THE DESIGN OF CAST-IN PLATES

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J.R. Henderson BSc, PhD, CEng, MIStructE

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SCI, Silwood Park, Ascot,
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T: +44 (0)1344 636525
F: +44 (0)1344 636570
E: reception@steel-sci.com

www.steel-sci.com

To report any errors, contact:
publications@steel-sci.com

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FOREWORD

The use of steel plates cast into concrete elements to connect steel beams to is a common construction method where a steel frame surrounds a concrete core. Despite the frequency of providing lateral stability to a steel frame in this way, hitherto there has been no design guide available in the UK to suggest a common approach. Consequently there may be discussions between project participants with limited common basis about design assumptions, the division of responsibility and the necessary arrangement of the connection to achieve the design intent. The purpose of this guide is to provide such a basis and identify issues that should be addressed in the design and construction process.

The guide was prepared by Richard Henderson of the SCI with guidance and input from a working party made up of the following individuals. Their contribution is gratefully acknowledged.

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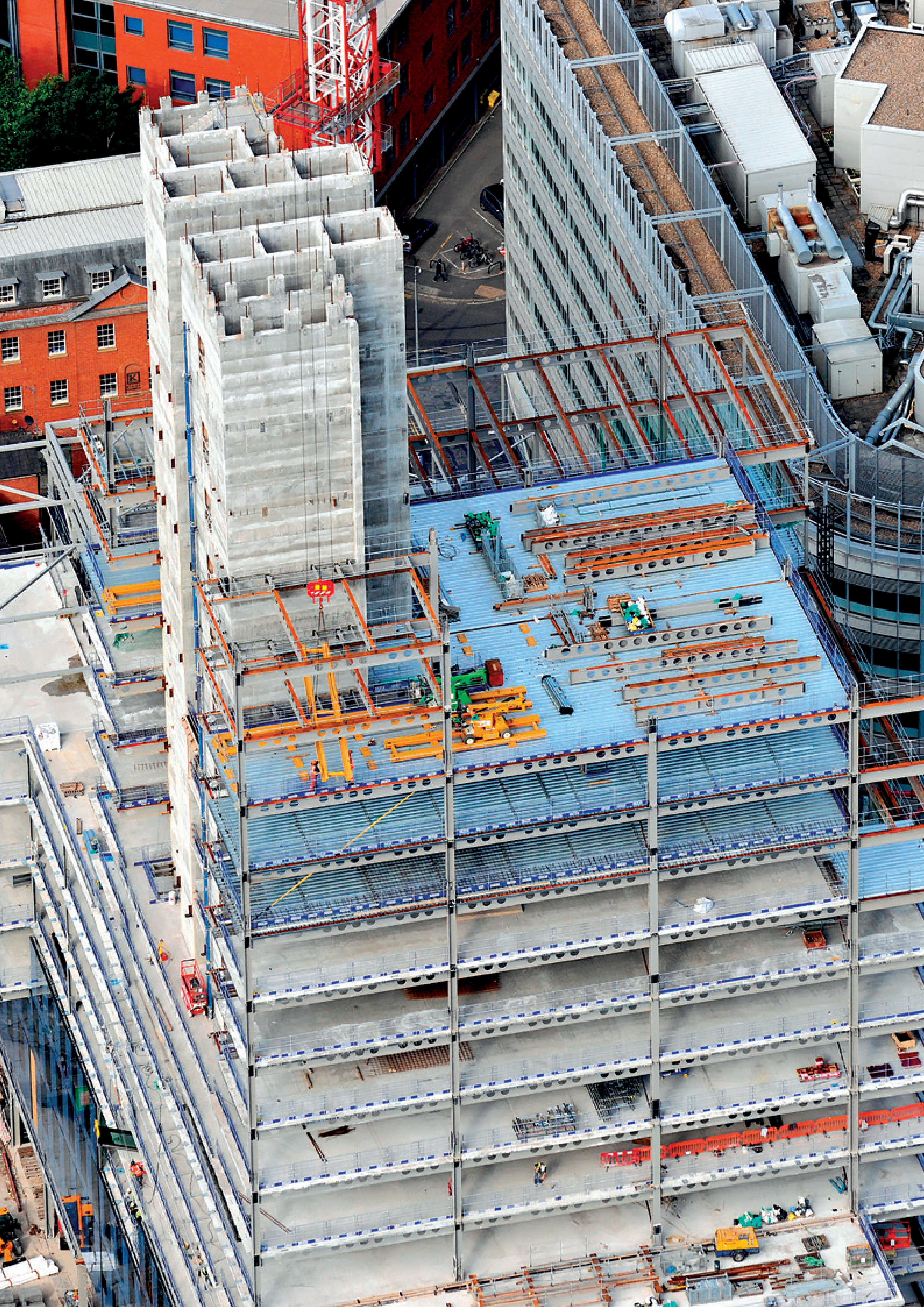

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SUMMARY

This publication provides guidance for the design of cast-in steel plates for connecting structural steel beams to concrete core walls. The guidance can be extended to connections of other elements (eg support brackets for services).

The guide provides a model for the design of simple connections that transfer shear force due to permanent and variable loads and a non-coincident axial tie force resulting from an accidental load case. It points out additional issues which must be considered where coincident shear forces and axial forces are to be dealt with. A sample design of a simple connection for a 610 serial size UB is presented. The design of punching shear reinforcement for the wall is included.

The guide discusses the responsibilities of the building structural engineer and the steelwork contractor and suggests where the responsibilities are best divided. It also considers the impact of deviations between the theoretical positions of the parts of the connection and their as-erected positions.

Issues relating to construction are discussed and some recommendations made.


WARRIOR

INTRODUCTION

A common form of building construction in the UK is to choose concrete cores to provide the lateral stability to steel framed buildings. The steel frames often consist of steel-concrete composite slabs and beams supported on steel columns using simple connections.

Loads are transferred through the floor structure to the concrete cores as:

- vertical forces from the permanent and variable actions on the floor;
- horizontal forces from wind on the building façade or the horizontal component of force from inclined columns;
- tie forces for robustness (independent of other forces).

Such force transfers necessarily require some steel beams to be connected to the concrete core structure. Until now, no design guidance for these types of connection has been published in the UK and designers have had no option but to develop their own approach from first principles. This has led to the adoption of various connection details and different contractual arrangements from project to project, with attendant discussion and potential for disagreement. The present guide has been produced with input from representatives of different parts of the construction industry to:

- propose a structural model for the design of the joint and its components and
- identify design responsibility for the different components.

As well as assuring the provision of acceptable details, it is hoped that the guide will reduce the time spent on discussion of such details and divisions of design responsibility.

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FUNCTIONAL REQUIREMENTS AND BEHAVIOUR

Many buildings are designed as 'simple construction' for which the global analysis assumes nominally pinned connections between beams and columns and resistance to horizontal forces is provided by bracing or cores. Consequently the beams are designed as simply supported and columns are designed for moments arising from a nominal eccentricity of the beam to the column. This design approach is accommodated by the Eurocodes. A 'simple' joint model, in which the joint may be assumed not to transmit bending moments, may be assumed if the joint is classified as 'nominally pinned'.

In a building designed with nominally pinned connections, any connections to a concrete core are required to carry shear and to be sufficiently flexible to behave as nominal pinned connections. The 'Green Book', SCI publication P358 [1], sets out design rules for simple joints between steel elements. Connections of steel elements to concrete cores, shear walls and other elements are required to behave in a similar way. Tie forces for robustness can normally be transferred to the core wall effectively through steel reinforcement in the floor slab. However, where this is not possible, in addition to carrying simple shear, the beam connections are required to carry the tie forces in a separate load case.

It is relatively common for steel to concrete connections to be required to carry coincident shear and axial forces (distinct from the separate tying forces). These connections do not conform to the simple connections discussed in the 'Green Book' because that publication does not deal with connections carrying such a combination of forces. These connections may arise where core structures are surrounded by voids for building services risers so that wind loads applied to the building cladding are transmitted to the core walls through the floor beams instead of through the floor slab. Also, any floor beam supported on an inclined column will experience an axial force in the same load case as the shear force. More significantly, if a vertical column becomes inclined at a particular floor level, the horizontal component of the inclined force is usually transferred into a floor beam (see Figure 2.1). The axial force in the beam may then be transferred into the lateral load resisting structure.

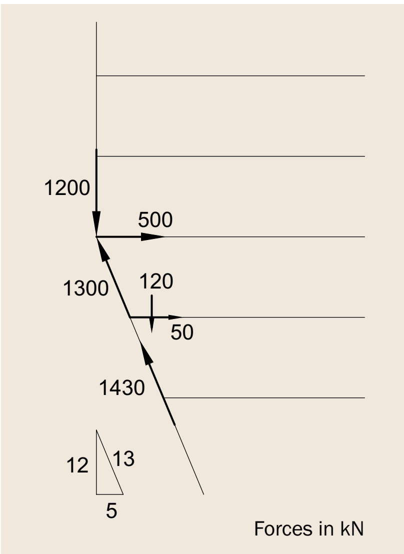


Figure 2.1 Axial forces in beams connected to inclined columns

The most common form of connection is a steel plate cast into the wall to which the steel beams of the floor structure can be connected. Shear studs are usually provided on the back of the plate to resist the vertical reaction from the floor beam. Axial forces in the beam are resisted by reinforcing bars which are generally welded to the plate and transfer the load through bond with the concrete. The plate, shear studs and reinforcing bars are embedded in the concrete structure and are referred to as 'cast-in' or 'embedment' plates.

After the plates have been cast into the core wall and steel erection is under way, the position of the plates is surveyed and an element is welded to the face of the plate (often a fin plate) to receive the connecting floor beam. The deviation of the fin plate from its 'true' position on the cast-in plate is determined mainly by the accuracy of construction of the concrete structure and cast-in plate. The construction tolerances specified for the concrete give a guide to the eccentricities which must be allowed for in the design of the plate and the shear connection.

Where the connection is 'simple' as defined in the 'Green Book', the fin plate geometry can be as set out in the standard fin plate connections.

2.1 Nominally pinned connections in simple construction

A 'simple' connection transfers the reaction at the end of the beam to the cast-in plate in shear. The shear force is transferred to the fin plate by bolts and the line of applied shear is assumed to be through the centroid of the bolt group. This line is eccentric to the steel to concrete connection and to the centre of the wall. The cast-in plate and its welded connection to the fin plate must resist the eccentricity moment due to the offset of the bolts from the face of the plate.

The steel to steel connection detailing rules in the 'Green Book' result in a joint which is sufficiently flexible to deform and allow the beam to take up an end slope which is consistent with a simple support. Rotations are achieved by plastic elongations in the bolt holes in the fin plate and/or beam web and by shear deformation of the bolts.

The standard arrangement for fin plates in the 'Green Book' is a 10 mm thick fin plate in S275 material with two 8 mm fillet welds provided to eliminate the possibility of weld failure. This arrangement entails an eccentricity moment being applied to the supporting column. Nominal moments are used in the column verification determined by assuming the shear force acts at a lever arm of 0.1 m to the face of the column web or flange.

The behaviour of simple steel to steel connections with the arrangement set out in the 'Green Book' has been validated by tests on beams up to 610 mm deep. Additional restrictions on the connection arrangement on beams deeper than 610 mm are set out in the 'Green Book' to preserve the nominal pinned behaviour and prevent excessive deformation at the extreme bolt positions. The limitations identified are that the span to depth ratio of the beam should not exceed 20 and the vertical distance between the centres of the top and bottom bolts in the joint should not exceed 530 mm. The maximum number of bolt rows is therefore limited to eight if the standard pitch of 70 mm is adopted. The nominal gap between the beam compression flange and the column is 10 mm for beams up to and including 610 mm deep and increased to 20 mm for deeper beams.

Cast-in plates are similarly subject to an eccentricity moment determined by the distance of the centroid of the bolt group from the vertical plane being considered. For the checks on the moment resistance of cast-in plates, the lever arm is taken as the distance from the bolt centroid to the back of the cast-in plate. The minimum gap between the end of the beam and the face of the concrete wall must be achieved to allow the rotation to occur, so the nominal gap must allow for the construction tolerances.

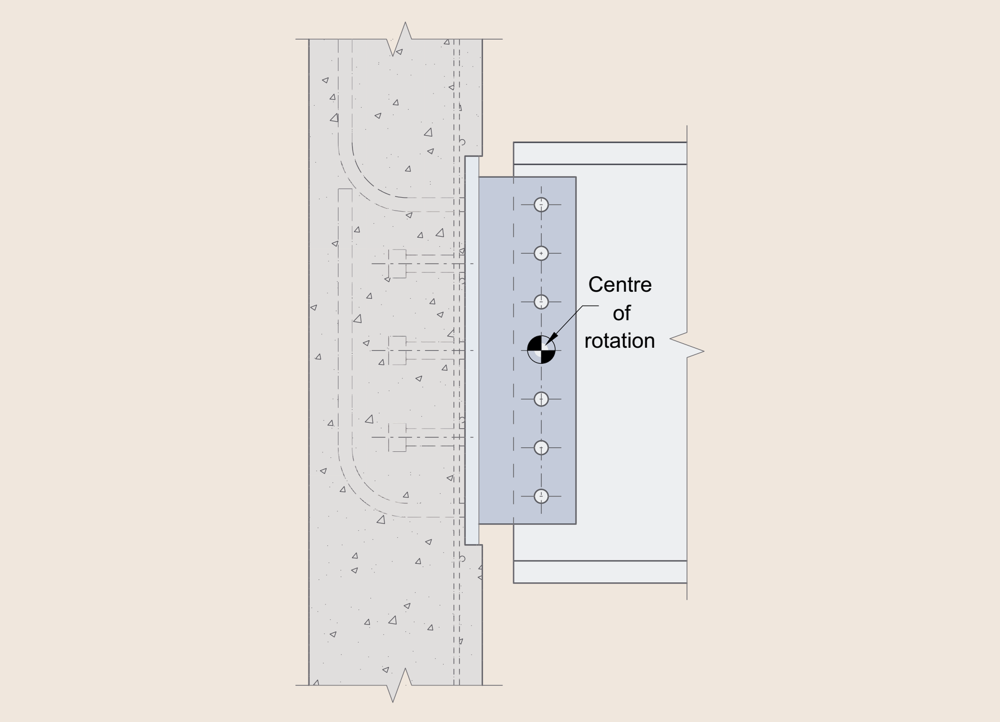


Figure 2.2 Centre of rotation of joint

The shear force and eccentricity moment applied to the cast-in plate must be transferred into the concrete core wall. The shear is transferred through shear studs welded to the back of the plate and the moment is transferred as a push-pull between a compression zone at the bottom of the fin plate and reinforcing bars welded to the plate near the top of the fin plate. The reinforcement is assumed not to contribute to transfer of shear into the wall. However, since the cast-in plate is rigid in plane, the reinforcement will be subject to shear force and so a reduced tension resistance of the bars should be used. The concrete wall must be checked for punching shear resulting from the eccentricity moment.

The reason for assuming that the shear studs make no contribution to the pull-out resistance of the cast-in plate is that there is no satisfactory interaction formula for the resistance of shear studs in combined shear and tension available in Eurocodes. A formula is proposed by Johnson [2] but stud tensions higher than a tenth of the stud shear resistance are outside the scope of Eurocode 4 [3].

2.1.1 Tie forces

To satisfy robustness requirements, the connection has to be able to transfer tie forces into the concrete core. If the floor slab continues up to the core wall, the tie force can be transferred from the steel beam into the floor slab and thence into the core wall via reinforcement. This arrangement is preferable because the tie force is less concentrated than if it arrives at the core wall in the beam, so the resistance to tying is likely to be easier to develop.

Where it is not possible to provide the necessary tying resistance through the floor slab, a load case is checked to confirm the steel connection can satisfactorily transfer the tie force. The axial force in the beam is applied to the cast-in plate through the fin plate and is transferred to the reinforcement welded to the back of the plate. The force is resisted by bond between the concrete core and the reinforcement. A sufficient anchorage length of bar is provided to develop the tensile resistance of the bar. This arrangement may result in the need to provide shear reinforcement in the core wall around the cast-in plate to prevent failure under this load case. Bars turned down are preferred because the failure mode is more ductile than for turned-up bars.

The line of action of the tie force is not considered. It is assumed to be sufficient to provide adequate resistance. Bars to resist tying can be provided at the level of the top of the fin plate only or at both top and bottom. In the latter case, the fin plate and cast-in plate acting together transfer the tying force in bending by spanning vertically between the bars.

2.2 Nominally pinned connections with coincident shear and axial forces

Where it is necessary for a connection to carry coincident shear and axial forces, satisfactory detailing to avoid the development of bending moments in the connection is likely to be difficult to achieve with a fin plate or end plate connection. In the 'Green Book' simple connection model, the holes in either the fin plate or beam web (or both) are assumed to elongate to accommodate the end-slope of the simply supported beam, while carrying the vertical shear. The elongations cannot occur in this way if a horizontal load is also to be carried. Similarly, an end plate is assumed to be sufficiently flexible to deform and allow the beam end slope to occur. Again, the deformations cannot occur in this way while a horizontal force is being transferred. If axial forces are sufficiently large to require the mobilization of the beam flanges, this will also entail the possible development of a couple. Such a fixing moment could be avoided by the use of a physical pin but the eccentricity moment to be resisted by the cast-in plate could then be significant because of the physical size of the connection.

Where the axial forces are significant, it may be simpler to transfer them into the core wall by providing a through connection in the wall with a back plate or welded bar connected to plate washers within the cover zone on the opposite side of the wall to the cast-in plate. The transfer of the axial load can be achieved more directly by bearing on the concrete, instead of relying on welded reinforcement and bond.

A connection which transfers coincident shear and horizontal force and applies a modest eccentricity moment to the cast-in plate, and allows the beam to take up a simply supported end slope is required. The development of a fixing moment must be prevented, to avoid transferring a significant bending moment into the concrete core wall at the connection. This means the transfer of horizontal force through both flanges of the beam must not be allowed. The vertical reaction in the beam must be transferred to the cast-in plate while allowing rotation to occur.

If the axial force in such a connection is transferred through a horizontal plate, the vertical bending resistance is low and plastic rotation of the plate can be assumed. A bearing block will carry the vertical shear and also allow rotation. A torsional restraint is also required. A possible arrangement is to provide a landing cleat and a torsional restraint towards the top of the beam web. A horizontal force can be delivered to the core wall through the bottom flange of the beam and the landing cleat. If the torsional restraint provides no axial resistance, the possibility of applying a significant bending moment to the concrete core is practically eliminated. Figure 2.3 shows the proposed detail. The anchors resisting the tie force are provided at or above and below the level of the tension plate and the cast-in plate bends between them.

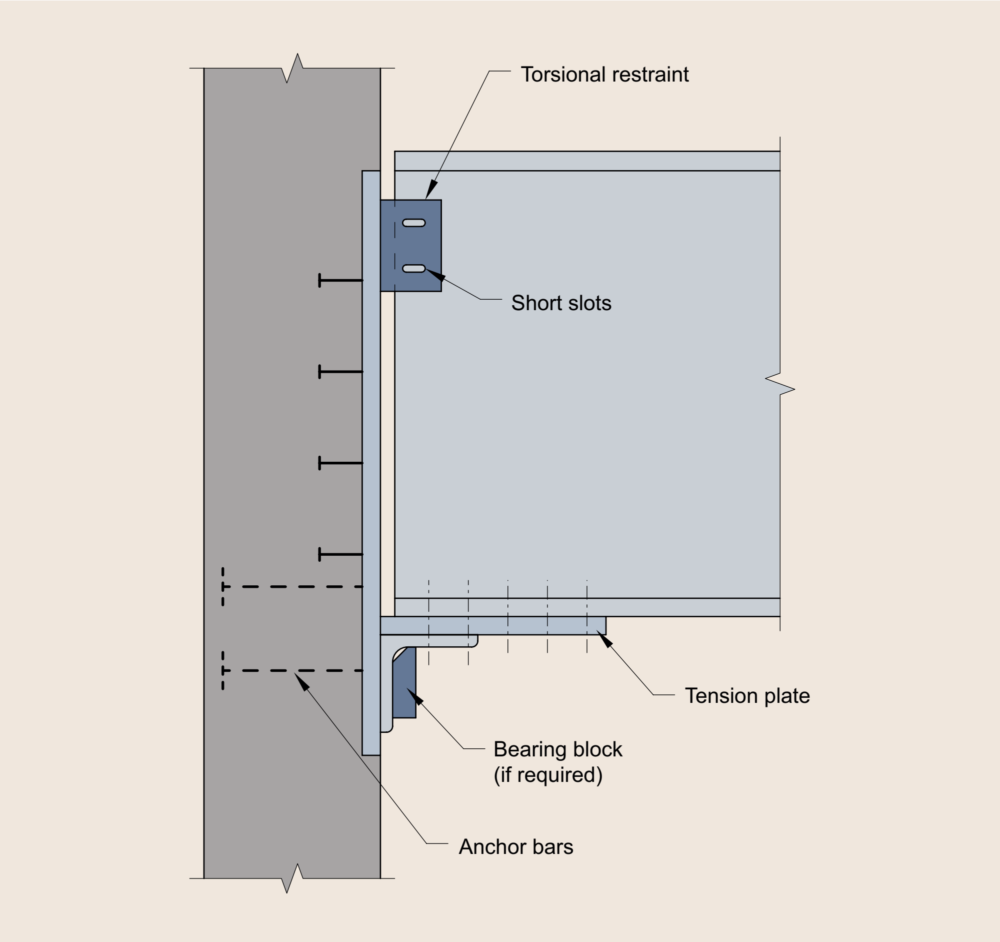


Figure 2.3
Connection detail to resist coincident shear and axial forces

The transfer of any axial force in the beam to the tension plate will result in a bending moment in the beam which reduces linearly to zero at the far support. For a tension of a magnitude where this detail would be appropriate, the additional bending moment at mid-span is unlikely to be large in comparison to the vertical load moment but it should be taken into account. A compression will result in a reduction in mid-span bending moment.

2.3 Alternative detailing to provide resistance to coincident shear and axial forces

Various modifications to the fin plate detail, required when it must carry shear and coincident axial force, are adopted by designers to ensure that behaviour remains that of a simple connection. The requirement for such modifications is not always identified at tender stage leading to unexpected additional work for the steelwork contractor. The modifications have varying degrees of effectiveness. Examples are:

- Provide ordinary holes near the beam neutral axis and slotted holes remote from the beam axis in a deep beam to use more than 8 bolt rows yet allow the beam to rotate.

  In this case, all the bolts are likely to be necessary to resist the tie force and the slots will need to be taken up before the tie force can be developed. The effect of this movement is uncertain.

- Allow some rotation to occur at the connection then provide preloaded bolts to prevent further rotation.

  In this case, it is not clear at what point the rotation is to be arrested or what the effect is of the subsequent variable load applied to the connection.

- Design for coincident axial and shear forces from first principles and use grade S355 material in the connection.

  In this case, the design of the connection for coincident axial and shear forces is outside the guidance in the 'Green Book' and the assumption of a pinned connection cannot be made with any confidence. Also, the possibility of weld failure must be considered because of the higher strength of the fin plate.

The important issue is that the structural engineer must be aware of the limitations of the design model for standard simple connections. It must not be assumed that the details from the 'Green Book' can be modified to transmit coincident shear and axial forces without also attracting a bending moment. (This applies equally to steel to steel connections). The concrete core wall must be designed to carry all the forces from the connection.

PROPOSED DESIGN MODEL

The proposed design rules are based on existing codes of practice, where such rules are available. The connection details proposed are chosen to fall within the principles of codified rules. Design of the steel side of the connection is based on Eurocode 3 [4] and the 'Green Book'. The resistance of shear studs is taken from Eurocode 4. The design of the concrete side of the connection and the determination of the tension resistance of reinforcement is based on Eurocode 2[5].

The following table lists the proposed design model assumptions and recommendations.

If the tension force is significant, it may be simpler and more convenient to use a symmetrical arrangement of through bolts as discussed in Section 2.2.

Table 3.1
Proposed design model

Item	Assumptions and recommendations	Notes
1	Shear studs are assumed to carry shear only (combined shear and tension is not verified).	The tension resistance of studs in concrete is only codified in relation to composite floor construction. The elastic failure is limited by crushing of the concrete beneath the projection of the head beyond the shaft. Design rules for shear studs subject to combined shear and tension are not codified.
2	The shear resistance of the studs should exceed the shear resistance of the bolts to ensure that shear failure will occur in the steel part of the connection.	Bolt shear will occur before failure of the studs.
3	Stud diameters are assumed to be either 19, 22 or 25mm. Studs should be long enough to extend inside the wall reinforcement cage.	The Bridge Guidance Notes (SCI P185) give advice on stud lengths.
4	The stud spacing is limited to a minimum of 5 times the stud diameter d in the direction of shear and a minimum of $2.5d$ in the orthogonal direction.	BS EN 1994-1-1 cl. 6.6.5.7(4).
5	Welded reinforcing bars are assumed to resist tension forces only and carry no shear.	The tension in the bars is due to eccentricity moment, tie forces or direct tension.
6	Use a minimum of two bars at the top of the cast-in plate; use four bars (two at the top and bottom) where the tie forces require it.	Tying is an accidental load case and the line of action of the tie force is not critical as long as the tie force can be developed.
7	The gap between welded reinforcement should exceed 6 times the bar diameter to allow a reduction factor on bond length of 0.7 (unless longer bars are acceptable).	BS EN 1992-1-1 Table 8.2: value of α_1. (Closer spacing results in a longer bar).
8	“Poor” bond conditions should be assumed for slip formed walls unless it can be shown “good” bond conditions exist.	This results in a lower design value for the ultimate bond stress. See BS EN 1992-1-1 clause 8.4.2(2).
9	Check the tension resistance of the reinforcement, reduced in the presence of shear.	The cast-in plate is assumed to be stiff in plane therefore the bars are subject to shear and tension although no contribution to shear resistance from the bars is assumed. Distribute the shear force in proportion to the area of the studs and bars.

CONTRACTUAL ARRANGEMENTS AND DESIGN RESPONSIBILITY

It is usual for there to be a structural engineer (organisation or individual) who has overall responsibility for the structural arrangement and stability of the building. This party will usually design the floor beams, floor plates and stability structure which involves determining their sizes and the structural actions transferred between the elements, and the general principles of the connections e.g. whether pinned or fixed. It is usual for the structural engineer to be responsible for the design of any concrete core walls and the arrangement of the reinforcement in them where this form of stability structure is selected. The same party is in the best position to carry out detailed checks on the integrity of the concrete wall when subject to the design actions arising from the connections to steelwork and should do so.

It is important for the smooth execution of a project that the assumptions made by the structural engineer can be realized in practice. It is usual in UK construction for the connection design to be the responsibility of the steelwork contractor. This means there is a contractual as well as a physical interface between elements at the connection of floor beams to concrete core walls. If the structural engineer's assumptions about the connection detail are unrealistic, resolution of the design of the connection can be prolonged because practical connection details are likely to affect the concrete elements, design of which may already be completed. If this is so, significant costs and delays may be incurred.

If the axial force in a beam is high, the beam flanges may need to be connected to the supporting element as well as the web and this clearly entails the development of a bending moment. Attempts to transfer large axial forces through the beam web alone may result in web doubler plates and thick fin plates which do not comply with the limiting parameters for simple connections. Such details, if not considered by the structural engineer, are likely to result in bending moments transferred through the cast-in plates that have not been allowed for in the wall design.

The most effective way of achieving an efficient connection design is to divide the design responsibilities for the connection according to the level of information held by the relevant parties. The split of responsibility that requires least transfer of information is defined by the plane at the face of the concrete wall (see Figure 4.1). The structural engineer however must still understand the form of the steel connection, be aware of the limitations of the simple joint model and understand what loads will be exerted on the concrete.

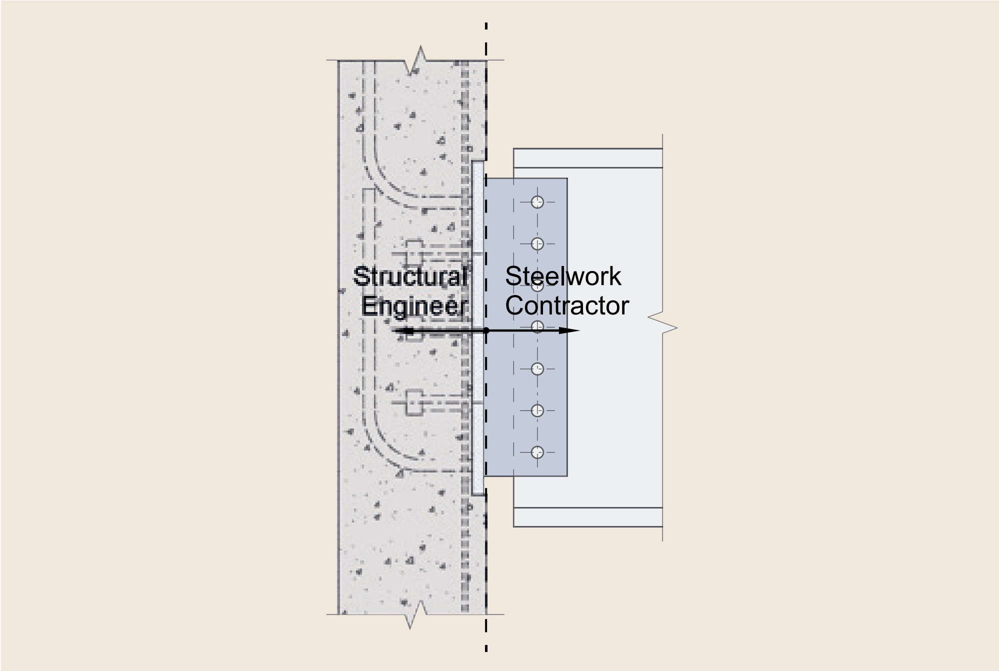


Figure 4.1
Split of design responsibility

The design resistance and setting out of the embedded elements (shear studs and reinforcement) depends on the concrete specification (strength grade) and on the arrangement of the wall reinforcement. Construction tolerances affect the way forces are applied to the cast-in plates and these are generally all specified by the structural engineer (See section 5).

These design responsibilities are reflected in the procedure given in Table 4.1, which sets out the suggested design procedure for the cast-in plate and the checks on the supporting wall.

Table 4.1
Design procedure

Item	Activity	Code reference	By whom
1	Assess incoming beam size and design forces: do the parameters conform to "simple" construction? (ie involve shear force and tie force only in separate load cases).		Structural Engineer
	Simple Design		
2	Determine the maximum deviations for detailing the connections.	NSCS/NSSS	Structural Engineer/Main Contractor
3	Determine the eccentricity moment: See Section 2.1.		Structural Engineer
4	Determine the number of shear studs (19 mm, 22 mm or 25 mm diameter). See Appendix A for a table of resistances.	BS EN 1994-1-1 Clause 6.6.3.1	Structural Engineer
5	If the tie force cannot be taken by the slab rebar, choose two or four bars to resist the tie force and detail them. See Appendix A for a table of resistances.	BS EN 1992-1-1 Clauses 3.1, 8.4	Structural Engineer
6	Check the tension in the top bars from the eccentricity moment (resistance reduced in the presence of shear force: see Section 2.1).		Structural Engineer
7	Check the bearing resistance of the concrete for the compression resulting from the eccentricity moment (see section 2.1).	BS EN 1992-1-1 Clauses 3.1, 6.7	Structural Engineer
8	Determine the cast-in plate size and thickness. It must be deep enough for the fin plate to be welded to it (allowing for deviations).	P358 gives fin plate sizes	Structural Engineer
9	Check the concrete wall for punching shear due to the eccentricity moment.	BS EN 1992-1-1 Clauses 6.4.4, 6.4.5	Structural Engineer
10	Check the concrete wall for punching shear due to the tie force The behaviour is similar to shear force in a concrete slab supported by a column.	BS EN 1992-1-1 Clauses 6.4.4, 6.4.5	Structural Engineer
11	Select fin plate detail from the 'Green Book'.	P358	Steelwork Contractor
12	Confirm the cast-in plate thickness as a function of bending due to a tensile tie force applied to the fin plate (check as a T stub).	P358	Steelwork Contractor
13	Check the bending resistance of the fin plate under the tie force. Fin plate and cast-in plate act together as a T section spanning between bars (See section 2.1.1).	BS EN 1993-1-1	Steelwork Contractor
	Coincident shear and axial force		
14	Determine the design principles of the connection and the design actions. Is the joint to be designed as nominally pinned?		Structural Engineer
15	Check the principles can be realized (can a practical connection be detailed to suit the analysis assumptions – zero bending moment for example).		Structural Engineer
16	Design concrete side of connection (steps as for simple design)	BS EN 1992-1-1	Structural Engineer
17	Design the steel side of connection in accordance with the principles determined by the structural engineer	BS EN 1993-1-1, Clause 1-8	Steelwork Contractor

CONSTRUCTION TOLERANCES AND THEIR IMPLICATIONS

Construction of a concrete core wall (like all building activities) is an imperfect process which will result in a structure that is not in its nominal position. The concrete contractor's work is subject to a specification which sets out the maximum allowable deviations of the built structure from the ideal. The National Structural Concrete Specification (NSCS) often forms the basis for the specific project specification. Fabrication and erection of structural steelwork are independent processes which are also subject to a specification with a similar function. The National Structural Steelwork Specification (NSSS) usually forms the basis for the project specification for the steelwork. Where the steel and concrete elements come together, the effect of the deviations is clearly seen. The result is that the connection details do not follow their nominal arrangements and the transfer of forces through the joint is affected by eccentricities resulting from the deviations from the nominal position.

According to the NSCS the permitted deviation due to inclination of a concrete wall from any vertical plane through its intended design centre at base level varies with height and is the smaller of 50 mm and $H/(200\sqrt{n})$ where H is the free height at the location of interest and n is the number of storeys. The NSSS allows the deviation of a column centre line relative to a vertical line through the column centre at its base to be $H/(300\sqrt{n})$. Deviations of cast-in fixing plates from their theoretical positions are $\pm$10 mm in any direction according to both national specifications. The two deviations (plan position of concrete element and cast-in plate) are independent of each other and it is unlikely that the maximum theoretical deviations in both occur simultaneously. The maximum combined deviation of a cast-in plate can be determined from the square root of the sum of the squares (SRSS) of the individual deviations. The permitted deviations of the structure at a given height allowed by the national specifications are clearly different and deviations to be considered in the design of the cast-in plates must be agreed. The structural engineer and the main contractor will play the leading role in this activity.

Cast-in fixings are set out relative to a grid position during construction independently of the position of the concrete wall (unless the position is close to the edge) hence the inclusion of deviations of ±10 mm in the national specifications. Such tolerances are difficult to achieve on-site and larger tolerances are often agreed.

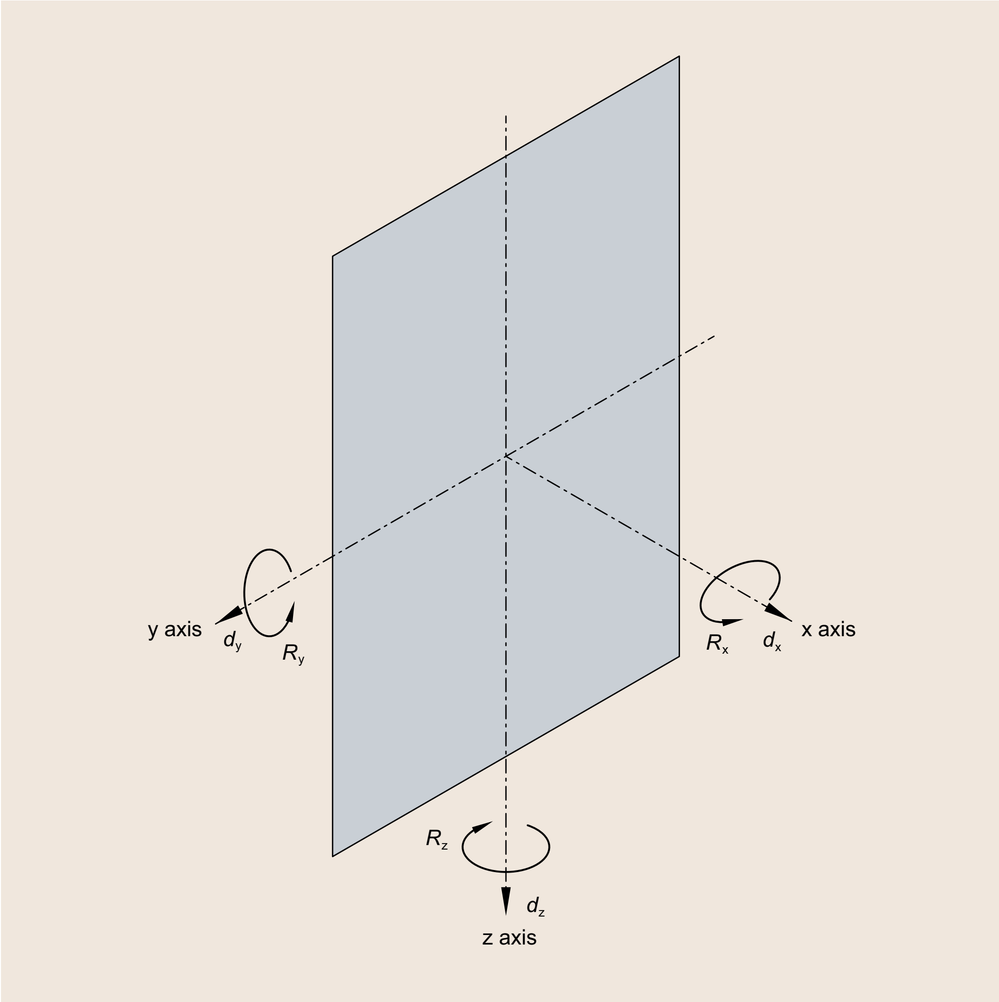


Figure 5.1
Positional and rotational deviations

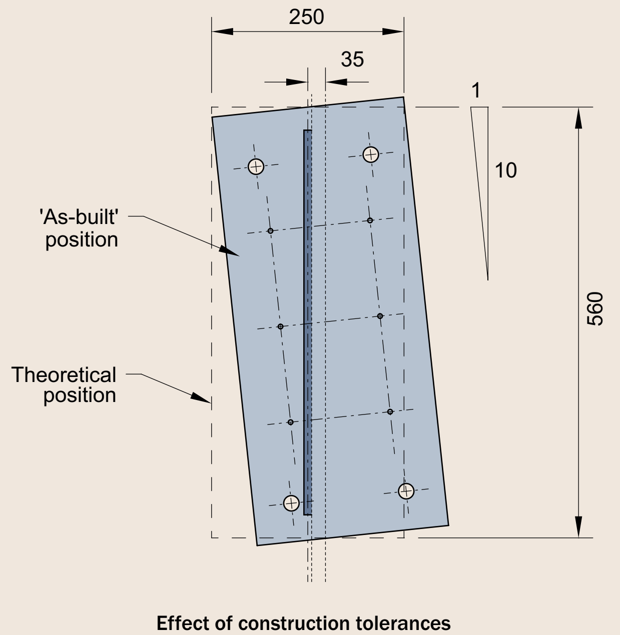
Deviations in the position of a cast-in plate relative to the connecting steelwork can be in any of six degrees of freedom, three positional and three rotational (see Figure 5.1). The effects of the deviations on the internal forces in the connection vary in significance. Of the positional deviations, vertical deviations (d_z) do not affect the distribution of shear force between shear studs and are not significant provided the bottom of the welded attachment still fits on the cast-in plate. Horizontal deviations parallel to the face of the wall (d_y) bring the line of action of the shear force closer to one vertical row of shear studs than the other, thereby increasing the force in the nearer row of studs. Deviations perpendicular to the face of the wall (d_x) increase or reduce the eccentricity moment applied to the face of the cast-in plate.

Typical maximum deviations in plan position (d_x and d_y) relative to the connecting steelwork considered in the design of the cast-in plate are ±35 mm, due to inclination of the core wall and deviation of the cast-in plate. The effect of a 35 mm sideways offset of a fin plate from the vertical centreline through a cast-in plate is profound if the vertical lines of shear studs are close together. If there is little margin between the resistance of a stud and the design shear force for a theoretical arrangement, a 35 mm offset is likely to result in the resistance of the stud being exceeded. It is therefore necessary to design for the worst case out of position rather than nominal positions.

The effect of the d_x deviation (perpendicular to the wall) is accommodated by detailing the fin plate after the positional survey of the corresponding cast-in plate. The beam will usually have already been fabricated and the length of the beam must be shorter than the nominal length to allow for the potential deviation in the core position. The minimum gap between the end of the beam and wall must be maintained to allow the beam to rotate. The actual length of the fin plate will be either shorter or longer than the nominal length, depending on the results of the survey. The design of the fin plate must allow for the deviation because it is of the same order of magnitude as the nominal eccentricity.

Once detailed, the fin plate is manufactured and welded with a two-sided fillet weld. The cast-in plate must be long enough to allow for any vertical deviations such that the fin plate does not project over either end.

Rotational deviations about the axes parallel to the face of the concrete wall (R_y and R_z) can usually be neglected because they do not impact the flow of forces significantly. Rotational deviations about the axis perpendicular to the face of the concrete wall (R_x) affect the distribution of shear forces between the shear studs. However, the likely lateral component of the deviation results in an increase in design force which is less than for a sideways offset.



Effect of construction tolerances
Sideways deviation and rotation
(Illustration only)

Figure 5.2 Construction tolerances

CONSTRUCTION DETAILS AND HEALTH AND SAFETY ISSUES

6.1 Welding: Studs and Reinforcement

It is assumed that studs will be welded using electric arc stud-welding equipment as described in SCI publication P300 [6] and elsewhere. The maximum diameter of studs usually welded with this type of equipment is 25 mm.

Welding of reinforcement is required to achieve the tension strength of the bar and the design strength of the weld is taken from BS EN 1993-1-8 [7]. A satisfactory butt weld is difficult to achieve because any circumferential preparation results in a point or a small diameter flat disc at the centre of the bar. Two circumferential fillet welds can be provided by drilling the cast-in plate and providing a counter sink on the outside face. See Figure 6.1.

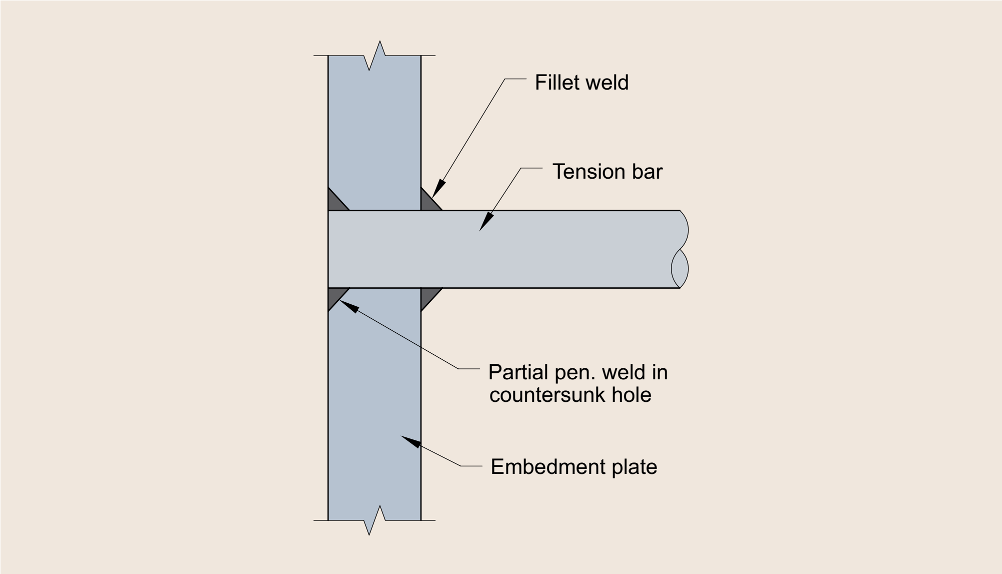


Figure 6.1 Suggested weld detail – cast-in plate to reinforcement

6.2 Thermal Expansion of the Cast-in Plate

A fin plate is welded to a cast-in plate after the concrete core or shear wall has been cast. If design is in accordance with the 'Green Book', the fin plate is 10 mm thick and two 8 mm site fillet welds are used to connect the fin plate to the cast-in plate. The site welding results in a temporary increase in the temperature of the cast-in plate. If an interpass temperature for the weld of 250 °C is used as a basis for determining the thermal expansion, the expected free expansion of a 250 mm wide plate is about 0.7 mm.

In practice the actual temperature rise will be less than this because of cooling by conduction into the concrete and by radiation and the thermal expansion is likely to be too small to cause any cracking of the concrete round the edge of the plate.

6.3 Cast-in Plate Material Grade

The grade and sub-grade of material for the cast-in plate can be the same as for other steelwork in the building of similar thickness. The fin plate material needs to be S275 to conform to the guidance in the 'Green Book'. The eccentricity moment applies a tension to the face of the cast-in plate through the fin plate welds and the cast-in plate deforms in bending to transfer the force to the reinforcing bars. The deformation experienced by the cast-in plate relieves the restraint in the welded joint and there is no need to specify plate with throughthickness properties. The tie force is also transferred through the fin plate welds to the face of the plate but this is defined as an accidental load case. The plate material grade should not be selected for an accidental load case.

6.4 Installation

Cast-in plates may weigh 40 kg or more for primary beams so handling for installation may need to be by means of a form-mounted mechanical hoist. Lifting eyes should be provided on the cast-in plate to facilitate handling where appropriate.

If the cast-in plate arrangement is not symmetrical, the lack of symmetry should be made obvious to reduce the chance of installation upside-down.

In slip-formed construction, it may be preferable not to detail the tie bars projecting downward below the bottom of the cast-in plate to avoid a potential clash with concrete already poured in the core wall. In addition, it is good practice to set the face of the plate inside the face of the wall (say 5 mm) so that the sliding form will not catch on the bottom of the plate and carry it upwards.

6.5 Painting and Fire Protection

The cast-in plate and fin plate will need to receive a site-applied corrosion protection system if corrosion protection of the steelwork is specified. Parts of the connection (e.g. the fin plate) will also require fire protection as for the connecting beam.
An assessment may be carried out to determine if the cast-in plate requires fire protection. As the plate is only exposed to fire on one face, an assessment may show that protection is not required.

CONCLUSIONS

It is hoped that the proposed design model will provide a standard approach for the design of a cast-in plate supporting a fin plate connection to a beam in simple construction. The detailed design of connections which are required to carry coincident shear and axial load has not been dealt with in this guide as these are outside the scope of connections covered by the 'Green Book' for simple connections. The example in the Appendix demonstrates the verification of a typical detail required by the proposed approach.

REFERENCES

[1] *Joints in steel construction: Simple joints to Eurocode 3.* SCI publication P358.
SCI and BCSA, 2014.

[2] Johnson, R.P. and Buckby, R.J.
Concrete structures of steel and concrete Volume 2: Bridges, Second Edition.
Collins, 1986.

[3] BS EN 1994-1-1:2004,
Eurocode 4: Design of composite steel and concrete structures – Part 1-1: General rules and rules for buildings.
(Incorporating corrigendum April 2009).
BSI, 2005.

[4] BS EN 1993-1-1:2005+A1:2014,
Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings.
(Incorporating corrigenda February 2006 and April 2009).
BSI, 2005.

[5] BS EN 1992-1-1:2004+A1:2014,
Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings.
(Incorporating corrigenda January 2008, November 2010 and January 2014).
BSI, 2004.

[6] Rackham, J.W.; Couchman, G.H. and Hicks, S.J.
Composite slabs and beams using steel decking: Best practice for design and construction
(Revised Edition), MCRMA Technical Paper No. 13, SCI publication P300.
MCRMA and SCI, 2009.

[7] BS EN 1993-1-8:2005,
Eurocode 3: Design of steel structures - Part 1-8: Design of joints. (Incorporating corrigenda December 2005, September 2006, July 2009 and August 2010).
BSI, 2005.

CREDITS

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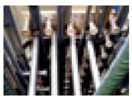

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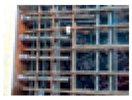

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APPENDIX A: DESIGN CALCULATIONS FOR CAST-IN PLATE

A.1 Resistances of shear studs and reinforcement

In the tables below resistances of arrays of shear studs and tying reinforcement have been calculated for various stud diameters and bar diameters. These have been provided as an aid to scheme design of a connection. Stud strengths are based on the steel strength which governs for strength class 30 and above. An ultimate strength of 450 MPa has been assumed. The tying resistance is calculated for the accidental load case with $\gamma_m = 1.0$. The tension resistance is for the permanent load case with $\gamma_m = 1.15$. Resistances are in kN.

Table A.1 Shear stud resistances – Concrete Strength Class 40

Stud diameter (mm)	19	22	25
Array		Resistance	
1	81.7	109	141
2	163	219	283
2 x 2	327	438	565
2 x 3	490	657	848
2 x 4	653	876	1131
3 x 3	735	985	1272
2 x 5	817	1095	1414
3 x 4	980	1314	1696
4 x 4	1307	1752	2262
4 x 5	1633	2190	2827

Table A.2 Reinforcement tying resistance – Steel grade B500

Bar diameter	10	12	16	20	25
No. of bars			Resistance		
1	39.3	56.6	100	157	245
2	79	113	201	314	491
3	118	170	302	471	736
4	157	226	402	628	982
6	236	339	603	942	1473

Table A.3 Reinforcement tension resistance – Steel grade B500

Bar diameter	10	12	16	20	25
No. of bars			Resistance		
1	34.1	49.2	87.4	137	213
2	68.3	98.3	175	273	427

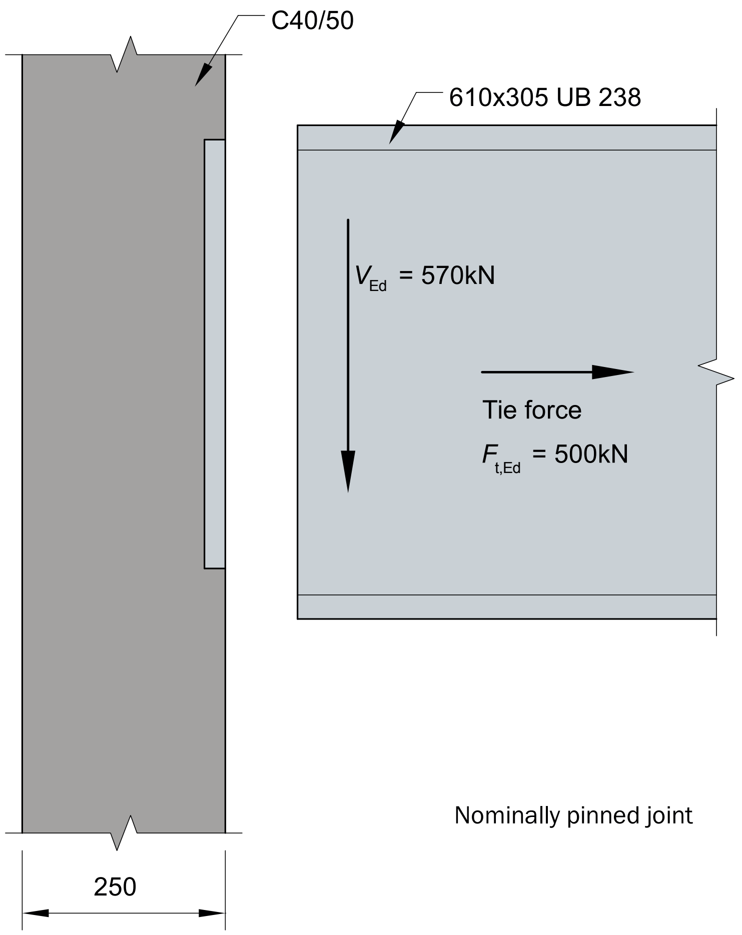
A.2 Calculations

An illustrative verification of a sample cast-in plate is presented below.

SCI Steel Knowledge
Silwood Park, Ascot, Berks SL5 7QN
Telephone: (01344) 636525
Fax: (01344) 636570

CALCULATION SHEET

Job No. CDS 361		Sheet 1 of 23	Rev
Title Cast-in Plates			
Subject Sample Design for UB610			
Client	Made by JRH	Date Mar 2017	
	Checked by DGB	Date Sep 2017	



Positional deviations of cast-in plate

Positional deviations: Level ± 10 mm; Plan: ± 35 mm parallel and perpendicular to the wall surface.

Choose the fin plate arrangement

From P358 Table G.18: choose a 500 deep × 10 thick fin plate with 7 rows of M20 bolts. Steel grade of fin plate: S275

Shear resistance = 576 kN > 570 kN OK

Tying resistance = 748 kN > 500 kN OK

Choose initial shear stud arrangement

From Appendix A Table A.1

Shear studs on cast-in plate: choose a 2 × 3 array of 25mm diameter studs: shear resistance = 848 kN to allow a margin of resistance to cover a non-uniform distribution resulting from the fin plate installation deviations. Assume two columns of 3 studs spaced 130 mm horizontally and 125 mm vertically.

Choose initial reinforcement arrangement

From Appendix A Table A.2

Reinforcement on cast-in plate for bending and tying resistance: choose four bars, two at the top and two at the bottom of the fin plate. Allow a margin of resistance to cover a non-uniform distribution for installation deviations: choose 25 mm diameter bars, B500 grade: tension resistance = 982 kN > 500 kN — OK

Cast-in plate: adopt steel grade S355; initially assume a 25 mm thick plate: yield strength = 345 MPa for $16 < t \leq 40$ mm.

Concrete strength is C40/50.

Confirm the shear stud arrangement

Shear resistance of studs (25 mm ϕ)

Design shear resistance of a single stud is the smaller of:

$P_{\mathrm{Rd}} = (0.8 f_{\mathrm{u}} \pi d^2/4)/\gamma_{\mathrm{v}} = 0.16 f_{\mathrm{u}} \pi d^2$ — EN 1994-1-1 Cl 6.6.3.1

and $P_{\mathrm{Rd}} = 0.29 \alpha d^2 \sqrt{f_{\mathrm{ck}} E_{\mathrm{cm}}}/\gamma_{\mathrm{v}} = 0.232 \alpha d^2 \sqrt{f_{\mathrm{ck}} E_{\mathrm{cm}}}$

The value of the partial factor $\gamma_{\mathrm{v}} = 1.25$ — EN 1994-1-1. NA.2.3

and $\alpha = 0.2(h_{\mathrm{sc}}/d + 1)$ for $3 \leq h_{\mathrm{sc}}/d \leq 4$ and $\alpha = 1$ for $h_{\mathrm{sc}}/d > 4$

where h_{sc} is the stud height and d the stud diameter; h_{sc} is assumed to be 100 mm.

Strength of shear studs: $f_{\mathrm{u}} = 450$ MPa

$f_{\mathrm{ck}} = 40$ MPa; $E_{\mathrm{cm}} = 22[(f_{\mathrm{cm}})/10]^{0.3}$ GPa $f_{\mathrm{cm}} = f_{\mathrm{ck}} + 8$ (MPa) — EN 1992-1-1 Table 3.1

$E_{\mathrm{cm}} = 22 \times [48/10]^{0.3} = 35.2$ GPa

Minimum resistance of studs in steel and concrete (6 studs)

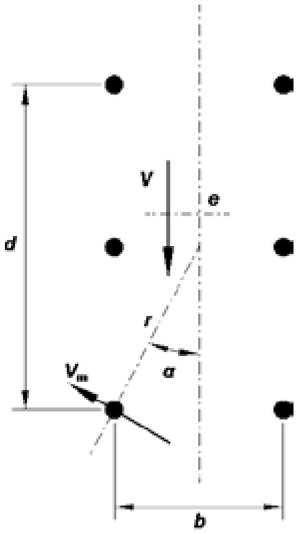
$$\mathrm{MIN} \begin{bmatrix} 6 \times 0.16 \times 450 \times \pi \times 25^2 \times 10^{-3}\ \mathrm{kN} \\ 6 \times 0.232 \times 1.0 \times 25^2 \times \sqrt{40 \times 35.2 \times 10^3} \times 10^{-3}\ \mathrm{kN} \end{bmatrix}$$

$$= \mathrm{MIN} \begin{bmatrix} 848\ \mathrm{kN} \\ 1032\ \mathrm{kN} \end{bmatrix} = 848 > 570\ \mathrm{kN}$$ — OK

Minimum resistance per stud = 848/6 = 141 kN

Consider positional deviations

A horizontal deviation of 35 mm parallel to the face of the wall from the nominal position is considered in the calculation.



Moment due to eccentricity

$M_T = Ve = 570 \times 35 \times 10^{-3} = 20.0$ kNm

Polar moment of stud group

$I_p = \Sigma r^2 = I_x + I_y$

$$= 4\left(\frac{d}{2}\right)^2 + 6\left(\frac{b}{2}\right)^2$$

Substituting values:

$$I_p = 4 \times \left(\frac{250}{2}\right)^2 + 6 \times \left(\frac{130}{2}\right)^2 = 87850 \text{ mm}^2$$

Shear in the extreme stud due to the moment

$$V_m = \frac{M_T r}{I_p}$$

$$r = \sqrt{\left(\frac{d}{2}\right)^2 + \left(\frac{b}{2}\right)^2} = \sqrt{125^2 + 65^2} = 141 \text{ mm}$$

$$V_m = \frac{20.0 \times 141 \times 10^3}{87850} = 32.1 \text{ kN}$$

$\sin\alpha = 65/141$ $\cos\alpha = 125/141$

Horizontal component =

$$V_m \cos\alpha = \frac{32.1 \times 125}{141} = 28.5 \text{ kN}$$

Vertical component = $32.1 \times 65/141 = 14.8$ kN

Max shear in corner stud =

$$\sqrt{(570/6 + 14.8)^2 + 28.5^2} = 113 \text{ kN} < 141 \text{ kN}$$

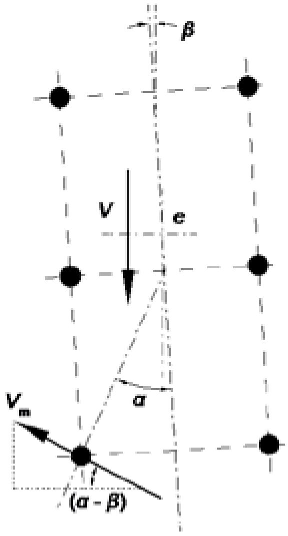
OK

Consider the effect of additional rotational deviation

If a rotational deviation β is present in addition to a lateral one, the moment with respect to the centroid of the stud group is the same:

$M_T = Ve$

The I_p of the stud group is the same and the magnitude of the maximum shear force V_m is the same. The vertical and horizontal components are modified by the rotational deviation.



For the stud shown, the horizontal component = $V_m \cos(\alpha - \beta)$; the vertical component = $V_m \sin(\alpha - \beta)$

A worse case occurs where α and β are additive:

Illustration: let $\beta = \tan^{-1}(1/10) = 5.71°$

$\alpha = \tan^{-1}(65/125) = 27.47°$

$V_m \cos(\alpha + \beta) = 26.9$ kN $V_m \sin(\alpha + \beta) = 17.6$ kN

Max shear =

$\sqrt{(570/6 + 17.6)^2 + 26.9} = 116\text{ kN} < 141\text{ kN}$ OK

The shear stud arrangement is satisfactory.

Reinforcement arrangement and bar size

Eccentricity moment on the cast-in plate

The nominal eccentricity from the bolt row to the back of the cast-in plate perpendicular to the face of the wall:

Eccentricity e_p = plate thickness + end distance of web + nominal gap

Positional tolerance = ±35 mm; inset for slip forming = 5mm

Minimum gap = 10 mm

Nominal gap = 10 + 35 mm

Maximum gap = 10 + 2 × 35 = 80 mm

Maximum eccentricity for 25 mm thick plate:

$e_p = 25 + 5 + 80 + 40 = 150$ mm

Maximum bending moment due to eccentricity

$M = 570 \times 0.150 = 85.5$ kNm

Vertical lever arm ℓ for tension - compression couple

The eccentricity moment is resisted by a tension in the top bars balanced by a compression at the bottom of the fin plate. The distance between the tension and compression forces is less than the depth of the fin plate. Set the top bars 40 mm below the top of the fin plate. Initially assume 40 mm depth of fin plate is required to resist the compression force.

Depth of fin plate = 500 mm

Lever arm $\ell = 500 - 40 - 40/2 = 440$ mm

Tension/compression =

$$\frac{85.5}{0.44} = 194 \text{ kN}$$

Required steel area of fin plate to resist 194 kN

$$A = \frac{194 \times 10^3}{275} = 705 \text{ mm}^2$$

Depth of fin plate to resist thrust in bearing:

$$d = \frac{705}{10} = 70.5 \text{ mm}$$

Lever arm $\ell = 500 - 40 - 70.5/2 = 425$ mm

Tension/compression = 85.5/0.425 = 201 kN

This is considered to be sufficiently close that further iteration is not required.

Confirm the bar size

Try a 20 mm diameter bar. From Table A.3, the resistance in tension is 137 kN.

Consider the eccentricity of loading due to the horizontal deviation of 35 mm. Using a simple beam model and a bar spacing of 150 mm (the gap between bars is greater than 6ϕ which allows a reduced bond length), the maximum force in a bar is:

$$F_{t,Ed} = \frac{201 \times (150/2 + 35)}{150} = 147 \text{ kN}$$

This exceeds the resistance of a 20 mm diameter bar. Adopt 25 mm diameter bars. Use a lateral bar spacing of 155 mm (Note: the gap between bars < 6ϕ)

For the persistent load case, $\gamma_s = 1.15$ — EN 1992-1-1 Cl 2.4.2.4(1) Table NA.1

$$F_{\mathrm{t,Rd}} = \frac{A f_{\mathrm{yk}}}{\gamma_{\mathrm{s}}} = \frac{\pi \times 25^2 \times 500 \times 10^{-3}}{4 \times 1.15} = 213 > 147\ \mathrm{kN}$$ OK

Resistance of 2 bars = 213 × 2 = 426 > 201 kN OK

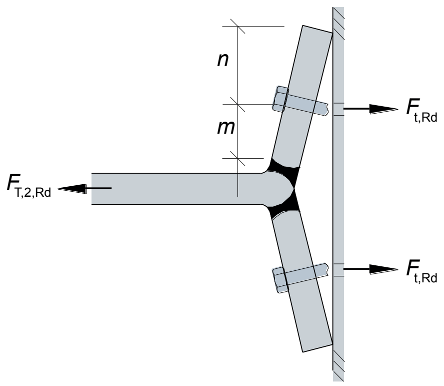
Bending of the cast-in plate

Check the resistance of the cast-in plate in bending for the eccentricity moment tension.

Tension in top bars due to the eccentricity moment = 201 kN

Use the resistance model for end-plate design from BS EN 1993-1-8 Cl 6.2.4

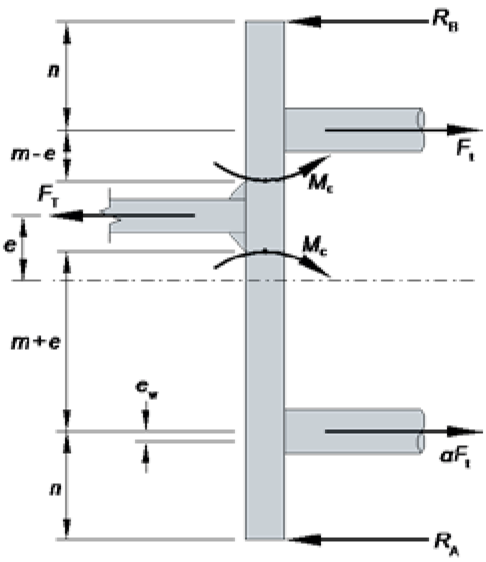
Consider Mode 2 failure: bolt failure with yielding of the flange. In this case the bars are substituted for bolts.



$$F_{\mathrm{T,2,Rd}} = \frac{2M_{\mathrm{pl,2,Rd}} + n\sum F_{\mathrm{t,Rd}}}{(m+n)}$$

The formula for resistance of the plate is derived using symmetry about the vertical centre-line and by considering half the plate and taking moments about the edge of the plate. The prying force at this position does not feature in the formula. The fin plate is not centred between the bars so the Mode 2 formula given above is not directly applicable.

Consider an eccentricity e relative to the centre line



$$M_{\mathrm{C}} + R_{\mathrm{A}}\left(\left(n + (m + e)\right)\right) - \alpha F_{\mathrm{t}}(m + e) = 0 \qquad \text{(i)}$$

$$M_{\mathrm{C}} + R_{\mathrm{B}}\left((n + (m - e))\right) - F_{\mathrm{t}}(m - e) = 0 \qquad \text{(ii)}$$

Determine the effective width of the T stub using the approach in SCI publication P398 Table 2.2. Choose the minimum effective length of the possible yielding patterns: consider i) corner yielding away from a stiffener or flange: $l_{eff,nc} = 2m + 0.625e_2 + e_x$

ii) circular yielding: $l_{erff,cp} = 2\pi m$ iii) side yielding: $l_{eff,nc} = 4m + 1.25e_2$

Use the average value of m on each side of the fin plate ie assume the fin plate is centred

$$m = \frac{w - t_{\mathrm{fp}} - 2 \times 0.8 \times s}{2}$$

where w is the bar spacing, t_{fp} is the thickness of the fin plate and s is the weld leg length.

From above, the bar spacing w = 155 mm.

$$m = \frac{155 - 10 - 2 \times 0.8 \times 8}{2} = 66.1 \text{ mm}$$

Edge distance e_2 = 0.5 × (plate width –w) = 0.5 × (250 – 155) = 47.5 mm

End distance e_x = 70 mm

i) $l_{eff,nc} = 2 \times 66.1 + 0.625 \times 47.5 + 70 = 232$ mm

ii) $l_{erff,cp} = 2 \times 3.14 \times 66.1 = 415$ mm

iii) $l_{eff,nc} = 4 \times 66.1 + 1.25 \times 47.5 = 324$ mm

The minimum effective length is 232 mm

Consider a 25 mm thick plate

$$M_{\mathrm{pl,Rd}} = \frac{0.25 \times 232 \times 25^2 \times 345}{1.0 \times 10^6} = 12.5 \text{ kNm}$$

(γ_{m} = 1.0 for the persistent load case)

n is the minimum of the cast-in plate edge distance (e_2) and 1.25m

$n = e_2$ = 47.5 mm and n < 1.25 × 66.1, so n = 47.5 mm

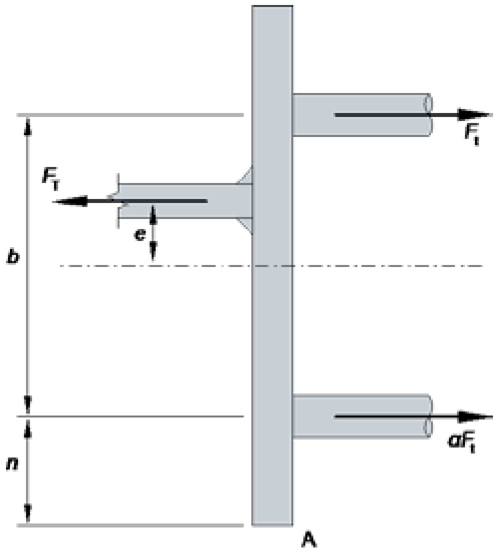
Substituting for M_{C} in equation (ii) with F_{t} = 213 kN; eccentricity e = 35 mm

$m + e = 101.1$ mm; $m - e = 31.1$ mm

$$12.5 \times 10^3 + R_{\mathrm{B}} \times (47.5 + 31.1) - 213 \times 31.1 = 0$$

$$R_{\mathrm{B}} = \frac{213 \times 31.1 - 13.6 \times 10^3}{78.6} = -74.8 \text{ kN}$$

This negative value indicates there is no prying force on one edge of the plate. Failure in Mode 2 does not occur with a 25 mm diameter bar and 25 mm plate.



Take moments about end A. For equilibrium:

$$F_T(n + (b/2 + e)) - F_t(\alpha n + (n + b)) = 0$$

Assuming the plate is rigid, the force in the bars is proportional to the distance from A

$$\alpha = \frac{n}{(n + b)}$$

n = 47.5 mm; b = 155 mm; e = 35 mm; α = 0.235

Substituting values:

$$F_T = \frac{213 \times (0.235 \times 47.5 + (47.5 + 155))}{(47.5 + (155/2 + 35))} = 284 > 201 \text{ kN}$$ OK

Check the bending moment at the face of the fin plate is less than the cast-in plate resistance:

$$M = F_t(m - e)$$

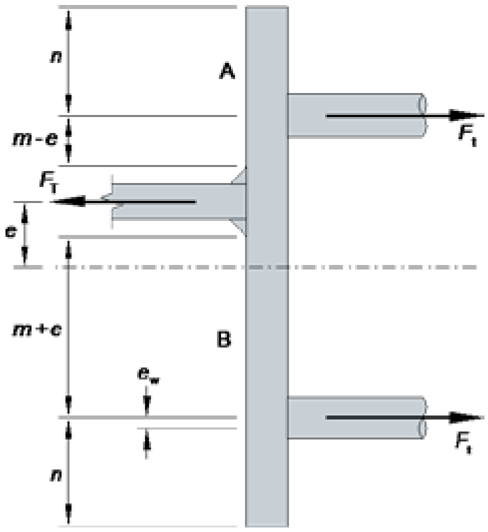
$$M = 213 \times (66.1 - 35) \times 10^3 = 6.62 < 12.5 \text{ kNmm}$$ OK

Check Mode 1 failure:

The calculation Model for Mode 1 failure considers a vertical centre line through the cast-in plate and fin plate giving the formula:

$$F_{\mathrm{Rd,T,1}} = \frac{(8n - 2e_{\mathrm{w}})M_{\mathrm{pl,1,Rd}}}{2\,\mathrm{mn} - e_{\mathrm{w}}(m+n)}$$

In this case, consider a vertical centre-line through the fin plate, offset by eccentricity e from the centre of the plate and consider each part, A and B separately:



$e_{\mathrm{w}} = d_{\mathrm{w}}/4$ where d_{w} is the bar diameter (See SCI publication P398 page 10 resistances of T-stubs).

$$F_{\mathrm{Rd,T,1A}} = \frac{(4n - e_{\mathrm{w}})M_{\mathrm{pl,1,Rd}}}{2\,\mathrm{n}(m-e) - e_{\mathrm{w}}((m-e)+n)}$$

$$F_{\mathrm{Rd,T,1B}} = \frac{(4n - e_{\mathrm{w}})M_{\mathrm{pl,1,Rd}}}{2\,\mathrm{n}(m+e) - e_{\mathrm{w}}((m+e)+n)}$$

$$F_{\mathrm{Rd,T,1}} = F_{\mathrm{Rd,T,1A}} + F_{\mathrm{Rd,T,1B}}$$

$$F_{\mathrm{Rd,T,1A}} = \frac{(4 \times 47.5 - 6.25) \times 12.5 \times 10^3}{2 \times 47.5\ \times (66.1 - 35) - 6.25 \times ((66.1 - 35) + 47.5)} = 932\mathrm{kN}$$

$$F_{\mathrm{Rd,T,1A}} = \frac{(4 \times 47.5 - 6.25) \times 12.5 \times 10^3}{2 \times 47.5\ \times (66.1 + 35) - 6.25 \times ((66.1 + 35) + 47.5)} = 265\mathrm{kN}$$

$$F_{\mathrm{Rd,T,1}} = F_{\mathrm{Rd,T,1A}} + F_{\mathrm{Rd,T,1B}} = 932 + 265 = 1197\ \mathrm{kN}$$

The Mode 1 resistance is greater than 285 kN calculated previously so Mode 1 is not critical.

A 25 mm thick plate is adequate. Adopt a plate 25 thick × 560 long × 250 wide

Check the cast-in plate under the tying load case

Bar strength in tying:

$$F_{\mathrm{t}} = \frac{A f_{yk}}{\gamma_{\mathrm{s}}} = \frac{\pi \times 25^2 \times 500}{4 \times 1.0} \times 10^{-3} = 245\ \mathrm{kN}$$

(γ_{s} for reinforcing steel = 1.0 for the accidental load case)

EN 1992-1-1 Cl 2.4.2.4

Substituting the bar strength in tying into the Mode 2 formula:

$$F_{\mathrm{T}} = \frac{245 \times \left(0.235 \times 47.5 + (47.5 + 155)\right)}{\left(47.5 + \left(\frac{155}{2} + 35\right)\right)} = 327 > \frac{500}{2} = 250\ \mathrm{kN}$$

OK

Check bending of the cast-in plate

Plate resistance is based on the ultimate strength and $\gamma_{mu} = 1.1$:

$$M_{pl,Rd} = \frac{0.25 \times 232 \times 25^2 \times 470}{1.1 \ \times 10^6} = 15.5 \text{ kNm}$$

$$M = F_t(m - e)$$

$$M = 245 \times (66.1 - 35) \times 10^3 = 7.62 \ < 15.5 \text{ kNm}$$

25 mm thick plate is acceptable for the tying load case.

Resistance of the combined cast-in plate and fin plate to bending in the vertical plane

Check the fin plate in bending at the face of the cast-in plate.

The nominal eccentricity moment at the back of the plate calculated above for determining bar sizes is 85.5kNm. This moment is reduced by the assumed plate thickness

$e = 155 - 25 = 130$ mm.

$M = 85.5 \times 130/155 = 71.7$ kNm

The fin plate resistance is:

$M_{Rd} = f_y\, Z/\gamma_m = 275 \times 500^2 \times 10\ /\ (1.0 \times 6) \times 10^{-6} = 115 > 71.7$ kNm   OK

Tying load case

The fin plate and cast-in plate acting as a Tee spans between the top and bottom tie bars when resisting tie forces.

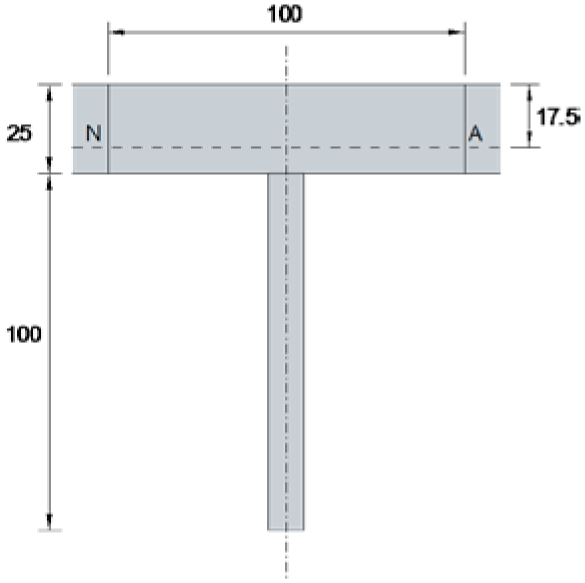
Assume the span is the distance between the top and bottom bars = 0.44 m

The tie force is transferred through the bolts (7 rows of bolts) and so approximates a uniform load

$$M = \frac{WL}{8} = \frac{500 \times 0.44}{8} = 27.5 \text{ kNm}$$

Consider the cast-in plate and fin plate acting as a Tee. The cast-in plate forms the flange. The assumed flange width is $b = 100$ mm (a wider flange could be chosen).

The fin plate forms the stem of the Tee. The minimum length of fin plate occurs if the cast-in plate is not set in 5 mm for slip-forming and the deviation of the wall results in the minimum gap between the wall and the beam end of 10 mm. The standard edge distance for fin plates in the Green Book is 50 mm. Length of Tee stem = 10 + 40 + 50 = 100 mm. The cross section of the Tee resisting the moment is:



Locate the plastic centroid:

Assumed area of Tee section: A = 25 × 100 + 100 × 10 = 3500 mm^2

A/2 = 1750 mm^2. This area is less than the assumed flange area so the neutral axis is in the flange.

Distance from top of flange =

$$y = \frac{A/2}{b} = \frac{1750}{100} = 17.5 \text{ mm}$$

Plastic modulus

Take area moments about the neutral axis:

Flange (1)	1750 × 8.75 =	1.53×10^4
Flange (2)	750 × 3.75 =	2.81×10^3
Web	1000 × 57.5 =	5.75×10^4

$\Sigma = 7.56 \times 10^4$ mm^3

$$M_R = \frac{410}{1.1} \times 7.56 \times 10^4 \times 10^{-6} = 28.2 \text{ kNm} > 27.5\text{kNm}$$ OK

(accidental load case: f_u = 410 MPa for the fin plate and γ_{mu} = 1.1)

Bearing resistance of concrete at bottom of fin plate

Compression at bottom of fin plate

$F_{c,td}$ = 201 kN

$$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c}$$ EN 1992-1-1 Cl 3.1.6(1)

α_{cc} = 0.85; γ_c = 1.5 (persistent load case) EN 1992-1-1 Table NA.1

$$f_{cd} = \frac{0.85 \times 40}{1.5} = 22.7 \text{ MPa}$$

Vertical dimension for bearing assuming a spread of 2.5 to 1 through the cast-in plate

$d = 73 + 5 \times 25 = 198$ mm

Horizontal dimension = $10 + 2 \times 8 + 5 \times 25 = 151$ mm

$$F_{Rd,u} = \frac{198 \times 151 \times 22.7}{1.5} \times 10^{-3} = 452 > 201 \text{ kN}$$ OK

Detailing of reinforcement for eccentric moment

Bond strength

Bond strength f_{bd} between reinforcement and concrete is given by:

$f_{bd} = 2.25\,\eta_1\,\eta_2\,f_{ctd}$ — EN 1992-1-1 Cl 8.4.2

where f_{ctd} is the design value of the concrete tensile strength and η_1 and η_2 are coefficients related to bond quality and bar diameter respectively.

$f_{ctd} = \alpha_{ct}\,f_{ct\,k,0.05}/\gamma_c : f_{ct\,k,0.05} = 0.7\,f_{ctm}$ where f_{ctm} is the mean tensile strength

$f_{ctm} = 0.3\,f_{ck}^{(2/3)}$ for concrete grade less than or equal to C50/60 — Cl 3.1.6(2)

$\alpha_{ct} = 1.0$ — Table NA.1

For the persistent load case, $\gamma_c = 1.5$.

$f_{ctd} = 1.0 \times 0.7 \times 0.3 \times 40^{(2/3)}/1.5 = 1.64$ MPa — Cl 3.1.6(3)

$\eta_1 = 0.7$ (cannot show good bond conditions)

$\eta_2 = 1.0\ \phi \leq 32$

$f_{bd} = 2.25 \times 0.7 \times 1.0 \times 1.64 = 2.58$ MPa

Bond length to resist 213 kN (bar strength)

$$\ell_{b,req} = \frac{F_t}{f_{bd}\,\pi\,\phi}$$ Cl 8.4.3(2)

$$\ell_{b,req} = \frac{213 \times 10^3}{2.58 \times \pi \times 25} = 1051 \text{ mm}$$

$\ell_b = \alpha_1\,\alpha_2\,\alpha_3\,\alpha_4\,\alpha_5\,\ell_{b,req}$ — Cl 8.4.4

The α_i are coefficients for various effects given in Table 8.2 of Cl.8.4.4(2)

Bar spacing: the distance between bars is $(155 - \phi) = 130$ mm ($= a$)

$c_d = \min(a/2, c_1)$ (c_1 (side cover) $> a/2$)

$= 65$ mm $< 3\phi$ Therefore $\alpha_1 = 1.0$ — Table 8.2

α_2: effect of concrete cover:

$\alpha_2 = 1 - 0.15 \times (77.5 - 3 \times 25)/25 = 0.985$

$\alpha_3 = 1.0$ (main steel not known)

$\alpha_4 = 1.0$ (no welded transverse bars)

$\alpha_5 = 1.0$ (no transverse pressure assumed)

ℓ_{bd} = 1051 × 1.0 × 0.985 = 1035 mm

Geometry of bent bar

The bar anchorage length will extend past the end of the bend by more than 5ϕ (= 125 mm) so calculate the minimum mandrel diameter to avoid concrete crushing on the inside of the bend.

Min mandrel diameter: $\phi_{m,min} = F_{bt}((1/a_b + 1/(2\phi))/f_{cd}$ — 8.3.3(3) Eqn 8.1

F_{bt} is the tensile force in the bar from ultimate loads.

a_b is half the centre to centre distance between bars.

$$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c}$$

EN 1992-1-1 3.1.6(1)

α_{cc} = 1.0 γ_c = 1.5 (persistent load case) — Table NA.1

$$f_{cd} = \frac{1.0 \times 40}{1.5}$$

= 26.7 MPa

Minimum mandrel diameter:

$\phi_{m,min} = 213 \times 10^3(1/77.5 + 1/50)/26.7$ = 262 mm; use 265 mm

Bend length along centre line of bar

$$\frac{\pi \times (265 + 25)}{4} = 228 \text{ mm}$$

Straight length = 1035 – 228 = 807 mm

Reduced tension resistance of reinforcement in the presence of shear

The bars are assumed to provide all the tension resistance required to carry the eccentricity moment. However, the cast-in plate is subject to a coincident vertical shear and the connections to the back of the plate all experience the shear force. The tension resistance of the bars will be assumed to be reduced in the presence of shear force.

Distribute the shear force in proportion to the area of bars and studs.

Shear carried by reinforcement per bar:

$$V = \frac{570}{4} \times \frac{4\,A_r}{6\,A_{st} + 4A_r}$$

$$A_r = \frac{\pi}{4} \times 25^2;$$

$$A_{st} = \frac{\pi}{4} \times 25^2; \text{ substituting: } \frac{4 \times 25^2}{6 \times 25^2 + 4 \times 25^2} = 0.4$$

$$V = 0.4 \times \frac{570}{4} = 57.0 \text{ kN/bar}$$

Von Mises failure criterion for uniaxial stress and shear stress:

$$\sigma_y = \sqrt{\sigma_x^2 + 3\tau^2} \Rightarrow \sigma_x = \sqrt{\sigma_y^2 + 3\tau^2}$$

Rearranging in terms of resistance:

$$F_{\mathrm{t,Rd,red}} = \sqrt{{F_{\mathrm{t,Rd}}}^2 - 3{V_{\mathrm{Ed}}}^2}$$

$$F_{\mathrm{t,Rd}} = \frac{\pi\, 25^2}{4} \times \frac{500 \times 10^{-3}}{1.15} = 213\ \mathrm{kN}$$

($\gamma_s = 1.15$ for persistent load case)

$$F_{\mathrm{t,Rd,red}} = \sqrt{213^2 - 3 \times 57^2} = 188\ \mathrm{kN}$$

Maximum tensile force due to eccentricity moment = 147 kN <188 kN — OK

Detailing of reinforcement for tying action

Use four bars: check 25 mm diameter bar

Force/bar:

$$\frac{500}{4} = 125\ \mathrm{kN/bar}$$

$$F_{\mathrm{t,Rd}} = \frac{A f_{\mathrm{yk}}}{\gamma_{\mathrm{s}}} = \frac{\pi \times 25^2 \times 500}{4 \times 1.0} = 245\ \mathrm{kN} > 125\ \mathrm{kN}$$ OK

($\gamma_s = 1.0$ for the accidental load case) — EN 1992-1-1 Table 2.1N

Bond strength

Bond strength f_{bd} between reinforcement and concrete (a different value applies for the tying load case):

$f_{\mathrm{bd}} = 2.25\ \eta_1\ \eta_2\ f_{\mathrm{ctd}}$ — EN 1992-1-1 Cl 8.4.2

where f_{ctd} is the design value of the concrete tensile strength and η_1 and η_2 are coefficients related to bond quality and bar diameter respectively.

$f_{\mathrm{ctd}} = \alpha_{\mathrm{ct}}\, f_{\mathrm{ct\,k,0.05}}/\gamma_{\mathrm{c}}$: $f_{\mathrm{ct\,k,0.05}} = 0.7\, f_{\mathrm{ctm}}$ where f_{ctm} is the mean tensile strength

$f_{\mathrm{ctm}} = 0.3\, f_{\mathrm{ck}}^{(2/3)}$ for concrete grade less than or equal to C50/60 — Cl 3.1.6(2)

$\alpha_{\mathrm{ct}} = 1.0$ — Table NA.1

For the accidental load case, $\gamma_{\mathrm{c}} = 1.2$.

$f_{\mathrm{ctd}} = 1.0 \times 0.7 \times 0.3 \times 40^{(2/3)}/1.2 = 2.05$ MPa — Cl 3.1.6(3)

$\eta_1 = 0.7$ (cannot show good bond conditions)

$\eta_2 = 1.0$ ($\phi \leq 32$)

$f_{\mathrm{bd}} = 2.25 \times 0.7 \times 1.0 \times 2.05 = 3.23$ MPa

Bond length to resist 245 kN (bar strength)

$$\ell_{b,req} = \frac{F_{\mathrm{t}}}{f_{\mathrm{bd}}\, \pi\phi}$$ Cl 8.4.3(2)

$$\ell_{b,req} = \frac{245 \times 10^3}{3.23 \times \pi \times 25} = 966 \text{ mm}$$

$\ell_b = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_4\ \alpha_5\ \ell_{b,req}$ — Cl 8.4.4

The α_i are coefficients for various effects given in Table 8.2 of Cl 8.4.4(2)

Bar spacing: distance between bars is $(155 - \phi) = 130$ mm ($= a$)

$c_d = \min(a/2, c_1)$ (c_1 (side cover) $> a/2$)

$= 65$ mm $< 3\phi$ Therefore $\alpha_1 = 1.0$ — Table 8.2

α_2: effect of concrete cover:

$\alpha_2 = 1 - 0.15 \times (77.5 - 3 \times 25)/25 = 0.985$

$\alpha_3 = 1.0$ (main steel not known)

$\alpha_4 = 1.0$ (no welded transverse bars)

$\alpha_5 = 1.0$ (no transverse pressure for tying load case)

$\ell_{bd} = 966 \times 1.0 \times 0.985 = 952$ mm

This length is less than for the persistent load case. The persistent load case governs.

Welding reinforcement to cast-in plate

Weld detail to achieve 245 kN resistance (bar strength).

Design strength of weld:

$$f_{v,wd} = \frac{f_u/\sqrt{3}}{\beta_w \gamma_m}$$

The plate has the lowest strength

$f_u = 470$ MPa: $\beta_u = 0.9$

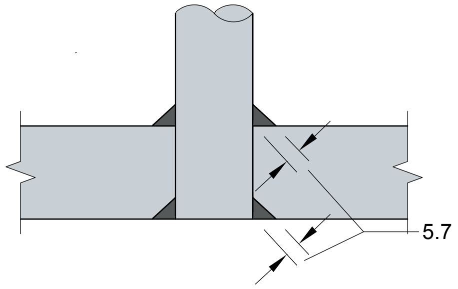
γ_m for tying: use 1.1

$$f_{v,wd} = \frac{470/\sqrt{3}}{0.9 \times 1.1} = 274 \text{ MPa}$$

Assume two welds round the bar will be provided (see detal).

$$a = \frac{245 \times 10^3}{2 \times 274 \times \pi \times 25} = 5.69 \text{ mm}$$

Provide a throat of 5.7 mm



Shear resistance of concrete wall

Checks:

- at the cast-in plate perimeter u_0
- at the control perimeter, u_1
- potentially at a further perimeter in order to identify where shear reinforcement is not required, u_{out}

Punching shear

EN 1992-1-1: 2004 cl 6.4
cl 6.4.3

The maximum design shear stress is given as:

$$v_{Ed} = \frac{\beta\, V_{Ed}}{u_i d} \qquad (6.38)$$

d = effective depth of wall = $(d_y + d_z)/2$ ie the average effective depth of bars in horizontal (y) and vertical (z) directions.

u_i = length of the perimeter being considered (6.4.4(2))

$$\beta = 1 + \frac{k\, M_{Ed}}{V_{Ed}} . \frac{u_i}{W_i}$$

(6.39 with suffix denoting perimeter)

W_i is the plastic moment of the perimeter relative to the axis about which the bending moment acts. (Figure 6.19)

The basic control perimeter is $2d$ from the edge of the cast-in plate

For eccentric loading (substituting for β):

$$v_{Ed} = \frac{V_{Ed}}{u_i d}\left(1 + \frac{k\, M_{Ed} u_i}{V_{Ed}\, W_i}\right)$$

$$= \frac{V_{Ed}}{u_i d} + \frac{k\, M_{Ed}}{W_i d}$$

(6.51 with no reduction for uplift pressure)

v_{Ed} has a direct component and a bending component.

Tie force = V_{Ed} = 500 kN

Consider the bending component only for the eccentricity moment and the direct component only for the tie force as these are separate load cases.

Shear stress on plate perimeter due to the eccentricity moment

Moment M_{Ed} = 85.5 kNm

$$v_{Ed} = \frac{k\, M_{Ed}}{W_0 d}$$

Parameters

k is taken from Table 6.1 in Cl 6.4.3(3)

EN 1992-1-1 Cl 6.4.3(3)

Plate dimensions: depth c_1 = 560 mm; width c_2 = 250 mm

$$c_1/c_2 = \frac{560}{250} \simeq 2.0 \Rightarrow k = 0.7$$

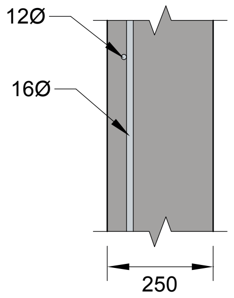
$$W_0 = c_1\left(\frac{c_1}{2} + c_2\right) \text{ at the plate perimeter}$$ (6.41 with d = 0)

$$= 560\left(\frac{560}{2} + 250\right) = 2.97 \times 10^5 \text{ mm}^2$$

Assumed wall details:

Effective depth: assume cover = 30 mm

Assume 16 mm diameter bars at 200 mm centres vertically and 12 mm diameter bars at 200 mm centres horizontally to estimate notional reinforcement ratios for this example.



d = 250 – (30 + 12 + 8) = 200 mm. At the plate perimeter, the effective depth is taken to the back of the plate ie d = (200 – 30) = 170 mm. It is conservative to use this value for the basic control perimeter u_1.

Shear stress due to moment:

$$v_{\text{Ed}} = \frac{0.7 \times 85.5 \times 10^6}{2.97 \times 10^5 \times 170}$$

= 1.19 MPa

At the cast-in plate perimeter, the maximum punching shear stress should not be exceeded: $v_{\text{Ed}} < v_{\text{Rd,max}}$. 6.4.3(2)

$$v_{\text{Rd,max}} = 0.5 v f_{\text{cd}}$$ NA Table NA1 6.4.5(3) 6.6.3(6) (6.6N)

$$v = 0.6\left(1 - \frac{f_{\text{ck}}}{250}\right)$$

f_{ck} = 40 MPa

$$f_{\text{cd}} = \frac{\alpha_{\text{cc}} f_{\text{ck}}}{\gamma_{\text{c}}} = \frac{0.85 \times 40}{1.5} = 22.7 \text{ MPa}$$

(γ_{c} = 1.5 for the permanent load case)

$$v_{\text{Rd,max}} = 0.5 \times 0.6\left(1 - \frac{40}{250}\right) \times 22.7 = 5.72 \text{ MPa}$$

1.19 < 5.72 MPa OK

Shear stress on the basic control perimeter u_1 due to bending moment

To check if punching shear reinforcement is necessary, determine the value of the shear resistance assuming there is no shear reinforcement $v_{Rd,c}$ (which is not less than (v_{min}+ $k_1\sigma_{cp}$)) and check if the design shear stress $v_{Ed} < v_{Rd,c}$.

$$v_{Rd,c} = C_{Rd,c}\, k\, (100\, \rho_1 f_{ck})^{1/3} + k_1\sigma_{cp} \geq (v_{min} + k_1\sigma_{cp})$$ Cl 6.4.4 (6.47)

Parameters

$$C_{Rd,c} = \frac{0.18}{\gamma_c}$$ UK NA to EN 1992-1-1

$$v_{min} = 0.035\, k^{3/2}\, f_{ck}^{\ 1/2} \quad \text{and} \quad k_1 = 0.1$$

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0$$

$$\rho_l = \sqrt{\rho_{ly} \cdot \rho_{lz}} \leq 0.02$$

Substituting values

$$C_{Rd,c} = \frac{0.18}{1.5} = 0.12;\ k = 1 + \sqrt{\frac{200}{170}} = 2.08 \text{ ie use } 2.0$$

(γ_c = 1.5 for persistent load case)

v_{min} = 0.035 × 2.0$^{3/2}$ × 40$^{1/2}$ = 0.626 MPa

Reinforcement ratios for the vertical and horizontal reinforcement in the wall on the side remote from the cast-in plate:

$$\rho_{ly} = \frac{5 \times \pi \times 16^2}{4 \times 250 \times 10^3} = 4.02 \times 10^{-3}$$

$$\rho_{lz} = \frac{5 \times \pi \times 12^2}{4 \times 250 \times 10^3} = 2.26 \times 10^{-3}$$

$$\rho_l = \sqrt{4.02 \times 10^{-3} \times 2.26 \times 10^{-3}} = 3.01 \times 10^{-3} < 0.02$$ OK

Assume σ_{cp} = 0 because under some load cases, the vertical axial compressive stress in the wall may be small or zero

$v_{Rd,c}$ = 0.12 × 2 × (100 × 3.01 × 10^{-3} × 40)$^{1/3}$ + 0

= 0.55 MPa < v_{min} ie use v_{min}

Shear resistance of the wall with no shear reinforcement: $v_{Rd,c}$= 0.626 MPa

Design shear stress on the basic control perimeter u_1:

$$v_{Ed} = \frac{k\, M_{Ed}}{W_1 d}$$

$$W_1 = \frac{c_1}{2} + c_1 c_2 + 4c_2 d + 16d^2 + 2\pi d c_1$$ (6.41)

$$W_1 = \frac{560^2}{2} + 560 \times 250 + 4 \times 250 \times 170 + 16 \times 170^2 + 2\pi \times 170 \times 560$$

$$W_1 = 1.53 \times 10^6 \text{mm}^2$$

$$v_{\text{Ed}} = \frac{0.7 \times 85.5 \times 10^6}{1.53 \times 10^6 \times 170} = 0.230 \text{ MPa}$$

0.230 < 0.626 MPa — OK

(By inspection this is also less than $v_{\text{Rd,max}}$)

The shear stress on th e control perimeter is smaller than the limiting shear stress – no shear reinforcement required for this load case.

Shear stress on plate perimeter due to tie force

An estimate of the shear reinforcement required for tying action will be made, assuming the tie force is carried into the wall through the cast-in plate. Where there is a concrete slab present up to the wall, it will be preferable to transfer the tie force into the wall through the slab. This will avoid the need to carry out shear and bending checks on the wall local to the connection due to tying action, and the provision of punching shear reinforcement and possibly bending reinforcement. No bending checks on the wall will be carried out in this example.

The tie force is a direct force = 500 kN

$$v_{Ed} = \frac{V_{\text{Ed}}}{u_{\text{o}} d} \leq v_{\text{Rd,max}}$$ 6.4.5(3)

The effective depth d = 170 mm

u_{o} is the plate perimeter = 1620 mm

$$v_{\text{Ed}} = \frac{500 \times 10^3}{1620 \times 170} = 1.82 \text{ MPa}$$

$$v_{\text{Rd,max}} = 0.5 v f_{\text{cd}}$$

NA Table NA1
6.4.5(3)
6.6.3(6) (6.6N)

$$v = 0.6\left(1 - \frac{f_{\text{ck}}}{250}\right)$$

f_{ck} = 40 MPa

$$f_{\text{cd}} = \frac{\alpha_{\text{cc}} f_{\text{ck}}}{\gamma_{\text{c}}} = \frac{0.85 \times 40}{1.2} = 28.3 \text{ MPa}$$

(γ_{c} = 1.2 for the accidental load case)

$$v_{\text{Rd,max}} = 0.5 \times 0.6\left(1 - \frac{40}{250}\right) \times 28.3 = 7.13 \text{ MPa}$$

1.82 < 7.13 MPa — OK

Shear stress on the basic control perimeter u_1 due to tie force

Using the value of $v_{Rd,c}$ already calculated for a wall with no shear reinforcement, adjusting for $\gamma_c = 1.2$ for the accidental load case:

$$v_{Rd,c} = \frac{0.55 \times 1.5}{1.2}$$

= 0.688 MPa

This value is greater than v_{min} = 0.626 MPa already calculated so $v_{Rd,c}$ = 0.688 MPa

The basic control perimeter u_1 is at $2d$ from the edge of the plate. It is conservative to use d = 170 mm.

$$u_1 = (560 + 250) \times 2 + 2\pi(2.0 \times 170)$$ 6.4.2

= 3756 mm

Shear stress on the control perimeter

$$v_{Ed} = \frac{500 \times 10^3}{3756 \times 170} = 0.783 \text{ MPa}$$

$$v_{Ed} > v_{Rd,c}$$

Shear reinforcement is required.

Provision of shear reinforcement

The punching shear resistance with reinforcement must be less than k_{max} times the shear resistance without reinforcement. 6.4.5(1) NA Table 1

$$v_{Rd,cs} \leq k_{max} v_{Rd,c} \text{where } k_{max} = 2.0$$

The punching shear resistance with reinforcement $v_{Rd,cs}$ must be at least equal to v_{Ed}.

Substituting v_{Ed} for $v_{Rd,cs}$

$$\frac{v_{Ed}}{v_{Rd,c}} = \frac{0.783}{0.688} = 1.14 < 2.0$$

It will be possible to achieve adequate shear resistance by adding shear reinforcement.

Consider the basic control perimeter u_1

Area of shear reinforcement on the control perimeter:

$v_{Rd,cs} = 0.75\, v_{Rd,c} + 1.5\, (d/s_r)\, A_{sw}\, f_{ywd,ef}\, [1/(u_1 d)] \sin \alpha$ (6.52)

s_r is the radial spacing of the shear reinforcement (mm).

α is the angle between the shear reinforcement and the plane of the wall

A_{sw} is the area of shear reinforcement; $f_{ywd,ef}$ is the effective design strength

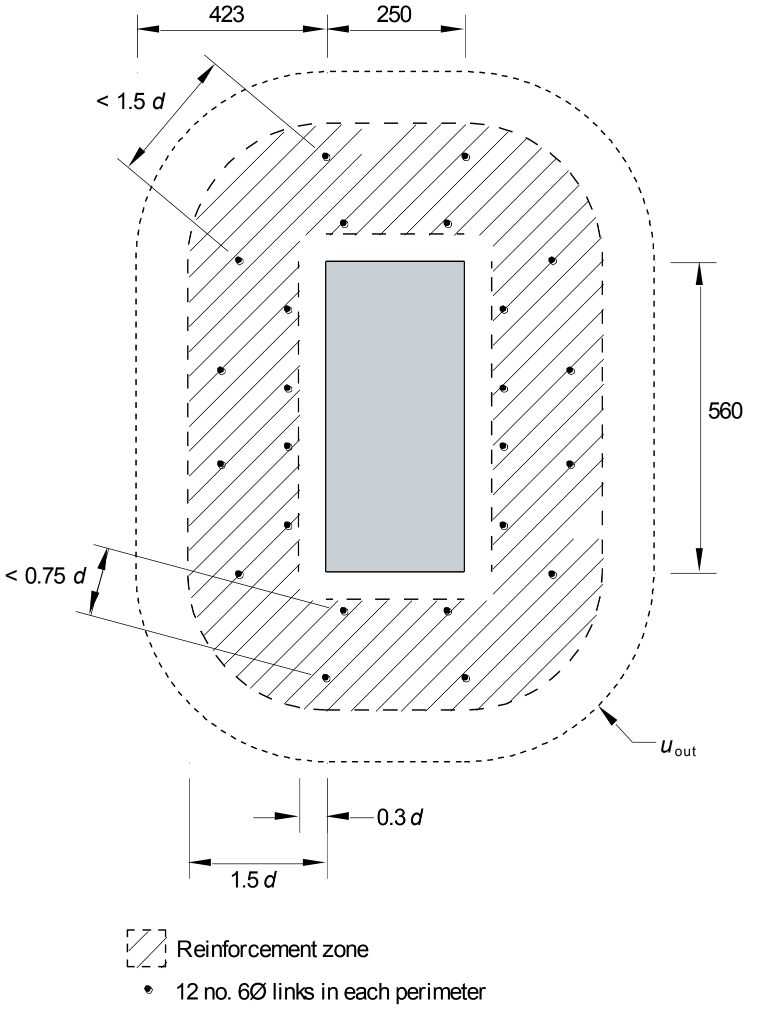
Parameters

$v_{Rd,cs}$ = 0.783 MPa (required shear resistance on control perimeter)

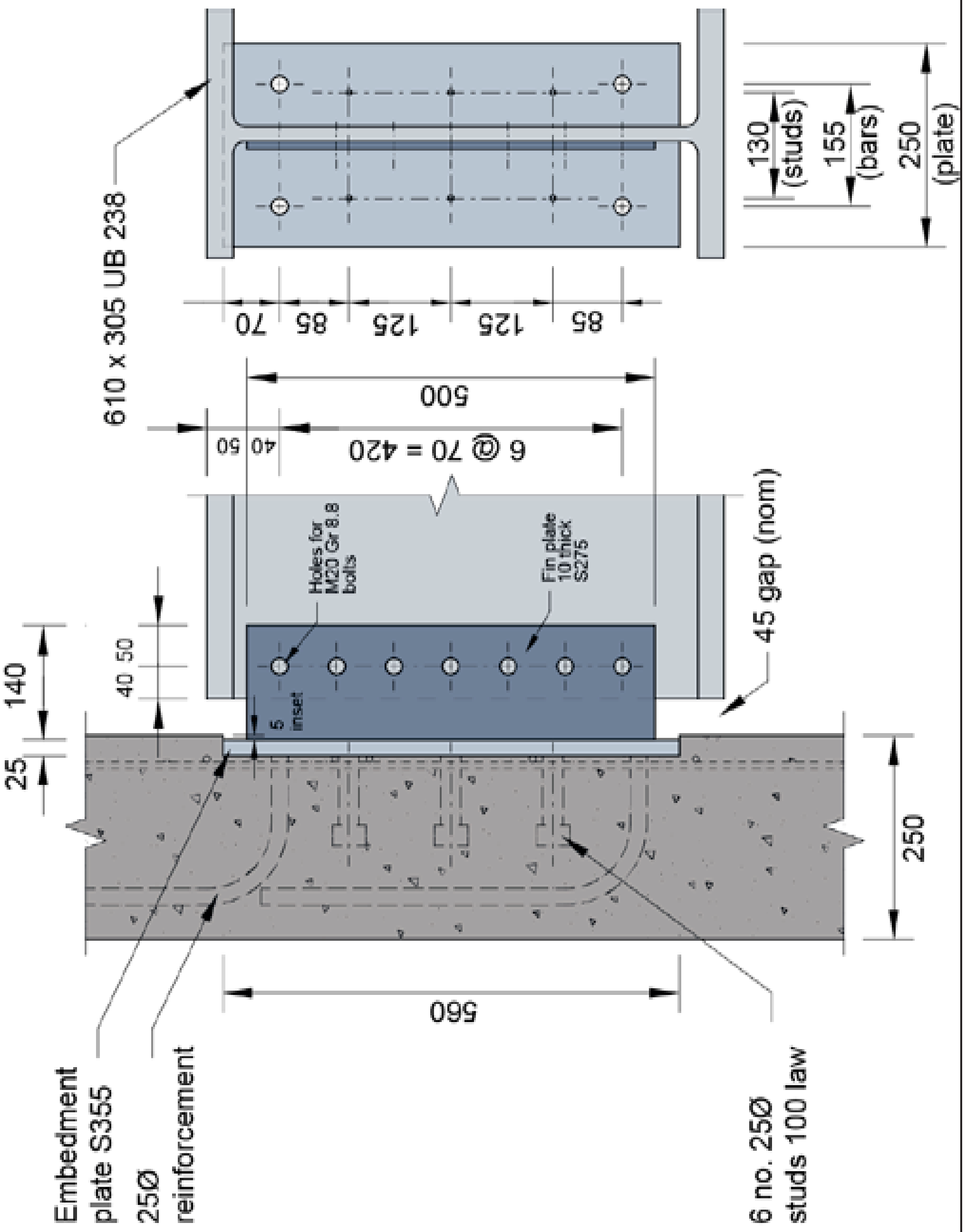
$v_{\text{Rd,c}} = 0.688$ MPa	
Let $(d/s_r) = 1.33$ (radial spacing = 0.75 d)	
$f_{\text{ywd,ef}} = 250 + 0.25 \times 170 = 293$ MPa	6.4.5(1)
$\alpha = 90^\circ \Rightarrow \sin\alpha = 1.0$	
Rearranging and substituting values	
$(0.783 - 0.75 \times 0.688) = 1.5 \times 1.33 \times A_{sw} \times 293 \times [1/(3756 \times 170)] \times 1.0$	
$A_{sw} = \dfrac{0.267 \times 638520}{584}$	
$= 292\ \text{mm}^2$	
No. of legs n for 6ϕ bars:	
$n = \dfrac{292}{28.3} = 10.3$ ie minimum 11 legs	
Use 12 legs for ease of detailing.	
Maximum spacing	Cl 9.2.2(6)
$s_{\text{lmax}} = 0.75d\,(1 + \cot\alpha)$	9.6N
$= 0.75 \times 170 = 127$ mm	
Perimeter u_{out} at which shear reinforcement is no longer required	
The length of the perimeter u_{out} is given by:	Clause 6.4.5
$u_{out} = V_{Ed}/(v_{Rd,c}.d)$	(6.54)
$= 500 \times 10^3/(0.688 \times 170)$	
$= 4275$ mm.	
Detailing of shear reinforcement	
According to the UK National Annex: "$k = 1.5$ Unless the perimeter at which reinforcement is no longer required is less than 3d from the face of the loaded area/column. In this case the reinforcement should be placed in the zone 0.3d to 1.5d from the face of the loaded area/column. The first perimeter of reinforcement should be no further than 0.5d from the face of the loaded area/column"	NA Table NA.1
The distance kd of the outermost perimeter of punching shear reinforcement within the perimeter u_{out} should be not greater than kd	6.4.5(4)
u_{out} = plate perimeter + 2πr	
$r = \dfrac{u_{out} - 2(250 + 560)}{2\pi} = 423$	
Distance from plate edge = 423mm = 2.49 $d < 3d$	
Radial spacing of bars < 0.75 d = 127 mm	9.4.3(1)
Use 12 no. 6 mm ϕ bars in each perimeter.	

Spacing round perimeter ≤1.5d within the first control perimeter (2d from the loaded area)

9.4.3(1)



Connection arrangement



THE DESIGN OF CAST-IN PLATES

Building structures are frequently designed with concrete lift and stair cores surrounded by steel beams and columns. The beams are supported by steel plates cast into the concrete core walls. This guide discusses the technical issues involved in connecting the steel and concrete elements together. A model and a procedure for the design of cast-in plates is proposed including the allocation of design responsibility. The guide includes example calculations.

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SCI
Silwood Park, Ascot, Berkshire. SL5 7QN UK
T: +44 (0)1344 636525
F: +44 (0)1344 636570
E: reception@steel-sci.com
www.steel-sci.com