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Structural Properties of Semi-Rigid Joints in Steel Frames

Analyse d'assemblages semi-rigides dans des cadres métalliques

Eigenschaften halbsteifer Knoten in Stahlrahmen

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(refer also to page 31)

SUMMARY
This paper describes the various methods of modelling the moment-rotation relationship of beam-to-column connections. Detailed rules are given and explained for the strength of sub-components and the calculation of rotational stiffness, moment capacity and rotational capacity as has been laid down in Eurocode No. 3 – Design of Steel Structures.

RÉSUMÉ
Cet article décrit les diverses méthodes de modélisation des courbes moment-rotation caractéristiques des assemblages poutre-colonne. Des règles de calcul détaillées de la résistance des parties d'assemblage sont fournies, ainsi que les méthodes de calcul de la rigidité rotationnelle des assemblages et de leur capacité à reprendre des moments de flexion, telles qu'elles apparaissent dans l'Eurocode No. 3 – Dimensionnement des structures en acier.

ZUSAMMENFASSUNG
Der Beitrag stellt verschiedene Methoden zur Beschreibung der Momenten-Rotationsbeziehung von Stützen-Riegelverbindungen vor. Zur Bestimmung der Festigkeit jedes Bauteils werden Formeln angegeben. Die Berechnung der Steifigkeit, der Momententragfähigkeit und der Verdrehfähigkeit wird mit den im Eurocode No. 3 angegebenen Formeln durchgeführt.

1. INTRODUCTION

In structural steelwork the joints between members play an important role. From an economic point of view the costs for design and fabrication form a considerable part of the total costs. From a structural point of view the properties of the joints fundamentally influence the response of the structure to actions. This is clearly illustrated in the IABSE Survey "Analysis and design of steel frames with semi-rigid connections", prepared by ECCS-TWG 8.2 [1]. In the past, when 'working stress' design was normally used joint design was based on rather simple though not necessarily economical assumptions.
The joints were assumed to behave either as hinges (simple construction) or as infinitely stiff (rigid construction). The forces on the joints then followed from an elastic analysis of the structure. The parts of the joints such as end plates and angles, welds and bolts, could subsequently be dimensioned. Even now this design procedure seems to be used in the majority of cases.
However, the introduction of limit state design, including practical rules for plastic design, requires a more realistic treatment of the joints. When using these methods, the designer is confronted directly with the fact that for a better insight into topics such as the stability of columns and frames and for a minimum cost design of members and joints proper understanding of the behaviour of joints is essential.
Another factor is that modern computer programs, now available to the majority of designers, allow a more sophisticated treatment of joints without an appreciable increase in calculation costs. Finally, also the use of automatic NC drilling and sawing equipment in the fabricators shop influences the cost relationship between various forms of connections, leading to a need to minimise the number of welded stiffeners.

Modern design codes, e.g. Draft Eurocode 3, allow the design of the structure to be based on the actual load deformation characteristics of the joints. It is, of course, essential that the designer be able to determine those characteristics, preferably in the form of a mathematical model.
This report reviews available information on this point. It is drafted in liaison with ECCS-TWG 8.2 and should be considered as complementary to [1] dealing with analysis and design of steel frames.
Therefore emphasis is given to beam-to-column connections in steel frames, although much of the information is of more general use.

2. DEFINITION OF PROPERTIES OF JOINTS

2.1 Structural parameters

A steel frame, whether considered as a plane or spatial structural system, essentially consists of linear members joined together by connections (Fig. 2.1).

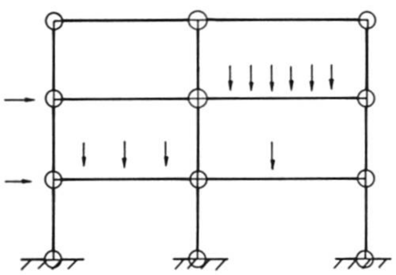


Figure 2.1. Combination of linear members and connections in a plane structural system.

The response of the system to actions (loads) is in general influenced by the structural properties of both members and connections. The relevant properties of these elements are strength, stiffness and deformation capacity (ductility).

Assuming for the present that bending is dominant, all three parameters can be presented in a moment-rotation curve of the type illustrated qualitatively in Fig. 2.2.

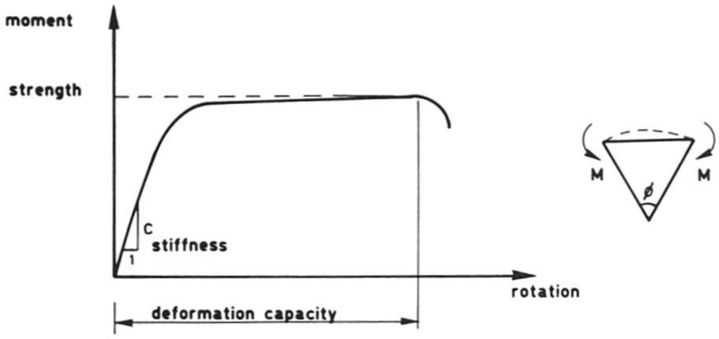


Figure 2.2. Presentation of structural parameters in a M-ϕ curve.

The properties of a member with parts in compression are dependent upon section geometry and the yield stress of the steel mainly due to the effect of local buckling.
In most codes the different types of cross-section are classified dependent on their respective moment-rotation properties.
Depending on the class of the cross-section different analysis models may be used. Fig. 2.3 shows different forms of moment-rotation characteristics of cross-sections.

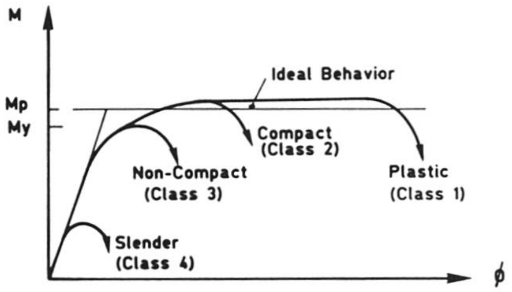


Figure 2.3. Moment-rotation relationships of cross-sections.

As an example the possibilities for ultimate limit state verification according to the Draft Eurocode 3 are given in Table 1.

Class of member	Capacity of cross-section	Analysis of system
Class 1	plastic	plastic
Class 2	plastic	elastic
Class 3	elastic	elastic
Class 4	reduced stress or effective section	elastic

Table 1.

The structural properties of connections can also be presented in a M-ϕ diagram. In Fig. 2.4 a set of M-ϕ curves for connections with different types of behaviour are shown.

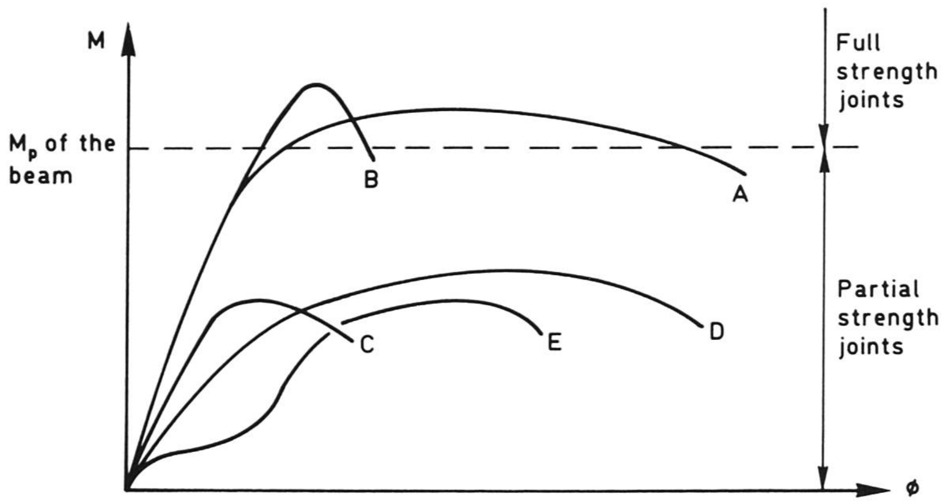


Figure 2.4. Moment-rotation curves of beam-to-column connections.

For the use of plastic design the connections can be classified in two categories:

- Full strength joints (Connections A and B in Fig. 2.4)
  The moment capacity is not less than that of the member. A plastic hinge will not be formed in the connection but in the member adjacent to the connection. In theory no rotation capacity is required for the connection. If the connection has little deformation capacity (connection B) an extra reserve of strength should be required to account for possible overstrength effects in the member.

- Partial strength joints (Connections C, D and E in Fig. 2.4)
  The moment capacity is less than that of the member. A plastic hinge will be formed in the connection, so sufficient rotation capacity is required. For this reason connection C is unsuitable. Curve E is typical for a bolted bearing type connection, showing slip due to the clearance of the holes.

In all cases connections have to meet requirements for minimum stiffness (rigidity).
For a more detailed treatment of the requirements reference is made to chapter 5 of the Survey on Frames [1]. This explains how, especially when using partial strength joints in plastic design, information on all three parameters of the M-ϕ relation is required.
In elastic design traditionally two categories of connections were considered:

- Nominally pinned connections (Fig. 2.5)
  The connections are assumed to transfer only the end reaction of the beam (vertical shear force and eventually normal force) to the column.

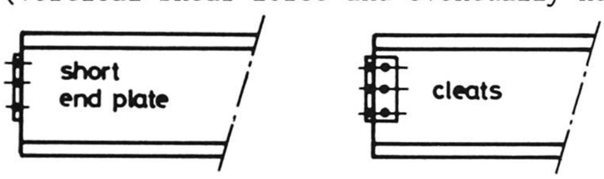


Figure 2.5. Nominally pinned connection.

They should be capable of accepting the resulting rotation without developing significant moments, which might adversely affect the stability of the column. In many countries it is common practice to design structures on a simply supported basis and then to provide connections which are in effect semi-rigid. A typical example is the flush end plate detail shown in Fig. 2.6.

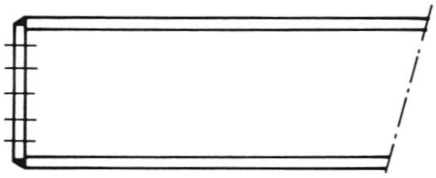


Figure 2.6. Flushed end plate connection.

- Rigid connections (Fig. 2.7)
  Rigid connections are used to transfer moments as well as end reactions. Design assumes that the joint deformation is sufficiently small (stiffness large), that any influence on the moment distribution and the structure's deformation may be neglected.

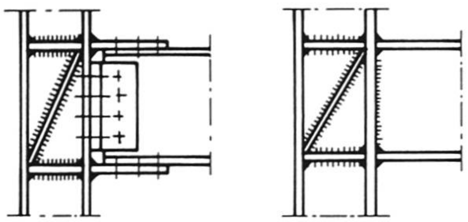


Figure 2.7. Rigid connections.

Practical realisation of the assumptions of rigid construction often leads to relatively expensive connection details. Thus in cases where stiffness is not required for stability reasons (e.g. in braced frames), the use of nominally pinned connections is usual. However, practical forms of this type will transfer some moments and contribute to the structure's stiffness. On the other hand, rigid connections can be simplified and made less expensive by the omission of stiffeners and other parts and accepting some flexibility.
To fill the gap between pinned and rigid connections, a third category is defined and accepted in most modern codes.

- Semi rigid (semi flexible) connections (Fig. 2.8)
  These connections are designed to provide a predictable degree of inter-action between members based on actual or standardized design M-ϕ characteristics of the joints.

Figure 2.8. Semi rigid (semi flexible) connections.

In conclusions it seems to be evident that there is a need to present information on the M-ϕ relations of joints in a form which can be readily used by designers.
This report provides a survey of current knowledge and indicates needs for further studies.

2.2 Connection test data

At present connection M-ϕ data can only reliably be obtained from actual tests since theoretical approaches cannot properly allow for the complex interplay of all these factors present in a real connection e.g. slip, localised plasticity, bolt preload etc. Recently prepared surveys [2, 3, 4] of available test data for

connections between beams and columns permit the existance of suitable information on particular connection types to be checked. In the case of ref. [4] digitised versions of over 300 experimentally obtained M-ϕ curves covering 7 different connection types are provided. Also ref. [28] contains a data base of steel beam-to-column connection characteristics. These may be used directly. On the other hand ref. [2] and [3] provide tabulated details of available data, together with some indication of both the effects of different variables on the M-ϕ characteristic and an assessment of those items requiring further study.
Because of the current interest in semi-rigid joint action it is known that test series aimed at remedying some of the more important deficiencies are currently in progress (Liege, Milan, Sheffield).
Ref. [3] identified the 12 types of beam to column connection between I-section members illustrated in Fig. 2.9 as being used at least "sometimes" in practice. The amount and quality of available test data for each type is variable, with the most important variations being:

(i) Number of tests available - none for the bottom seat and web cleat arrangement to over one hundred for extended end plates.
(ii) Number of variables present in the joint - some connections contain so many components that complete coverage would require an enormous number of tests.
(iii) Method of defining M and ϕ - this will be a function of the test set-up and measurement systems employed; frequently it is not clear from the original source exactly how ϕ is defined.
(iv) Quality of reporting - it is suspected (and indeed known in certain cases) that M-ϕ data were obtained during the testing but were not provided in the report.

Fortunately those arrangements which appear to be most popular (Fig. 2.9 type 2, 6/7 and 9) appear to have received the most attention. Thus whilst gaps in coverage inevitably exist, it is possible to use the available data to make reasonable assessments of the M-ϕ performance of these types in most cases.

2.3 Analytical methods for the description of the M-ϕ relationship

Inclusion of joint flexibility effects into the analysis of the behaviour of members or complete frames requires that joint M-ϕ curves be capable of representation in a convenient mathematical format. Table 2 lists some of the schemes which are available to describe the M-ϕ data referred to in the previous section. Early (pre-computer) schemes using linear representations only cover initial connection stiffness. As such they are not suitable for full-range analysis of behaviour, seriously over-estimating the connections stiffness at all but the lowest levels of joint rotation.
Certain forms of connection, e.g. web side plate, do actually possess M-ϕ characteristics which correspond quite closely to a bilinear curve. Although some studies of column strength based on attainment of the peak load suggest that using a bilinear M-ϕ curve might lead to acceptable results, it is by no means certain that such an M-ϕ representation will always prove sufficient.
The most important development in modelling M-ϕ curves was the contribution of Frye and Morris [5] who first suggested the use of polynomials and who employed curve-fitting techniques to obtain best fit solutions. Analytical difficulties associated with the negative slope of polynomial curves which can occur with certain sets of data, prompted the later use of the B-spline technique [6].
Several alternative sophisticated models have also been proposed in recent years, e.g. Ramberg-Osgood formula [7], Richard [8] and Yee and Melchers [9]. An appraisal of the accuracy of several of the available techniques is provided in ref. [4]. In table 2 the various methods of modelling M-ϕ relationship of connections are summarized.
More recently an ingenious semi-analytical method of modelling the M-ϕ response of certain forms of connection has been proposed by Tschemmernegg [10]. This splits the connection into the joint itself and the connection means from the beam to the joint as indicated in Fig. 2.10. The whole of the connection,

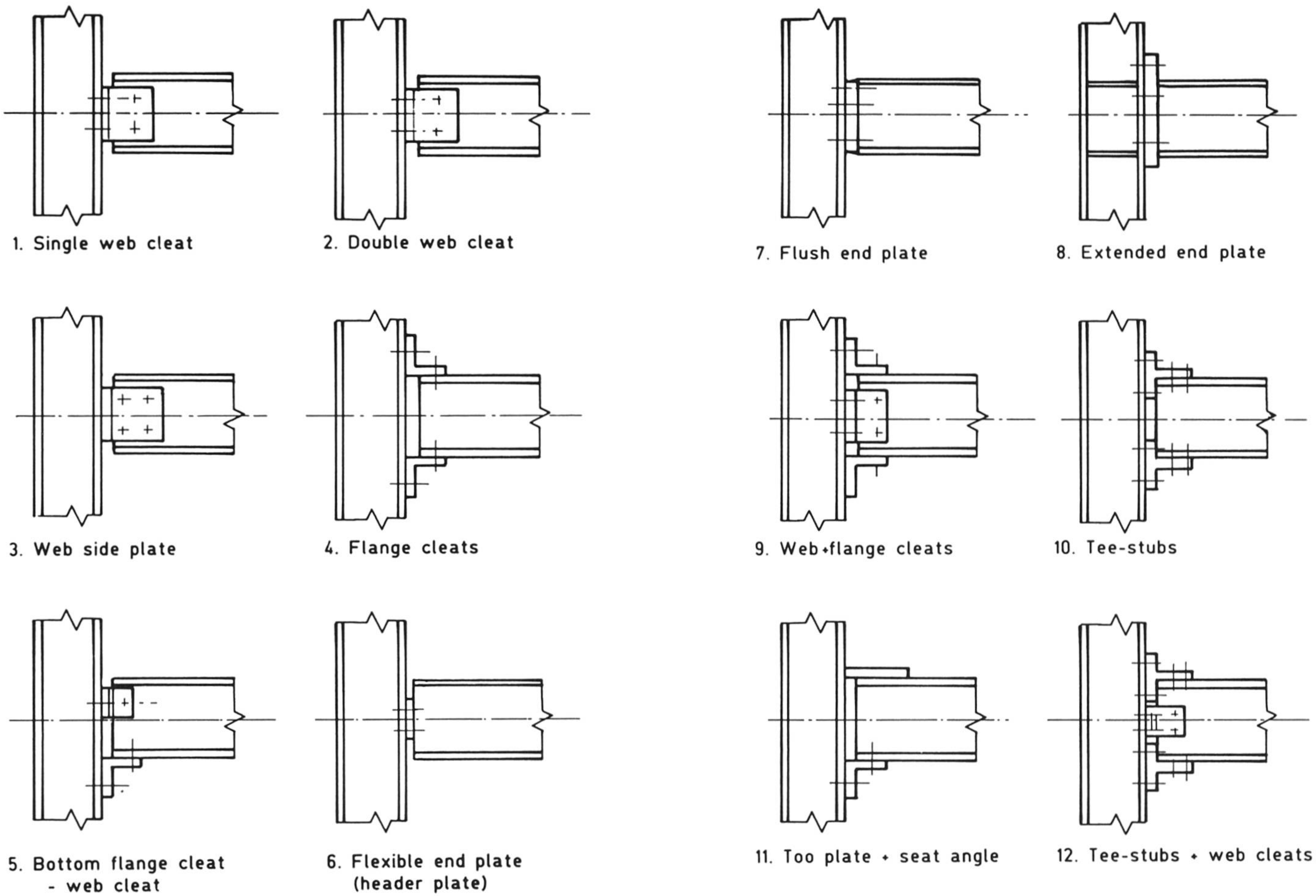


Figure 2.9. Common types of beam-to-column connections.

consisting of joint and connection means, is represented by the spring model with elastic-plastic springs as shown in Fig. 2.11.
Inside the joint, load introduction springs and a shear spring are assumed to act in series, while the connection means spring acts between the joint and the beam. The connection means spring and the load introduction spring resist the applied moments M_A and M_{A1} respectively, while the shear spring resists the moment difference $\Delta M = M_A - M_{A1}$. The capacity of the connection is taken as that of the weakest of the springs.
Fig. 2.12 shows that inside the joint, either the load introduction spring or the shear spring might be the controlling factor. It is assumed that the capacity of the connection means is sufficient.
It also shows how the connection capacity may be influenced by the presence of stiffeners.
In a connection several arrangements of connection means can exist, each of which will have a different spring characteristic. By adopting a completely stiff joint the spring characteristic of any of the different connection means can be determined.
The advantage of this model is that the individual contributions to the joint deformation and thus its flexibility of the shear spring, load introduction spring and connection means spring can be combined so that each of the principal factors which influence M-ϕ behaviour may be considered separately. In Innsbruck elastic-plastic characteristics of both the shear springs and the load introduction spring were determined experimentally. These were then combined with suitable connection means stiffnesses to provide the complete connection characteristic.

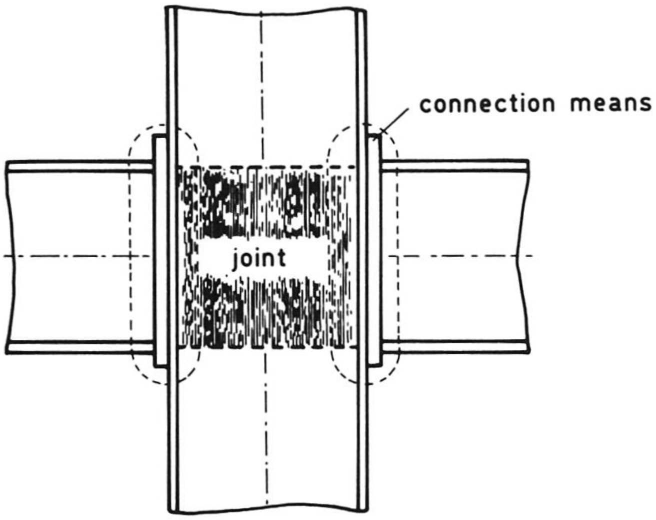


Figure 2.10. Beam-to-column connection.

Type of model	References	Year	Advantages	Disadvantages
- Linear	Baker Rathbun	1933	1. Simple to use 2. Stiffness matrix only requires initial modification	Inaccurate at high rotation values
- Bilinear	Lionberger & Weaver Romstad & Sumbramanian	1969 1970	1. Simply to use 2. Curve follows M-ϕ curve more closely than linear model	Inaccurate at some rotation values
- Polynomial	Sommer Frye & Morris [5] Radziminski et.al.	1970 1975 1982	Produce a close approximation to the shape of the M-ϕ data	1. Can produce inaccurate (even negative connection tangent stiffness values 2. Nonlinear requires iterative evaluation
- Cubic B-spline	Jones, Kirby & Nethercot [6]	1980	1. Produces a very close approximation to the M-ϕ data shape 2. Produces accurate values of connection stiffness	1. Nonlinear requires iterative evaluation 2. Requires special numerical procedures for evaluation
- Richard formula	Richard et.al. [8]	1980	Produces a good fit to the test data for single angle connections untried for other types but should be suitable	1. Nonlinear requires iterative evaluation 2. Requires weighted least squares evaluation
- Ramberg-Osgood	Ang & Morris [7]	1984	Produces a good fit to a variety of test data similar to type 3	1. Nonlinear requires iterative evaluation 2. Requires weighted least squares evaluation
-	Yee & Melchers [9]	1986	Semi-empirical based on theoretically determined limits	1. Nonlinear requires iterative evaluation

Table 2. Methods of modelling M-ϕ relationships of connections.

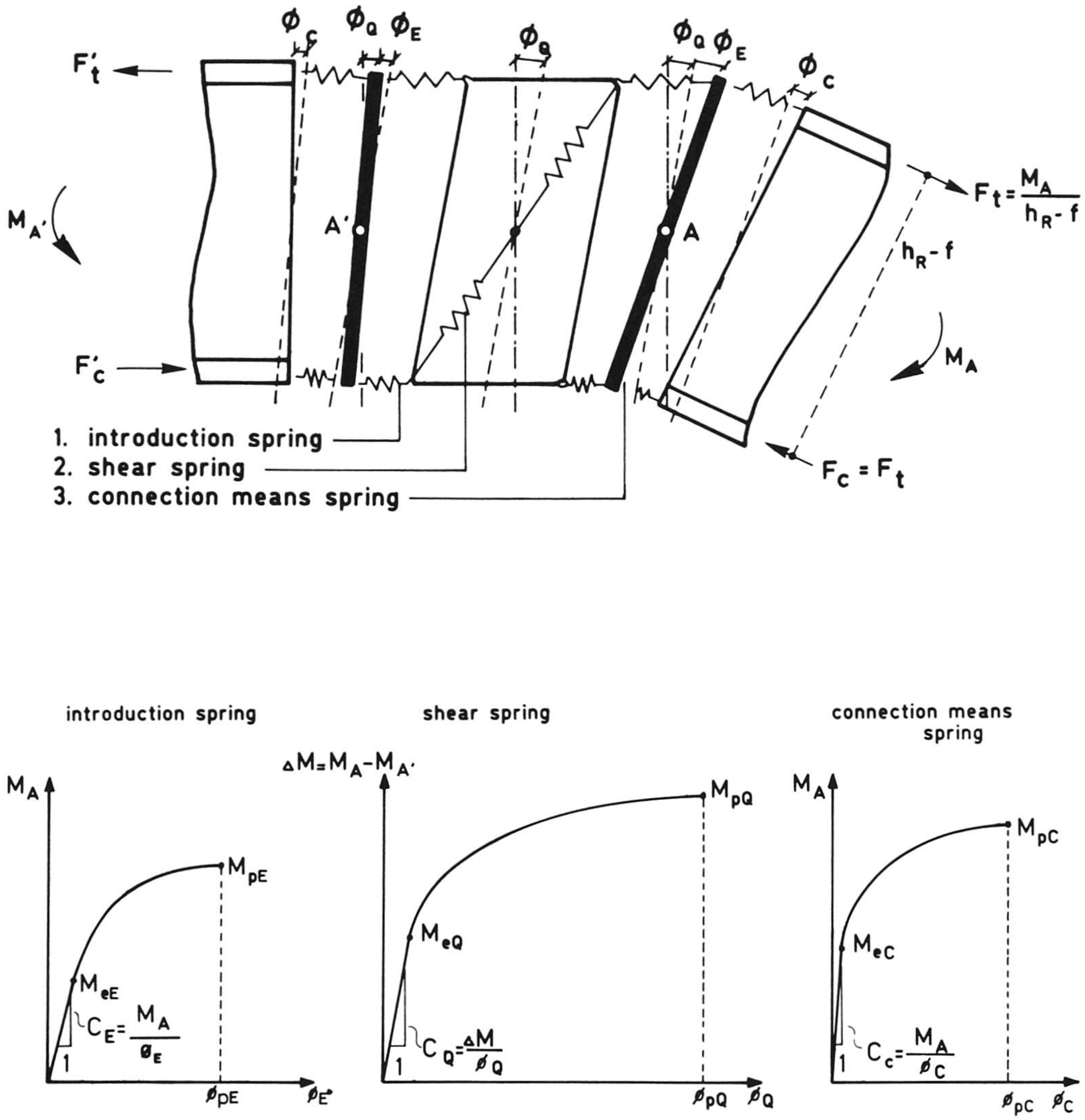


Figure 2.11. Spring-model.

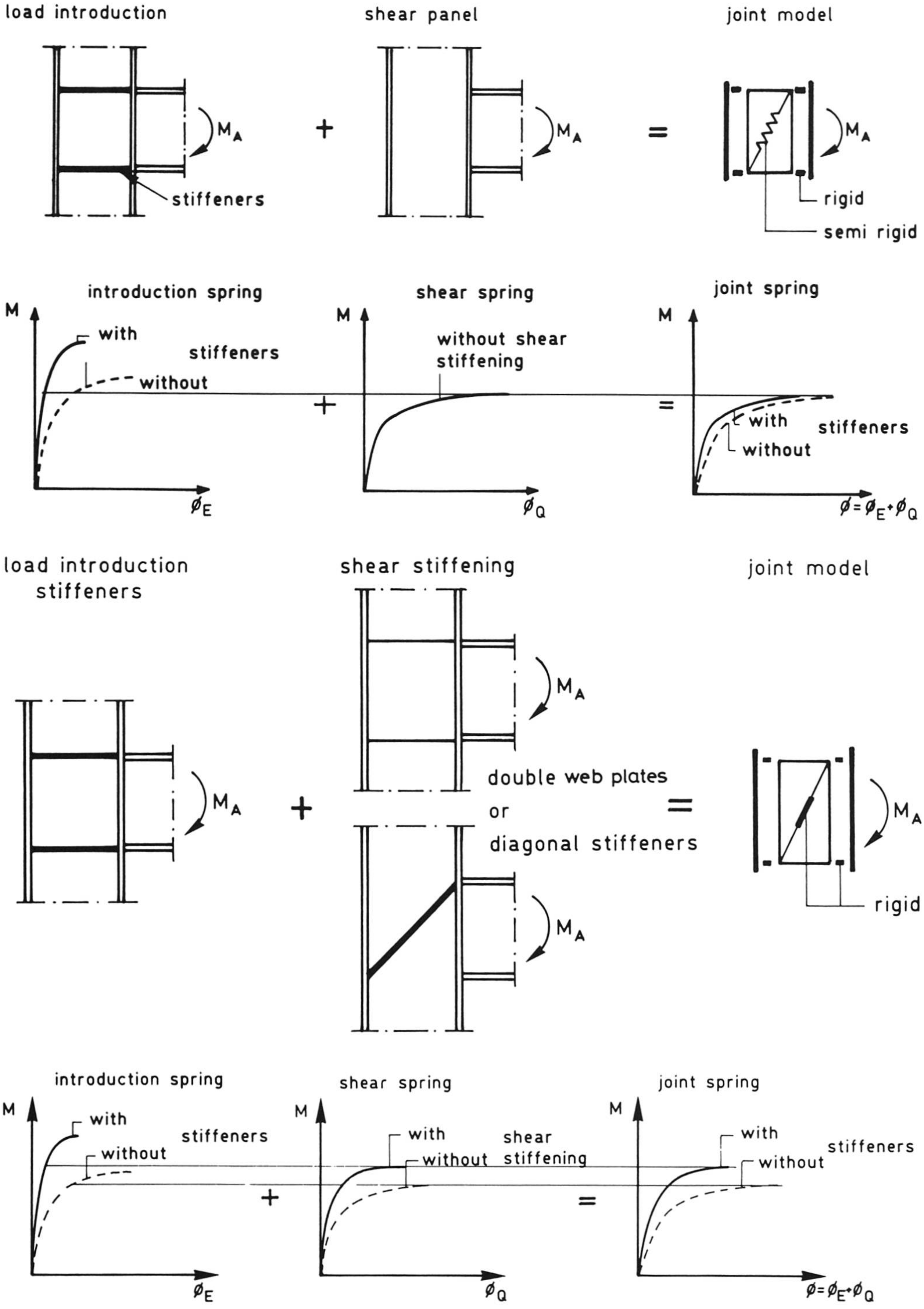


Figure 2.12. Influence of Stiffeners.

3. PROPERTIES OF SUB COMPONENTS

In this chapter the sub components will be discussed against the background of the strength and stiffness of the connection of which they form a part.

3.1 Fasteners

3.1.1 Strength

The strength of fasteners is dependent on the way they are loaded. The ultimate design strength can be taken from the Draft Eurocode 3 [11], which covers bolts up to and including grade 10.9.
Clearly the forces in the bolts due to the design load should not exceed the design capacity of the bolts. Where fasteners are used to carry an applied tensile load, they should be proportioned to resist the additional force due to prying action [23].
In the Draft Eurocode 3 [11] requirements are given for maximum end and edge distances and maximum pitch to prevent local buckling of compressed parts between the fasteners and to guard against corrosion at the interfaces.

Also requirements for minimum distances are given based on structural considerations.
In connections subjected to a shear force the bolts have to transmit the force by bearing and shear.
The bearing capacity of a bolt is in fact dependent on the edge distance e_2, the end distance e_1 or the distance p_1 (pitch) between bolts in a bolt group.
The design capacities of the bolts are only valid when the end distance e_1 from the centre of a fastener hole to the adjacent end of the part in the direction of load is not less than 1.2 d.

The bearing stress αf_u is experimentally determined as a function of the end distance e_1 or the pitch p_1 and with a constant edge distance e_2 = 1.5 d.
In Figs. 3.1 and 3.2 the resulting design rule for the factor α is presented and does only apply when the edge distance e_2 from the centre of a fastener hole to the adjacent edge of any part at right angles to the direction of load is not less than 1.5 d (where d is the hole diameter), and when the pitch is not less than 2.2 d.

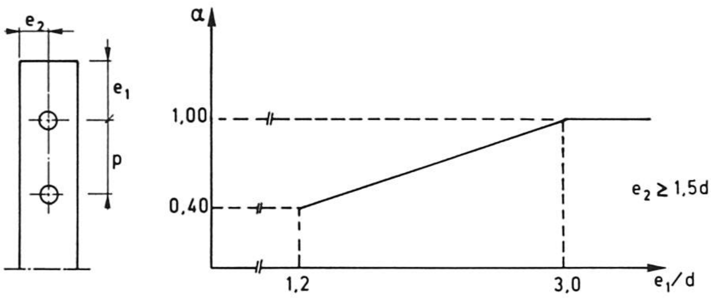


Figure 3.1. Factor α as a function of the end distance.

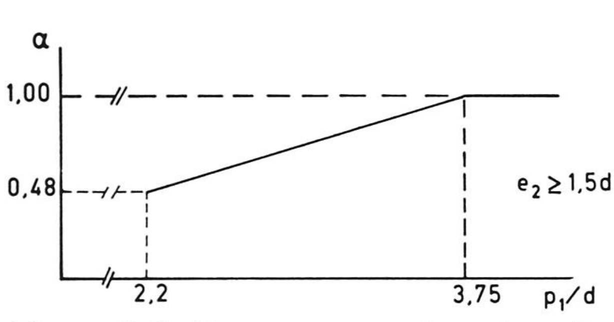


Figure 3.2. Factor α as a function of the pitch.

Normally an end distance $e_1 = 2$ d is used but for one-bolt connections it is sometimes advantageous to use a shorter end distance.
The minimum edge distance e_2 is taken as 1.5 d because there are not sufficient test results available from tests with a smaller edge distance.
A smaller edge distance can lead to failure of the net section prior to reaching the bearing stress according to the given formula. To allow for this it is suggested in [26] to multiply the factor with a factor 2/3 in case of an edge distance $e_2 = 1.2$ and so $1.2 \leq e_2 < 1.5$ d is permitted.
The design capacity of a bolt in shear or in tension has traditionally been based on the yield stress of the bolt material.

In the Draft Eurocode 3 [11] the capacity of the bolt is based on its ultimate strength, expressed in terms of the tensile strength f_u of the bolt material. This is more in accordance with physical reality since experimental evidence shows that bolts of normal quality fail in tension when the average stress in the critical cross-section reaches f_u.
Recently evaluations of test results have been carried out to obtain strength functions and suitable model factors for bolted connections [27] resulting in the following design resistances for bolts.

Tension resistance

$$F_{td} = \frac{0.9\, f_{ub}\, A_s}{\gamma_{Mb}}$$

Bearing resistance

$$F_{bd} = \frac{2.5\, \alpha\, f_u\, d_n\, t}{\gamma_{Mb}}$$

where α is the smallest of:

$$1.0,\ \frac{e_1}{3d},\ \frac{p_1}{3d} - \frac{1}{4} \text{ or } \frac{f_{ub}}{f_u}$$

Shear resistance per shear plane

if the shear plane passes through the unthreaded portion of the bolt

$$F_{vd} = \frac{0.6\, f_{ub}\, A}{\gamma_{Mb}}$$

if the shear plane passes through the threaded portion of the bolt

$$F_{vd} = \frac{0.6\, f_{ub}\, A_s}{\gamma_{Mb}} \text{ if } f_{ub} \leq 800\ \text{N/mm}^2$$

$$F_{vd} = \frac{0.5\, f_{ub}\, A_s}{\gamma_{Mb}} \text{ if } f_{ub} > 800\ \text{N/mm}^2$$

A model factor $\gamma_M = 1.25$ is recommended.

Bolts subject to both shear and tensile forces (F_v and F_t respectively) shall in addition satisfy the following interaction expression:

$$\frac{F_v}{F_{vd}} + \frac{F_t}{1.4\, F_{td}} \leq 1.0$$

When high-strength bolts in slip-resistant connections are used the resistance to slip can be taken as:

$$F_{sd} = k_s\, m\, \mu\, F_p/\gamma_{Ms}$$ with $\gamma_{Ms} = 1.25$ for ultimate limit state
and $\gamma_{Ms} = 1.10$ for serviceability limit state

where:
F_p = the preloading force = $k_p\, f_{ub}\, A_s$
μ = the slip factor (extensively specified in the Draft Eurocode 3 [11])
m = the number of friction interfaces
k_s = a factor dependent of the size of the holes in the plies connected by the bolt, varying between 1.0 for cases where the holes in all the plies are of normal size to 0.7 for long slotted holes with the long direction of the slot not perpendicular to the direction of the load.
f_{ub} = the specified ultimate tensile strength of the bolt material
A_s = the tensile stress area of the bolt
k_p = a factor dependent of the tightening method (k_p = 0.8 if the combined method is used and k_p = 0.6 if the torque controlled tightening using normal calibration or k_p = 0.7 if torque controlled tightening using more careful calibration or if turn of the nut method is used)
d_n = nominal diameter of the bolt
t = plate thickness
f_u = the specified ultimate tensile strength of the plate material
e_1 = end distance
p_1 = pitch

If a slip-resistant connection is subjected to tension loading in addition to the load tending to produce slip, the friction capacity per bolt should be taken as:

$$F_{sd} = k_s\, m\, \mu\, (F_p - 0.8\, F_t)/\gamma_{Ms}$$

where:
F_t = the externally applied factored tensile load

In the combination of tension and shear the preloading force F_p need not be reduced with the total external tensile force F_t. Because of the relative stiffness ratio between the bolt in tension and the plates in compression the contact force F_c between the plates is not reduced by the full external tensile force F_t (see Fig. 3.3).

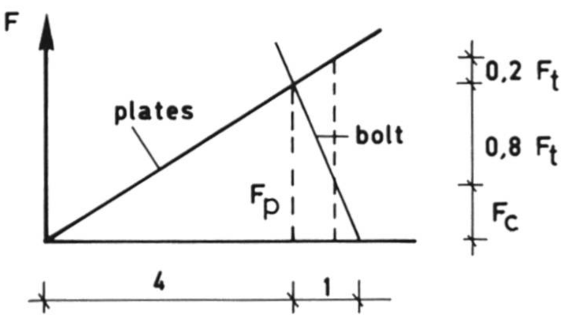


Figure 3.3. Load deformation curve of a preloaded bolt in a plate package.

Assuming a stiffness ratio for plates to bolt of 4 : 1 the contact force F_c can be calculated as:

$$F_c = F_p + 0.2\, F_t - F_t = F_p - 0.8\, F_t$$

With respect to fabrication details such as clearance of holes, the use of washers, tightening procedures for preloaded bolts etc., the Draft Eurocode 3 [11] provides detailed guidance on these matters.

3.1.2 Stiffness

The deformation of bolts acting in tension is negligible compared with the deformation of the surrounding parts such as column flanges and end plates which deform due to bending. Pretensioning the bolts leads to a decrease of deformation of these surrounding parts.

In bearing type connections with normal bolts a deformation will occur due to slip over the clearance of the holes.
Only high-strength bolts in slip resistant connections will not produce deformation in this way.

3.2 Fillet welds

3.2.1 Strength

The ultimate design strength of fillet welds can also be calculated in accordance with the Draft Eurocode 3 [11] and is dependent on the ultimate tensile strength of both the base material and the weld material.

3.2.2 Stiffness

The deformation within the welds themselves is negligible compared with the deformation of the surrounding parts such as the end plate, which will deform due to bending.

3.3 Parts

3.3.1 Strength

Tension side of a connection
- Design of bolts
  The bolts in the tension zone must be proportioned to resist any force Q due to prying action, which can occur, in addition to the applied tensile force T, as indicated in Fig. 3.4.

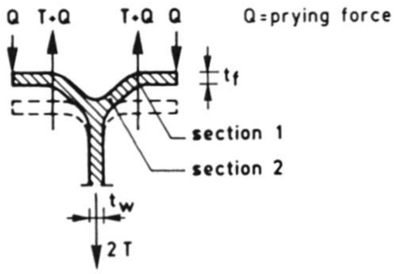


Figure 3.4. Prying forces.

- Column flange
  In order to understand the behaviour of the flange of a column in the tension zone of a connection, it is useful to consider the behaviour of T-stubs bolted together with four bolts (see Fig. 3.5). From tests [22] it appears that the sum of the forces in the four bolts is greater than the external force acting on the test piece. This is due to the deformation of the T-stubs, in which the edges of the flange are subjected to "prying-forces" which increases the load in the bolts. The magnitude of the "prying-forces" depends on the stiffness ratios between the flanges on the T-stubs and the bolts. This leads to three possible failure modes.

The first mode is dominant when the flanges are heavy in comparison with the bolts. Then flanges of the T-stubs separate from one another due to the plastic deformation of the bolts. The failure load is equal to the sum of the failure loads of the bolts (see Fig. 3.5a).
The second mode is dominant when the stiffness ratio of the flange and the bolts is such that "prying-forces" can develop. In this situation the bolts fail and yield lines develop in the flanges near the fillet between the flange and the web of the T-stubs (see Fig. 3.5b).
The third mode is dominant when the flanges deflect in such a way that yield lines develop in the flanges near the bolts and the fillet between the flanges and the web (see Fig. 3.5c).

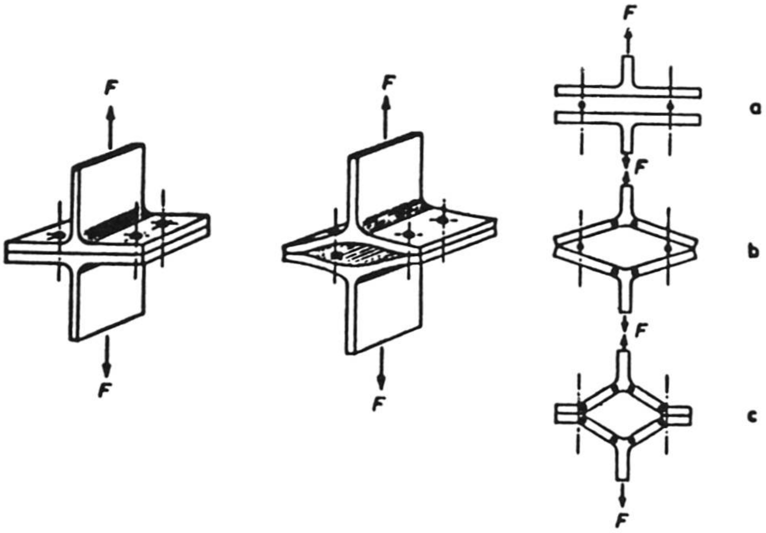


Figure 3.5. Tests on T-stubs.

Formulae may be derived [23] to determine the failure load of the column flange, the bolts and the end-plate of the beam in the tension zone.

From tests [22] it appears that the tension zones of the connections may act as shown in Fig. 3.6 with the actual behaviour pattern depending on:
- the ratio of bolt capacity to plate strength;
- the distribution of the bolt forces.

Plate strength depends on plate thickness and location of the bolts with respect to the plate supports.

The column flange is supported by the column web and the stiffener, a situation comparable to the beam web and beam flange supporting the end plate. Thus the strength of the end plate and the column flange may be determined by the same method of calculation.

Tests [22] have shown that in the region between the beam flanges a yield line pattern develops at first around the bolt which is located nearest to the tension flange of the beam and the beam web. If another bolt is added, the yield lines extend to it, but the force in the first bolt remains the same as that corresponding to the original yield line pattern. This effect has been confirmed by bolt force measurements in tests [12] and [13]. The consequences of this for design are indicated schematically in Figs. 3.6 and 3.7.

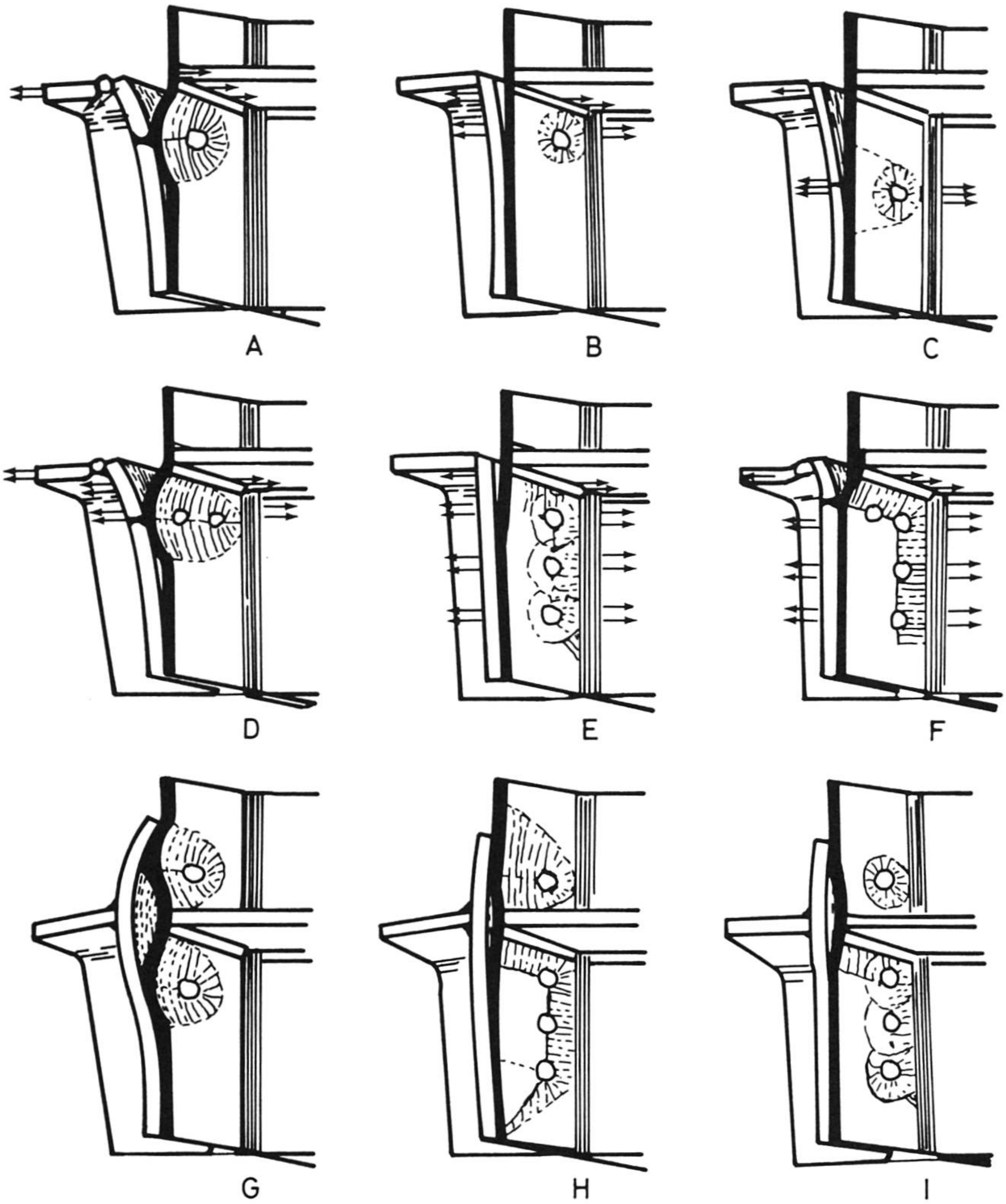


Figure 3.6. The effectiveness of added bolts depends on the yield line pattern formed by the first bolt.

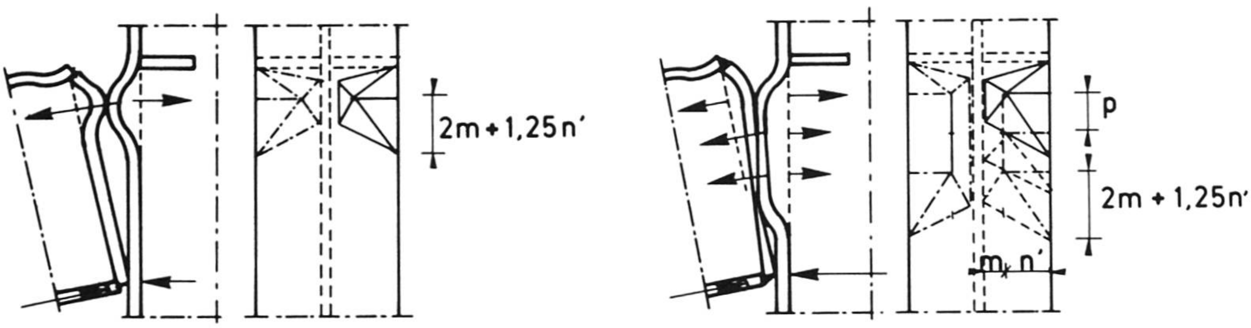


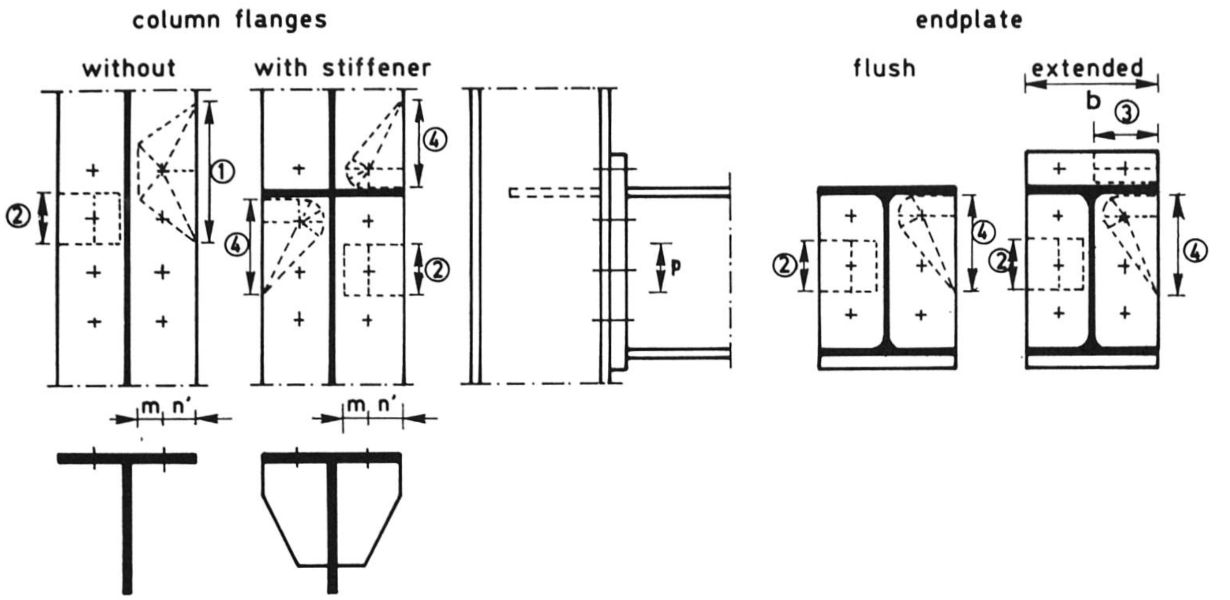
Figure 3.7. Extension of the yield line pattern proceeds from the first row of bolts. The effective length is limited by the dimensions of the end plate or column flange.

It is possible that a second bolt, e.g. as shown in Fig. 3.6D, does not contribute, because it has been placed within the yield mechanism formed by the first bolt.
To start with, the design strength of the plate with one bolt should be determined, as indicated in Figs. 3.6A, 3.6B and 3.6C. Next, the effect of adding a bolt should be considered.
This can be done with formulae and diagrams of Fig. 3.8 and 3.9 at first. The effective length of a mechanism (b_m) of a plate with one bolt is determined from the formulae and diagram of Fig. 3.8.

The effective length of a mechanism involving more bolts is equal to the sum of the effective lengths found with Fig. 3.8. The design strength of the combination of plate and bolt group is the lowest value given by formulae (a), (b) or (c) of Fig. 3.9.
The results of these formulae are schematically shown in the diagram of Fig. 3.8.

The maximum force that a bolt row can transmit is equal to the difference between the design strengths of two groups with and without the bolt row under consideration.

The force must be determined from the difference between the two bolt groups because the addition of a bolt may change the type of failure mechanism, e.g. from a mechanism with bolt failure to a mechanism with complete yielding of the end plate or column flange.
It has been shown [22] that the design strength of the end plate and column flange can be determined independently of each other and that the lower strength is the governing strength with regard to the bolts.



1	column flange without stiffener	first bolt row	$b_m = 4\ m_1 + 1.25\ n'$
2	column flange or end plate	second bolt row	$b_m = p$
3	extended part of end plate	per side per bolt	$b_m = 4\ m_1 + 1.25\ n' < \frac{b}{2}$
4	column flange with stiffener	first bolt row	$b_m = \frac{\alpha m_1}{2}$ (see diagram α)
5	all bolt rows		$b'_m = \Sigma\ b_m$ (of the individual bolts)

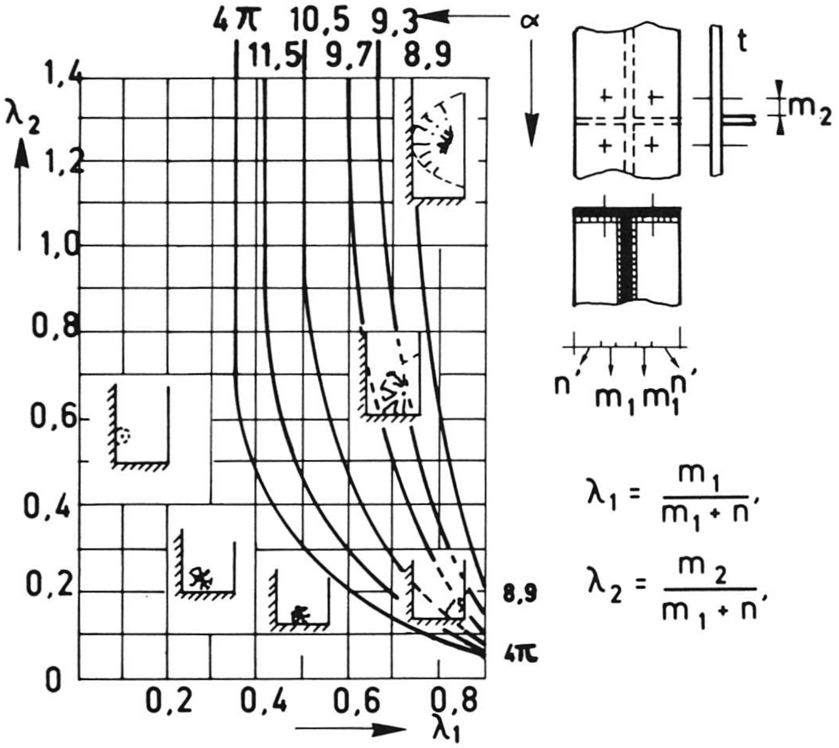


Figure 3.8. Effective length b_m for determining the strength of a part of plate or column flange according to the formulae in Fig. 3.9.

Formulae for T-stubs analogy of column flange or end plate

$$F^* = \frac{4\,b'_m\,m_p}{m}$$ complete plate yielding (a)

$$F^* = \frac{2\,b'_m\,m_p + \Sigma\,B^*_t\,n'}{m + n'}$$ with $n' < 1.25\ m$ yielding of bolts and plate (b)

$$F^* = \Sigma\,B^*_t$$ bolt failure (c)

where:

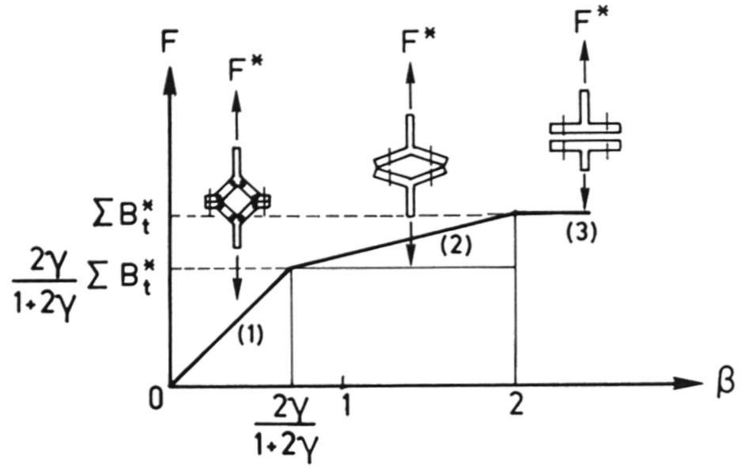
m_p = plastic moment per unit length of the column flange or of the end plate

b'_m = total effective length of column flange or end plate

$\Sigma\,B^*_t$ = the smallest value of the sum of the design tensile force F_{td} and the sum of the design punshing shear forces of all the bolts in the tension zone of the connection.

m, n' = see Fig. 3.8

n' = the smallest value of the distance between the centre of the bolts and the edge of the column flange or the edge of the end plate, but not more than 1.25 m



(a) complete yielding of plate

$$F^* = \beta\,\Sigma\,B^*_t$$

(b) yielding of plate and bolts

$$F^* = \frac{\beta + 2\,\gamma}{2 + 2\,\gamma}\,\Sigma\,B^*_t$$

(c) bolt failure

$$F^* = \Sigma\,B^*_t$$

with $\beta = \dfrac{4\,b'_m\,m_p}{m\,\Sigma\,B^*_t}$ and $\gamma = \dfrac{n'}{m}$

Figure 3.9. Similarity between the design methods for the T-stubs and the end plate or column flange with bolt row is evident on rearranging the formulas for the T-stubs.

In this procedure it is assumed that the pitch of the bolts is not large enough to develop isolated yield line patterns per bolt. So the pitch must be smaller then 4 m + 1.25 n'.

Compression side of a connection

The design resistance of an unstiffened column web in the compression side of a connection can be determined by the following formula for web crippling [24]:

$$F^*_c = f_y\,t_w\,s\,(1.25 - 0.5\,\frac{\sigma_n}{f_y}) < f_y\,t_w\,s$$

where:
$s = t_{fl} + 2\ a \sqrt{2} + 2\ t_e + 5\ (t_e + r)$
f_y = nominal yield stress of column material
σ_n^y = actual longitudinal compression stress in column web due to factored load

and the geometrical parameters according to Fig. 3.10.

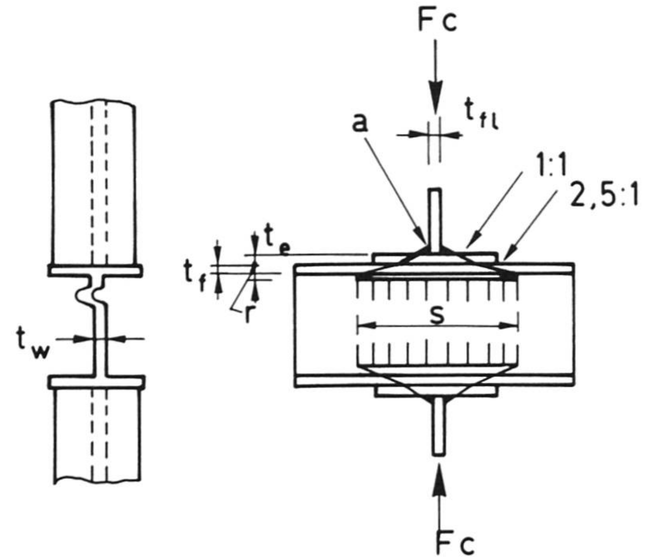


Figure 3.10. Unstiffened column web subject to compression.

Shear panel of the column

The ultimate strength of the shear panel, as shown in Fig. 3.11 without shear stiffeners is:

$F_s^* = 0.58\ f_y\ t_w\ (h - 2\ t_f)$

where:
h = the depth of the column

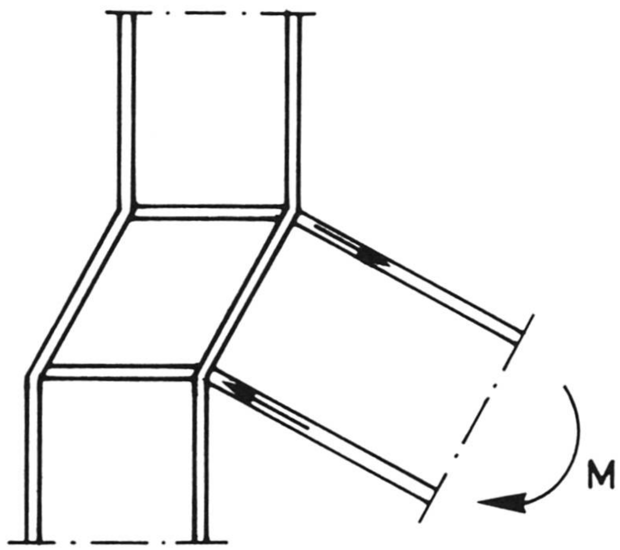


Figure 3.11. Shear panel failure.

3.3.2 Stiffness and rotational capacity

Bending of components in connections such as column flanges and end plates in the elastic range will normally determine the connection stiffness and in the plastic range it will determine the rotation capacity. Premature brittle failure of welds or bolts must therefore be avoided. This can be achieved assuring that F^* is less than $0.9\ \Sigma\ B_t^*$.

4. END PLATE CONNECTIONS

As described in section 2.2 the moment rotation characteristic of the connection can be determined by assuming it to be composed of sub-components each of which has its own specific load deformation characteristic.
Here this approach will be followed as far as possible. However, due to a lack of detailed research information on the behaviour of some sub-components it cannot consistently be followed.

When the three main items such as stiffness, design moment resistance and rotation capacity is calculated a design moment rotation characteristic is available which may be considered as the property of a fictitious rotational spring connecting the centre lines of the column and the connected beam at the point of intersection, as indicated in Fig. 4.1.

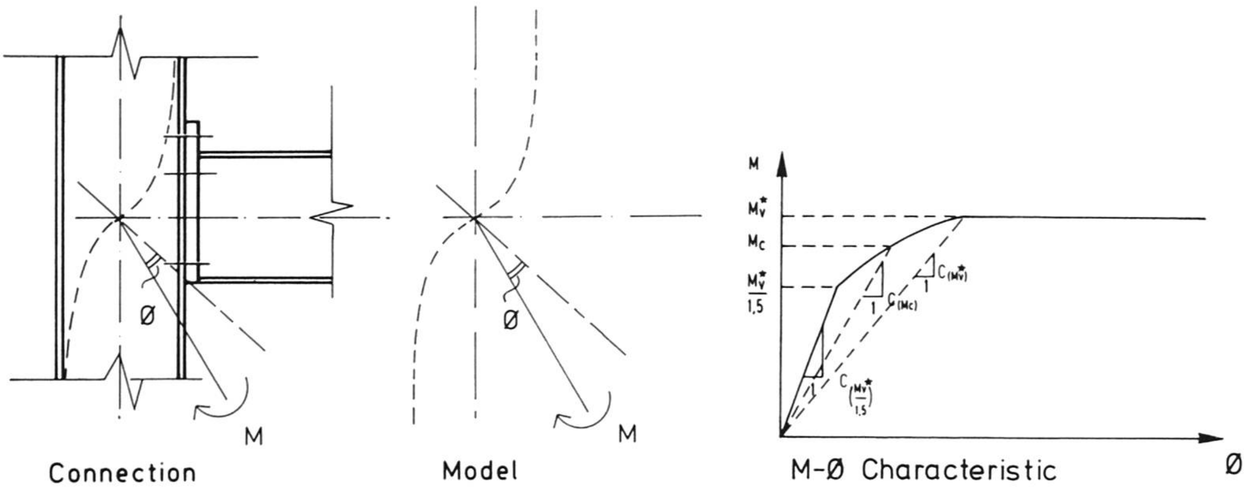


Figure 4.1. Modelling the connection by a rotational spring.

4.1 Stiffness

The stiffness C of end plate connections may be approximated with the following formula [25]:

$$C = E\, h_1^2 \left[\sum_{i=1}^{n} \left(\frac{F_i}{F_i^*}\right)^2 \frac{1}{C_i}\right]^{-1} \frac{M_v^*}{F_{b1}\, h_1}$$

where:

- C = the secant stiffness with respect to a certain moment level M_c in the connection
- E = Young's modulus
- h_1 = distance between the first bolt row beneath the tension flange of the beam and the centre of reaction on the compression side
- F_i = actual force in component i of the connection due to the moment level M_c in the connection, but may not be taken smaller than $F_i = F_i^*/1.5$
- F_i^* = ultimate design strength of component i of the connection
- M_v^* = ultimate moment capacity of the connection
- C_i = stiffness factor of component i as given below
- F_{b1} = force in the first bolt row beneath the tension flange of the beam due to M_v^*

These stiffness factors for the various components are [25]:

		where:
Shear in column web	: $C_1 = 0.24\ t_w$	t_w = thickness of column web
Tension in column web	: $C_2 = 0.8\ t_w$	
Compression in column web	: $C_3 = 0.8\ t_w$	
Tension in column flange without stiffeners	: $C_4 = \dfrac{t_f^3}{4\ m^2}$	t_f = thickness of column flange; m = distance between bolt and web
Tension in column flange with stiffeners	: $C_4 = \dfrac{t_f^3}{12\ \lambda_2\ m^2}$	λ_2 = as in Fig. 3.8
Tension in bolt row	: $C_5 = \dfrac{2\ A_s}{l_b}$	A_s = tensile stress area; l_b = length of a bolt
Tension in end plate	: $C_6 = \dfrac{t_e^3}{12\ \lambda_2\ m^2}$	t_e = end plate thickness

The basic principle of this formula is that the joint flexibility is the summation of the flexibilities due to:
a. shear deformation of the column web;
b. compressive deformation of the column web;
c. tensile deformation of the column web, column flanges, bolts and end plate.

The deformation on the tension side is assumed to be independent of the number of bolt rows employed. This means that the stiffness of a connection can first be calculated with one bolt row. The actual stiffness of the connection can then be found by multiplying this result by the quotient of the strength of the actual and of the notional connection.
The flexibility of each component of the connection is expressed by the reciprocal value of a stiffness factor C_i. This factor can be multiplied by a quadratic term if the load on the component is lower than its design strength. The deformation of a component is assumed to increase linearly with the load up to 67% of the design strength of the component and then to increase quadratically. Because the stiffness factor C_i is given in conjunction with the design strength of the component, this means that at 67% of the strength the stiffness is 2.25 times as great as that which is calculated for the design strength of the component.
In determining the stiffness of the notional connection with one bolt row it is, for calculating the load on the compression side or the shearing zone, necessary to take account of the force which equilibrates the sum of the forces of all the bolt rows, however.

4.2 Design moment resistance

The design resistance of the specific critical zones can be calculated on the basis of chapter 3.
The design resistance of the column flance is often not the same as the design resistance of the end plate within the same connection.
In order to calculate the design moment resistance of the connection both distributions of bolt forces should be made in equilibrium.
The distribution of bolt forces can be calculated by determining the design resistance of the column flange or end plate with the complete bolt group and subtract the design resistance with the bolt group minus the lowest boltrow. This difference is the design resistance of the zone with the lowest boltrow.
By repeating this procedure the complete distribution of bolt forces can be

determined. Now both distributions can be made in equilibrium, starting from the highest boltrow.
In this process it is allowed to "shift" bolt force capacity to a lower boltrow, as long as the total of the boltrow forces does not exceed the design resistance of the bolt group.
In calculating the design moment resistance the boltrow forces are multiplicated with their distances to the centre of compression.
This can be done starting from the highest boltrow, until horizontal equilibrium is reached between the lowest design resistance resulting from the specific critical zones. Lower positioned boltrows can then be ignored.
When the final distribution of forces in the boltrows is known the web of the beam, right behind the end plate can be checked as well as the welds between end plate and web in the tension and in the compression zone.

Tests [14] have shown that the extended part of the end plate may only be taken into account in calculating the design moment capacity if the plate strength to bolt strength ratio β of the bolt group beneath the tension flange of the beam is less than $\beta = 2\ \gamma/(1 + 2\ \gamma)$.

4.3 Rotational capacity

The deformation capacity of a connection may be due to:

a. yielding of the column web shear;
b. yielding of the column web on the compression side of the connection;
c. yielding of the column flange or end plate on the tension side of the connection.

In general, the shear deformation mentioned in point a. will provide the largest component of the deformation capacity. However, it cannot occur in symmetrically loaded connections. In that case the phenomenon mentioned in point c. must be presumed.
From tests [12], [13] and [14] it follows that considerable deformation capacity is obtained from the tension side of the connection if:

$$\beta < \frac{2\ \gamma}{1 + 2\ \gamma}$$

for either:

a. the whole bolt group in the case of column flanges without stiffeners; or
b. the bolt groups in the parts of the column flange above and below the stiffener; or
c. the bolt group in the end plate, provided that the part of the plate extending outside the flanges also deforms sufficiently.

In these cases complete yielding occurs of one of the plate components mentioned.
In the intermediate range $2\ \gamma/(1 + 2\ \gamma) < \beta < 2$ the bolt strength will be sufficient to cause yielding at the transition from plate to web, but the bolts will fail before the prying force at the edge of the plate becomes so large that yielding of the plate at the bolts also occurs. The deformation then remains limited. From the test results [14] an approximate formula for the deformation capacity in that situation has been deduced as:

$$\phi = \frac{10.6 - 4\ \beta}{1.3\ h}$$

where:
h = the distance in mm between the first bolt row from the tension flange and the centre of compression

If $\beta > 2$, no signifficant plastic deformation of the plate occurs. The connection behaves elastically up to failure of the bolts. The deformation will

in that case have to be supplied by the elongation of the bolts. Safe values for the maximum elongation are 2 mm for 8.8 bolts and 1 mm for 10.9 bolts.
It is, however, important to avoid having the deformation capacity provided by failure of the bolts. To ensure that this condition is satisfied, it is essential that $\beta < 2$. For design purposes an extra safety factor on this limit for β should be included; $\beta < 1.75$ is therefore proposed.

4.4 Haunches

It is sometimes necessary to increase the stiffness of the connection. For that purpose, as the formula for the spring stiffness indicates, the most effective measure consists of increasing the distance h_s between tension and compression side (see Fig. 4.2). This may be achieved quite simply by installing a thick haunch plate under the beam, as shown in the righthand part of Fig. 4.3.
It is necessary to check that this inclined part will not buckle prematurely.
If no stiffeners are required for transmitting the compressive force into the web of the beam or the column, this method of increasing the depth is in better agreement with the pattern of forces than the solution with a haunch as shown on the left in Fig. 4.3 since the transmission of force there is concentrated at the end plate and beam flange and then spreads into the haunch. With the inclined thick haunch plate the force is distributed over the height (or depth) of the connection. A design method for the plate type of haunch and the connecting welds is proposed in ref. [15].

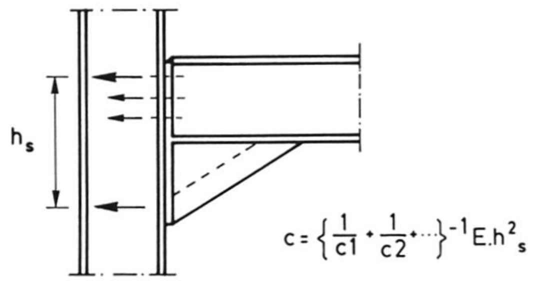


Figure 4.2. Increase of lever arm by means of a haunch.

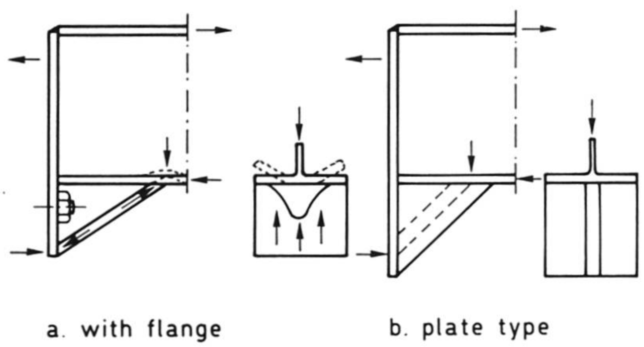
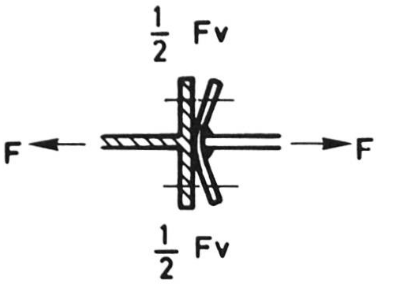


Figure 4.3. Types of haunch and force patterns.

4.5 Influence of pretension in the bolts

Another possible way of increasing the stiffness of the connection is by pretensioning the bolts. This in only effective if the contact pressure due to the prestress is aligned with the column web and beam web. Such alignment is in practice quite likely to occur. As a result of shrinkage of the welds the end plate will tend to deform as shown in Fig. 4.3. If the contact pressure is greater than the maximum force on the tension side of the connection, this force will only cause a reduction of the contact pressure. Thus the deformations due to bending of the end plate, bending of the column flange and tension in the bolts will not contribute to the flexibility of the connection (see Fig. 4.4).



If $F_v > F$

$$\frac{m_k^2}{t_{f_k}^3} + \frac{\ell_b}{A_s} + \frac{m_e^2}{t_e^3} = 0$$

Figure 4.4. Pretensioning of bolts and end plate deformations.

4.6 Backing plates

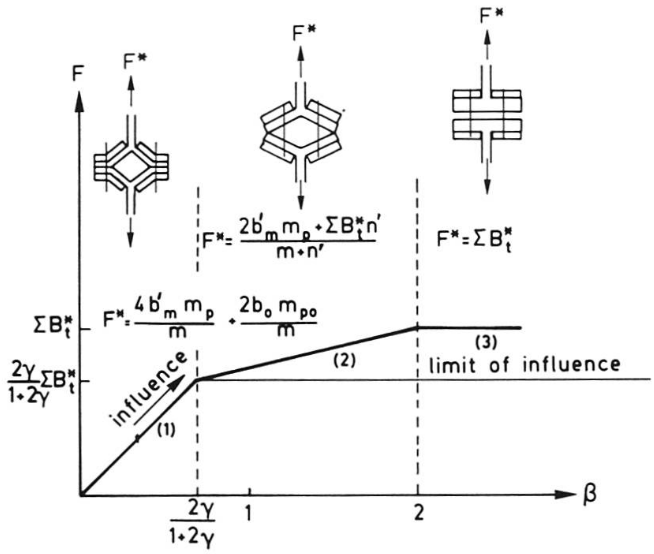
The column flanges can be strenghtened with backing plates as shown in Fig. 4.5, as a result of which, the strength of the column flange:

$F_p^* = 4\ b_m\ .\ m_p/m$

is increased by a factor $(1 + b_o\ .\ m_{po}/2\ b_m\ .\ m_p)$

where:
b_o = length of the backing plate < b_m
m_{po} = plastic moment per unit length of backing plate
b_m = effective length of column flange or T-stub flange
m_p = plastic moment per unit length of column plate

In this factor and in the formulae given in Fig. 4.5 it is clear that the plastic moment at the web-to-flange transition is not increased by the backing plates. Hence it follows that there is no point in using these plates if $\beta > 2\ \gamma/(1 + 2\ \gamma)$, because then yielding occurs only at the web-to-flange transition and failure of the bolts is the governing condition. In the case where $\beta < 2\ \gamma/(1 + 2\ \gamma)$ the strength of the connection can be substantially increased, e.g. by a factor 1.5, by providing a backing plate of length equal to the effective length and a thickness equal to that of the column.



$$(1)\ F = \beta\ (1 + \frac{b_o\ m_{po}}{2\ b_m\ m_p})\ \Sigma\ B_t^*$$

$$\text{with } \beta = \frac{4\ b'_m\ m_p}{4\ \Sigma\ B_t^*} \text{ and } \gamma = \frac{n'}{m}$$

Figure 4.5. Effect of backing plates.

5. ANGLE CONNECTIONS

5.1 Web angle (angle cleats)

The reaction force of the beam is assumed to act at the end of the beam. The bolt group connecting the angle cleats to the web of the beam has to transmit the reaction force plus a bending moment (see Fig. 5.1A).

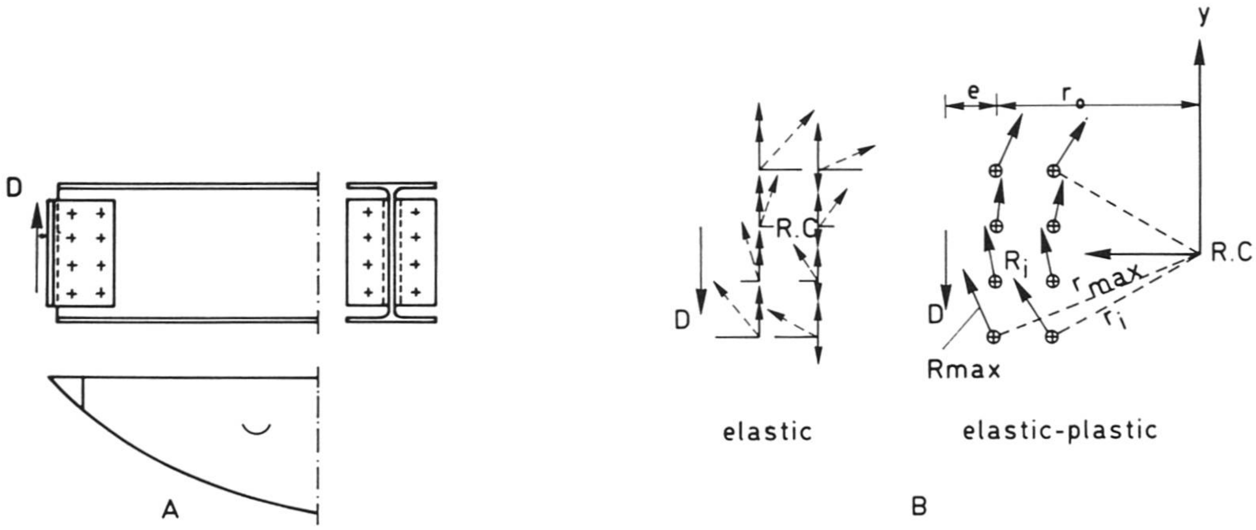


Figure 5.1. Web angle connection

If the design is in accordance with clause 6.2.3.2.1 of the Draft Eurocode 3 [11], i.e. based on elastic theory, a distribution of forces as shown in the left-hand diagram of Fig. 5.1B is assumed. The vertical shear force is assumed to be equally distributed over the bolts. This means that plastic deformation is taken into account implicitely, because it is not possible to drill the holes in the cleats and in the beam with equal dimensional tolerances. In determining the distribution of forces due to the bending moment it is assumed that the forces are distributed in proportion to the distances from the centre of rotation. It is also assumed that the centre of rotation coincides with the centroid of the bolt group. The dotted arrows indicate the resultants of the forces thus determined. It appears that the bolts which are located far from the reaction force, but close to the assumed centre of rotation, are very inefficiently utilized.
If an elasto-plastic analysis is performed in accordance with Chapter 6 of the Draft Eurocode 3 [11], a distribution of forces as shown in the right-hand diagram of Fig. 5.1B is appropriate.
In some cases the maximum capacity may turn out to be as much as 20% larger than that found by elastic analysis. The precondition is, however, that the outermost bolts have adequate deformation capacity so that sufficient redistribution of the internal forces can occur leading to the force distribution assumed in the calculation.
It appears questionable whether this would indeed be possible in case of bolts with 2 mm clearance holes. To reduce this clearance is not justified because the gain due to larger loads would then be offset by higher cost of erection.
Tests [16] conducted to verify the design method described by Fisher and Struik [17], the formulae for which are given in Fig. 5.2, have shown good agreement with the calculated values, justifying the conclusion that an elasto-plastic approach gives satisfactory results, provided that the actual bolt load deformation relation is known.

$$r_i^2 = x_i^2 + y_i^2$$

$$\Delta_i = \frac{r_i}{r_{max}} \Delta_{max}$$

$$\downarrow$$

R_i from experimentally determined relation R - Δ

$$\Sigma F_x = 0 \rightarrow \sum_{i=1}^{n} R_i \frac{y_i}{r_i} = 0$$

$$\Sigma F_y = 0 \rightarrow \sum_{i=1}^{n} R_i \frac{x_i}{r_i} - D = 0$$

$$r_o \rightarrow r_i \rightarrow \underset{\displaystyle D}{\overset{\displaystyle r_i}{\downarrow}}$$

$$\Sigma M = 0 \rightarrow \sum_{i=1}^{n} r_i R_i - D\,(e + r_o) = 0$$

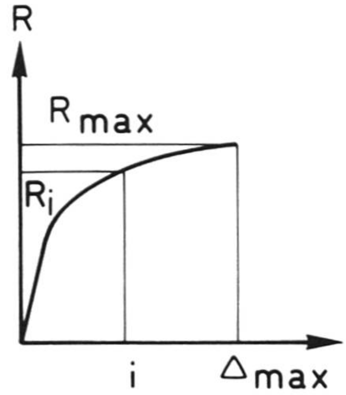


Figure 5.2. Elastic-plastic design method for web angles.

5.2 Web and flange angles

The moment rotation characteristic of this type of connection is similar to that of an end plate connection.
The ultimate design strength of the flange angle may be calculated in the same way as for the flange of a T-stub, provided that the plastic moment is assumed to occur in the leg connected to the beam flange.
Slip of the flange cleat should be avoided by pretensioning of the bolts or using a small bolt clearance (< 1 mm).
The web cleats transfer the vertical reaction of the beam as described in the previous section.

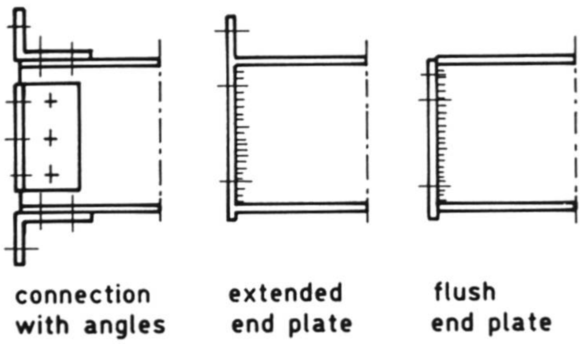


Figure 5.3. Moment connections.

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Structural Properties of Semi-Rigid Joints in Steel Frames

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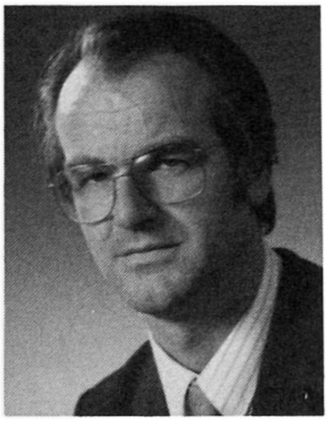

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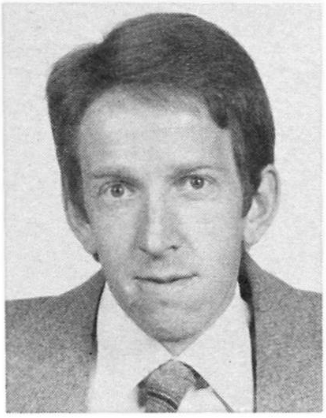

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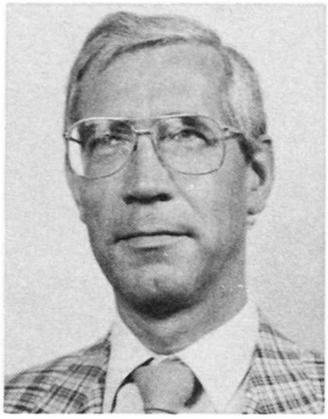

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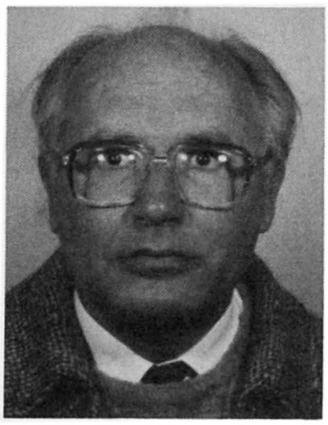

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