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Experimental Investigation And Design Of Fin Plate Connection

Detailed of fin plate connection

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Abstract- Fire hazards and full - scale structural tests have indicated that steel connections could be subjected to large deformations and fracture in fire. This is not currently considered in design approaches because the connections are assumed to heat up more slowly than the structural frame members, and therefore retain more relative strength. A project at the Universities of Sheffield and Manchester is currently investigating the robustness of common types of steel connections when subjected to fire. This paper reports on the part of the results on the fin plate connections. The results illustrate that bolts have their strength reduced faster than hot rolled steel with increase of temperature, and failure of fin plate joints is quite often controlled by bolt failure in shear. As a result of bolt shear fracture, fin plate connections have unexpectedly low resistance and ductility when subject to elevated temperatures and large rotations.

Keywords: Idea Statica, Fin Plate Connection, Bolt, Weld, Friction type connection.

I. INTRODUCTION

In this articles, the most frequent forms of fin plate connections will be shown first, their advantages and disadvantages will be then specified and some explained in detail. Beam to column connections are not further considered, even if they are possible in principle. Furthermore, operational questions should not be neglected. Unfortunately, it happens, when designing steel connections, that the practicability of the connection is not or insufficiently taken into account. Therefore, these possible issues should also be discussed in this article.

In the second part of the articles, the calculation and design of fin plate connections according to AS 4100 will be shown and explained with an example calculated with RF-/JOINTS Steel - Pinned. This article focuses on some designs which are sometimes skipped in design practice. There are many reason for this, the two most frequently are probably the following. The calculation should not be too complex, because it is, as is well - Known, “only” about a connection of a secondary beam. Or the designing engineer lacks knowledge about the necessity of one or other design.

The present article should explain the necessity of one design or the other. Finally, in the event of damage, the spurious argument “It has always worked without this design in the past!” will not be very helpful. When performing design, it becomes obvious very quickly how complex it is to design a pinned fin plate connection and how useful a design program like RF-JOINTS steel-pinned can be to enable an economical design according to the relevant standards within a reasonable processing time.

In the last section, it will be shown which big disadvantage it is to use fin plate connections and other pinned connections the often considerable reduction of the critical buckling moment. Most pinned connections and in particular fin plate connections cannot be considered as lateral and torsional restraints, which may lead to a considerable reduction in resistance for beams at stability risk.

Current design codes generally consider that steel connections will be heated more slowly than beams or columns in fire situations, and are therefore less likely to be the critical components in fire design

II. Definition

Fin plate connections are a popular form of pinned steel connection and are commonly used for secondary beams in steel structure. They can be used easily in beam structures arranged on the top edge (for example, working platforms). Manufacturing expenditures in the workshop as well as the onsite assembly costs are normally manageable. The design seems to be completed easily and quickly, but it has to be put into perspective to a certain extent in the following text. Moreover, this connection type is basically possible as a pinned beam-to-beam or pinned beam-to-column connection; the former case is the more common one in design practice.

III. CBFEM versus Component method

The weak point of standard component method is in analyzing of internal forces and stress in a joint. CBFEM replaces specific analysis of internal forces in joint with general FEA

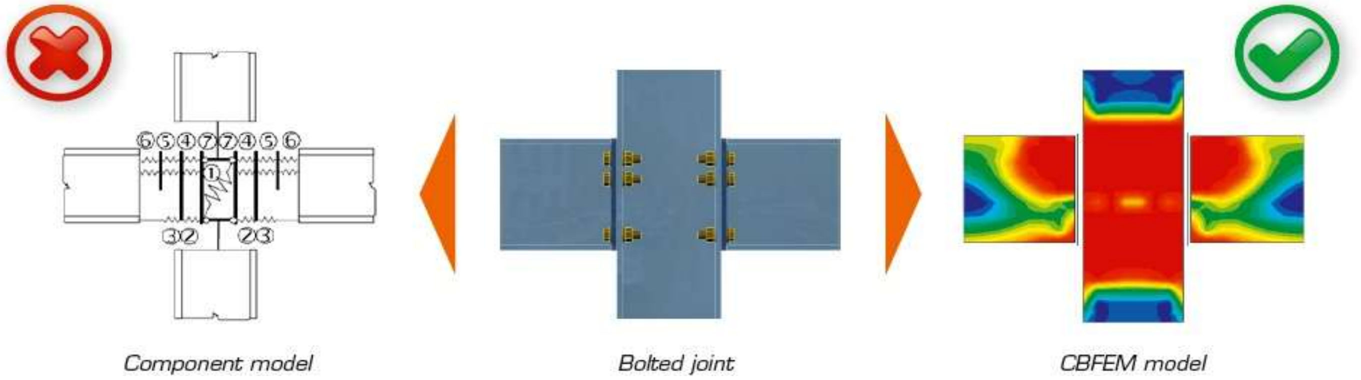

Fig. 1 - CBFEM

Check methods of specific components like bolts or welds are done according to standard component method.
For the fasteners-bolts and welds-special FEM components had to be developed to model the welds and bolts behaviour in joint. All parts of 1D members and all additional plates are modelled as plate/walls. These elements are made of steel (metal in general) and the behaviour of this material is significantly nonlinear.
The real stress-strain diagram of steel is replaced by the ideal plastic material for design purposes in building practice. The advantage of ideal plastic material is that only yield strength and modulus of elasticity must be known to describe the material curve. The granted ductility of construction steel is 15%. The real usable value of limit plastic strain is 5% for ordinary design.
The stress in steel cannot exceed the yield strength when using the ideal elastic-plastic stress-strain diagram.

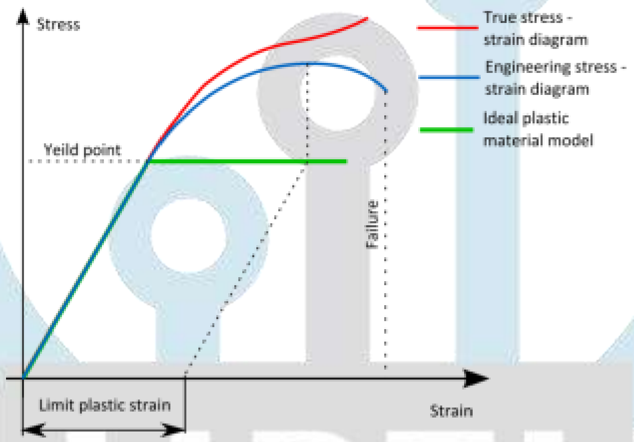

Fig. 2 - Real tension curve and the ideal elastic-plastic diagram

CBFEM method aims to model the real state precisely. Meshes of plates/walls are not merged, no intersections are generated between them, unlike it is used to when modelling structures and buildings. Mesh of finite elements is generated on each individual plate independently on mesh of other plates.
Between the meshes, special massless force interpolation constraints are added. They ensure the connection between the edge of one plate and the surface or edge of the other plate.
This unique calculation model provides very good results-both for the point of view of precision and of the analysis speed. The method is protected by patent.
The steel base plate is placed loosely on the concrete foundation. It is a contact element in the analysis model- the connection resists compression fully, but does not resist tension.

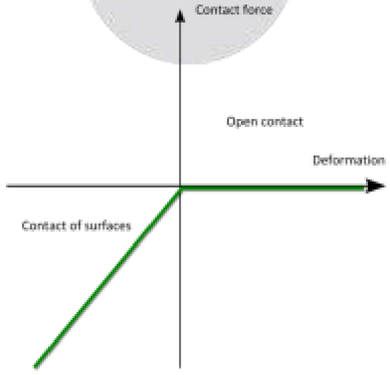

Fig. 3 - Stress-strain diagram of contact between the concrete block and base plate

Welds are modelled using a special elastoplastic element, which is added to the interpolation links between the plates. The element respects the weld throat thickness, position and orientation. The plasticity state is controlled by stresses in the weld throat section. The plastic redistribution of stress in welds allows for stress peaks to be redistributed along the longer part of the weld.

Bolted connection consists of two or more clasped plates and one or more bolts. Plates are placed loosely on each other. A contact element is inserted between plates in the analysis model, which acts only in compression. No forces are carried in tension. Shear force is taken by bearing. Special model for its transferring in the force direction only is implemented. IDEA Statica Connection can check bolts for interaction of shear and tension. The bolt behavior is implemented according to the following picture.

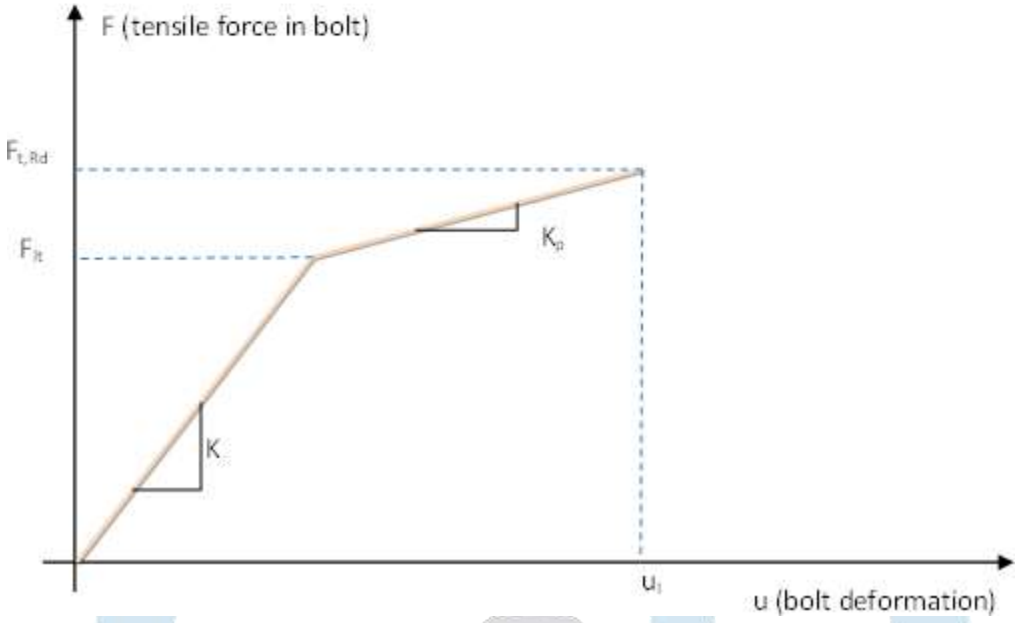

Fig. 4 - Bolt – Tension

Symbol explanation:

- K – linear stiffness of bolt,
- Kp – stiffness of bolt at plastic branch,
- Flt – limit force for linear behaviour of bolt,
- Ft, Rd – limit bolt resistance,
- Ul – limit deformation of bolt.

IV. Loads

End forces of member of the frame analysis model are transferred to the ends of member segments. Eccentricities of members caused by the joint design are respected during load transfer.
The analysis model created by CBFEM method corresponds to the real joint very precisely, whereas the analysis of internal forces is performed on very idealized 3D FEM 1D model, where individual beams are modelled using centerlines and the joints are modelled using immaterial nodes.

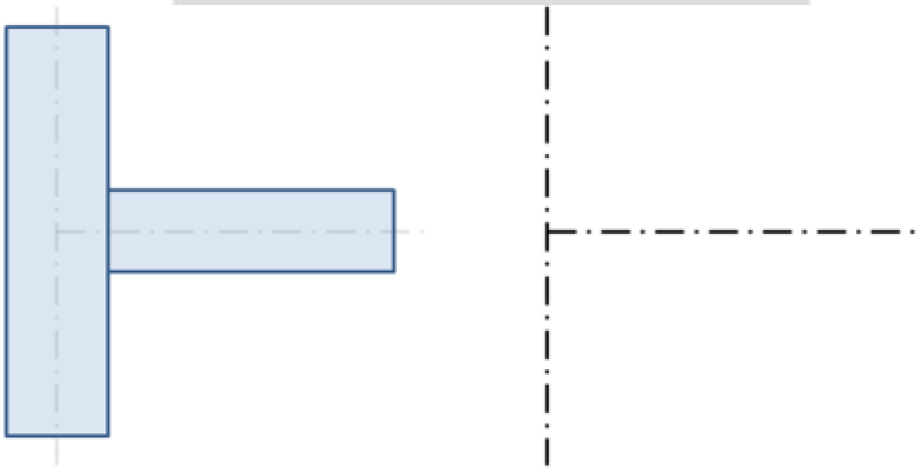

Fig. 5 - Joint of a vertical column and a horizontal beam

Internal forces are analyzed using 1D members in 3D model. There is an example of courses of internal forces in the following picture.

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V. Welds

Fillet welds - The resistance of fillet weld is determined according to AS 4100-2020 – Cl. 9.6.3.10. Plastic redistribution in weld material is applied in Finite Element Modelling. The most stressed element is checked.
Butt welds - The resistance of butt weld is assumed as that of the base metal and is not checked.

VI. Bolts

Tensile resistance of bolts - The tensile resistance of a bolt is assessed according to AS 4100-2020 – Cl. 9.2.2.2.
Shear resistance of bolts - The shear resistance of a bolt is assessed according to AS 4100-2020 – Cl. 9.2.2.1. Each shear plane of a bolt is checked separately. When the bolt threads are intercepted by a shear plane, the minor diameter area of the bolt is used instead of nominal plain shank area of the bolt.
Combined tension and shear in bearing type connection - The resistance of a bolt loaded by combined tension and shear is assessed according to AS 4100-2020
Ply in bearing - The resistance developed at the bolt in a bolted joint subjected to bearing and shear is assessed according to AS 4100-2020. The value *a*e is determined in the direction of the shear load. Only the worst case for each bolt is shown.
Friction-type connections - The slip resistance of a bolted joint is assessed according to AS 4100-2020 – Cl. 9.2.3. A tension and shear interaction is also assessed.

VII. Concrete design bearing resistance in compression

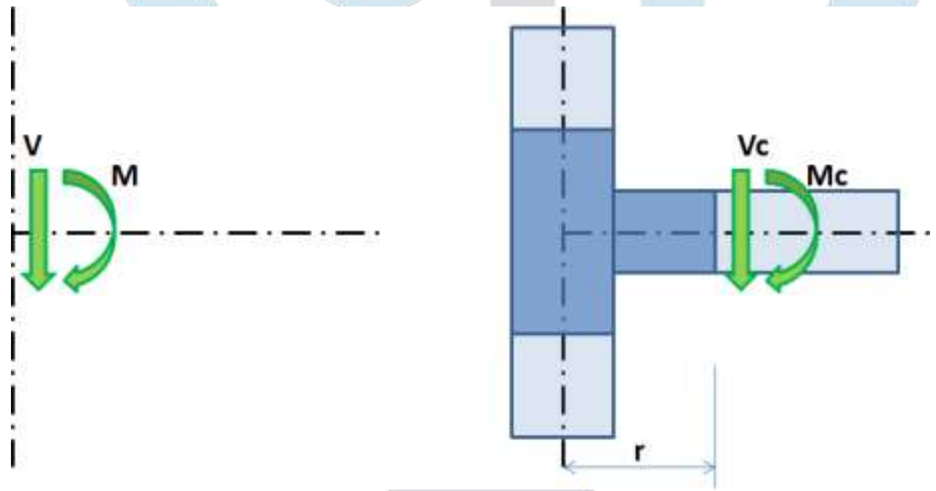
Concrete bearing surface is checked according to AS3600 – Cl. 12.6. The load on the bearing area is distributed upon the supporting surface (geometrically similar lower area of the frustum having its slopes of 1 vertical to 2 horizontal) of the concrete when the foundation block is larger than the base plate. The average stress under the base plate in contact with the foundation pad is compared to the concrete design bearing resistance in compression.

VIII. Transfer of shear at the base plate

The shear action at the base plate is assumed to be transferred from the column to the concrete foundation by:

[1] Friction between base plate and concrete/grout
[2] Shear lug
[3] Anchor bolts

The shear capacity of friction between base plate and concrete and of shear lug is calculated according to Gianluca Ranz, Peter



Kneen: Design of Pinned Column Base Plates, Journal of the Australian Steel Institute, vol. 36, no. 2, September 2002

Fig. 6

IX. Anchors

Anchor rods are designed according to AS 5216:2018 and SA TS 101:2015. The following resistances of anchor bolts are evaluated:

[1] Steel resistance of anchor in tension, N_{tf}
[2] Concrete breakout resistance in tension, $N_{\mathrm{Rk,c}}$
[3] Concrete pullout resistance, $N_{\mathrm{Rk,p}}$
[4] Concrete side-face blowout resistance, $N_{\mathrm{Rk,cb}}$
[5] Steel resistance of anchor in shear with or without lever arm, $V_{\mathrm{Rk,s}}$
[6] Concrete edge resistance in shear, $V_{\mathrm{Rk,c}}$
[7] Concrete pryout resistance of anchor in shear, $V_{\mathrm{Rk,cp}}$
[8] Combined tension and shear loading

Steel failure of anchor in tension - Steel resistance of anchor in tension is determined according to AS5216:2018

Concrete cone failure of anchor in tension - Concrete cone resistance is designed according to the Concrete Capacity Design (CCD) in AS 5216:2018 – Cl. 6.2.3. In the CCD method, the concrete cone is considered to be formed at an angle of approximately 34° (1 vertical to 1.5 horizontal slope). For simplification, the cone is considered to be square rather than round in plan. The concrete breakout stress in the CCD method is considered to decrease with an increase in size of the breakout surface. The check is provided for a single anchor or for a group of anchors if they are located close to each other and subjected to tension.

Concrete pullout failure of anchor in tension - Concrete pullout resistance of anchor is defined in SA TS 101:2015 – Cl. 6.2.3. Concrete pullout resistance for other types of anchors than headed is not evaluated in the software and has to be specified by the manufacturer.

Concrete blowout failure - Concrete side-face blowout resistance of headed anchor in tension is defined in AS 5216:2018 – Cl. 6.2.7. A single anchor or anchor group is evaluated.

Steel failure of anchor in shear - The steel resistance in shear is determined according to AS 5216:2018 – Cl. 7.2.2.2 and AS 4100-2020 – Cl. 9.2.2.1. If mortar joint is selected, shear force with lever arm is assumed and the bolt is checked according to AS 5216:2018 – Cl. 7.2.2.3. The characteristic flexural strength of the fastener is assumed according to ETAG 001 – Annex C.

Concrete edge resistance of anchor in shear - Concrete edge resistance of an anchor in shear is designed according to AS 5216:2018 – Cl. 7.2.3. Individual anchors or anchor groups, in case of their proximity, are checked in the direction of the shear force. Only the most critical case is shown. The anchors closest to the edge at one base plate are assumed to transfer the whole shear force of the base plate.

Concrete pryout resistance of an anchor in shear - Concrete pryout resistance is designed according to AS 5216:2018 – Cl. 7.2.4. In the calculation of concrete cone failure, all anchors are assumed to be loaded by tension.

Interaction of tensile and shear forces - Interaction of tensile and shear forces is assessed according to AS 5216:2018 – Cl. 8. The steel interaction resistance of the fastener is based on AS 4100. The concrete interaction resistance is checked according to AS 5216:2018 – Cl. 8.2.1.

Anchors with stand-off - Anchor with stand-off is designed as a bar element loaded by shear force, bending moment and compressive or tensile force. These internal forces are determined by finite element model. The anchor is fixed on both sides, one side is 0.5×d below the concrete level, the other side is in the middle of the thickness of the plate. The buckling length is conservatively assumed as twice the length of the bar element. Plastic section modulus is used. The bar element is designed according to AS 4100. Interaction of shear force is neglected because the minimum length of the anchor to fit the nut under the base plate ensures that the anchor fails in bending before the shear force reaches half the shear resistance and the shear interaction is negligible (up to 7 %)

Interaction of bending moment and compressive or tensile force is conservatively assumed as linear. Second order effects are not taken into account.

[1] Shear resistance is checked according to AS 4100-2020 – Cl. 5.11
[2] Tensile resistance is checked according to AS 4100-2020 – Cl. 7
[3] Compressive resistance is checked according to AS 4100-2020 – Cl. 6
[4] Bending resistance is checked according to AS 4100-2020 – Cl. 5.1
[5] Linear interaction for combined bending moment and axial force is used.

X. Design of Fin Plate Connections

[1] In this section, the most common forms of fin plate connections as beam-to-beam connections will be shown and evaluated with regard to their advantage and disadvantage in calculation and design, as well as in manufacture and assembly. The point mentioned only represent suggestions and make no claim to completeness

[2] Basically, fin plate connections can be used both as beam to column connections and beam to beam connections whereas beam to column connections will not be discussed in any further detail here. In most cases, other pinned connection types are more favourable for beam to column connection

[3] The main problem with fin plate connections during assembly is that it is impossible to specifically compensate for manufacturing tolerances such as overlength or shortlength of the beam, as is possible with short end plate connections and double angle connections. The beams are here specifically manufactured with a short length according to the maximum tolerance which corresponds to the standard. The short length thus obtained can be then offset on-site with supplied backing plates.

[4] The fact that this method is not possible with fin plate connections ensures that they should not be arranged between two columns, because it may happen that the beam can only be assembled by force. Apart from this fact that the assemblers have problems with it, effects due to restraint are introduced to the structural system which should be actually avoided.

[5] A big advantage of fin plate connections in, in general, the lower fragmented structure compared with most other pinned connection types. No small parts are necessary except bolts.

[6] In this following, the two most common types of fin plate connections will be shown as beam to beam connection. This is regarded as a connection with a "long" or "short" fin plate.

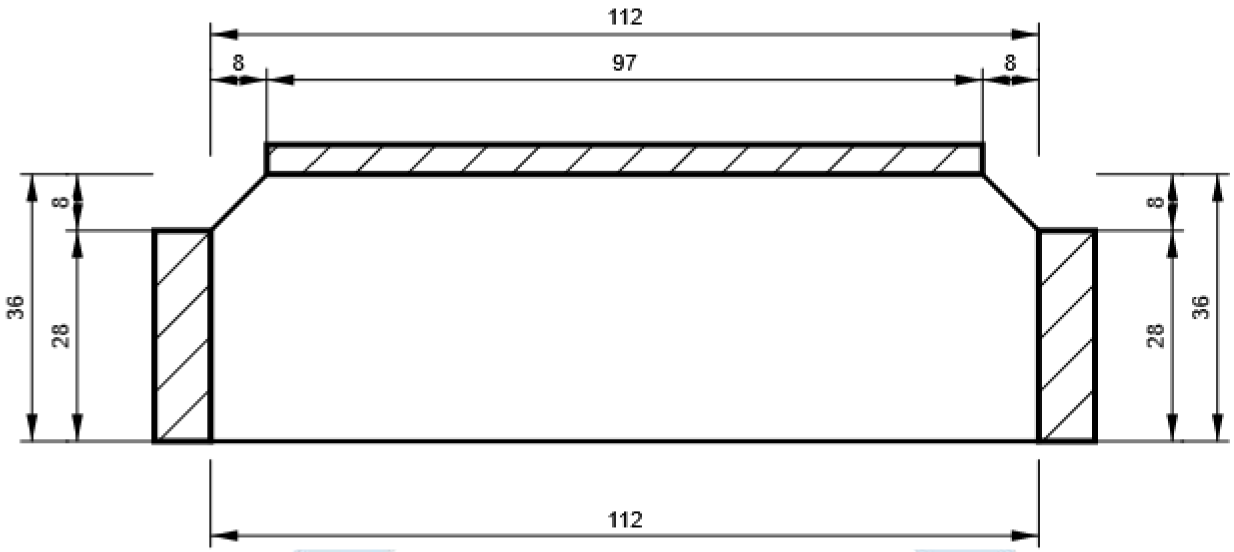


Fig. 7 - STIFF – 1

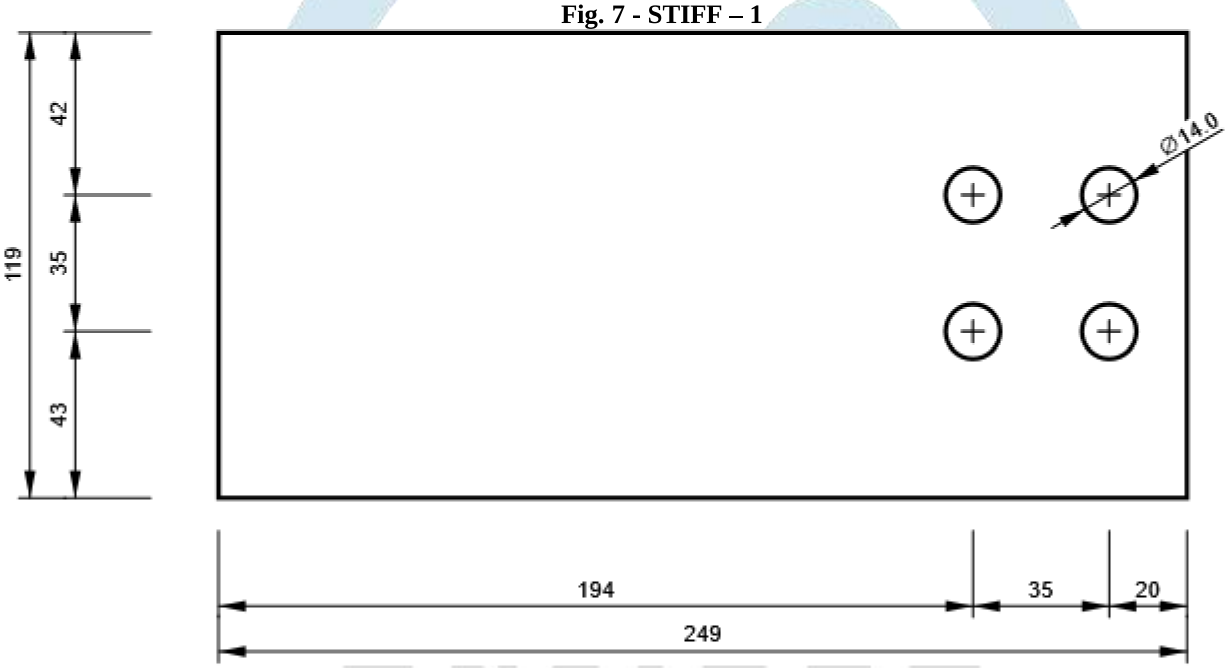


Fig. 8 - M2, I-Section (UB 127 x 76 x 13)

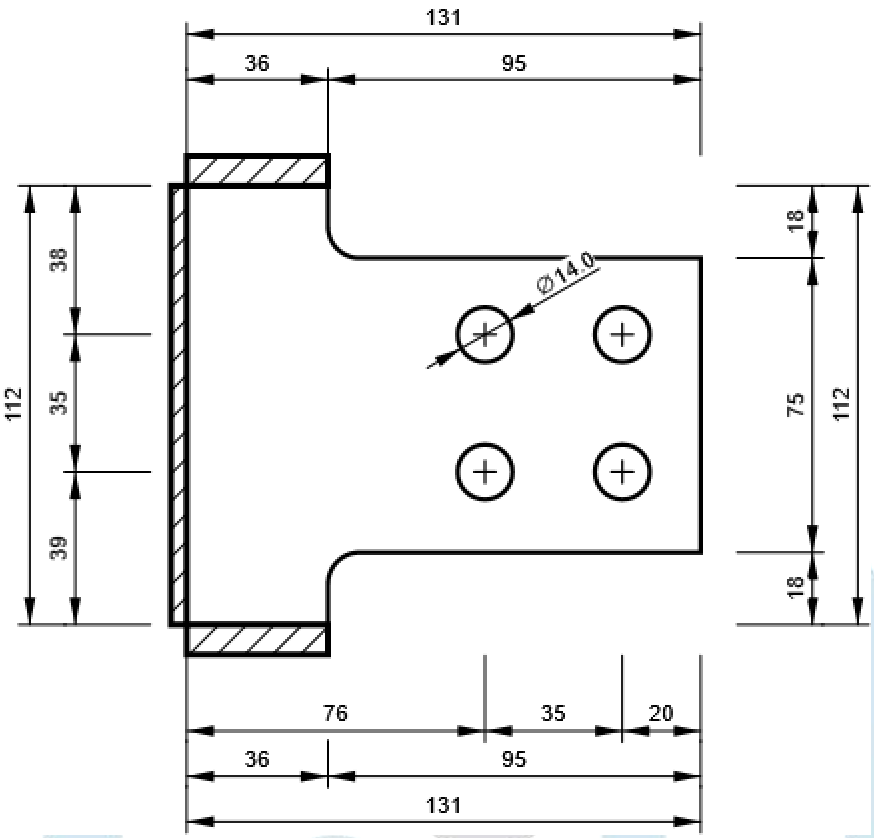


Fig. 9 - SP – 1

Beam-to-beam connection with a “long” fin plate:

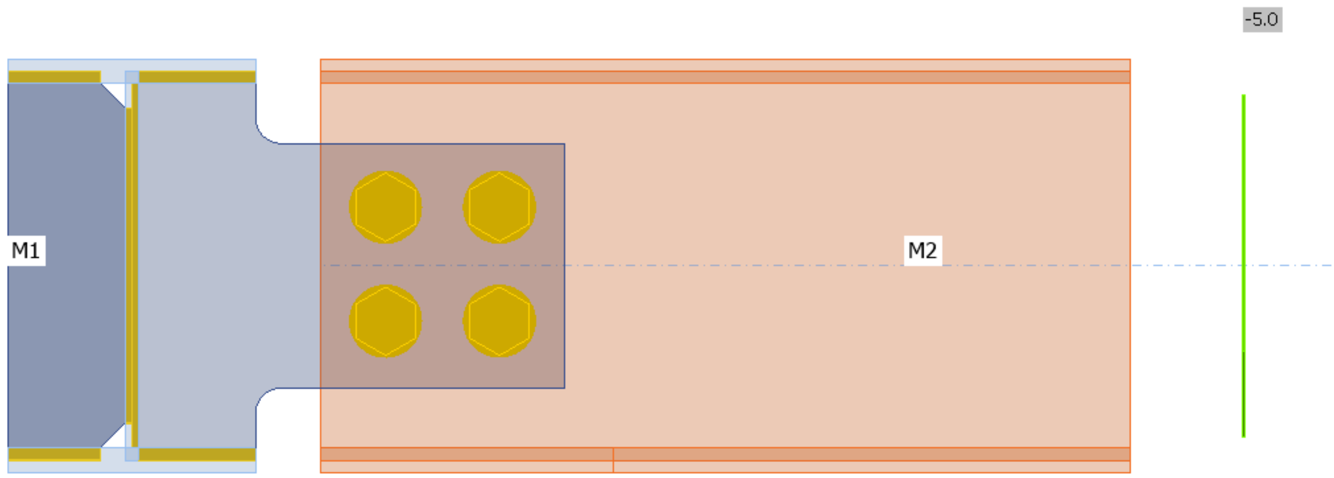


Fig. 10

DETAILED RESULT FOR M1-TFL 1 / SP1 - 1

Weld resistance check (AS 4100-2020 - Cl.9.6.3.10)

$$\phi v_W = \phi \cdot 0.6 \cdot f_{uw} \cdot t_t = \quad 997.9\text{kN/m} \geq \quad v_w^* = 73.5\text{kN/m}$$

Where:

$\phi = 0.80$ – resistance factor for welded connections
$f_{uw} = 490.0$ MPa – nominal tensile strength of weld metal
$t_t = 4$ mm – design throat thickness
$v_w^* = 73.5$ KN – design force per unit length of weld

DETAILED RESULT FOR M1-W 1 / SP1 - 1

Weld resistance check (AS 4100-2020 - Cl.9.6.3.10)

$$\phi v_W = \phi \cdot 0.6 \cdot f_{uw} \cdot t_t = \quad 997.9\text{kN/m} \geq \quad v_w^* = 91.1\text{kN/m}$$

Where:

$\phi = 0.80$ – resistance factor for welded connections
$f_{uw} = 490.0$ MPa – nominal tensile strength of weld metal
$t_t = 4$ mm – design throat thickness
$v_w^* = 91.1$ kN – design force per unit length of weld

DETAILED RESULT FOR M1-BFL 1 / STIFF1A - 1

Weld resistance check (AS 4100-2020 - Cl.9.6.3.10)

$$\phi v_W = \phi \cdot 0.6 \cdot f_{uw} \cdot t_t = \quad 997.9\text{kN/m} \geq \quad v_w^* = 52.8\text{kN/m}$$

Where:

$\phi = 0.80$ – resistance factor for welded connections
$f_{uw} = 490.0$ MPa – nominal tensile strength of weld metal
$t_t = 4$ mm – design throat thickness
$v_w^* = 52.8$ kN – design force per unit length of weld

XI. Influence of Connections on the Stability of the Component

Finally, the biggest problem that occurs with fin plate connections, as well as with other pinned connection types, is mentioned the partially large deviation from a lateral and torsional restraint. This deviation is in contrast to rigid connections, often incorrect and thus of significant importance in respect of safety

The following text will not show the one correct design method for this problem, but rather, how a design engineer might handle it.

In steel construction, it often occurs that the engineer in charge of the beam design does not design all connections of the structure. He will usually design the column bases as well as the main connections of the structure and hand over the secondary connections to the executing company. The engineers and designers working for this company knows best which connections suit for their technology in manufacture and assembly.

Within the framework of connection design, the responsible designer has to verify the design of the secondary beam for the lower connection stiffness anyway, which is then only possible, if he has some tolerances from the original planning. How this design might look like is the decision of the designing engineer. Unfortunately, the engineering standards offer no assistance.

The design of the beam taking into account the lower connection stiffness can be basically performed in two ways either by direct consideration in the support by using a torsion spring or by means of table values or diagrams to determine the elastic critical buckling moment

The second case is the preference provided that an equivalent member design should be performed manually. Since the design process is not covered in particular by this article, it is only referred to useful references. Highly recommendable are the articles and unfortunately, the corresponding research report of DASt has not yet been published which might be the most useful source in concerning this difficulty.

The possibility to consider an appropriate torsion spring is certainly the solution approach if the design should be performed with the general method according to the calculation according to the second order torsional buckling theory. To get these spring parameters, a calculation with the FE method is possible.

XII. Conclusion

Fin plate connections are versatile and commonly used in steel structures for shear and axial force transfer. However, careful consideration of design parameters like bolt and weld strength, plate thickness, and temperature effects is crucial for ensuring a safe and efficient connection

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