the connection rotations, see Fig 4. Inadequate stiffening of the eaves connection clearly produces a shortfall in the design capacity of the frame.

5. YIELDED ZONES IN CONNECTION

The frame had been pre-coated with a brittle lacquer, which cracked when subject to plastic strains; the photograph in Fig 1(a) depicts the various areas where yielding had occurred in the connection region at the point when the test was terminated. The shear yielding in the column web is clearly visible, as well as the buckled web/flange in the compression region, on the level with the inclined haunch flange. The advantage of theoretical simulations is that the yielded zones can readily be defined. To illustrate this capability, the yielded zones for the eaves connections **A** and **D** are given in Fig 6.

Inspection of connection **A** shows the complete yielding of the column web within the connection zone, which mirrors the test evidence, see Fig 1(a). The plastic tensile strains in the outer column flange extend above the corresponding position of the horizontal web stiffeners in connection **D**. In the latter connection zone, the shear yield is contained within the web bounded by the stiffeners and the inner column flange. The yielded region above the short horizontal projection of the Morris stiffeners arose due to the high tension loading from the bolts in that region.

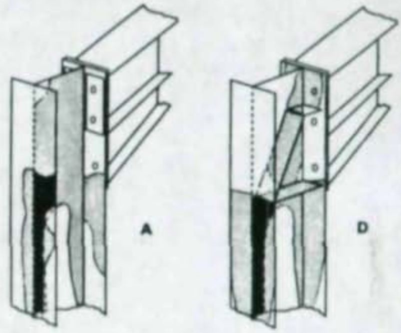

Fig 6 Yielded zones in connection

The diagonal stiffeners channel the tension down to the outer flange; note that the plastic tensile strains in this region do not spread above the horizontal stiffeners. There is more plasticity in the column below the haunch for the frame **D**, as it is carrying 20% more load capacity.

6. CONCLUSIONS

In the context of eaves connections for portal frames, the conventional philosophy with respect to unrestrained connection rotation does not apply, as the frame design requires limited rotation at the eaves connection. This is necessary, in order that the assumed plastic collapse mechanism can develop at, or above, the ultimate design capacity of the frame. In the context of plastic design, the requirement for adequate rotation is with respect to the designated hinge positions. Inadequate stiffening at eaves connections can result in a reduction in the load carrying capacity of a portal frame.

7. REFERENCES

Davies, J M, Engel, P, Liu, T C H and Morris, L J (1990), *Realistic Modelling of Portal Frame Behaviour*. The Structural Engineer, London, Vol 68, No 1, pp 1-6.

Horne, M R and Morris, L J (1981), *Plastic Design of Low Rise Frames*, Constrado Monograph, London, re-issued by Collins in 1985.

Liu, T C H (1988), *Theoretical Modelling of Steel Portal Frame Behaviour*, PhD Thesis, University of Manchester, England.

Morris, L J and Plum, D R (1988), *Structural Steelwork Design to BS5950*, Longman Technical, London.

ANALYSIS OF FRAMES FOR STABILITY

Richard Kohoutek[1]

Abstract

This paper is concerned with the global dynamic and static behaviour of steel structures considering semi-rigid connections. Due to the relationship between static stability and natural frequencies, the dynamic stability and static stability can be analysed using the deformation method. The same method can be used to determine a distribution of forces for both, the static and dynamic load.

1. INTRODUCTION

In Australia, the Australian Institute of Steel Construction produced "Standardized Structural Connections" which was first published in 1978, second edition 1981, and third edition 1985. The sustained interest in the connections was verified by the "Survey of Research Needs for Metal Structures" reported by (Pham and Mansell, 1985), which was conducted during 1983. The first two research topics in the Survey with the highest ratings were: total structure behaviour (67) and connection behaviour (49) out of the total 344. The third was service performance (36) and the fourth fatigue (28) followed by dynamic behaviour (21). The interest in semi-rigid connections is clearly shared by other countries as indicated by published papers and, notably, two recent monographs (Bjorhovde, 1988; Chen 1987).

There is an intrinsic relationship, defined by a partial differential equation for a bar (Kohoutek, 1990), between the static and dynamic behaviour of a bar, which allows findings of the dynamic behaviour to be utilized also in the static analysis. However, this presentation concentrates on the dynamic component of the system of ordinary differential equations replacing the partial differential equation (Kohoutek, 1990).

First, a simple structure, in an extreme limit of a fixed/fixed beam, the variation in a bending moment and an effective length due to changes in the performance of the connections will be shown. The results of static and dynamic analyses is given below. Using the same method, expansion to analyses of frames can be made readily.

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2. PROBLEM FORMULATION

2.1 Behaviour of Connection

In this workshop[1], the difference between a connection and a joint will be made. The joint comprises the detailing of an assembly, such as welds and bolts. The connection comprises some part of a beam and a column of an assembly[2].

Both static and dynamic analyses recognise some variation from complete rigidity, commonly expressed by a relationship between a moment M and a rotation of connection $\xi = \delta + \zeta$, where ζ is the rotation maintaining moment through a connection to other members (Kohoutek, 1985). The rotation δ is a free flexing or rotational slack which reduces moment stiffness of the connection.

2.2 Behaviour of Beam

The analysis follows in principle (Firt, 1974), which assumes a completely rigid "core" of a connection and a semi-rigid joint (a semi-rigid hinge) of a bar to the core. The whole semi-rigid performance is attributed to the connection, though it is clear that the highly stressed part of a beam is also contributing to the semi-rigidity of an assembly.

The dynamic deformation method, assuming completely rigid connections or hinges, (sometimes called the slope deflection method in its static version) used for analysis in this study has been described elsewhere (Kohoutek, 1985, 1985, Kolousek, 1973). The method is typically used for dynamic analysis of frames for classical resonance and forced vibrations.

The partial differential equation for a transverse vibration, with an axial force $N(t)$ is

$$EI \frac{\partial^4 w(x,t)}{\partial x^4} \pm N(t) \frac{\partial^2 w(x,t)}{\partial x^2} + m \frac{\partial^2 w(x,t)}{\partial t^2} = 0. \qquad (1)$$

Equation (1) can be replaced by a system of ordinary differential equations, identifiable with static stability, free vibrations and parametric resonance (Kohoutek, 1990). The complete solution of free vibration is

$$v(x) = C_1\exp(k_1x) + C_2\exp(k_2x) + C_3\exp(k_3x) + C_4\exp(k_4x) \qquad (2)$$

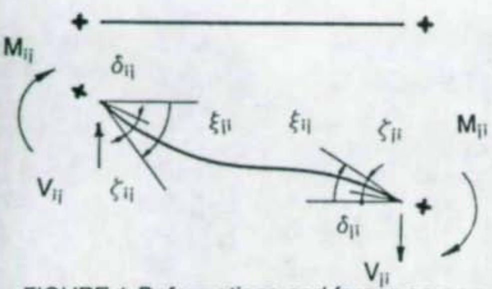
or,

$$v(x) = A\cos(\lambda x/l) + B\sin(\lambda x/l) + C\exp(-\lambda x/l) + D\exp(-\lambda(l-x)/l) \qquad (3)$$

where A,B,C,D are constants, and $\lambda = l\,(m\theta^2/EI)^{0.25}$, where m is mass per unit length; θ is excitation frequency, EI, flexural stiffness, and l length of a beam.

[1] AISC terminology is reversed (Australian Institute of Steel Construction)

[2] for further discussion see *Preamble* in *Dynamic Tests of Semi-Rigid Connections*, in this volume.



The homogeneous differential equation (1) is valid for a beam of constant mass m and constant flexural stiffness EI. Tha beam may have any kind of supports. The beam geometrical relationships are shown in Figure 1.

Systems composed of several beams will lead to evaluation of $4n$ constants, where n is the number of bars.

FIGURE 1 *Deformations and forces on a bar with semi-rigid connections.*

This is, in principle, the same as the deformation method (slope deflection) for static frame analysis. While in the static method the stiffness is a constant related to a support condition, span and EI, in a dynamic analysis the stiffness is also related to frequency and changes with a change in frequency.

The main advantage of this method is that it can provide all the information necessary for design purposes, whilst still being relatively comprehensive and easy to apply to both static and dynamic problems. Other advantages include the fact that (for dynamics) it uses a continuous model to describe the structure. Consequently, the dynamic modelling should be inherently better than the lumped mass approach, with an advantage of utilising the knowledge of slope deflection method in engineering.

2.3 Solution of Differential Equation

The complete solution of equation (1) for free vibration is (3). The boundary conditions applied for semi-rigid connection on each end of a beam, are $(x = 0,\ x = l)$, $v(0) = 0$ $v(l) = 0$:

$$-(1-\Gamma_{ij})v''(0) + \Gamma_{ij}\, v'(0) = 0 \quad (1-\Gamma_{ji})v''(l) + \Gamma_{ji}\, v'(l) = 0 \tag{4}$$

where the factor of rigidity Γ, $\Gamma_{ij} \neq \Gamma_{ji}$ in general, is zero for a hinge or a unit for complete fixity.

The nontrivial solutions in the form of (3) which satisfy the boundary conditions (4), lead to the expression for the eigenvalues (natural frequencies):

$$0 = 2\lambda^2 (EI\lambda\, (l\, (\Gamma_{ij}\, (-(2\,\Gamma_{ji} \sin\lambda + \cos\lambda) + 2\,\Gamma_{ji}\cos\lambda + \sin\lambda) + (-\Gamma_{ji} + \exp(-2\lambda)\, (\Gamma_{ji} + \Gamma_{ij}\, (1 - 2\,\Gamma_{ji}))) \cos\lambda) + (2\, (EI\lambda\, (1 - (\Gamma_{ij} + \Gamma_{ji}\, (1 + \exp(-\lambda)\, \Gamma_{ij}) + \exp(-2\lambda)) + \Gamma_{ij}\Gamma_{ji} + \exp(-2\lambda)\, (\Gamma_{ij} + \Gamma_{ji})) - \exp(-2\lambda)\, \Gamma_{ij}\, l\, \Gamma_{ji}) + \exp(-2\lambda)\, l\, (\Gamma_{ij} + \Gamma_{ji})) \sin\lambda) + l\,\Gamma_{ji}\, (\Gamma_{ij}\, l\, (-(1+\exp(-2\lambda)) \cos\lambda + 2\exp(\lambda)) + EI\lambda \sin\lambda))/l^4 \tag{5}$$

with all parameters defined. From Figure 1 the relationship between ξ_{ij} and ζ_{ij} , for rotations, is

$$\xi_{ij} = \delta_i + \zeta_{ij}, \text{ and } \quad \xi_{ji} = \delta_j + \zeta_{ji}, \tag{6}$$

and subsequently

$$M_{ij} = -\frac{EI}{l}\Gamma_{ij}\,\zeta_{ij}, \quad \text{and} \qquad M_{ji} = -\frac{EI}{l}\Gamma_{ji}\,\zeta_{ji} \tag{7}$$

Hence, from (6) and (7),

$$\xi_{ij} = \delta_i - \frac{l\,M_{ij}}{EI\,\Gamma_{ij}}, \quad \text{and} \qquad \xi_{ji} = \delta_j - \frac{l\,M_{ji}}{EI\,\Gamma_{ji}}. \tag{8}$$

where Γ_{ij} and Γ_{ji} are constants representing the semi-rigidity of a connection for the particular bar/connection assembly.

The common view, and the one adopted here, is that only the moment is primarily affected, while the shear force is carried through the connection. Equation (8) fully describes this class of a semi-rigid connection.

3. DYNAMIC ANALYSIS

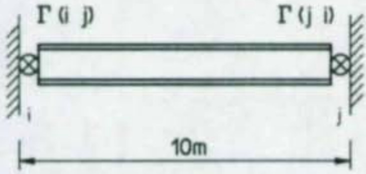


FIGURE 2. *Beam with semi-rigid connections.*

The structure can be modelled as a beam with two semi-rigid connections at the end supports *i* and *j*. The analytical model allows an arbitrary value for semi-rigidity to be entered where the coefficient of rigidity $\Gamma(i,j)$ is in the range of *0* for a hinge to *1.0* for a complete fixity.

If the frequency of the oscillation coincides with the natural frequency of a beam, resonance will occur and large amplitudes of vibrations can be expected on the real structure. Consequently, serviceability problems or, in time, fatigue can be expected because of the large forces generated by the beam's inertia. Therefore, the analysis of the beam to determine such frequency, which is related to the physical properties, is of interest.

Analysing the beam with the assumption of uniformly distributed mass and of flexural stiffness produces the results given in Table 1. Numerical solutions of (5) are in the first column for the natural frequency. As can be seen from Table 1, the connection parameter Γ makes a substantial variation of several design parameters, such as critical load due to the effective length and the first natural frequency.

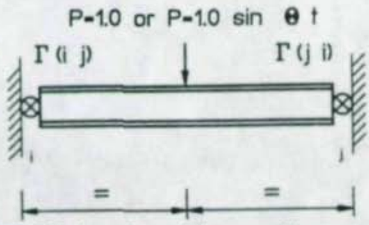
The second column of the first natural frequencies includes the effect of shear deformations and rotary inertia on the flexural deformations of the beam. Within each section, where the cross section parameter is constant, the type of connection at the ends makes some tuning of a desired property possible. A designer can influence the selection of suitable natural frequency, effective length, or critical load by choice of suitable connections at the ends together with other traditional design parameters such as the cross section and the material of the beam.

TABLE 1. *Variation of natural frequencies and effective lengths related to support conditions. The actual length for this calculation is assumed to be 10 m.*

Overall dimensions mm	Sectional properties	Natural frequency in Hz Inclusion of deformations No shear	With shear	Effective length m	Support end conditions Fixed = 1.0
400x640	$A = 74{,}000\ mm^2$	46.46	44.43	5.0	Fixed/fixed
	$I_x = 4946 \times 10^6 mm^4$	37.29	28.03	7.2	Γ=0.75 /0.75
581 kg/m	$r_x = 258.5\ mm$	28.89	23.64	8.5	Γ=0.5 /0.5
	$I_y = 748.6 \times 10^6 mm^4$	24.40	21.52	9.4	Γ=0.25 /0.25
	$r_y = 100.6\ mm$	20.47	20.26	10.0	Hinge/hinge

The static and dynamic characteristics Γ of some standard connections are being evaluated experimentally, with some results for the first batch of connections reported in this volume.

3.1 Static - Special Case of Dynamic



The example investigated in this section will use the sectional properties and span of the example above. The beam is loaded by a single point load as shown in Figure 3. All values are relative to the same beam with fixed/fixed ends for the static load $P = 1.0$ in Table 1.

FIGURE 3. *Loading of beam with semi-rigid connections.*

Because this analysis does not include damping, the ratios are only indicative of an increase in the bending moment, however, a damping can be included in this model.

The beam above will be analysed for static and dynamic loading of the same amplitude of the load $P = 1.0$, loaded around the *x-axis* of the cross section. The ratio of dynamic moment over static moment at the midspan for different frequencies and various support conditions is shown in Table 2.

The frequency of the load will gradually increase from $\theta = 10\ Hz$, which stands for a quasi-static load, to higher frequencies with large variation in the moment under the load. The quasi-static character is shown by virtually no change in the first column for *10* Hz in Table 2. When the first column is considered, the moment should increase from *1.00* to *2.00* between fixed/fixed beam and simply supported beam, or more precisely from *Pl/8* to *Pl/4*. The large numbers in some parts of Table 2 are due to the frequency of the load being close to the natural frequency of the beam, as can be verified by comparison with Table 1.

TABLE 2. *Variation of the dynamic moment / static moment under the load, dependant on the load frequency and support conditions. The actual length for this calculation is assumed to be 10 m. The first column for each frequency does NOT include the influence of shear deformations and rotary inertia, where the second column includes those influences. Factors are only indicative of an order of amplitude because no damping was considered.*

Load frequency										Support end conditions
10 Hz		20 Hz		30 Hz		40 Hz		50 Hz		Fixed = 1.0
1.04	1.04	1.16	1.17	1.49	1.57	2.98	3.94	4.05	2.25	Fixed/fixed
1.32	1.35	1.56	1.84	2.79	8.16	5.40	1.20	0.74	0.25	Γ=0.75 /0.75
1.65	1.69	2.36	3.11	152.	3.17	1.02	0.46	0.24	4.11	Γ=0.5 /0.5
2.04	2.08	4.51	6.64	2.27	1.44	0.42	0.26	4.25	3.76	Γ=0.25 /0.25
2.51	2.52	34.27	64.39	1.03	0.97	0.19	0.17	6.64	7.91	Hinge/hinge

3.2 Frame Analysis

The dynamic deformation method (Kohoutek, 1985,) can be used to assemble a stiffness matrix of a frame structure. The principal aim is the same as that of the slope deflection method, to achieve an equilibrium in each connection of a structure. However, the elements of a matrix are functions of load frequency. The stability of such a system (eigenvalues), natural frequencies for the load considered here, can be found by evaluating only properties of a stiffness matrix; it is independent of load. The structure considered is shown in Figure 4, where each semi-rigid connection can attain values from Γ = *0* (hinge) to Γ = *1.0* (fixed). The limit cases are a mechanism and a fully fixed frame. The fully fixed frame will be investigate first, by evaluating the frequencies which provide non-trivial solution of the matrix (eigenvalues). The plot, which contains roots and asymptote is shown in Figure 5.

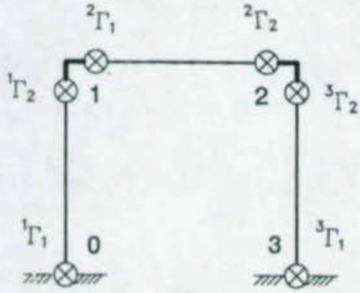


FIGURE 4. *A simple frame with semi-rigid connections*

A similar calculation was also performed for some variation in some semi-rigid connections, with results summarised in Table 3.

Comparing variations in the natural frequencies (stability problem) shows considerable move of frequencies. When one considers the results of the dynamic tests elsewhere in this volume, this could be used successfully as a design parameter to achieve a desired frequency, or by a modification of some connections on the existing frame to move a resonant frequency at will.

TABLE 3. *Variation of the first natural frequency (Hz), due to change in connections.*

Variation in connections of the beam $^2\Gamma_1$ and $^2\Gamma_2$				Columns $^1\Gamma_2$ $^3\Gamma_2$
1.0 1.0	0.75 0.75	0.5 0.5	0.25 0.25	Fixed = 1.0
46.07	45.37	40.06	34.80	Fixed/fixed
42.09	41.55	38.78	34.25	Γ=0.75 /0.75
38.62	38.28	-	33.83	Γ=0.5 /0.5
35.68	35.52	35.38	33.49	Γ=0.25 /0.25
33.22	33.22	33.22	33.22	Hinge/hinge

A similar process can be employed to evaluate static stability. The variation is in new formulae, which include the factors of semi-rigid connections. The resulting stiffness matrix is constant, as for standard rigid connections, but parameter dependent. As for rigid joints, the roots of this stiffness matrix are critical loads of the structure under investigation.

The third stabilty problem of the partial differential equation (1), parametric resonance, can be also solved (Kohoutek, 1986), including the influence of semi-rigid connections.

4. CONCLUSIONS

A method is presented for analyses of frames with semi-rigid connections. The dynamic deformation method used for the analysis is suitable for free or forced vibrations. The method incorporates all influences such as semi-rigidity, axial force, rotary inertia and shear force deformations into one expression with improved numerical stability desirable for computer modelling.

The boundary conditions play an important role in all dynamic analyses. The correct estimation of the boundary conditions significantly influences the determinations of the static critical load (Segal and Barnuch 1980, Sweet et al 1977), natural frequencies (Firt 1974, Kohoutek 1985), and dynamic stability (Kohoutek 1984).

Generalized boundary conditions, such as those applying at a connection in the frame, will have a major influence on *all the critical design parameters* above. The derivation of modified frequency functions for linear semi-rigid connections permits the dynamic deformation method to be used in the investigation of dynamic stability problems, as well as in static stability and natural frequencies, which are only special cases of dynamic stability.

5. ACKNOWLEDGEMENT

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6. REFERENCES

1. Ackroyd, M. H. (1987). "Simplified Frame Design of Type PR Construction," Amer.ISC, Eng. J., Vol. 24, No. 4, pp 141-147.
2. Bjorhovde, R. et al. (1988). "Connections in Steel Structures," Elsevier Science Publishers.
3. Chen, W.F. (1987). "Joint Flexibility in Steel Frames," Elsevier Science Publishers.
4. Clough, R.W. and Penzien, J. (1975). "Dynamics of Structures," International Students Edition, McGraw-Hill, Kosaido, Japan.
5. Ellingwood, B. (1989). "Serviceability Guidelines for Steel Structures," Amer.ISC, Eng. J., Vol. 26, No. 1, pp 1-8.
6. Firt, V. (1974). "Stabilita a kmitani konstrukci s netuhymi spoji," (Stability and Vibrations of Structures with Semi-Rigid Joints). Academia-Praha.
7. Gorenc, B.E. (1974). "Design of Compression Members," Aust.ISC, Steel construction, Vol. 8 No. 1.
8. Hogan, T.J., Firkins, A., Thomas, I.R. (1985). "Standardized Structural Connection," Aust.ISC, (first edition).
9. Hogan, T.J., Firkins, A. (1983). "Standardized Structural Connections-five years on," Aust.ISC, Steel construction, Vol. 17 No. 2.
10. Hogan, T.J., Firkins, A., Thomas, I.R. (1985). "Standardized Structural Connection," Aust.ISC, (third edition).
11. Howlett, J.H. et al. (1981). "Joints in Structural Steelwork," Proceedings of the International Conference held at Teesside Polytechnic, Cleveland 6-9 April 1981. Pentech Press.
12. Jones, S.W., Kirby, P.A., Nethercot, D.A. (1983). "The Analysis of Frames with Semi-Rigid Connections - A State-of-the-Art Report," Journal of Constructional Steel Research, Vol. 3, No. 2, (Static).
13. Kohoutek, R. (1984). "The Dynamic Instability of the Bar under Tensile Periodic Force," Transactions of Canadian Society for Mechanical Engineers, Vol.8, pp 1-5.
14. Kohoutek, R. (1985). "Dynamic Analyses of Frames with Semi-Rigid Joints," I.E. Aust. Proceedings of Metal Structures Conference, held at Monash University, Melbourne, 23-24 May, L. Pham, ed., pp 7-10.
15. Kohoutek, R. (1985). "Analysis of Beams and Frames," Chapter 4 in "Analysis and Design of Foundations for Vibrations," pp 99-156; P. Moore, ed., 512pp, 1985, published by A.A.Balkema.
16. Kohoutek, R. (1986). "Classical and Parametric Resonance of a Frame - Case Study," Proceedings of The Tenth Australasian Conference on Mechanics of Structures and Materials, held at Adelaide University, 20-22 August, G. Sved, ed., pp 229-232.
17. *Kohoutek, R. (1988). "Serviceability Performance of Steel Joints,"* Aust.ISC, Steel Construction, Vol. 22, No. 3.

18.Kohoutek, R. (1990). "Static and Dynamic Stabilities for Structures with Semi-Rigid Joints," Proceedings of The Twelfth Australasian Conference on Mechanics of Structures and Materials, held at Queensland University of Technology, 24-26 September, G.H.Brameld, ed., pp 191-196.

19.Kolousek, V. (1973). "Dynamics in Engineering Structures," Praha-Academia, London-Butterworth.

20.Lay, M.G., (1973). "Effective Length of Crane Columns," Aust.ISC, Steel construction, Vol. 7 No. 2.

21.Leon, R. T. et al. (1987). "Semi-Rigid Composite Steel Frames," Amer.ISC, Eng. J., Vol. 24 No. 4, pp 147-155.

22.Pham, L., Mansell, D. S.(1985). "Survey of Research Needs for Metal Structures," I.E. Aust. Proceedings of Metal Structures Conference, held at Monash University, Melbourne, 23-24 May, L. Pham, ed., pp 172-177.

23.Segal, A. and Barnuch, M. (1980). "A Nondestructive Dynamic Method for the Determination of the Critical Load of Elastic Columns," Experimental Mechanics, pp 285-288.

24.Sweet, A.L., Genin, J. and Mlakar, P.F. (1977). "Determination of Column-buckling Criteria Using Vibratory Data," Experimental Mechanics, pp 385-391.

An Inelastic Analysis and Design System for Steel Frames with Partially Restrained Connections

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ABSTRACT

This paper describes a method for modeling semi-rigid beam-column connections in the design of steel framed structures. The proposed method has been implemented in a workstation-based computer program, CU-STAND, which has been developed in related research at Cornell University on inelastic analysis and design of two- and three-dimensional steel structures. An example is presented to investigate the sensitivity of limit state behavior in a low rise frame with PR connections. Included is discussion of two connection classification schemes recently proposed for steel structures.

BACKGROUND

While the inelastic behavior of structures with semi-rigid connections has long been recognized by engineers, convenient methods for nonlinear analysis and design which find widespread use have yet to be developed. The result has been reluctance by engineers to design structures with partially restrained (semi-rigid or Type III) connections. Evidence that this is changing is the development of design specifications with greater emphasis on connection effects and inelastic design methods. For example, detailed provisions for connection design and modeling in the latest draft version of Eurocode No. 3: Design of Steel Structures (ECCS 1990), represent perhaps the most comprehensive effort yet to address connection behavior in a design specification. Included in EC3 are specific quantitative guidelines for classifying connections as rigid or semi-rigid based on the moment-rotation response of the connections.

The success of efforts to include connection effects in design will depend in large part on the development of reliable and practical computer-aided analysis methods for practicing engineers. This paper is a report of one effort to address this need.

CONNECTION MODEL

In this work, the inelastic connection behavior is modeled through zero-length rotational springs to account for nonlinear moment-rotation behavior of the connection about the major- and minor-bending axis of the connected member. As described below, for design purposes the model is calibrated to existing experimental data.

Moment-Rotation. The nonlinear moment-rotation (M-θ) response is given by the following equation which is based on the previous work by Richard and Abbott (1975) and Chen and Kishi (1989):

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$$M = \frac{(K_e - K_p)\theta}{\left[1 + \left[\frac{(K_e - K_p)\theta}{M_o}\right]^n\right]^{1/n}} + K_p\theta \qquad (1)$$

The parameters, K_e, K_p, and M_o, are related to the behavior as shown in Fig. 1, and n controls the transition between K_e and K_p. To allow for unloading and moderate load reversals associated with nonproportional loading and inelastic force redistribution, the following extension to Eq. 1 was proposed by Hsieh (1990):

$$M = M_a - \frac{(K_e - K_p)(\theta_a - \theta)}{\left[1 + \frac{(K_e - K_p)(\theta_a - \theta)}{2M_o}^n\right]^{1/n}} - K_p(\theta_a - \theta) \qquad (2)$$

Calibration to Existing Data. For use in design, standardized values of the parameters in Eqs. 1 and 2 were determined. Further explanation of the procedure used is given by Hsieh (1990) and Shen (1990); here, in Figs. 2a-c, the process is illustrated for the case of top- and seat-angle with double web angle (TSAW) connections. First, Eq. 1 is curve-fit to experimental data and a comparison of typical results is shown in Fig. 2a. The experimental values used in this work are taken from data previously collected by Goverdhan (1983) and assembled in a computerized database by Kishi and Chen (1986). Next, the curves from Fig. 2a are normalized by the value of moment at 0.02 radian, resulting in the set of curves shown in Fig. 2b. The choice of moment to normalize the data is somewhat arbitrary, but was chosen on the basis that: (1) it resulted in a convenient consolidation of curves as shown in Fig. 2b, and (2) for many connection types the moment at 0.02 radian corresponds fairly well to the nominal connection strength. Finally, using the set of curves from Fig. 2b., mean and upper/lower bound curves were determined and are shown in Fig. 2c. The upper and lower bound curves represent a deviation from the mean curve of two standard deviations which statistically encompasses roughly 95% of the expected responses. The parameters for the normalized curves in Fig. 2c are given in Table 1. The same procedure was followed for several other connection types (e.g. end plate, top- and seat-angle, etc.) and based on these an additional curve representing the average of all types was determined. This is termed the AVERAGE curve. Parameters for this curve are also given in Table 1.

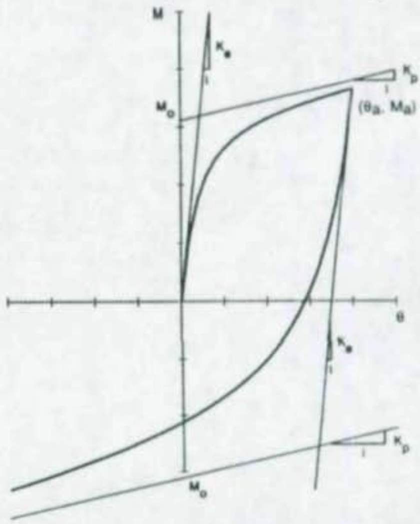

Figure 1. Moment-Rotation Model for Connections.

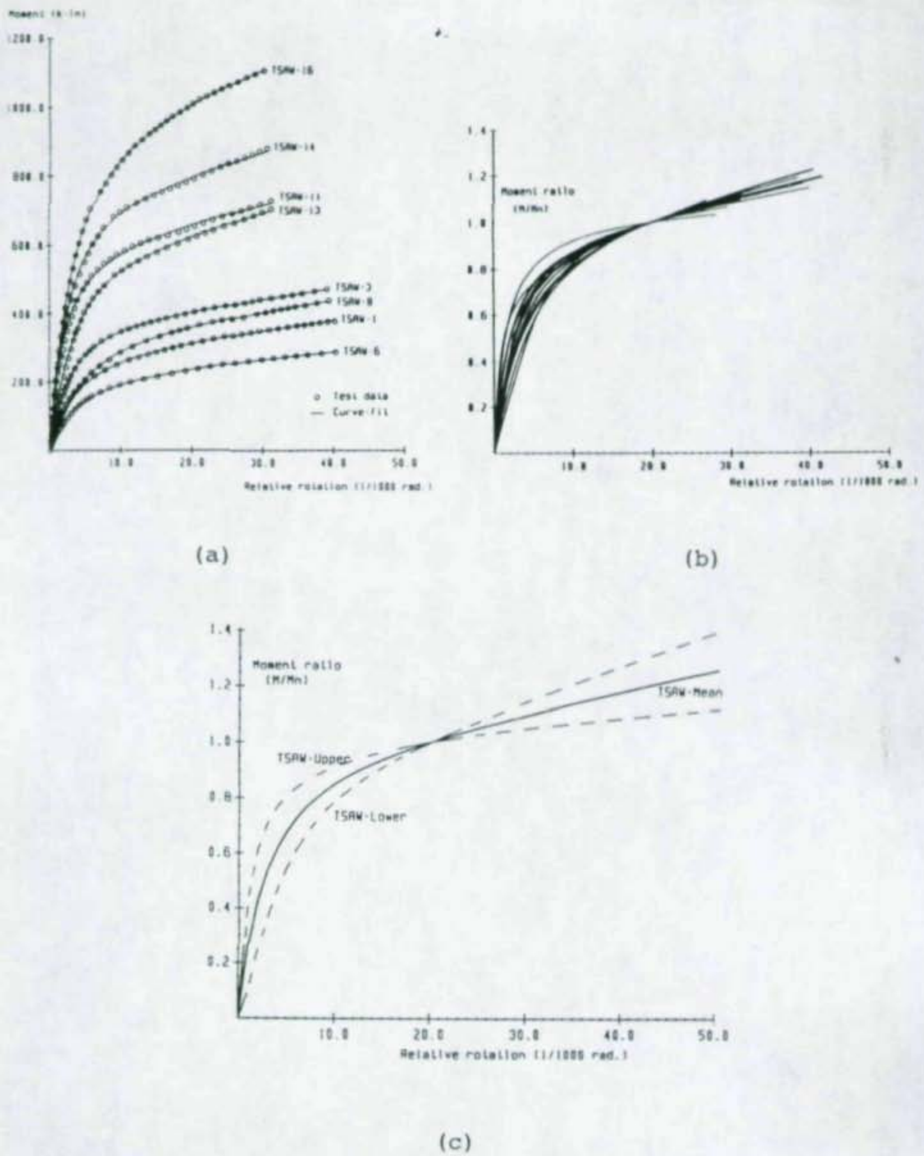

Figure 2. Calibration of Model Parameters for TSAW Connections.
(a) Curve Fit to Experimental Data, (b) Normalization by M_{cn},
(c) Mean, Upper, and Lower Bound Curves.

Table 1. Normalized Connection Response Parameters

Curves		K_e'	K_p'	M_o'	n
TSAW	Upper	436	4.1	.93	1.6
	Mean	266	7.6	.90	1.4
	Lower	132	12.0	.80	2.0
AVERAGE		200	4.0	1.0	1.4

COMPUTER IMPLEMENTATION

The connection model was implemented in an existing program, CU-STAND, for the inelastic analysis and design of two- and three-dimensional steel structures which includes the ability to accurately model the geometric and material nonlinear response of steel framed structures. Geometric nonlinear behavior is included through an updated Lagrangian procedure including element geometric stiffness matrices. Material nonlinearities are handled through a concentrated plasticity yield surface to model the elastic-perfectly plastic response of cross sections under axial loads and biaxial bending; this is in addition to the inelastic connection response. Further details regarding CU-STAND and the connection implementation are described by Hsieh, et.al. (1989, 1990).

EXAMPLE CASE STUDY

The two story frame shown in Fig. 3 was design using the AISC-LRFD Specification (1986) for the loads shown. Member forces were calculated using a 2nd-order analysis which took direct account of nonlinear geometric and connection response. All members were assumed to be braced against out-of-plane displacements.

The beam-column connections were modeled as TSAW connections where the connection behavior was defined by the AVERAGE curve parameters listed previously in Table 1. Note that the AVERAGE curve is in fact very similar to the mean curve for TSAW connections (Fig. 2c). The connection strength, M_{cn}, which is used to scale the normalized moment-rotation response in the analysis was taken as 0.4 times the plastic moment of the connected beams, M_p.

To investigate the limit state response and sensitivity to the connection model parameters, several inelastic analyses were conducted. Except where noted, proportional loading was used and the inelastic limit points were detected within CU-STAND by formation of

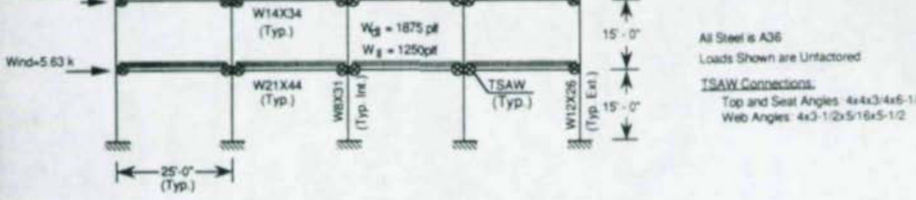

Figure 3. Two Story Planar Frame.

negative terms on the diagonal of the reduced global stiffness matrix. For assessing limit state behavior, the plastic member cross-section strengths were factored by the AISC-LRFD resistance factors for tension, compression, and bending (ϕ=0.9, 0.85, and 0.9, respectively). This results in a yield surface based on the "design" rather than "nominal" member capacities.

Inelastic Response. The inelastic strength limit state was investigated for factored loads under combined gravity loads ($1.2D + 1.6L_{floor} + 0.5L_{roof}$) and combined gravity and wind loads ($1.2D + 0.5L + 1.3W$). The moment diagrams and hinge locations for the two loading cases are shown in Figs. 4a&b. Under gravity loads the first hinges formed in the beams at loads between 1.05 and 1.07 of the full factored load. The inelastic limit point was reached at an applied load ratio of 1.33 times the factored load. The wind load combination did not control the strength design, and as indicated in Fig. 4b, the first hinge formed at 1.49 times the factored load and the limit point was reached at a load ratio of 1.64.

Connection Classification and Rotations. The peak rotations under the two loading combinations are shown in Fig. 5. In this figure, the connection moment and rotation is normalized following the convention used in EC3. Superimposed on the moment-rotation curve in Fig. 5 are lines corresponding to connection classification guidelines proposed in EC3 and by Bjorhovde et.al. (1990) for unbraced frames. The region to the left of each line corresponds to connections classified as rigid and to the right as semi-rigid. It is worth noting that the EC3 classification for "rigid" connections seems quite restrictive and as reported by Anderson (1990) excludes most details which are not fully welded moment connections with column stiffeners. The classification proposed by Bjorhovde is based on a value of θ_p using a fixed beam length related to the section depth (i.e., $\theta_p = [5d]M_p/EI$). In Fig. 5, the Bjorhovde values are plotted based on the span/depth ratio (=14.3) of the floor beam. As evident from Fig. 5, the connections in this example are clearly semi-rigid according to either Eurocode 3 or Bjorhovde.

Peak connection rotations under gravity and gravity plus wind loading are indicated in Fig. 5 and are compared to the minimum limit for required connection ductility proposed by Bjorhovde et. al. (shown by the descending dashed line). Rotations are shown for service loads ($1.0D + 1.0L$ and $1.0D + 0.2L + 1.0W$) and for full factored and inelastic limit point loads. Under service and full factored loading and prior to any hinge formations, the maximum connection rotations are relatively modest ($\bar{\theta} \leq 0.22$; $\theta \leq .009$ radian). At the limit point under gravity loading, the calculated peak connection rotations exceeded 0.100 radian which are far in excess of the minimum ductility proposed by Bjorhovde and the limits of most reported test data. One way to account for this in the analysis is to simply limit the connection rotations to a specified value based on the minimum ductility. For example, the applied load ratio corresponding to the point where peak connection rotations exceeded the minimum ductility specified by Bjorhovde ($\theta \leq 0.030$ radian for this frame) was 1.18 which is 11% less than the inelastic limit point of 1.33. Under the wind load combination the peak rotations at the limit point were roughly 0.040 radian. In this case, the limit point would only be reduced about 1% (from 1.64 to 1.62) to meet the ductility requirement.

Sensitivity Analyses. Results of several analyses to investigate sensitivity of the response to assumed connection parameters are summarized in Tables 2 and 3. Case 1 is the basic case reported above where the AVERAGE connection curve is used and M_{cn} = 0.4

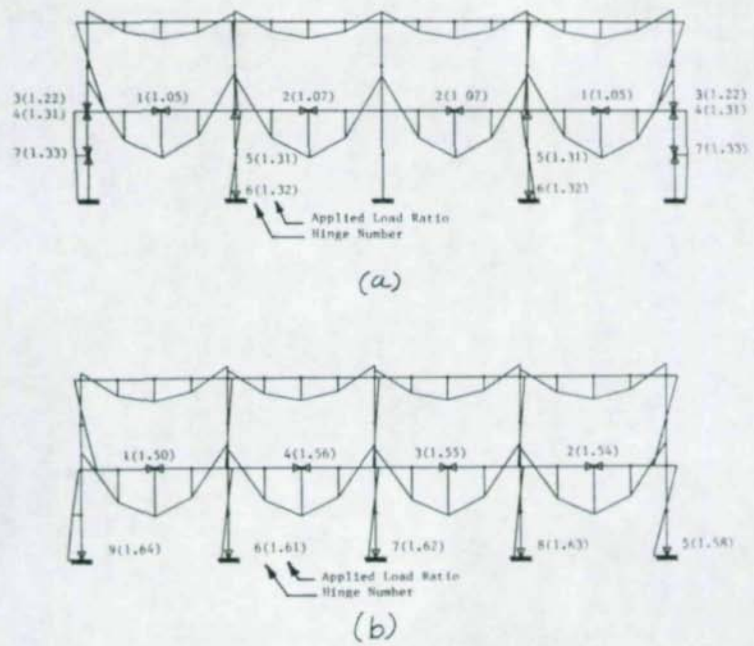

Figure 4. Bending Moment Diagram and Hinge Locations
(a) Under Gravity Loads, (b) Under Gravity Plus Wind Loads.

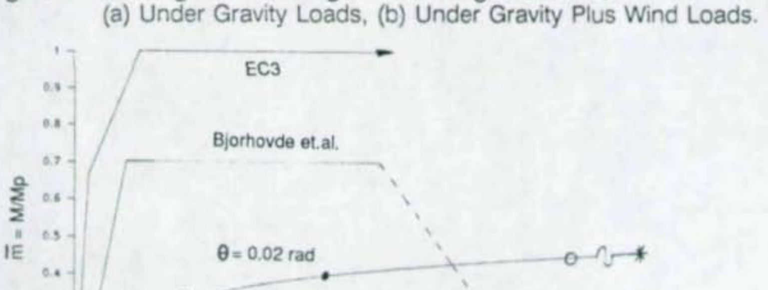

Figure 5. Comparison of Connection Behavior with EC3 (1990) and Bjorhovde et. al. (1990) Classification Schemes.

M_p. Cases 2 and 3 (TSAW-Upper and TSAW-Lower with M_{cn} = 0.4 M_p) are used to compare differences due to the shape of the response curve (see Fig. 2c). Cases 4 and 5 (AVERAGE curve with M_{cn} = 0.3 M_p and 0.5 M_p, respectively) are used to compare the effect of connection strength. Finally, in case 6 rigid connections are assumed. By comparing the applied load ratios (Table 2) between cases 1, 2, and 3, it is apparent that variations in the shape of the connection curve have a small effect on the calculated strength. All of the load ratios for these cases are within ±8% of the AVERAGE case. As evident from the roof drifts reported in Table 3, the deflection response is more sensitive to the curve shape, but even here the differences are within ±10% of those for the AVERAGE case.

Variations in connection strength have a greater effect on the response, but percentage wise the overall effect is less than the change in connection strengths. Comparing cases 1, 4, and 5, the differences in connection strengths are ±25% and the resulting inelastic limit points are within ±8% and drifts within ±23% of the AVERAGE case. The comparison between cases 1, 4, and 5 also indicates that there is a consistent increase in the overall inelastic limit point with increasing connection strength. Finally, a comparison of cases 1 and 6, indicates that by neglecting the connection flexibility, the inelastic limit point for the controlling gravity load case is overestimated 17% and the roof drifts are underestimated by up to 40%.

Nonproportional Loading. To check whether proportional loading results in the most critical evaluation of the limit states, several analyses were made for case 1 under nonproportional and reverse loading. Using the service load combination per Table 3, the dead and live loads were applied and held constant after which the wind load was applied. Under this sequence the wind drift was approximately 15% less than under the

Table 2. Comparison of Applied Load Ratios.

		Connection Model					
		AVE	TSAW (.4M_p)		AVE		Rigid
Loading	Criteria	.4M_p	Upper	Lower	.3M_p	.5M_p	
		1	2	3	4	5	6
1.2D + 1.6L_F + 0.5L_R	1st Hinge	1.05	1.09	1.03	0.99	1.10	1.08
	Limit Point	1.33	1.31	1.44	1.22	1.41	1.56
1.2D + 0.5L + 1.3W	1st Hinge	1.49	1.52	1.47	1.41	1.56	1.47
	Limit Point	1.64	1.62	1.70	1.54	1.73	1.90

Table 3. Comparison of Roof Drifts (inches).

	Connection Model					
	AVE	TSAW (.4M_p)		AVE		Rigid
Loading	.4M_p	Upper	Lower	.3M_p	.5M_p	
	1	2	3	4	5	6
1.0D + 0.2L + 1.0W (Service)	0.55	0.49	0.57	0.64	0.50	0.37
1.2D + 0.5L + 1.3W (Full Factored)	0.82	0.77	0.84	1.01	0.73	0.49
1.2D + 0.5L + 1.3W (Limit Point)	3.58	3.77	3.33	4.23	3.34	2.17

proportional loading case. Moreover, upon subsequent cycling of the wind load the structure reached an elastic shakedown state after the 2nd cycle and subsequent elastic wind drifts were 30% less than in the proportional loading case.

To evaluate the inelastic limit point under nonproportional loading, the factored gravity plus wind load combination was used. First, the factored gravity loads (1.2D + 0.5L) were applied up to a load ratio of 1.64 which corresponds to the limit point under proportional loading. Next, the factored wind load (1.3W) was increased until the limit point was reached at a load ratio of 1.73 which is 5% over the proportional loading value.

SUMMARY AND CONCLUSIONS

To address the growing desire to include nonlinear connection effects explicitly in analysis and design, a method has been proposed where standardized curves are used to model moment-rotation response. Included is a proposed AVERAGE curve for use in design. While this curve may be less precise than more complicated models, the results of a limited sensitivity study indicate that, compared to the effect of other assumptions made in design, the accuracy of the overall response is not significantly affected. Also indicated is the potential need for imposing limits on the maximum connection rotations permitted for inelastic analysis/design methods. Finally, the results substantiate trends which have been reported previously: (1) that increasing connection strength increases the load at the inelastic limit point, and (2) for design purposes, proportional loading gives a conservative prediction of both strength and serviceability limit states.

REFERENCES

AISC (1986), Load and Resistance Factor Design, 1st Edition, AISC, Chicago.

Anderson, D.(1990), "Design of Unbraced Frames," Chapter 4 of Analysis and Design of Steel Frames with Semi-rigid Joints, Draft report by ECCS TWG 8.1/8.2, TC8, April 22, 1990.

Bjorhovde, R., Colson, A, Brozzetti, J. (1990), "Classification System for Beam-to-Column Connections," Jl. of Struct. Engrg., ASCE, 116(11), Nov. 1990, pp. 3059-3076.

Chen, W.F., Kishi, N. (1989), "Semirigid Steel Beam-to-Column Connections: Data Base and Modelling," J. Struct. Engrg., 115(1), pp. 105-119.

ECCS (1990), Eurocode No. 3: Design of Steel Structures, EC3: Part 1:19xx, Draft Issue, April 1990.

Goverdhan, A.V. (1983), "A Collection of Experimental Moment-Rotation Curve and Evaluation of Predicting Equations for Semi-Rigid Connections," M.S. Thesis, Vanderbilt Univ., Nashville, TN.

Hsieh, S-H (1990) "Analysis of Three-Dimensional Steel Frames with Semi-Rigid Connections," M.S. Thesis, School of Civil and Environmental Engrg., Cornell University, Ithaca, NY.

Hsieh, S-H, Deierlein, G.G., McGuire, W., Abel, J.F. (1989), "Technical Manual for CU-STAND," Struct. Engrg. Report 89-13, School of Civil and Environmental Engineering, Cornell University, Ithaca, NY.

Kishi, N., Chen, W. F. (1986), "Data Base on Steel Beam-to-Column Connections," CE-STR-86-26, School of Civil Engineering, Purdue University, West Lafayette, IN.

Richard, R.M., Abbott, B.J. (1975), "Versatile Elastic-Plastic Stress-Strain Formula," Jl. of the Engrg. Mech. Div., 101(EM4), Aug. 1975, pp. 511-515.

Shen, Y-J (1990), "Computer-Aided Design of Steel Frames with Partially Restrained Connections," M.S. Thesis, School of Civil and Environmental Engineering, Cornell University, Ithaca, NY.

Technical Papers on

ECONOMY OF DESIGN

ECONOMY OF SEMI-RIGID FRAME DESIGN

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André Colson[2]

Abstract

The paper presents a study of the economical impact of utilizing semi-rigid concepts in the design and fabrication of steel frames. To make the comparisons complete, the evaluations include data for structures using flexible, semi-rigid and rigid connections. Two typical frames have been examined: one an unbraced, three story, two bay structure with rigid and semi-rigid joints; the other a braced, four story, four bay frame flexible, semi-rigid and rigid connections. Cost evaluations were provided by French and American fabricators. The results indicate that the cost benefits may be substantial for low-rise, unbraced frames; the findings are mixed for braced systems. Albeit a study of limited scope, the potential benefits indicate that a systematic and detailed study need to be undertaken, to gage the practical impact.

1. INTRODUCTION

Semi-rigid design has been recognized as a structural engineering concept for a long time, although design codes have not offered many guidelines in the way of specific criteria or other requirements. In principle, the "types of construction" that formed the backbone of American steel design specifications for the past 40 years and more (AISC, 1961, 1986), established a range of structural response that was uniquely tied to the kinds of connections that were used to attach the structural members to each

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other[3]. Thus, Type 1 referred to rigid frame or moment resistant construction, Type 2 was simple or pinned framing, and Type 3 was identified as semi-rigid. With the advent of LRFD, it was recognized that most, if not all connections offered a measure of stiffness and moment capacity, and the designations were changed to Type FR (= Fully Restrained) and Type PR (= Partially Restrained). FR is identical to Type 1; PR combines Types 2 and 3.

The old American specifications gave explicit rules for the design of structures classified as Types 1 and 2. Type 3 was allowed, but it was left entirely to the designer to demonstrate why and how a particular detail would respond in a fashion less than rigid, but with a certain moment capacity. The situation is essentially unchanged regarding the use of Types FR and PR under the LRFD rules.

The European situation has been somewhat different, primarily because it is not until relatively recently that a common design document has appeared. It is not known whether any earlier, individual national codes provided criteria for semi-rigid design. However, with the advent of Eurocode 3 (EC 3) (European Community, 1990), although still in draft form, it is clear that the aim of the European Community is to offer a uniform basis for structural design in the member countries.

As it stands, EC 3 gives what is currently the most explicit criteria anywhere in the world, for the design of frames with semi-rigid connections. Thus, recommendations are provided for frame analysis approaches and connection modeling, using certain classifications and simplifications that are meant to make the procedure somewhat more palatable to the design profession. This notwithstanding, the general reaction on the part of engineers has been one of reserve, or, in other words, negative.

In the United States, for a number of years structures have been designed on the basis of the concepts of "Type 2 construction with wind moment connections", where the connections are assumed to behave as pins under gravity load, but offering stiffness and moment capacity under lateral load (Disque, 1964, 1975). For example, many of the high-rise buildings that were constructed in New York City in the 1960's and 70's were designed in this fashion. Effectively semi-rigid, the framing members and the connections were nevertheless dealt with as simple. Thus, although the code framework for semi-rigid design has been available, in actual fact it has not been taken into account ***per se***.

Finally, one of the key obstacles to the acceptance of semi-rigid criteria has been the lack of practical design tools available to the profession. Semi-rigid design is much too complicated for manual procedures, but the rapid development of computer hardware

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[3] The first AISC Specification was issued in 1923. This, and all subsequent, editions was based on allowable stress design (ASD) principles. The first American limit states code for steel was the 1986 version. ASD and LRFD are both currently accepted as design philosophies in the US.

and software has brought the technique close to practical realization. Thus, computer programs are now available or under development, in Europe as well as North America, that can be used by designers. Certain American firms are actually performing designs that are in agreement with the general criteria of the code (Ioannides, 1988; Lindsey, 1988). Obviously, this is done to provide more economical structures, which is the bottom line control factor.

2. SCOPE OF STUDY

This paper will provide examples of designs of typical frames, using the range of connection representations that are acceptable, and to assess realistic fabrication costs. Although necessarily limited in scope, the intent is to demonstrate that it can be financially advantageous to utilize semi-rigid concepts for many structures.

Two types of structures have been analyzed, as follows:

(1) An unbraced, three story, two bay frame

(2) A braced, four story, four bay frame

The structural analyses were performed with the computer program PEP-Micro (Galéa and Bureau, 1990), which incorporates inelastic member behavior, structural and member second order effects, and non-linear connection response. The connection properties are input either in the form of an algebraic equation or as digitized data, including test results. For the frames of this study, the major parameters of the moment-rotation curves for the connections using web and flange angles were established using the mathematical model of Kishi et al. (1988). The braced frame was further analyzed through an optimization scheme (Colson and Fléjou, 1989).

The loads were set to be representative of French and American practice and codes, as were the frame dimensions. Eurocode 3 and the AISC LRFD Specification were used to size the members and the connections for the resulting designs. The cost figures that are presented are relative only, and reflect averages of the data provided by the two French and the two American companies. It is understood that regional differences may be important for connection configuration preferences and labor costs. However, the relative numbers are still of significance.

3. UNBRACED FRAME

3.1 Description of Structure

Due to the response characteristics of semi-rigid connections, it is felt at this time that their use ought to be limited to relatively low-rise structures that are subjected to static lateral loads. This is due to the potential problems of drift, which may be problematic in frames using more flexible connections, especially as the number of stories becomes large. A study by Ackroyd (1987) recommends limiting the use of semi-rigid concepts to frames 10 stories and less. Although it may be structurally feasible to maintain the drift within reasonable limits, the economies of such frames may be unacceptable. However, only a detailed examination of a range of frames and framing systems can offer a rational assessment of the relationship between connection characteristics and frame serviceability.

Finally, the questions of cyclic loading and the response of the connections, especially regarding cumulative plasticity, energy absorption, and the potential for low-cycle fatigue, have not been addressed in systematic fashion. A recent investigation by Nader and Astaneh (1991), however, offers significant input on the seismic response of semi-rigid connections. The results appear to indicate favorable behavior, and clearly point out that the energy-absorbing characteristics of such joints may offer potential advantages for structures in seismic regions.

The unbraced frame that was analyzed is shown in Fig. 1, with beam spans and story heights in meters. The structure is braced at each story in the direction perpendicular to the plane of the frame; the distance between adjacent frames is 6 meters (20 feet).

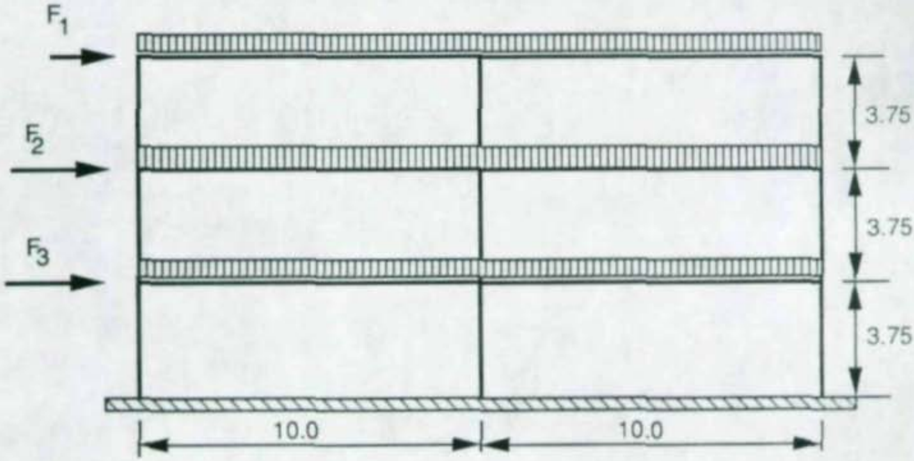

Figure 1 Unbraced Frame Used in Design Example

Plastic analysis was used for the structure, with the following gravity and wind loads:

Dead load:	9 kN/m (0.62 k/ft)
Live load:	13.25 kN/m (0.91 k/ft)

Wind loads (see Fig. 1): F_1 = 3.93 kN (0.88 kips)
$F_2 = F_3$ = 7.87 kN (1.77 kips)

The analysis was based on a column out-of-plumb of h/200, where h is the story height. For out-of-plane buckling of the columns, the system length of 3.75 meters (12.3 feet) was set equal to the buckling length (i.e. K = 1.0).

Serviceability limits were imposed for beam deflections (live load only) and story drift, as follows:

Story drift:	h/400 (28.1 mm = 1.1 in)
Beam deflection:	l/360 (27.7 mm = 1.1 in)

The relevant criteria of Eurocode 3 and the AISC LRFD Specification were satisfied for all structural alternatives.

3.2 Results for Unbraced Frame with Rigid Connections

All connections were assumed to be perfectly rigid. Recognizing current practice, it was decided to use end-plate moment connections with pre-tensioned bolts. Further, the column web was stiffened, at the level of the beam flanges, and a diagonal stiffener was also utilized. The latter is required under Eurocode 3; it is rarely used in structures of this kind in the United States, where doubler plates are more common. For the end-plate connections in question, furthermore, design by LRFD showed that neither diagonal stiffener nor doubler plates were needed. However, it was decided to retain the diagonal stiffener for the design by both codes, to obtain a more realistic comparison of the fabrication costs. Some details of the moment connection are shown in Fig. 2. In addition, the final end plate thickness was 30 mm (approximately 1-3/16 inch), and the connection needed 10 20 mm (3/4 inch) A 325 bolts (European designation 8.8). The column base connections were treated as fully fixed, using a sufficiently thick base plate and four anchor bolts.

The resulting column and beam sizes were, using American and European standardized shapes:

All columns:	W8x40	or	HEB 200
All beams:	W16x45	or	IPE 400

The frame elements that governed the design were the interior column at the first floor level and the second story of the exterior (windward side) column. A plastic hinge formed in the adjacent beam for each ultimate limit state.

At the serviceability limit loads, the maximum story drift was 9.5 mm (3/8 inch), or about one third of the allowable value. The maximum beam deflection was 11.2 mm (approximately 7/16 inch), or roughly 40 percent of the allowable.

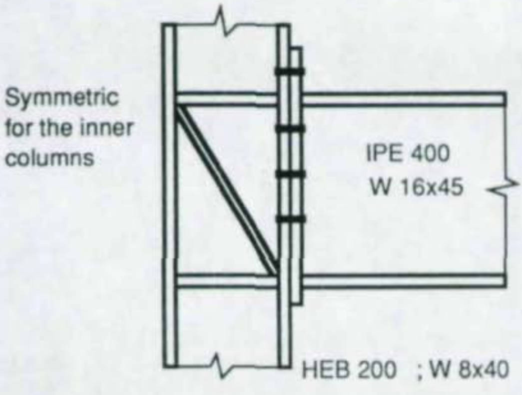

Figure 2 Rigid Connection Used for Unbraced Frame

3.3 Results for Unbraced Frame with Semi-Rigid Connections

Figure 3 shows the beam-to-column connection that was chosen. Using top and seat angles, along with a double web angle, this is a typical semi-rigid connection for frames. The moment-rotation relationship was established in accordance with the approach of Kishi et al. (Kishi et al., 1988). The flange angles were L120x120x12 (L5x5x1/2) with 22 mm (7/8 inch) A325 or 8.8 high strength bolts, and the web angles were L80x80x8 (L3-1/2x3-1/2x3/8) with 20 mm (3/4 inch) bolts. The bolts were not pretensioned.

The column base connections were basically the same as those used in the rigid frame example of Section 3.2, although they were assigned an initial stiffness of 20,000 kNm/rad (14,760 k-ft/rad), to reflect an average flexibility as determined in physical tests (Penserini, 1991).

The design of the frame aimed at retaining the same sizes of beams and columns as was used for the rigid frame. It is readily understood that this would be the outcome if the frame were governed fully by ultimate limit states, and if structural second order effects were inconsequential. The results showed that W14x38 (IPE 360) beams could be used, in contrast to the W16x45 (IPE 400) of the preceding example. However, it was decided to use the same beam sizes as in the rigid frame, to focus strictly on the economic impact of using less demanding connections.

The members that governed the frame design were the same as those of the rigid case, namely, the interior column at the first floor level, and the windward second story column. The number of plastic hinges was unchanged, as a direct consequence of the extensive redistribution of the bending moments in the more flexible semi-rigid frame. This also explains why the smaller beam size could have been used.

At the serviceability limit state the maximum story drift (top floor) was 16.8 mm (0.66 in), or approximately 60 percent of the allowable. The maximum beam deflection was 18.3 mm (0.72 in), or about 66 percent of the deflection limit.

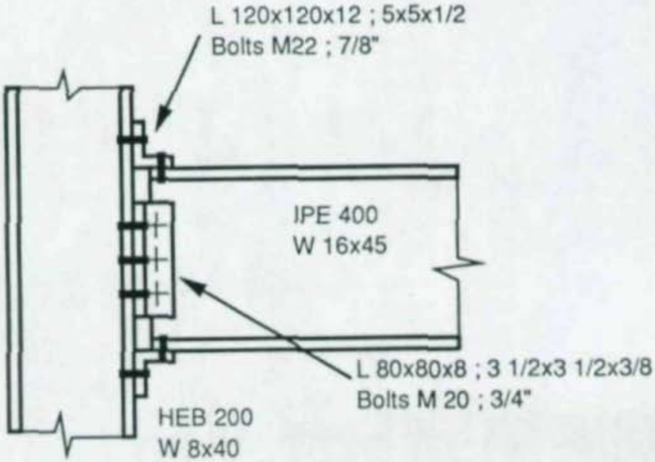

Figure 3 Semi-Rigid Connection Used in Unbraced Frame

3.4 Cost Comparison of Rigid and Semi-Rigid Frames

To evaluate the relative cost figures, it was decided to set the value of the rigid frame alternative as 100. The members and connection details of this case adhered to the requirements of Eurocode 3, and also to the LRFD Specification solution (although, as *noted earlier*, the use of a diagonal stiffener is not necessary under US criteria).

The relative costs of the semi-rigid frame example were 82 in France and 80 in the United States. Since the beam and column sizes were purposely chosen to be the same for both, the better economies of the semi-rigid frame can be attributed to the less costly connections. Finally, all strength and serviceability requirements were met.

4. BRACED FRAME

4.1 Description of Structure

The design was performed using a computer program that optimizes members and connections in semi-rigid frames. For a given geometry (beam spans, distances between adjacent beams) and set of loads, the program sizes the columns through a simple member force analysis procedure. The beam design is based on equalizing the positive and negative moments. Once the member sizes are known, the angles for the beam-to-column connections are chosen, taking into account bolt diameters and placement. Finally, the lengths of the angles are selected such that the connection stiffness and moment capacity are in agreement with those required by the beam optimization solution. That is, the beam span moment should be as close to the support moment as possible.

The braced frame that was utilized in this part of the study is illustrated in Fig. 4. The uniformly distributed gravity dead load and live load were both 15 kN/m (1.03 k/ft), giving a total factored gravity load of 42.75 kN/m (2.93 k/ft). Serviceability was set as a beam deflection limitation of (span)/300, equal to 20 mm (0.75 inch). The structure was analyzed elastically, that is, using the maximum factored moment in a member to select a suitable section modulus.

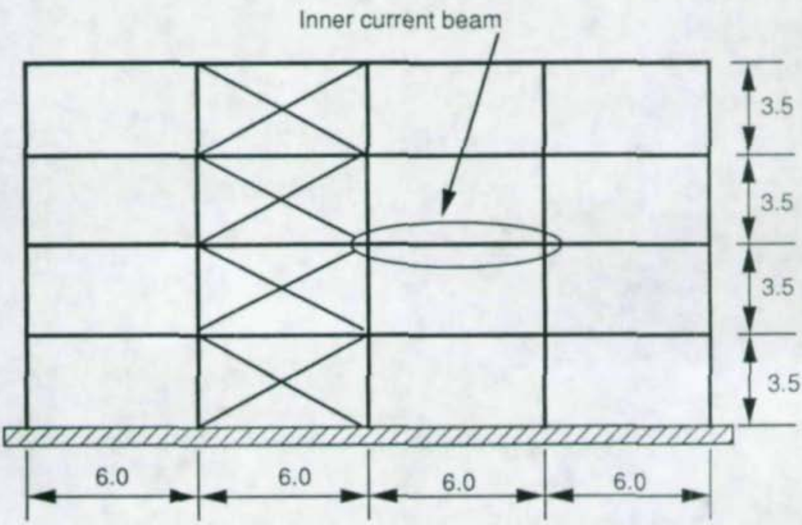

Figure 4 Braced Frame Used in Design Example

Three frame designs with different connections were prepared, as follows:

(1) Double web angle connections, which makes the frame simply connected (Type PR or 2). In current US practice single plate shear tabs

are often used as connections in this kind of construction.

(2) End-plate moment connections, which makes the frame rigid (Type FR or 1). Although this is not normally done for braced frames, with the possible exception of cases where the story height needs to be minimized, the example was included to provide a solution that would be the upper bound for the semi-rigid case.

(3) Top-and-seat angle and double web angle connections, which makes the frame semi-rigid (Type PR or 3).

The column sizes were dictated by the gravity loads, and were the same for all three frame designs. It was found that the shapes W8x58 or HEB 240 would be the most economical solution. The lateral loads were taken by the bracing system, which was identical for all three frames.

4.2 Results for Braced Frame with Simple Connections

The maximum beam moment is that of a simply supported beam, equal to $ql^2/8$, where q is the uniformly distributed load and l is the span. The final beam sizes were W14x38 or IPE 360, with web angles L120x120x12 (L5x5x1/2) and 24 mm (1 inch) non-pretensioned high-strength bolts (A325 or 8.8). The design is shown in Fig. 5.

4.3 Results for Braced Frame with Rigid Connections

The maximum beam moment is that of an elastic, fixed-end beam, equal to $ql^2/12$. This solution required a W12x30 or IPE 300 beam, along with a 15 mm (5/8 inch) end plate and 24 mm (1 inch) pretensioned quenched and tempered high-strength bolts (ASTM A490, which is comparable to the European designation 10.9). Details are shown in Fig. 6. It is again emphasized that this design example does not represent common practice; it is included only to provide a fully rigid comparison to the semi-rigid alternative.

4.4 Results for Braced Frame with Semi-Rigid Connections

The maximum beam moment is $ql^2/16$. The resulting design utilizes a W10x26 or IPE 270 beam, with top and seat angles L150x150x15 (L6x6x5/8) and web angles L120x120x12 (L5x5x1/2). The bolts are 24 mm (1 inch) A325 or 8.8 non-pretensioned high-strength bolts. A slightly lighter beam could have been used, but it was decided that this was a marginal call. The W10x26 (IPE 270) was therefore retained. The solution is illustrated in Fig. 7.

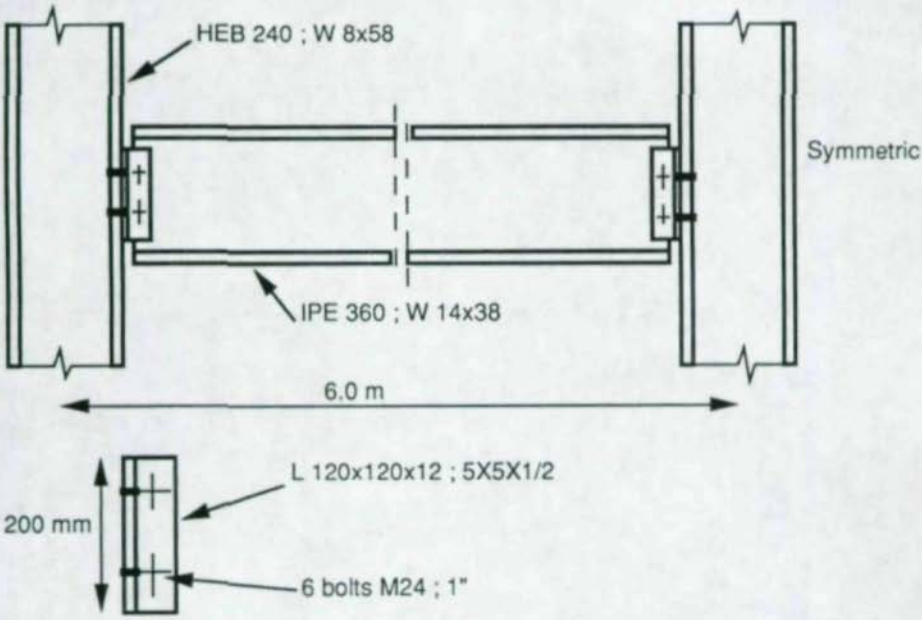

Figure 5 Braced Frame Beam with Simple Connections

4.5 Cost Comparison of the Three Braced Frame Solutions

Only the connection details and the beam sizes varied from example to example. Using the simply connected frame as the base, since this is representative of current practice for braced frames, its cost was assigned the value 100. The US and French relative cost data for the rigid and semi-rigid alternatives were found as:

(a) Case 2: Rigid Connections:	France:	120
	US:	115
(b) Case 3: Semi-Rigid Connections:	France:	96
	US	105

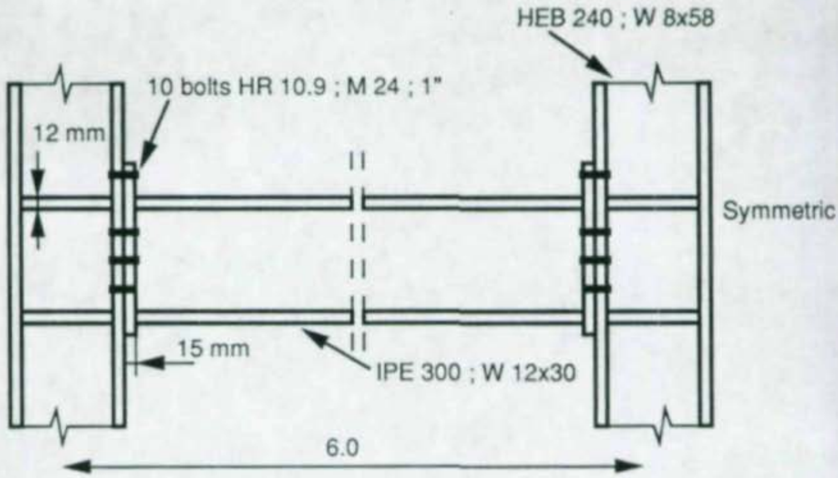

Figure 6 Braced Frame Beam with Rigid Connections

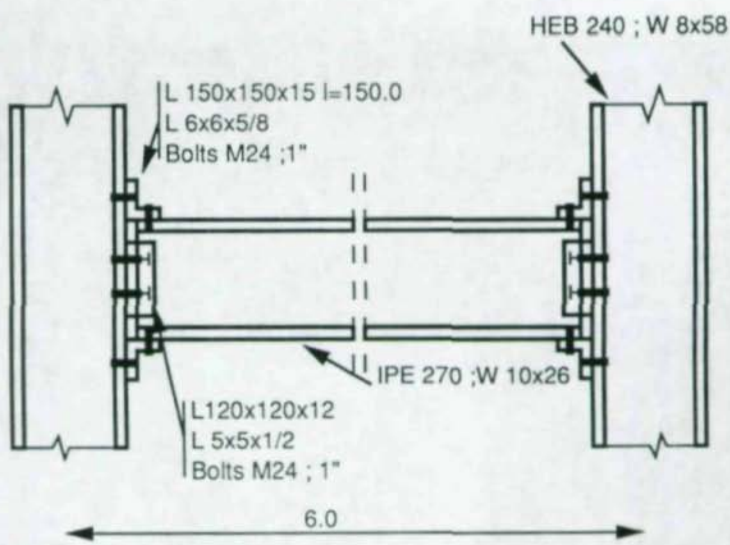

Figure 7 Braced Frame Beam with Semi-Rigid Connections

5. SUMMARY AND CONCLUSIONS

As a somewhat surprising solution, it was found that the unbraced frame benefited clearly from taking advantage of semi-rigid concepts. Although the member sizes did not change, significant savings were obtained by using less expensive connections. *Drifts and deflections were larger for the semi-rigid solution, but still well within the* limitations set for adequate performance.

For the braced frame, on the other hand, the cost picture is mixed. Comparing cases 1 (simple) and 3 (semi-rigid), it is evident that although smaller beams could be used for case 3, the more expensive connections effectively negate the benefit. It is possible that more clear-cut benefits can be gained from other framing systems and connection details, but this must be examined as part of a larger project.

Cost benefits of 20 percent for the unbraced frames indicate that it would be important to undertake a detailed, systematic investigation of a range of frame and connection types, along with cost evaluations for different geographic locales. Further, it would offer benefits that are not being accounted for in current practice. This will especially be the case for moment frames with a large number of bays. In any case, such a study should be performed by a practicing engineer, as opposed to a university researcher.

As indicated in the introduction, the purpose of this study was not to arrive at specific, precise cost figures for a range of framing systems. Rather, relative performance criteria were intended to establish the potential of semi-rigid concepts. Current US and European design codes allow for the use of such design procedures; the actual implementation has been hampered by the lack of analysis and design tools. Such are now becoming readily available, and practical utilization is therefore a matter of technology transfer and further code recognition.

6. REFERENCES

1. Ackroyd, M. H. (1987). "Simplified Frame Design of Type PR Construction", AISC Engineering Journal, Vol. 24, No. 4, December (pp. 141-146).

2. American Institute of Steel Construction (AISC) (1961). "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings", 6th Edition, AISC, New York, New York, U S A.

3. American Institute of Steel Construction (AISC) (1986). "Specification for the Load and Resistance Factor Design, Fabrication and Erection of Structural Steel for Buildings", 1st Edition, AISC, Chicago, Illinois, U S A.

4. Colson, A., and Fléjou, J. L. (1989). "Conception Automatique d'Assemblages Semi-Rigides pour les Structures à Noeuds Fixes", Construction Métallique, Vol. 26, No. 4.

5. Commission of the European Communities (CEC) (1990). "Eurocode 3: Design of Steel Structures", CEC, Brussels, Belgium, November (Draft).

6. Disque, R. O. (1964). "Wind Connections with Simple Framing", AISC Engineering Journal, Vol. 2, No. 3, July (pp. 101-103).

7. Disque, R. O. (1975). "Directional Moment Connections - A Proposed Design Method for Unbraced Steel Frames", AISC Engineering Journal, Vol. 12, No. 1, January (pp. 14-18).

8. Galéa, Y., and Bureau, D. (1990). "PEP-Micro: Programme d'Analyse Non-Lineaire des Structures à Barres", CTICM, St. Rémy-lès-Chevreuse, France.

9. Ioannides, S. A. (1988). "Frame Analysis Including Semi-Rigid Connections and P-Delta Effects", in "Connections in Steel Structures: Behavior, Strength and Design", edited by R. Bjorhovde, J. Brozzetti and A. Colson, Elsevier Applied Science Publishers, Ltd., London, England.

10. Kishi, N., Chen, W. F., Matsuoka, K. G., and Nomachi, N. G. (1988). "Moment-Rotation Relation of Top- and Seat-Angle with Double Web-Angle Connections", in "Connections in Steel Structures: Behavior, Strength and Design", edited by R. Bjorhovde, J. Brozzetti and A. Colson, Elsevier Applied Science Publishers, Ltd., London, England.

11. Lindsey, S. D. (1988). "Practical Considerations in the Design of Frames with PR Connections", in "Connections in Steel Structures: Behavior, Strength and Design", edited by R. Bjorhovde, J. Brozzetti and A. Colson, Elsevier Applied Science Publishers, Ltd., London, England.

12. Nader, M. N., and Astaneh, A. (1991). "Dynamic Behavior of Flexible, Semi-Rigid and Rigid Steel Frames", Journal of Constructional Steel Research, Vol. 18, No. 3 (pp. 179-192).

13. Penserini, P. (1991). "Caractérisation et Modélisation du Comportement des Liaisons Structure Métallique-Fondation", Ph.D. Dissertation, École Normale Superieure de Cachan, Université de Paris VI, February.

TEMPORARY FLEXIBLE CONNECTIONS

IN THE CONSTRUCTION PROCESS

ENRIQUE MARTINEZ-ROMERO[1]

Abstract

The design case of a 30-story building in Mexico City, which incorporated temporary flexible connections in the construction process, to attain an economical, safe and time effective excavation system of its five underground parking levels, is described herein.

The behavior of the flexible strut-beam connections to the bare steel core of composite columns, is analized by a three- dimensional finite-element computer program, and its results are summarized in this paper.

DESCRIPTION OF THE BUILDING'S FRAMING

A composite steel-concrete structural system was developed for this building, such that the bare steel framing could be used first during the excavation process of its five-underground parking levels, to resist the earth pressures, and later on to integrate the floor system. The same construction system with slight changes, was employed from the street level up to the 25th (roof) level. Both framings consisted of a modified version of the stub-girder system, as will be explained later.

The construction process started at street level, by drilling 1.20m in diameter, and about 20.0m deep holes on the ground at every column location. The bottom of the hole, which was suppossed to be about 1.50m under the lowest part of the building's foundation, received once it was clean from loose material, an unreinforced concrete slab about 20cm thick. (Fig.1)

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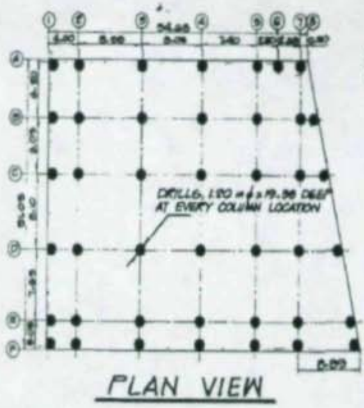

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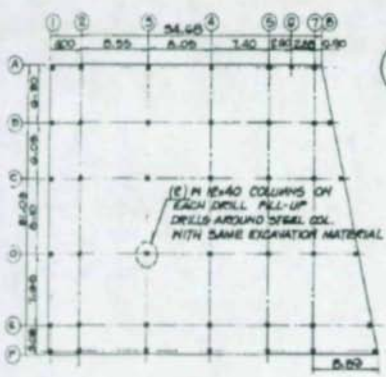
Light steel columns formed by two W 12 x 40 sections joined in parallel,were introduced until the bottom of each drill, and after being properly plumbed and aligned, were encased at the bottom in about 1.50m of plain concrete. Holes were then re- filled with the same excavation material up to ground level, and the column box filled up with concrete slurry at its entire length. (Figs. 2 and 3).



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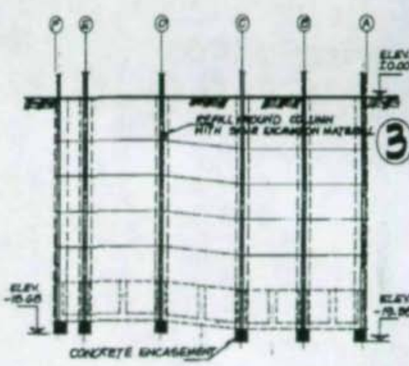

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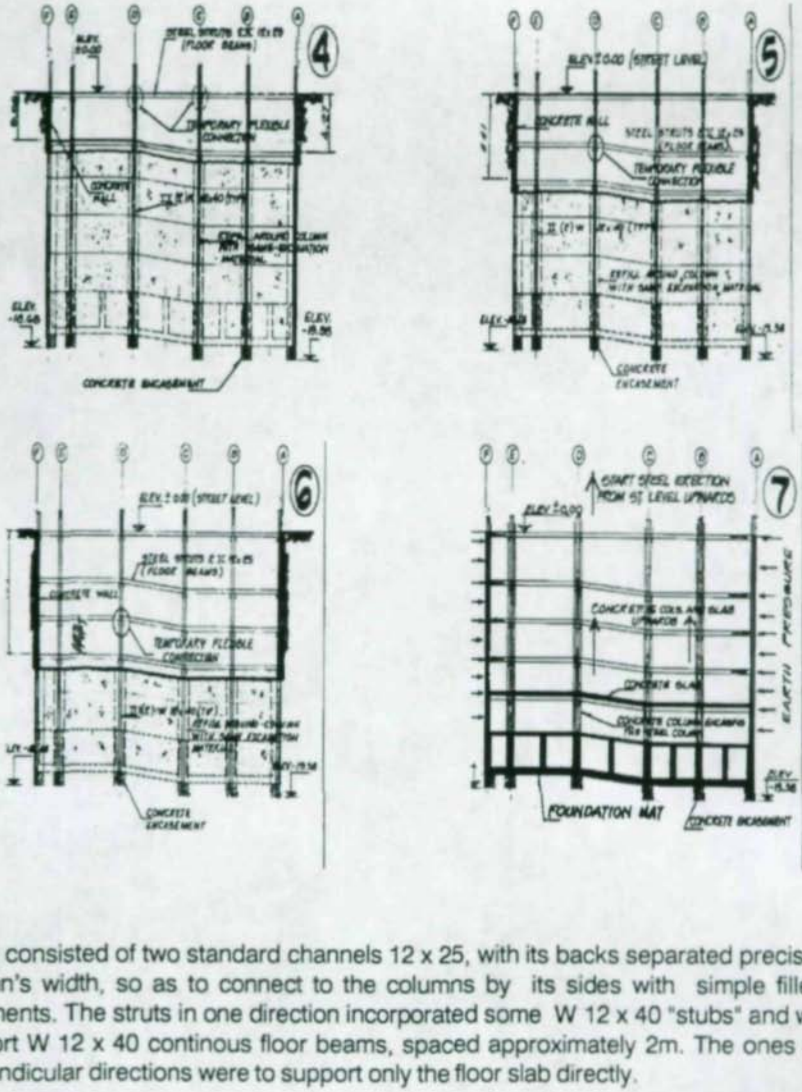
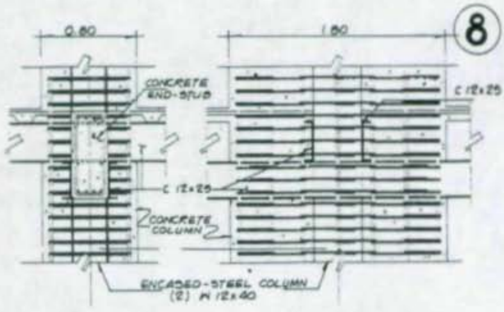
Such steel columns were intended to serve during the excavation process as the connecting points of the orthogonal struts at every underground level, which transmmitted the earth pressures acting on a poured-downwards concrete wall, from one side to the opposite one, and after the completion of the excavation and finishing of the perimetrical concrete wall and building's foundation, as the core of the building's composite columns. (Figs. 4 to 7).



Struts consisted of two standard channels 12 x 25, with its backs separated precisely the column's width, so as to connect to the columns by its sides with simple fillet field weldments. The struts in one direction incorporated some W 12 x 40 "stubs" and were to support W 12 x 40 continous floor beams, spaced approximately 2m. The ones on the perpendicular directions were to support only the floor slab directly.

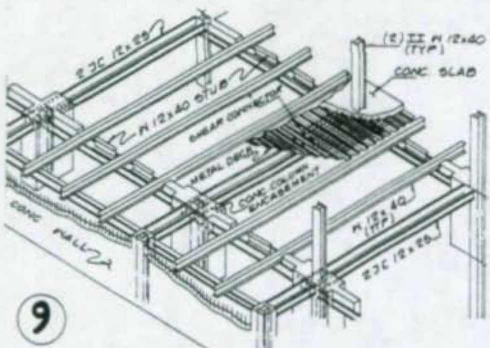
The floor slab consisted on 5cm of concrete poured on top of the high ribs of a QL-99 Robertson metal deck; and integrates to the floor beams and "stub" girders by means of some C3 x 5 shear connectors, 10cm long, field welded through the steel deck to the top-flanges of the channels strut-beams and floor beams. 300 kg/cm^2 (4267 psi) concrete strength was used for the slab.

The floor so structured presents a modified version of the so- called "Stub-Girder" construction system (Colaco, 1972), and features end-stubs and naturally, two-channel floor girders instead of the standard W sections. (Fig. 8).



SECTION A-A SECTION B-B

It is noteworth that the end-stubs will be formed of reinforced concrete in the space left between the two channels, and should be poured monolithically with the concrete column encasement of the steel column. (Fig. 9).



SCHEMATIC FLOOR FRAMING

Thus, the beam-to-column connection, flexible on its nature during the construction process, will be at a later stage, totally encased in reinforced concrete in order to assure a rigid-joint structural behavior, required both for strength and stiffness of the building.

The following paragraphs describe in detail the simple construction of the joint, its analysis and design, as well as its predicted behavior.

BEAM-TO-COLUMN JOINT DURING CONSTRUCTION

As explained before, the main purpose of the connection studied ahead, was to simplify the construction process, of the building particularly during the excavation, since its depth (almost 20m), originated severe earth pressures againstthe retaining walls, which could be either temporary or permanent. The cost and feasibility of several construction systems for the excavation were studied for this building. All of them resulted excesively expensive as compared with the one selected, particularly because of both,the reduced place for construction and importantly, because of the additional risk of adjacent tall buildings, founded above the bottom of the excavation.

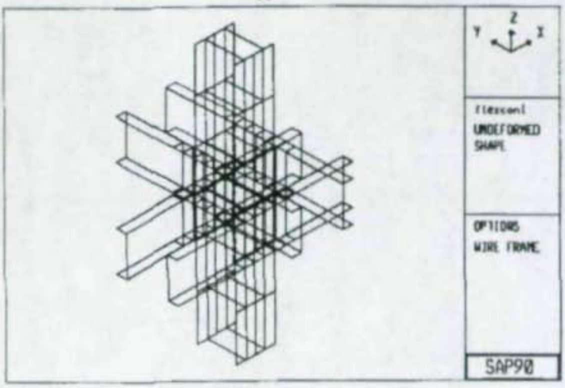
The joint was the simplest possible way of connecting the strut- beams to the steel column; i.e., by field fillet weldments between the channels and the column's flanges.

The struts were designed to carry the compression load from the retaining walls, together with the vertical (transversal) load given by the floor beams and construction load only. This was the design load for its temporary condition.

The intersection of the struts in orthogonal directions was greatly simplified by offsetting one strut above the other, being the ones at a lower level, parallel to the column's flanges, where the length of weld is larger and easier to make. Thus, the upper strut passes continuously by the column's sides (parallel to the column's webs), on top of the lower strut, and is attached to the column's flange-tips by a vertical fillet weld only.

Additionally, the upper struts deliver some part of its vertical load through direct contact between the horizontal channel's flanges, the amount of such a load being difficult to evaluate, since it depends on the relative stiffness of the parts in contact as well as on the weldments rigidity.

This joint was modeled for its study by a three-dimensional finite element mathematical model, and analised by the SAP90 program (Wilson, et al, 1989). Figure 10 shows the computer produced plot of the joint analised. The weldments between the channels and between the column and the channels are shown in darker line; those welds not visible are indicated in dotted darker line.



Shell elements were used to represent all parts of the model, even the weldments themselves. This modeling element worked mostly as a three-dimensional plate, and the 4-node element formulation is a combination of membrane and plate bending behavior, including two-way out-of-plane plate rotational stiffness components and a translational stiffness component in the direction normal to the plane of the element (Taylor, 1985; Batoz, 1982).

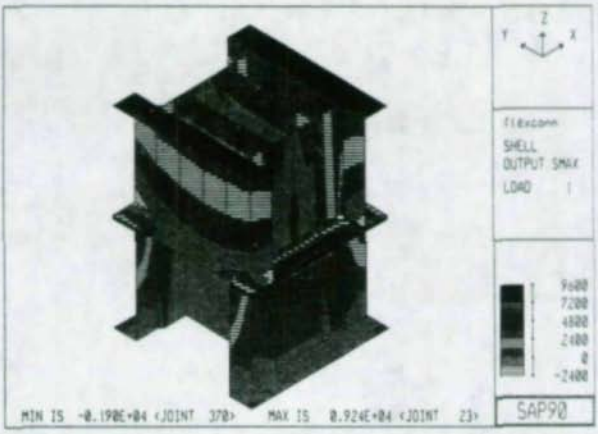
Proper restrains were assigned to some of the joints, at the top and bottom of the column. Similarly, restrains at both the ends of the struts and the vertical loading points were duly considered inasmuch as to represent in a realistic mode the joint behavior.

The model was analized for four loading combinations which incorporated the full axial compression of the strut as well as the axial compression on the column, originated by the earth pressures and weight of the framing (five levels), respectively. Additionally vertical loading was applied transversally to the struts, to represent the construction load. The load combinations included symmetrical and non-symmetrical loading. The magnitude of the loads used herein, were taken from a ETABS (Habibulah, 1989) three-dimensional analysis of the complete building, during the construction stage. (Martinez-Romero E., 1990).

Since the flexible joint defined above was not designed, to resist as such any other transversal load different than the self weight of the framing previously considered, (and the thrusts delivered by the retaining wall, of course), a different model of the joint, partially embeded on the encasement of concrete, was used to revise its working conditions under the weight of the slab and full live load. Such a model is not included here due to limitations of this paper.

RESULTS

Results from the analysis performed on the model described above, show dramatically realistic results which can be easily interpreted with the assistance of the graphic postprocessor SAPLOT (Habibulah, 1989), which by different colors and intesities, depicts the state of stresses in the various elements of the model. Figure 11 shows the computer plot of principal axial stresses SMAX on the elements, due to one of the loading combinations.



The values of the stresses were compared with the allowable stresses, (AISC, 1989). Further, the stresses on the "weld- elements" were also revised according to the same specifications and found them within permissible values.

An interesting option of the graphic postprocessor permitted to suppress all the shell elements except for those representing the fillet welds. Thus the same colored intensity plots were studied to analize the stress distribution on each weldment.

CONCLUSIONS

Results from the analyses performed demonstrate an adequate structural behavior of the flexible beam-to-column connection described herein. Importantly, close observations of the construction process indicated a very convenient and easy way to connect steel framing with this twofold purpose of serving to resist soil pressures and in a later stage, as floor beams.

The steel framing weight accomplished was 66.6 kg/m^2 (13.6 P.S.F.), which can be rated as quite low, considering the height and slenderness of the building, as well as the fact of being built on a highly seismic zone.

The building construction, as it is up to date (April 1991), shows up excellent construction speed and concreting sequence. The topping up of its 30th floor is expected to be reached next June.

Another remark to be made on this construction system is that by going five levels underground, it was possible to reach the hard soil and thus to reclasify the seismic loading of the building to a less demanding zoning, according to the Mexico City Building Code of 1987.

REFERENCES

1. Colaco, J.P. "A Stub-Girder System for High Rise Buildings", Engineering Journal, AISC, July 1972.

2. Wilson, E. and Habibullah, A. "SAP 90. A Series of Programs for the Static and Dynamic finite Element Analysis of Structures", July 1989.

3. Taylor, R.L. and Simo, J.C. "Bending Membrane Elements for Analysis of Thick and Thin Shells", Proceedings of the NUMEETA 1985 Conference, Swansea, Jan 7-11, 1985.

4. Batoz, J.L. and Tahar, M.B. "Evaluation of a New Quadrilateral thin Plate Bending Element", International Journal for Numerical Methods in Engineering, Vol. 18, 1982.

5. Habibullah, A., "ETABS. Three Dimensional Analysis of Building Systems". Computers and Structures, Inc., Berkeley, CA., 1989.

6. Habibullah, A., and Wilson E., "SAPLOT. An interactive Imput and Output Display Postprocessor for SAP90". Computers and Structures, Inc., Berkeley, CA., 1989.

7. Martinez-Romero, E. and Juarez A., "Data Calculations of FORUM building, in Mexico City". Enrique Martinez Romero, S.A., Mexico, D.F., 1990.

8. Departamento del Distrito Federal "Reglamento para las Construcciones en el D. F., 1987"

A COST COMPARISON OF SOME METHODS FOR DESIGN OF BRACING CONNECTIONS

W.A. Thornton[1]

Introduction

Bracing connections constitute an area in which there has been much disagreement concerning a proper method for design. Since about 1981, AISC has funded research at the University of Arizona (Richard, 1986) and, since 1985, NBS (Gross and Cheok, 1988, now the National Institute of Standards and Technology) has been involved in physical testing of three approximately full scale vertical bracing connection specimens.

The above work has not yet been distilled into a consistent method of design of connections of this type. For many years, designers have fallen back on elastic beam models to analyze gussets and their connections. This is still a viable approach (Kulak et al 1987, Blodgett, 1966). However, a simpler approach is to use equilibrium models for the gusset, beam, and column and to require that yield not be exceeded globally on any gusset edge or section, and also on any section in the column or beam. If we achieve this, then the Lower Bound Theorem of Limit Analysis indicates that we will have achieved a safe design. This approach was promulgated in a paper by the author (Thornton, 1984), and forms the basis of the "AISC Method" (AISC, 1984).

There are many possible equilibrium models, three of the simplest of which are presented here.

Equilibrium Models

Model 1. This is the most common and simplest of all equilibrium models. The force distribution on the gusset, beam, and column are shown in Fig. 1. As with all equilibrium models, this model guarantees that the gusset, beam, and column are in equilibrium under the brace load P. If the work point coincides with the gravity axes of the members, equilibrium is achieved with no connection induced couples in the beam, column, or brace. Model 1 has been referred to as the "KISS" method, or, sarcastically, "keep it simple, stupid!".

---

[1] Chief Engineer, Cives Steel Company, Roswell, Georgia USA

Model 2. This model is a direct extension of the model adopted by AISC (AISC, 1984) to bracing connections to column flanges. The force distributions for the gusset, beam, and column are shown in Figs. 2 and 3 and as for Model 1, these force distributions guarantee that the gusset, beam, and column are in equilibrium under the brace load P. If the work point coincides with the member gravity axes, equilibrium is achieved with no connection induced couples in the beam, column, or brace. Model 2 is a little more complex for calculations than Model 1, but, as will be shown, it yields cheaper designs. *Model 2 will be referred to as the AISC method.*

Model 3. This model is the result of the author's search for an equilibrium model for bracing connections which achieves equilibrium for all components of the connection with linear forces only, i.e., no couples. It is the most efficient (yields cheapest designs) of the three models presented here but is the most complex in terms of calculations required. Note that this is not a serious problem because when programmed for a PC, the computational aspect of this model (and of the other two) is of little moment.

The force distributions for the gusset, beam, and column are given in Figs. 4 and 5. As for Models 1 and 2, it can be shown by elementary calculations that these force distributions satisfy equilibrium for the gusset, beam, and column, and if the work point is at member gravity axes, with no connection induced couples anywhere.

The beam shear R in Figs. 4 and 5 is shown applied to the beam to column connection. If the shear is large, it may be desirable to distribute it to the gusset to column connection as well. In this case, the gusset serves as a haunch and the gusset to beam forces must be adjusted to effect the desired distribution of R.

Example:

Consider the example given in Fig. 6. The column is a W14x605, the beam a W18x106 and the brace a W12x65 with 450K at a bevel of $9^{9}16$ vertical to 12 horizontal, $\theta = \tan^{-1}(^{12}/9^{9}16) = 51.45°$. This connection was completely designed for each of the three models discussed above using the connection design principles given in (Thornton, 1984 and Gross and Cheok, 1988). Figs. 7, 8, and 9 give the completed designs for Models 1, 2, and 3 respectively. Table 1 gives costs for the designs in Figs. 7, 8, and 9 determined by a fabricator's estimating department.

Table 1 shows what is visually apparent when Figs. 7, 8, and 9 are compared, i.e., that Model 3 gives the most economic design, and Model 1, the "KISS" Method, gives a design which costs (approximately) 28-30% more.

Table 1, Cost Comparisons

Model	W14x605 Column		W18x119 Column	
	Cost[1] U.S.$	Cost Relative to Model 3, o/o	Cost[1] U.S.$	Cost Relative to Model 3, o/o
1	840	128	732	130
2	746	113	638	114
3	658	100	562	100

(1) Cost includes material of gusset, clip angles, gusset to column and beam to column, bolts, and labor for welding and drilling.

Table 1 includes a lighter column section, a W18x119, to assess the effect of drilling the heavy flange of the W14x605. It should be noted that the total costs as determined here could be widely different if estimated by other fabricators, but the costs given should be useful as a rough indicator of relative costs.

Summary

Three equilibrium models have been presented here. Of these models, 2 and 3 are superior to the very simple Model 1 in terms of cost. Of the three models, Model 3 produces gusset edge force orientations closest to those indicated by the extensive analytical work done at the University of Arizona (Richard, 1986), which provides further confidence in its use.

The three equilibrium models presented here, which provide internal force distributions between the beam, column, and gusset, eschew empirical factors of any kind. Models 1, 2, and 3 achieve their force distributions with absolutely no connection induced couples required in the beam, column, and brace when the working point is the gravity axis intersection of these three members. Since no empiricism is involved, these models are applicable, with engineering judgment and intuition, to a wide variety of similar connections.

References:

1. American Institute of Steel Construction, Engineering for Steel Construction, AISC, Chicago, Illinois, 1984, pp.7-55 through 7-62.

2. Blodgett, Omer, Design of Welded Structures, The James F. Lincoln Arc Welding Foundation, Cleveland, Ohio, 1966, pp. 5.9-24 through 5.9-29.

3. Gross, John and Cheok, Geraldine, Experimental Study of Gusseted Connections for Laterally Braced Steel Buildings, National Institute of Standards and Technology Report NISTIR 88-3849, Gaithersburg, Maryland, U.S.A., November, 1988.

4. Kulak, Geoffrey L., Fisher, John W., and Struik, John H.A., Guide to Design Criteria of Bolted and Riveted Joints, Second Edition, Wiley-Interscience, 1987, Chapter 15.

5. Thornton, William A., Bracing Connections for Heavy Construction, Engineering Journal, AISC, Third Quarter 1984, Vol.21, No. 3, pp.139-148.

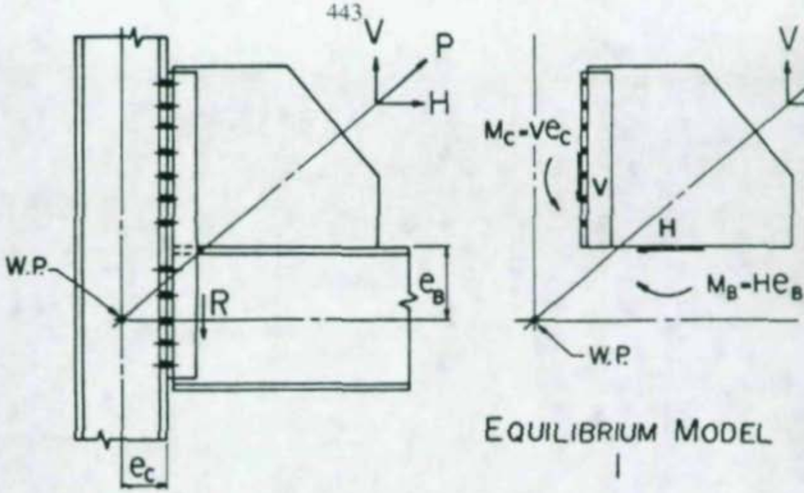


Fig. 1
The Simplest Equilibrium Model, Model 1

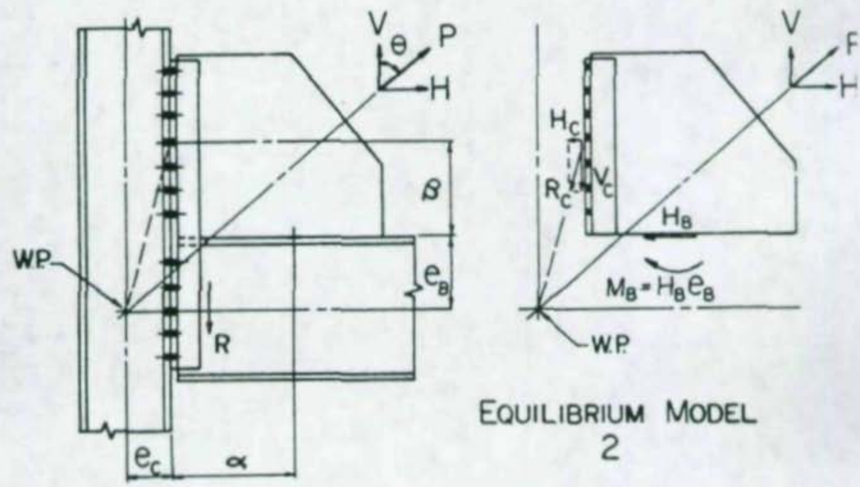


Fig. 2
Model 2, an extension of the AISC Model

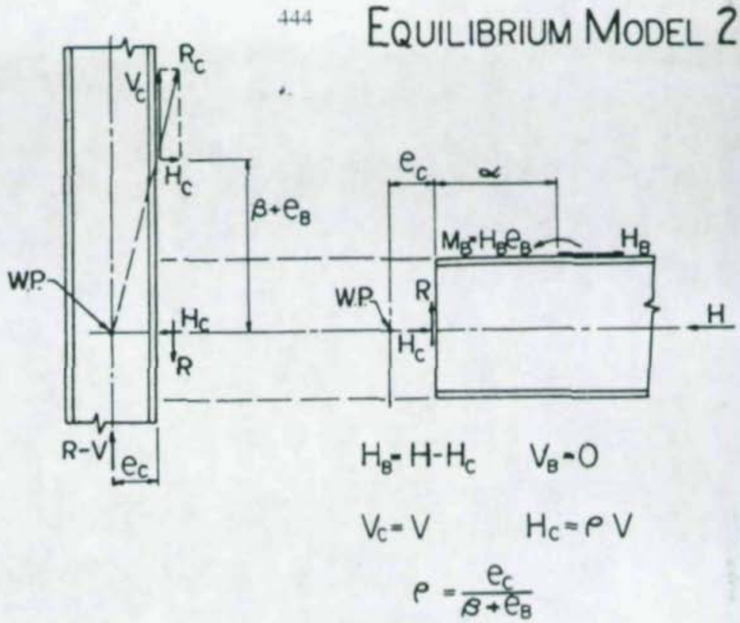


Fig. 3
Force Distributions for Model 2

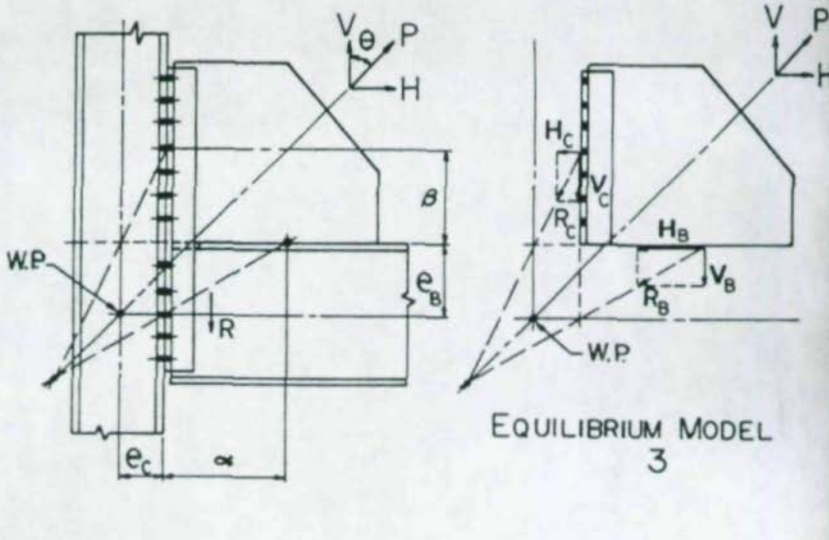


Fig. 4.
Model 3, an equilibrium model with no couples

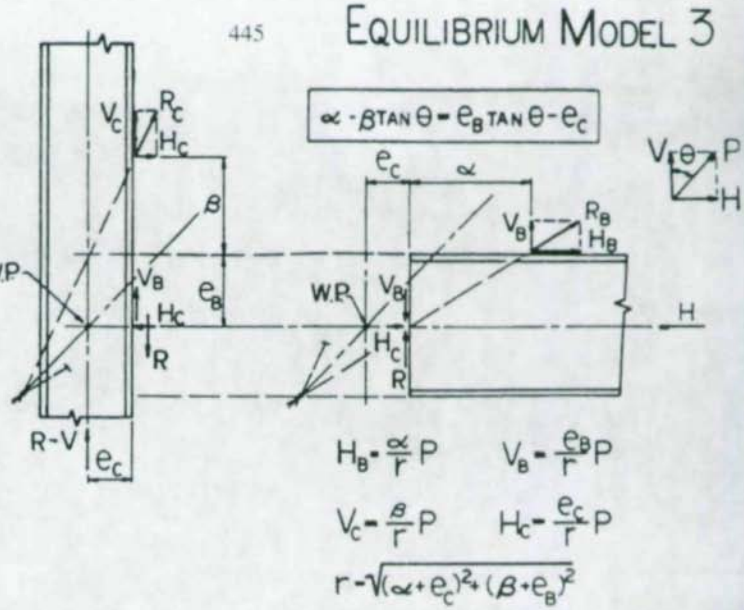


Fig. 5
Force distributions for Model 3

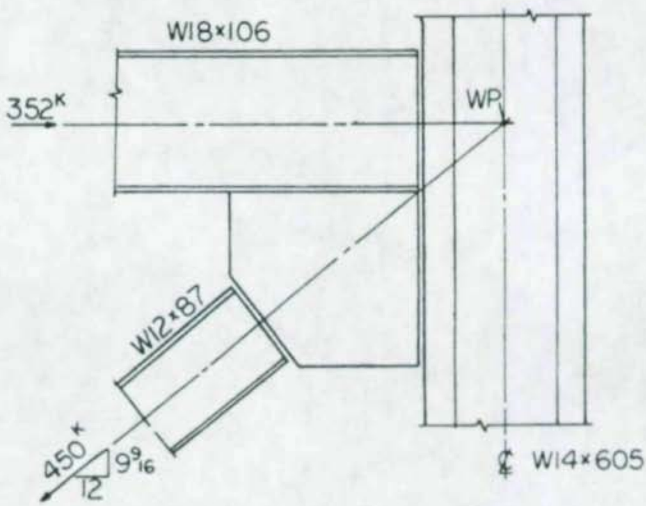


Fig. 6
Data for Illustrative Example connection

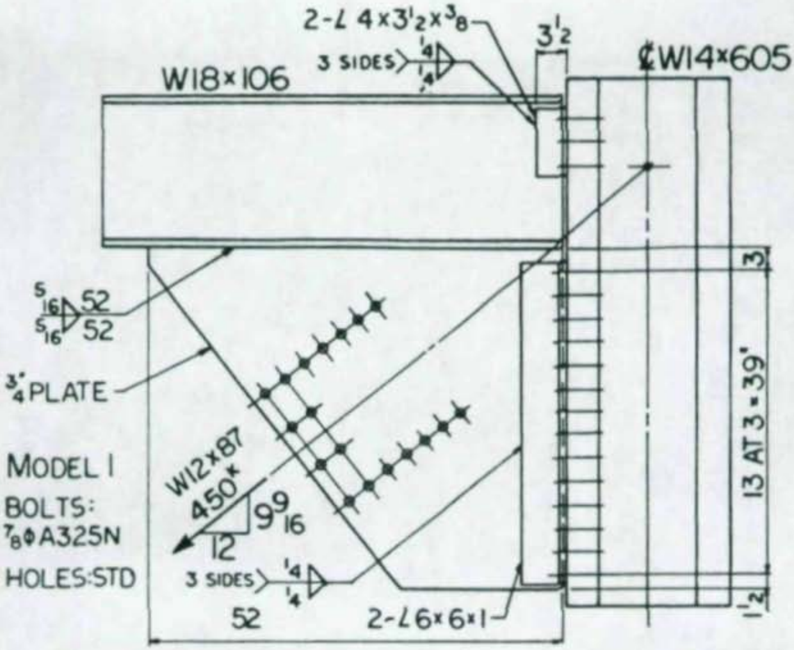


Fig. 7
Solution to Example connection using Model 1

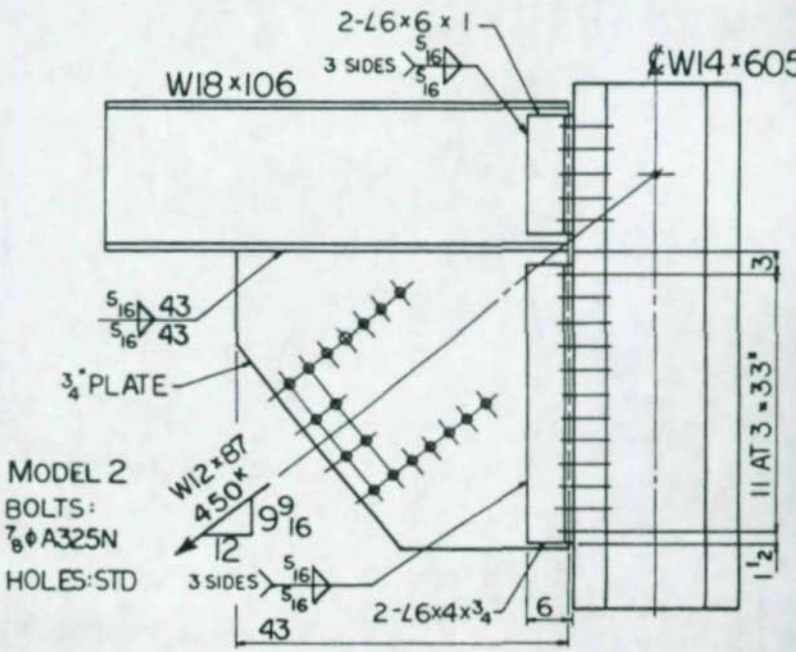


Fig. 8
Solution to Example connection using Model 2

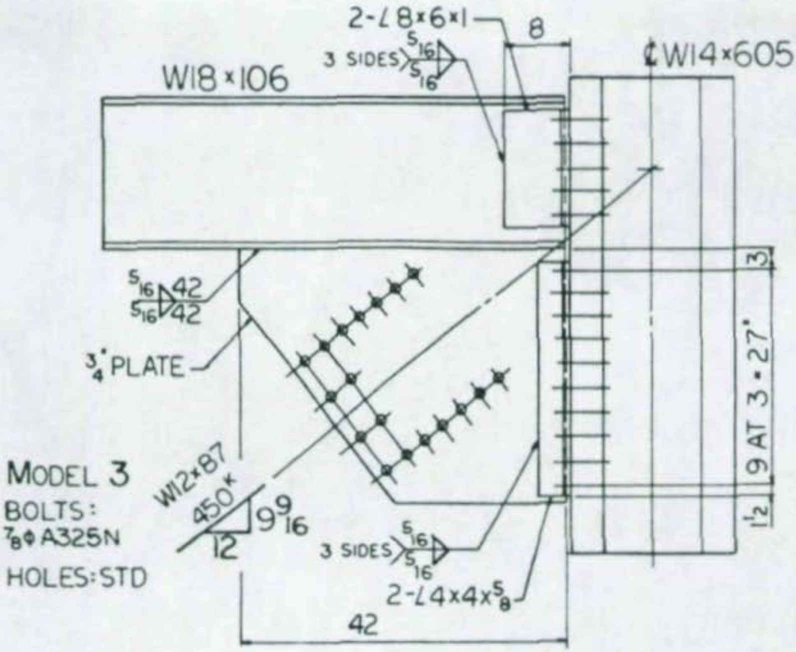


Fig. 9
Solution to Example connection using Model 3

Semi-Rigid Lightweight Steel Frame SKELETON

František Wald

Abstract

The paper is aimed at showing an application of the semi-rigid steel frame in Czechoslovakia. The constructional system is based on cold formed beams and columns, as well as semi-rigid joints and column bases. The test results of joints and column bases are included.

1. STRUCTURAL SYSTEM

The steel frame system SKELETON was designed for lightweight single-story buildings used as shopping centers, offices and schools. On Fig. 1. we can see the design of a shopping mall (Bílek at al., 1990).

Fig.1. Frame System SKELETON, shopping mall (Bílek at al.,1990)

Assistant Professor, Czech Technical University, Thákurova 7, 16629 Prague 6, Czechoslovakia

The structural modulus is range between 3.0 and 5.4 m and the construction height between 3.0 and 3.9 m. The roof is designed as flat or with inclination 30%. The columns are made of square hollow sections, 100x100x3.5 mm in size. The beams are designed as cold formed double C the height from 120 to 240 mm, sheet 4 mm. The joints are calculated with preloaded bolts M12.

2. FRAME DESIGN

The structure was designed according to the Czechoslovak standard (CSN 731401/1986). The frame was calculated to satisfy the sway rigidity under a service load. The semi-rigid joints and column bases were taken into account.The first order elastic model with the secant stiffness of joints (JASPART and MAQUOI, 1990) was used for the calculation. The second order elastic calculation (EUROCODE 3, 1989) including the nonlinear joint behavior as a power model was used to check the ultimate limit state requirements (Wald, 1990). The local stability of thin tubes limited the design. The stability check through the second order calculation with the initial stiffness of joints was found in a good agreement with the method based on a modified moment of inertia for beams (BARACAT and CHEN, 1990).

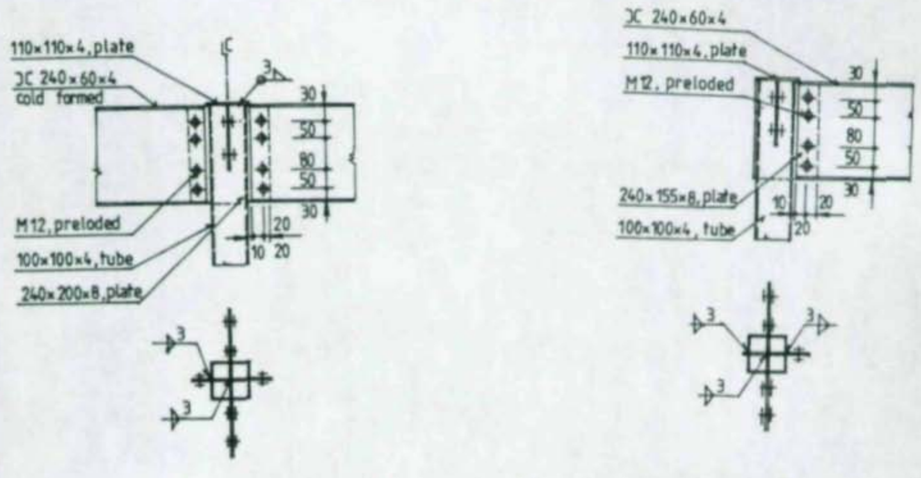


Fig.2. Beam-to-Column Joints

3. JOINTS

The joints are designed with 8 mm connection plate and preloaded high strength bolts M 12 (fyb=500MPa). Fig.2.. The contact surface was cleaned by wire brushing without any blastening. The semi-rigid behavior of joints was predicted from the polynomial expression (FRYE and MORRIS, 1983), interaction influence (ASTANEH, 1990) and experiments (KISHI and CHEN, 1989). The major details were tested in University Lab, Fig.3. The external beam-to column joints were loaded on a cruciform testing stand, Fig.4.

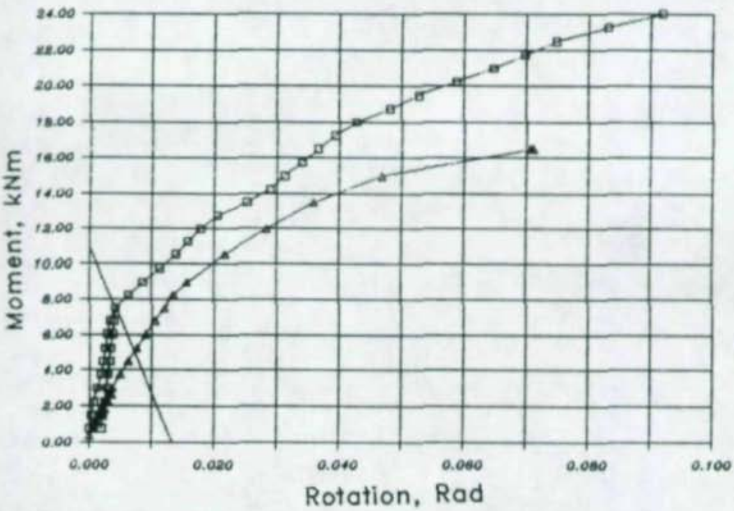


Fig.3. Moment-Rotation Diagram: ▫ Internal Column.▲External Column.-Beam-Line

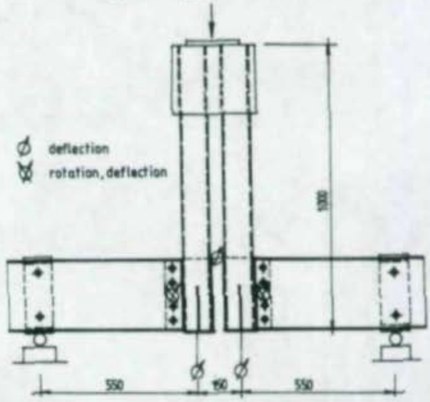
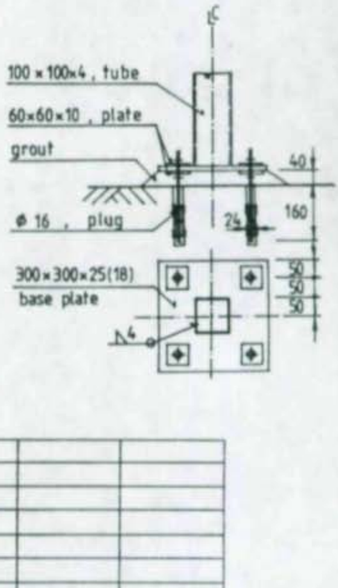


Fig.4. A Cruciform Test Setup for an External Beam-to-Column Joints

4. COLUMN BASES

The column bases were designed with a simple base plate 300x300x25mm without a stiffener, Fig.5. The base bolts, 18 mm, are anchored to a concrete block with steel plugs (Kotevni Technika Praha Comp.). The plugs were fixed through prestressing of wedge-shaped rings into a base concrete block. The stiffness was predicted from the exponential model (MELCHERS, 1987) and experiments (PICARD at al., 1987). The experiments were carried out to check the minimum thickness of the base plate, Fig.6. The test showed great sensitivity of the plug slip on the quality of concrete.

Fig.5. Column Base



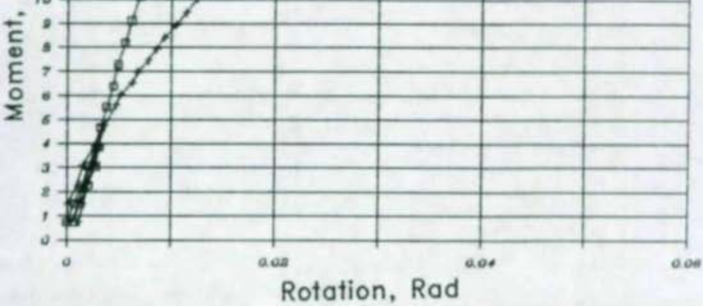


Fig.6. Moment-Rotation Diagram: ▫ Base Plate 24mm, + Base Plate 18mm.

5. CONCLUSION

The use of the lightweight constructional system SKELETON is limited on one side by the mobile house cell systems developed for similar purposes and on the opposite side by classical hot-rolled steel structures. This system seems to be competitive for a number of different types of low buildings through a proper complex detailing, including semi-rigid design, as well as erection advantages and speed owing to small weight.

Acknowledgement

The work described herein was carried out in the Lab of Czech Technical University . The author is indebted to thanks Prof. V. Bílek and Dr. Z. Langer, the designers of the complete system for the support. The sponsorship PULS Prague Comp. is gratefully acknowledged.

References

- ASTANEH A., (1989): Demand and Supply of Ductility in Steel Shear Connections, J.Construct. Steel Research, No 14, pp. 1-19
- BARAKAT M., CHEN W.F., (1990): Practical Analysis of Semi-Rigid Frames, Engineering Journal, No.2, pp. 54-68
- BÍLEK V., HORKÝ J., LANGER Z., LORENZ P., (1990): Skeleton, Light Structural System for Low Buildings, in Czech, CVUT, Prague
- FRYE M.J., MORRIS G.A.,(1975): Analysis of Flexibly Connected Steel Frames, Canadian Journal of Civil Engineering, Vol.2, pp. 280 - 291
- JASPART J.P., MAQUOI R., (1990): Guidelines for the Design of Braced Frames with Semi-Rigid Connections, in International Colloquium Stability of Steel Structures, Vol.II, Budapest, pp. 197-204
- KISHI N., CHEN W.F., (1989): Data Base of Steel Beam-to-Column Connections, AISC Engineering Journal, Vol.24, pp. 89-106
- MELCHERS R.E., (1987): Modelling of Column-Base Behavior, Connections in Steel Structures, ed. Bjorhovde R. at al., Elsevier Applied Science, London, pp. 150-158
- PICARD A., BEAULIEU D., PERUSSE B., (1987): Rotational Restraint of a Simple Column Base Connection, Canadian Journal of Civil Engineering, Vol.14.No.1, Feb., pp. 49 - 57
- WALD F., (1990): Semi-Rigid Frame Design, in Czech, Ing. stavby, No.6., pp.355-359

CURRENT RESEARCH NEEDS

FOR

CONNECTIONS IN STEEL STRUCTURES

INTRODUCTORY COMMENTS

The research and development need subjects that are given in the following were determined from the technical papers that were presented and the extensive discussions that took place during the workshop. The listing represents the essence of the work of the Research Reporters. Although no detailed explanatory notes are provided with each topic, it is felt that the subjects give a realistic reflection of the work that needs to be done.

The research topics are listed in the order of the technical sessions of the workshop, and are not prioritized in any way. Several of the topics are very broad in scope; these may entail a number of actual projects.

RESEARCH AND DEVELOPMENT NEEDS

1. **Bolts**

   (a) Tension-shear interaction for slip-critical joints, with respect to bolt preload

   (b) Long joint strength reductions: Effects of bolt diameter and grade of steel

   (c) Quality assurance of bolt materials and their installation, with respect to fitness for purpose

   (d) Applications of snug-tight bolts in shear connections and end-plate moment connections

   (e) Field monitoring of actual bolt preloads for various installation methods

2. **Welds and Local Strength Design Considerations**

   (a) Recognition of actual fillet weld strength for design, particularly for transversely loaded welds

(b) Strength of fillet welds with undermatched electrodes

(c) Shear lag effects in welded connections

(d) Cyclic loads on welded connections in construction with tubular members

3. **Predesigned and Special Connections**

(a) Initial stiffness of simple shear connections for serviceability checks

(b) Internal force and stress distribution in connections at service and ultimate limit states

(c) Design of semi-rigid connections, including seismic considerations

4. **Composite Connections**

(a) Rotation capacity demands and capabilities of connections in semi-rigid composite construction

(b) Moment redistribution criteria for semi-rigid composite construction

(c) Improved modeling of force transfer in composite connections using profile shapes

(d) Rotation capacity of beam web in negative moment regions, particularly considering the effects of compressive areas

(e) Further studies of classification schemes for beam-to-column connections

(f) Further studies of moment-rotation characteristics of composite connections

(g) Effects of residual stress and strain-hardening in composite connections under cyclic loads

5. **Semi-Rigid Connections**

(a) Reliability of semi-rigid connections, including representative parameters of variation for each connection type, and how to incorporate variability into design approaches and codes

(b) Further studies on the behavior and strength of the tension zone of end-plate and top- and seat angle connections

6. **Available Connection Software**

(a) Applications of expert systems concepts to the full range of connection behavior and design

7. **Global Behavior of Semi-Rigid Connections**

(a) Reliable methods of definition of moment-rotation curves for design applications

(b) Use of design vs. actual moment-rotation curves for various types of frames, considering serviceability and ultimate limit states

(c) Practical methods of connection classification, considering stiffness, strength and ductility, for a range of frame types

(d) Effects of joint geometry on strength and stiffness of tubular constructions

(e) Improved finite element modeling of connections, including hysteretic behavior of bolts

(f) Experimental and analytical models for structure-cladding interaction, including fastening systems and experimental verification of theoretical models

(g) Range of applicability of simple methods of frame stability

(h) Damping effects of cladding during seismic events

8. **Examples of Frame Design**

(a) Methods of calculating service load deflections that take stiffness of collateral elements into account

(b) Modification of connections to alter dynamic characteristics

9. Economy of Design

(a) Develop compendium of relative costs of different types of connections

(b) Cost comparisons of hot-rolled and cold-formed beams and columns in actual structural applications

(c) Design of gusset plates for compression

APPENDIX

Attendees of the

Second International Workshop

on

Connections in Steel Structures

Pittsburgh, Pennsylvania, U S A

April 10 to 12, 1991

WORKSHOP ATTENDEES

Pittsburgh, Pennsylvania, U S A

April 10 to 12, 1991

Dr. Bjørn Aasen
Norwegian Group of Steel Construction, Oslo, **Norway**

Dr. David Anderson
Department of Civil Engineering, University of Warwick, Coventry, **England**

Professor Jean Marie Aribert
Department of Civil Engineering, INSA - Rennes, Rennes, **France**

Professor Abolhassan Astaneh-Asl
Department of Civil Engineering, University of California, Berkeley, California, **U S A**

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