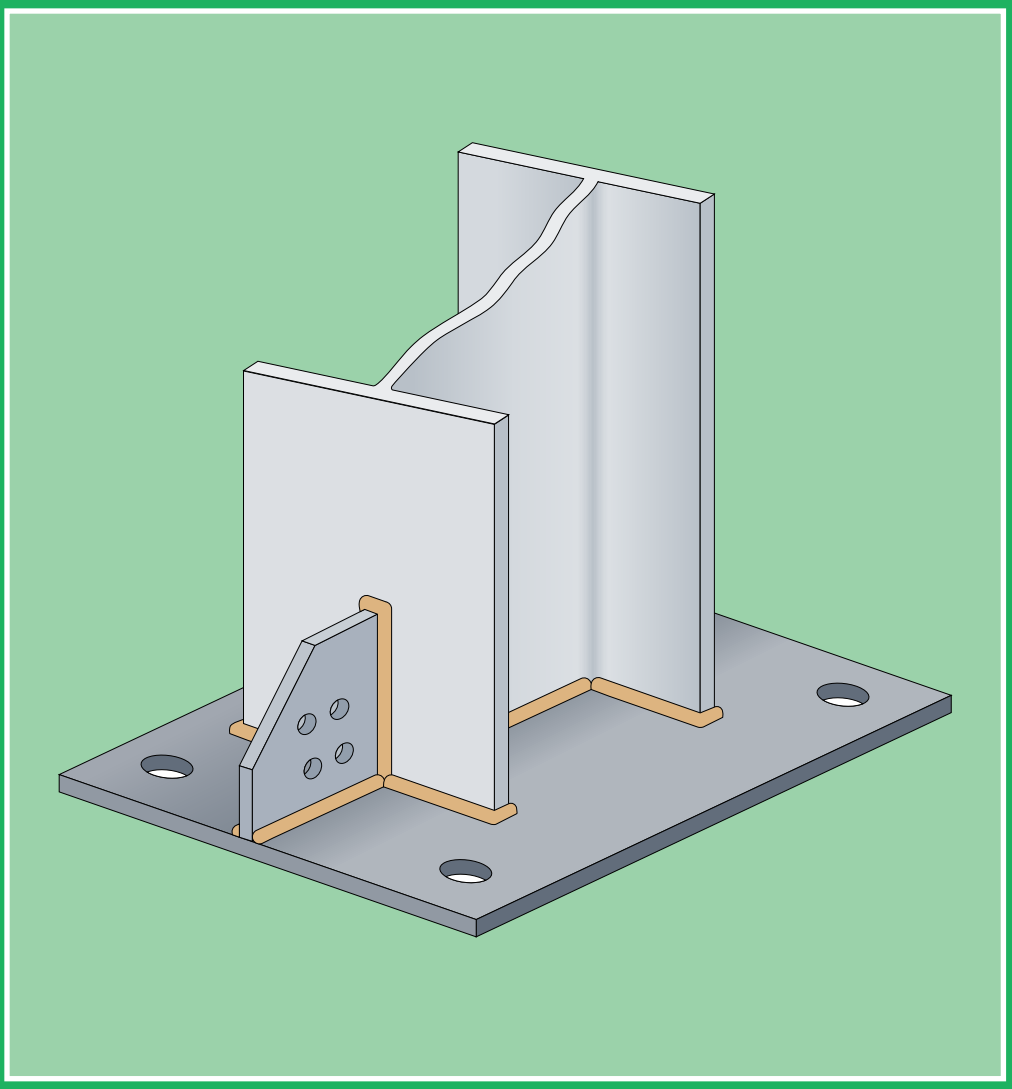
JOINTS IN STEEL CONSTRUCTION:
COLUMN BASES



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COLUMN BASES

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FOREWORD

This publication is one of a series of "Green Books" that cover a range or steelwork joints. This publication provides guidance on steelwork bases, designed in accordance with BS EN 1993-1-8 and the associated UK National Annex. Where appropriate, references are given to both BS EN 1993-1-8:2005 and BS EN 1993-1-8:2024.

The information on this publication supersedes that given in Joints in Steel Construction: Simple joints to Eurocode 3 (P358) and Joints in Steel Construction: Moment-resisting joints to Eurocode 3 (P398).

The significant changes include:

- Design guidance for column bases with shear stubs welded below the base plate.
- Guidance covering the verification of column bases in the temporary condition.
- Adoption of the Eurocode rule for the design shear resistance of anchor bolts (clause A.20 of BS EN 1993-1-8:2024 and clause 6.2.2(7) of BS EN 1993-1-8:2005)

The primary drafters of this guide were David Brown and Bogdan Balan.

This publication was produced with funding from BCSA under the guidance of the Connections Group, which was established in 1987 to bring together academics, consulting engineers and steelwork contractors to work on the development of authoritative design guides for steelwork connections.

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CONTENTS

FOREWORD III

1 INTRODUCTION 1
- 1.1 About this publication 1
- 1.2 Scope 1
- 1.3 Typical details covered by this publication 1
- 1.4 Structural requirements 2
- 1.5 Joint classification 2
- 1.6 Design responsibilities 3
- 1.7 Exchange of information 3
- 1.8 Practical considerations 3
- 1.9 Joint elements 4
- 1.10 Permitted deviations 7
- 1.11 Reinforced base details 8
- 1.12 Temporary conditions 9
- 1.13 Remedial work 10
- 1.14 Major symbols 11

2 BASE COMPRESSION 12
- 2.1 Introduction 12
- 2.2 Design procedures 12

3 BASE TENSION (UPLIFT) 18
- 3.1 Introduction 18
- 3.2 Design assumptions 18
- 3.3 Design procedures 20

4 BASE SHEAR 27
- 4.1 Introduction 27
- 4.2 Friction 27
- 4.3 Holding down bolt design resistance 27
- 4.4 Shear stub 28
- 4.5 Design procedures 30
- 4.6 Bases with bracing on the minor axis 39

5 BASES IN COMBINED AXIAL FORCE AND MOMENT 40
- 5.1 Introduction 40
- 5.2 Design assumptions 40
- 5.3 Minor axis moment 42
- 5.4 Bi-axial moment 42
- 5.5 Design procedures – major axis moment 43

6 REFERENCES 51

WORKED EXAMPLE 1 53
Column subjected to axial compression and shear

WORKED EXAMPLE 2 57
Column subjected to axial compression and shear (with shear stub)

WORKED EXAMPLE 3 65
Column subjected to combined axial compression and bending moment

WORKED EXAMPLE 4 69
Verification of the base detail in the temporary condition

INTRODUCTION

1.1 ABOUT THIS PUBLICATION

This publication provides procedures for designing holding down arrangements for steel columns in accordance with Annex D.3 of BS EN 1993-1-8:2024[1] and its accompanying National Annex[2]. At the time of writing (2026) the National Annex to BS EN 1993-1-8:2024 is still under development. References to the current standard, BS EN 1993-1-8:2005[3] are included where relevant.

The design procedures reflect UK practice, successfully employed over many decades. The guidance has been prepared under the direction of the Connections Group.

1.2 SCOPE

The scope of this publication is limited to the steel components in a base detail, with only passing reference to the foundation and its reinforcement. This publication does not address the resistance of the material under and around the foundation.

Designers should also note the paucity of references to BS EN 1992-4[4], which covers the design of fastenings for use in concrete. Although this standard contains useful information which may be helpful in the design of the base and its reinforcement, those verifications fall outside the scope – and the responsibility – of the steelwork designer. BS EN 1993-1-8:2024 discriminates between "anchor bolts" and "fasteners" in Annex A. The resistance of anchor bolts is addressed in clauses A.18 and A.20 whilst fasteners are addressed in clauses A.19 and A.21. The requirements relating to "fasteners" make a forward reference to BS EN 1992-4. This design guide provides guidance for "anchor bolts" rather than "fasteners". In keeping with usual UK practice, this guide refers to "anchor bolts" as "holding down bolts".

1.3 TYPICAL DETAILS COVERED BY THIS PUBLICATION

In UK practice, column bases generally consist of a single plate fillet welded to the end of the column and attached to the foundation with holding down bolts. Although two holding down bolts may be used for secondary steelwork or small columns, usually at least four holding down bolts are used, which are cast in the foundation within cones or sleeves to facilitate some lateral adjustment of the base during erection. Casting holding down bolts solidly with no opportunity for movement during erection is more common in other parts of Europe. If holding down bolts are cast without adjustment, a rigid template is essential, accurately positioned and restrained during the concreting operation.

Holding down bolts may be located outside the section profile, or, for larger columns, within the depth of the profile. After erection, the base is grouted, filling all voids and the gap under the base plate.

Unless the column is particularly large, or perhaps a fabricated column composed of welded elements, it is not necessary to machine the end of the column – a saw cut end is sufficient. Similarly, it is not generally necessary to machine or flatten the base plate.

Each holding down bolt is usually anchored at the bolt head with a plate cast into the concrete. For larger bases and forces, the holding down bolts may be connected to a single large anchor plate or to a fabricated grillage. Holding down bolts are generally only threaded for a portion of their length.

Holes in the base plate are usually oversize, facilitating some inclination or misalignment of the holding down bolts. Washer plates with normal holes may be placed over the holding down bolts and welded to the top of the base plate so that horizontal shear may be assumed to be equally distributed to all holding down bolts.

If normal clearance holes are used in a base plate (perhaps to ensure that all the holding down bolts can be assumed to transmit shear – see Section 4.3), the use of substantial templates to position the holding down bolts accurately is strongly recommended. Concreting operations may move holding down bolts which are not securely located.

Height adjustment is achieved by placing packs of appropriate thickness under the base plate.

When a base is subject to a significant shear, a base detail with a shear stub welded underneath the base plate is a

common solution. Guidance on the design of this type of detail is provided in this publication.

Typical base details are illustrated in Figure 1.1.

Particular care will be needed if steel bases are located close to the edge of a concrete foundation, or located on an a narrow substructure such as a plinth. The assumptions about the coefficient, α (see Section 2) must be reviewed, as the usual value may not be appropriate if the steel base is close to an edge or the foundation concrete is shallow. When the steel base is close to an edge, the design of the steel elements and holding down bolts should be coordinated with the design of the supporting concrete element, which will require careful attention to the reinforcement details, verified in accordance with BS EN 1992-4.

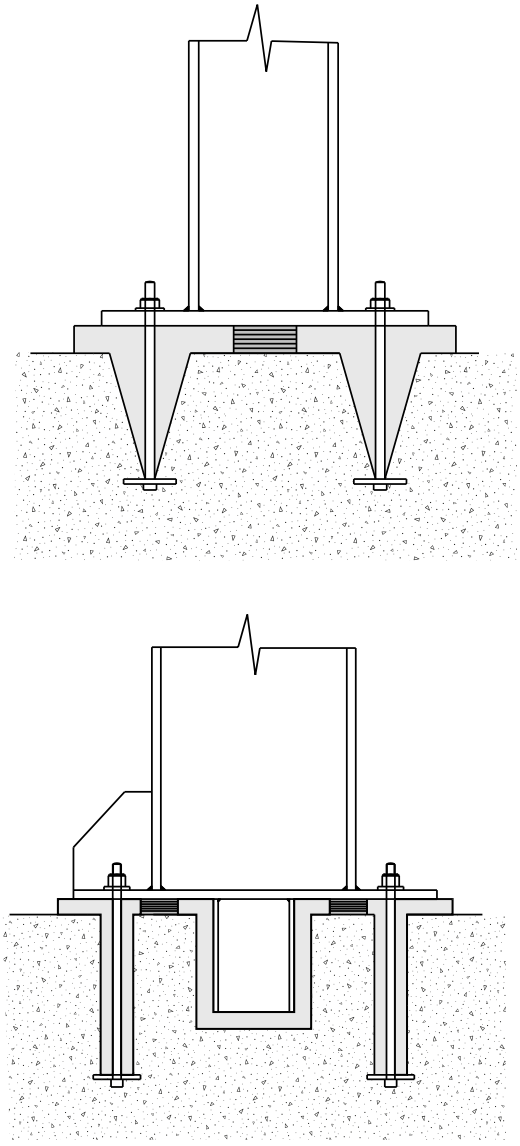


Figure 1.1 Typical base details

1.4 STRUCTURAL REQUIREMENTS

In the permanent condition, bases are primarily subject to axial compression. Bases may also be subject to bending moments and shear, in combination. Bases may be subject to tension when resisting overturning, or at the foot of a braced bay.

A base also has a critical function in the temporary condition, which should be considered at the design stage. During erection, a column may have to be self-supporting for a brief period, so the base detail and foundation must resist a bending moment. Temporary bracing is often required during construction, introducing shear, tension and compression to bases forming part of the temporary bracing system. Best practice is to consider the erection method during design since bases designed only for the permanent condition may not be adequate.

1.5 JOINT CLASSIFICATION

BS EN 1993-1-8 requires all joints to be classified by rotational stiffness and moment resistance. In the UK, practice in analysis is to assume the base to be either nominally pinned or rigid (*fixed* and *moment-resisting* are synonymous terms).

Rigid bases are often used to reduce deflections within a frame, such as when a portal frame supports an overhead crane. Steel details for rigid bases are not particularly difficult to design or fabricate – the more significant factor is the cost of the fixed foundation. For this reason, nominally pinned bases are generally preferred in the UK.

As specified in clause 7.3.2.1(2) of BS EN 1993-1-8:2024 (clause 5.2.2.1(2) of BS EN 1993-1-8:2005), joints may be classified in terms of rotational stiffness by calculations or on the basis of previous satisfactory performance in similar cases. UK practice is to recognise many decades of previous satisfactory performance in the classification of base details and generally not to classify by calculation of stiffness.

In the UK, bases which are not designed for a bending moment are typically classified as nominally pinned. The nominally pinned classification includes bases with holding down bolts located outside the profile and bases with a shear stub. The required rotation of nominally pinned bases is generally assumed to be accommodated within the foundation or supporting structure, rather than within the steel base detail. The nominally pinned classification also

covers bases which have been designed to carry moment in the temporary condition, and (for portal frames), bases designed to carry a bending moment in the fire limit state.

The UK's pragmatic approach to the classification of base details is endorsed by the Connections Group, was endorsed by the UK National Annex to BS EN 1993-1-8:2005[5] and is expected to be endorsed by the UK National Annex to BS EN 1993-1-8:2024.

In many cases, a fixed base, classified in UK practice as rigid, may appear hardly any different to a nominally pinned detail. The reality is that so-called nominally pinned bases may have considerable stiffness and bending resistance. The designer of the steel frame should consider the impact of base stiffness on the design of the frame. Generally, the impact of base stiffness on the frame is beneficial, compared to an assumption of a perfectly pinned base. There may be situations when the structure supporting the steelwork must be verified for the moment transferred by the nominally pinned base, or when special details must be adopted to minimise the stiffness and resistance of the steel base detail.

1.6 DESIGN RESPONSIBILITIES

This publication assumes usual UK practice to be:

- When the structural design of the frame is completed by a consulting engineer, the consulting engineer designs and details the bases. The steelwork contractor is responsible for the joint design excluding the bases. This practice has developed from the necessity to coordinate the locations of the holding down bolts with the reinforcement. In this situation, the consulting engineer should consider the base design for the temporary condition as discussed in Section 1.12. The steelwork contractor may review the details and propose alternatives.
- When the structural design and detailing is entirely the responsibility of the steelwork contractor (a "design and build" contract) the steelwork contractor will design and detail the steel bases including the size and position of the holding down bolts and – if required – a shear key. Those steel base details will be passed to the party responsible for the design of the foundations to design the reinforcement. There may be some co-ordination work between the parties as the design of the base and foundation develop.

1.7 EXCHANGE OF INFORMATION

The complete design of a base and foundation generally requires information supplied by different parties and is likely to involve some modification as the design is developed, alternative details are proposed and considered. Timely exchange of information is therefore important, particularly as the casting of the bases with the holding down bolts is early in the construction programme and once cast, the details cannot readily be modified.

The National Structural Steelwork Specification[6] (NSSS) requires the steelwork contractor to prepare a foundation plan, which includes the base locations and holding down bolt positions. This plan will be used by the main contractor to cast the holding down bolts in the correct locations.

Some coordination and development of the base and foundation design is likely in more heavily loaded bases, bases resisting moments, bases resisting high shear and steel bases on unorthodox foundations (e.g steel bases located close to a foundation edge). Early development of proposed details and exchange between the parties is strongly recommended.

1.8 PRACTICAL CONSIDERATIONS

The base plate, holding down bolts and welds must be designed to carry the permanent design loads to the foundations. In addition, the design should consider any temporary loading, including wind loads, lack of verticality and any moment due to eccentricity of connected components.

The following practical guidance should be considered.

1.8.1 Holding down bolts

- Holding down bolts are generally property class 8.8. Different property class holding down bolts of the same diameter should not be used on the same site, to reduce the risk of casting the incorrect holding down bolts.
- Holding down bolts should be a minimum of 300 mm long. Although a shorter bolt may be adequate in the final state (perhaps when the design load is only compression in combination with modest shear) the holding down bolts must be detailed to provide fixity in the temporary condition.
- Unless the column is secondary steelwork, where the holding down arrangements could be

post-fixed anchors, holding down bolts should be M20 minimum. Larger holding down bolts will often be required for moment-resisting bases.
- Anchor plates should be detailed as shown in Table 3.2.
- Appropriate specifications for holding down assemblies are given in Table 2.7 of the NSSS.

1.8.2 Setting out

- All setting out of holding down bolts should be on regular, simple geometry. It is recommended that the setting out of four holding down bolts is either identical in the two orthogonal directions (as might be the case for a square hollow section column) or distinctly different in the two directions. This is to reduce the risk of casting the holding down bolts out of position, or casting the entire bolt group in the wrong orientation.
- Unless the column section is very small, such as a channel or UB used as secondary steelwork, a minimum of four holding down bolts is strongly recommended. The recommendation means that the base will have some resistance to overturning in the temporary case. If four holding down bolts are provided, they can be loosened and tightened as appropriate to bring the column into the vertical position.
- For larger columns (typically 406 UB and larger) it is possible to detail a group of four holding down bolts within the depth of the section.
- For smaller columns and UC sections, four holding down bolts would normally be detailed outside the profile.
- Assuming the shims under a base are typically 100 mm square and the tops of the bolt cones are 100 mm diameter, the diagonal dimension between holding down bolts should be a minimum of 250 mm (orthogonal grid dimensions of at least 175 mm). These minimum dimensions leave a small platform of concrete between the holding down bolts on which to position the shims. Larger spacing is recommended if possible, to avoid the possibility of concentrated loads overstressing a relatively unconstrained block of concrete. Particular care is needed if the concrete is not fully cured when the steelwork is erected.

1.8.3 Base plates

- Base plates should be a minimum thickness of 15 mm for primary steelwork and 10 mm for secondary steelwork.
- If the base exceeds 700 mm × 700 mm, a 50 mm diameter grout hole to let air out from below the base plate should be provided for every 0.5 m^2 of base area. If grout is to be placed through the holes, the hole diameter should be increased to 100 mm.
- In compression, load will be transferred between the column and the base plate in direct bearing, so the weld need not be designed for that case. The weld will have to be verified for any shear, and for the overturning moment in the temporary case.
- A continuous weld may not be required for structural reasons, but is often provided to prevent corrosion if the detail is exposed.

1.9 JOINT ELEMENTS

1.9.1 Holding down bolts

Holding down bolts are normally manufactured in accordance with BS 7419[7] which covers:

- Holding down bolts with a square head and neck, and
- Holding down bolts with a hexagon head and round neck

For the square head type, bolt rotation during tightening is prevented by placing the square neck in a square hole in the anchor plate. For the hexagon head type, a small ‘keep’ flat is usually welded to the underside of the plate to bear against one of the hexagonal head flats. The holding down bolts should not be welded.

The embedded length of the bolt in the concrete will usually be in the region of 16 to 18 bolt diameters. The thread length must allow for tolerances and should be 100 mm plus the bolt diameter.

Holding down bolts are usually property class 8.8 (property class 4.6 is not commonly used). It is bad practice to specify different bolt strengths on one site, unless the holding down bolts are physically different in length or diameter, as confusion can easily lead to the incorrect bolt being cast into the foundation. M20 holding down bolts are generally the minimum diameter adopted, with larger diameters specified for bases subject to larger design forces and moments.

Although holding down bolts are normally used in the non-coated condition, they can be supplied:

- Electroplated to BS EN ISO 4042[8]
- Galvanised to BS EN ISO 10684[9]
- Sherardized to BS 7371-8[10]

1.9.2 Base plates

Base plates are fabricated from steel plate, in S275 or S355, though higher strength plate may be used. Base plates generally do not need machining or flattening.

1.9.3 Base plate toughness

In most cases, a steel base plate within a building will not be exposed to extremes of temperature, or to fatigue. For plain plate bases, it is recommended that the selection of a steel sub-grade is completed using P419[11].

Bases subject only to compression are designed assuming that the base plate is in bending, with compression on the upper surface and tension on the lower surface of the plate. The welds between the column and the base plate, which are important when assessing the risk of brittle fracture, are completed on the upper surface which is assumed to be in compression and therefore relatively low risk. If a shear stub is welded to the underside of the base plate, the welds are to a face subject to tension, increasing the risk of brittle fracture.

When specifying a steel sub-grade for a plain plate base without uplift, a "moderate" detail type and a compressive stress is an appropriate selection.

For plain plate bases subject to uplift in the permanent condition, or subject to bending moment, the design model assumes a tensile bending stress in the plate where the welds are located, so a more refined approach may be required.

When assessing the risk of brittle fracture the design value of the actions are much reduced compared to the usual ultimate limit state (ULS) values (see BS EN 1993-1 10[12]). In the absence of a more precise calculation of the bending stress in the plate, a "moderate" detail type and a tensile stress ratio of 0.3 is an appropriate selection for a base plate without stiffeners.

With very large welds – where the throat thickness of a fillet weld exceeds 25 mm – the possibility of lamellar tearing should be considered. With such large welds, the detail may become "high risk", as indicated in PD 6695-1-10[13]. In "high risk" situations, a steel with specified through thickness properties ("Z quality") may be required.

If a nominally pinned column is self-supporting in the temporary condition (see Section 1.12) the overturning moment will produce tension at the base of the column. This temporary situation need not be considered when assessing the risk of brittle fracture.

1.9.4 Hole sizes and washers

Clearance holes in the base plate should be 6 mm larger than the bolt diameter, to allow for adjustment. For base plates thicker than 60 mm, this clearance may need to be increased.

Table 2.7 of the NSSS specifies washers to BS EN ISO 7091[14], which are the same size and thickness as washers for ordinary bolted joints between members. Traditionally, Form G washers to BS 4320[15] have been used with holding down bolts in the UK. Form G washers have a larger outside diameter and are thicker than those to BS EN ISO 7091. A comparison is shown in Table 1.1. Form G washers to BS 4320 are recommended for use with the oversize holes generally used in the UK.

Washer plates are occasionally made from plate, especially if used with slotted or modified holes needed to overcome some bolt misalignment (see Section 1.13). Plate washers should have a clearance of 2 mm around the bolt and provide as much overlap of the base plate as a standard washer. Plate washers should be at least 4 mm thick according to 8.2.4 of BS EN 1090-2[16] (and 6.1.5 of the NSSS). If a hole has been enlarged to accommodate out of position holding down bolts, a thicker washer plate may be necessary.

1.9.5 Location tubes and cones

In UK practice, holding down bolts are commonly set into ready-made location tubes (usually wire mesh) or cones (usually polystyrene formers or cardboard). The diameter of the tube or the tops of the cones should be at least 100 mm or 3 times the diameter of the bolt, whichever is greater. The threaded portion of the holding down bolts should be protected during the concreting operation. As the concrete cures, the holding down bolts should be moved so that they can be freely adjusted as the steel is erected.

	Outside diameter (mm)		Thickness (mm)	
Bolt diameter	**Form G to BS 4320**	**BS EN ISO 7091**	**Form G to BS 4320**	**BS EN ISO 7091**
M20	60	37	5	3
M24	72	44	6	4

Table 1.1 Washer dimensions

Holding down bolts are occasionally cast solidly into the concrete foundation which leaves no opportunity for adjustment. If holding down bolts are to be solidly cast in, it is recommended that:

- Individual anchor plates are replaced with one single fabricated component.
- A steel template is used to keep the tops of the holding down bolts in position during casting.
- Time is allowed in the construction programme to survey the precise location of the holding down bolts before fabrication.

1.9.6 Base packs

Columns are normally erected on a central pile of steel levelling packs (typically 100 mm square) that are left permanently in position. Steel wedges may be temporarily placed around the edges of the base plate to assist with stability during erection. If a shear stub is used packs are generally placed around the edge of the base, not under the stub.

1.9.7 Grout

Filling the voids and space under the steel base plate is usually the responsibility of the main contractor and is undertaken after the steelwork contractor has confirmed that the steelwork has been erected, plumbed and aligned into its final position.

The grouting operation is an important part of the process to complete the base detail. The grout distributes the load from the base plate to the concrete, fills the void around the holding down bolts, provides the interface for the shear resistance of the base detail and – in the case of a shear stub – transfers that shear force from the steelwork to the surrounding foundation.

The grout should be mixed in accordance with the manufacturer's instructions and should be an appropriate product for the size of the void to be filled. The foundation under the base plate, including the voids around the holding down bolts, should be clean, dry and free from detritus. It is essential that the grout completely fills the space under the base plate, so air holes are recommended in any base plates larger than 700 mm × 700 mm. Holes in the base plate are also recommended if the base has a shear stub, so that the grout can be properly placed around the stub.

If a shear key is used, a 50 mm hole through the base plate on each side of the web of the shear stub should be provided so that grout can completely fill the void around the shear stub. The pocket around the shear stub should be detailed to be at least 50 mm larger than the stub to allow for the foundation being cast out of position and to ensure adequate grout filling all around the shear stub.

Depending on the nominal thickness of the grout layer, BS EN 1090-2 recommends using different types of cement-based grouts. UK practice is generally to use proprietary non-shrink cementitious grout.

1.9.8 Clearance under the base plate

A space between the top of the foundation and the base plate of between 25 mm and 50 mm is the normal clearance when using grout. This gap gives reasonable access for grouting the bolt pockets, which is necessary to prevent corrosion, and for thoroughly filling the space under the base plate.

1.9.9 Concrete strength

Typical concrete strengths are shown in Table 1.2, taken from BS EN 1992-1-1[17].

Concrete class	Cylinder strength (N/mm^2)	Cube strength (N/mm^2)	Design compressive strength (N/mm^2)
	f_{ck}	$f_{ck,cube}$	f_{cd}
C25/30	25	30	14.2
C30/37	30	37	16.8
C35/45	35	45	17.0
C40/50	40	50	22.7

Table 1.2 Concrete strength

Although not included in BS EN 1992-1-1, concrete class C32/40 may also be specified for foundations.

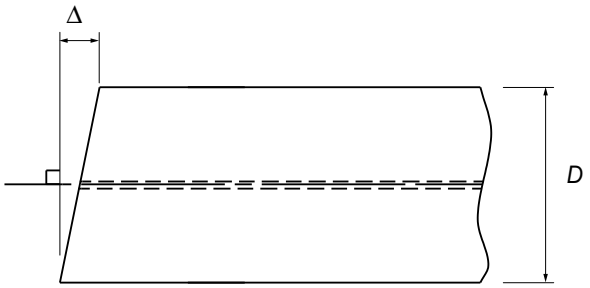
For design purposes, the lowest strength of either the grout or the concrete in the foundation should be used. In most situations, the design will be based on the strength of the concrete in the foundation, and non-shrink cementitious grout will be specified to be at least as strong. Other bedding materials such as epoxy grout may be specified.

1.10 PERMITTED DEVIATIONS

Permitted deviations (often referred to as tolerances) are given in BS EN 1090-2[16] and the NSSS. The following clause references are all from the NSSS.

1.10.1 Permitted deviations – steelwork

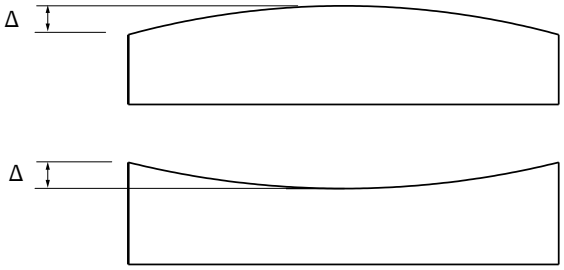
The end of a column is normally simply cut by a saw, which will provide a satisfactorily flat and accurate bearing surface. The squareness of the end with respect to the axis of the member is limited to $D/1000$ in clause 7.2.3 (Figure 1.2).



$\Delta = \pm D/1000$

Figure 1.2 Squareness of ends prepared for bearing

The flatness of a surface prepared for bearing is given in 7.3.7 and is shown in Figure 1.3.

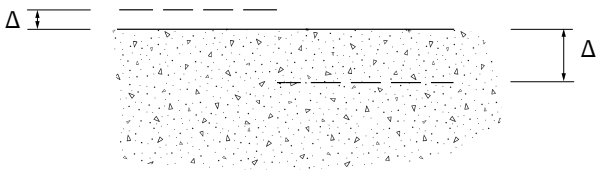


Maximum gap Δ between the surface and a straight edge laid in any direction is 0.5 mm

Figure 1.3 Flatness of surfaces prepared for bearing

1.10.2 Permitted deviations – concrete

The top of the concrete should be within the deviation given in 9.1.1, as shown in Figure 1.4.

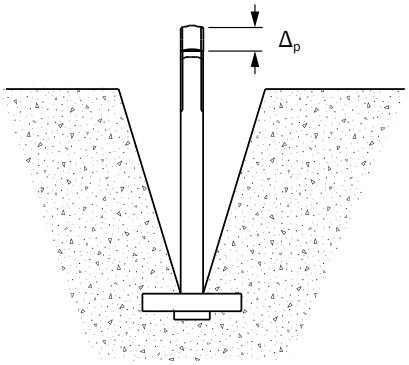


Δ = – 15 mm low, + 5 mm high

Figure 1.4 Foundation level

1.10.3 Permitted deviations – holding down bolts

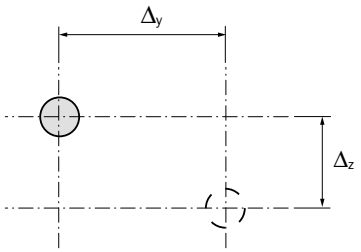
The protrusion of the holding down bolt with respect to its intended level is given in 9.1.3 and shown in Figure 1.5.



Δ_p = – 5 mm low, + 25 mm high

Figure 1.5 Holding down bolt protrusion

The position of individual holding down bolts on plan, with respect to the centre of the bolts group, is given in 9.1.3 and as shown in Figure 1.6.



Δ_y and $\Delta_z = \pm$ 10 mm

Figure 1.6 Plan position of holding down bolts

In addition to the deviations indicated in Figure 1.6, according to clause 9.1.3 of the NSSS the maximum horizontal deviation of the centre of a bolt group is 6 mm.

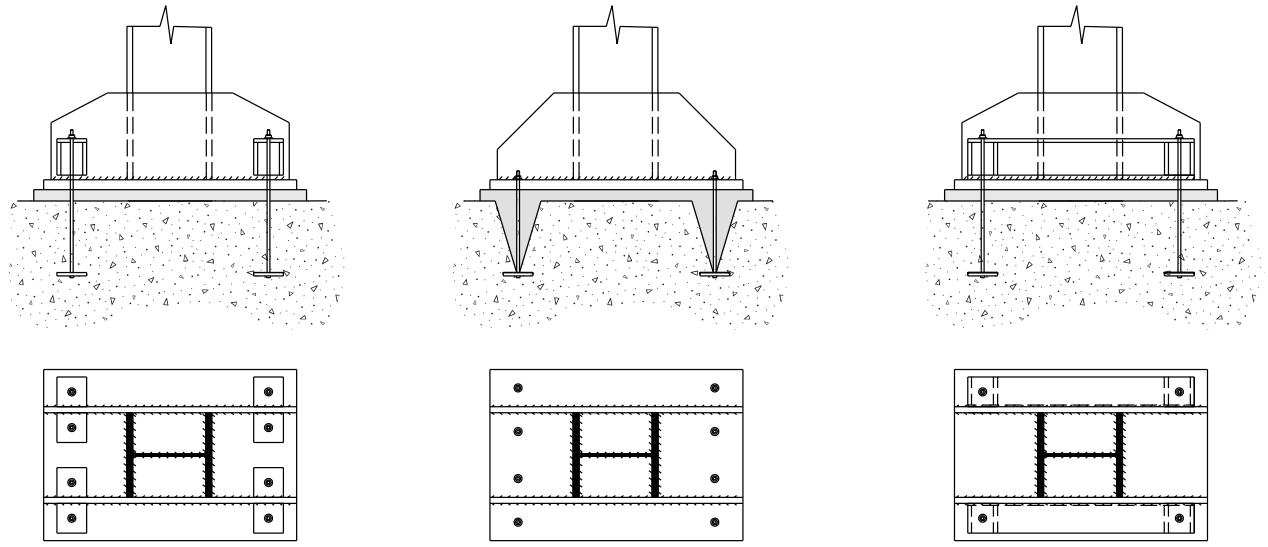


Figure 1.7 Reinforced base details

1.11 REINFORCED BASE DETAILS

Reinforced bases may be used to increase the bending moment resistance of the base or increase the effective area in compression. Typical details with bolt boxes, plate stiffeners or parallel flange channels are illustrated in Figure 1.7.

Unless a more refined analysis is undertaken, in addition to verifying the resistance of the stiffeners, good practice is to observe the Class 3 limitations presented in BS EN 1993-1-1[18]. The minimum gusset plate thickness t should be the larger of:

- $c/38\varepsilon$ where c is the distance between two flanges of the column
- $c/14\varepsilon$ where c is the dimension of an outstand, measured from either the horizontal or vertical support

$$\varepsilon = \sqrt{235/f_y}$$

The above recommendations are illustrated in Figure 1.8. The gusset plate has been cut so that at any point the maximum outstand from a support is $14t\varepsilon$.

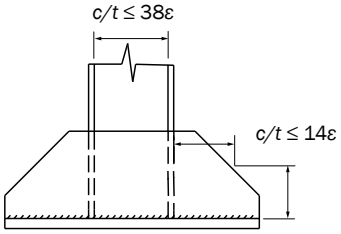


Figure 1.8 Limitations on gusset plate geometry

The general approach is to assume an area of uniform compression which loads each gusset outstand as shown in Figure 1.9.

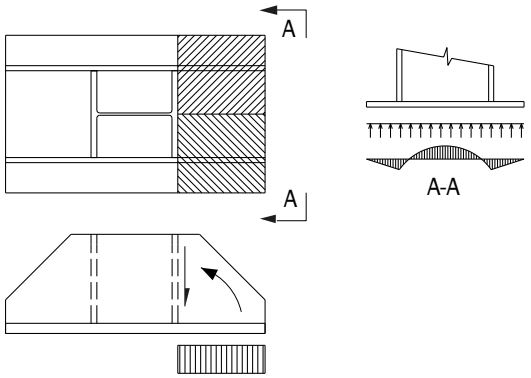


Figure 1.9 Design of bases with gusset plates

The load indicated in Figure 1.9 leads to combined shear and bending in the gusset. The moment resistance of the gusset may be reduced if the shear is “high”.

The base plate is verified by considering it as a continuous plate supported by the gusset plates, as indicated in Figure 1.9, section A-A.

The axial force and any bending moment in the column is decomposed into longitudinal forces in the vertical welds between the toes of the column flanges and gusset plates.

In the tension zone, the base plate and holding down bolts may be considered as a T-stub, following the principles presented in Section 3. The gusset plates must be verified for the bending moment produced by the tension in the holding down bolts.

1.12 TEMPORARY CONDITIONS

By preference, erection of steelwork generally commences with a “stiff box”, being a fully erected core of columns, beams and vertical bracing. Steelwork erection then continues, working away from the “stiff box”, thus ensuring that the erected steelwork is stable.

The first columns to be erected, together with any others which cannot be immediately stabilised, must rely on the base detail and foundation to prevent overturning. The overturning moment arises from wind loading, eccentric loads due to attachments to the column and the column being out of plumb. The overturning moment is resisted by tension in the holding down bolts, by compression on one side of the base and the foundation itself. The base is normally erected on base packs which act as a pivot point, with compression supported by wedges under the edge of the base plate and tension resisted by the holding down bolts. At least four widely spaced holding down bolts and a relatively thick base plate will increase the resistance of the base to overturning in the temporary condition. The wedges are removed before grouting, once the structure is stable.

Although temporary guys or struts can be used to secure single columns, the temporary supports can impede movement of machinery around the site and will require their own capable anchoring point at ground level. For orthodox steelwork structures, it is rarely convenient to use temporary supports. For less ordinary structures, a properly designed temporary support system with its own foundation arrangements and predetermined anchorage points on the erected steelwork will be required.

1.12.1 Design loading in the temporary condition

The *BCSA Guide to steel erection in windy conditions*[19] recognises two design situations:

1. The “working” condition, during active erection of steelwork in working hours, when high winds would lead to a cessation of activities in a safe manner, and
2. The “transient” situation, when erected steelwork may be left (for example over a weekend) and may experience higher wind forces.

The design loading when considering the temporary loads on a base are:

- The wind loading, considering the two conditions above (variable action).
- An amplification of the wind actions to allow for dynamic effects (an amplifier of 1.2 is recommended).
- The eccentricity due to the column being out of plumb (considered to be a permanent action).
- Any eccentricity due to attachments to the column (considered to be a permanent action).

Recommended practice is to assume the column is out of plumb at 1 in 100, with a minimum displacement of 100 mm at the top of the column.

Wind loading is site-specific so should be considered for the construction site. Alternatively, assumptions can be made to justify standardised base details within a specified scope (geographic location, column height etc).

Reference 19 recommends a minimum of 200 Pa in the “working” condition and a minimum of 550 Pa in the “transient” condition. The peak velocity pressure can be much higher than 550 Pa, depending on the location.

1.12.2 Verification of the temporary condition

The calculated overturning moment is resisted by the couple between tension in the holding down bolts and compression assumed at the wedges – the arrangement of wedges affects the lever arms assumed, so the design assumptions and implementation on site must be consistent.

For small bases, one pack may be placed centrally on each edge. For larger bases two packs may be placed on each edge, or pairs of packs placed to resist overturning in one axis only, when the column is stable in the other axis. Examples are shown in Figure 1.10.

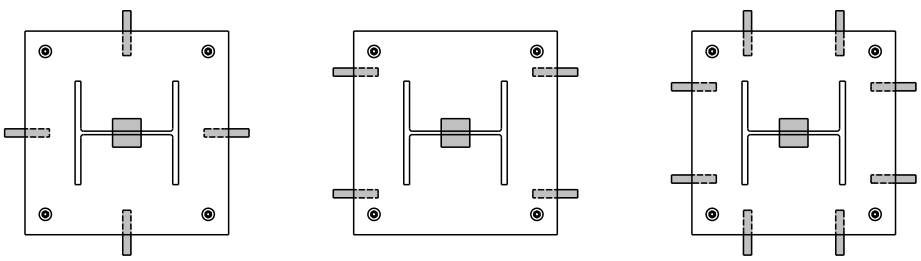


Figure 1.10 Alternative arrangement of packs

The contact between the edge of the base plate is a knife edge contact – it is generally assumed that the steel-on-steel contact can transfer the load, based on extensive previous satisfactory experience. The load from the contact point is assumed to be dispersed through the wedge to the concrete, assuming a maximum spread at 1:3.5 as shown in Figure 1.11, but limited by the physical dimensions of the wedge and the grout gap. If the grout gap is deep (possibly because the concrete is low), the dimensions of the wedge may be the limiting constraint.

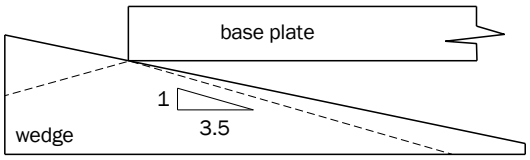


Figure 1.11 Distribution of load through wedge

The pull-out resistance of the holding down bolts and the resistance of the concrete under the wedges depend on the grade of concrete and the curing time since the concrete was poured. The concrete strength may be critical, so any assumptions about the concrete strength must be clearly specified and communicated so that erection only commences when the assumed strength has been attained. Particular care is needed if the concrete has cured for less than seven days.

If the bases are subject to significant shear in the temporary case before grouting is completed, measures include casting an upstand in the foundation, or fixing a temporary steel bracket to the foundation that the base plate can be packed against. Chemical anchors or expansion anchors may also be used to transfer shear.

A numerical design example covering stability in the temporary case is presented in Example 4.

1.13 REMEDIAL WORK

Holding down bolts are occasionally cast out of position laterally, vertically or in both directions. Breaking out the entire foundation and re-casting the concrete would be very unusual for financial and construction programme reasons, so other solutions are usually preferred. Possible solutions to the most common problems are discussed in the following sections.

1.13.1 Bolts out of position

Potential solutions include:

- Removing the base plate and rewelding in an offset position (or welding a new base plate with holes drilled to suit the bolt positions). This solution may be structurally satisfactory for nominally pinned bases after verifying that the base plate is still adequate after the offset. Verification of moment-resisting bases will be necessary if the base plate is offset. Shear resistance is unlikely to be affected. If the column and base plate are galvanised, this solution may not be preferred. Fabricating a new base plate is likely to have a detrimental impact on the progress of the steel erection.
- Drilling new holes, or slots in the base plate. This solution may be satisfactory for holding down bolts laterally out of position by a modest amount. Oversize holes or slots should be covered with a plate washer, minimum 4 mm thick, of the same grade as the base plate. Any horizontal shear is transferred via the plate washers, so these should be welded to the top of the base plate.
- Cutting the existing holding down bolts and installing expanding anchors or chemical anchors in new holes in the base plate. The shear resistance and moment resistance of the base (if applicable) must be verified as post-installed anchors are unlikely to match the resistance of the original holding down bolts.

If the holding down bolts are so far out of position that they clash with the column cross section, the possible solutions are breaking out and re-casting the foundation, use of post-installed anchors or a completely reconfigured detail at the bottom of the column.

1.13.2 Insufficient bolt projection

Potential solutions include:

- Using couplers to add a length of threaded bar of the same property class as the bolt. Depending on the precise arrangement, it may be necessary to drill an oversize hole in the base plate to allow the coupler to pass through. If the coupler protrudes, an assembly of washer plates will be required so that the nut can be tightened.
- Bolt extenders, which are large diameter bolts with a full length hole drilled and tapped to suit the thread of the holding down bolt. The bolt extender can be simply threaded over the holding down bolt and tightened as shown in Figure 1.12.

Larger holes are required in the base plate to suit the diameter of the extender, typically M30 extender for M20 holding down bolts and typically an M36 extender for M24 holding down bolts. Other sizes are available.

If the nut is partially engaged, reference [20] recommends that as long as at least half the threads in the nut are engaged, it can be assumed that the shear resistance of the holding down bolt is unaffected. Nuts should not be welded to holding down bolts.

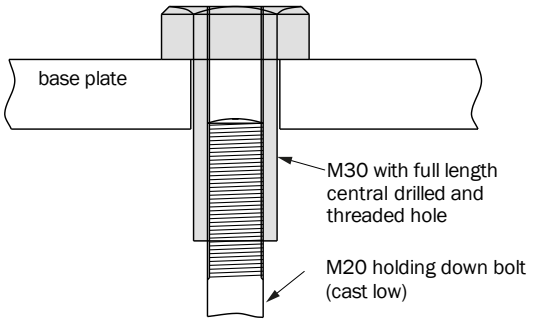


Figure 1.12 Use of bolt extenders

1.13.3 Holding down bolt protrusion too long

If the holding down bolt protrusion is so long that the nut cannot be tightened (since there is insufficient threaded length) the usual solution is to provide plate washers on top of the base plate.

1.13.4 Rotated bolt patterns

If the bolt group is rotated, the potential solutions are to remove, rotate and re-weld the base plate, to cut the holding down bolts and install expanding or chemical anchors, or to break out and re-cast the foundation.

1.14 MAJOR SYMBOLS

The major symbols used in this publication are listed below for reference purposes. Others are described where used.

γ_{M0} is the partial factor for the resistance of cross section ($\gamma_{M0} = 1.0$ as given in the UK NA to BS EN 1993-1-1[21])

γ_{M1} is the partial factor for the resistance of members to instability assessed by member checks ($\gamma_{M1} = 1.0$ as given in the UK NA to BS EN 1993-1-1)

γ_{M2} the partial factor for resistance of bolts and welds, ($\gamma_{M2} = 1.25$ as given in the UK NA to BS EN 1993-1-8)

A Area of element

a Design throat thickness

b Width of section (subscript c, p or n refers to column, plate, or shear stub)

b_{eff} Effective width of a base plate T-stub in compression.

c Additional bearing width

d Diameter of a bolt

d_0 Hole diameter

$d_{eff,n}$ Effective depth of a shear stub

d_n Total depth of a shear stub

f_y Yield strength of steel (subscript c, p or n refers to column, plate, or shear stub)

f_{jd} Design bearing strength of foundation joint

f_u Ultimate tensile strength (subscript c, p or n refers to column, plate, or shear stub)

f_{ub} Ultimate tensile strength of a bolt

h Depth of section (subscript c, p or n refers to column, plate, or shear stub)

n Number of holding down bolts

s Leg length of fillet weld

t Thickness (subscript f, w, p or g refers to flange, web, plate or grout)

BASE COMPRESSION

2.1 INTRODUCTION

In compression, the design force is assumed to be distributed over an effective area of steel-to-concrete contact that is determined by the strength of supporting material (the weaker of the concrete or grout) and the resistance of the base plate to bending.

The design model for the axial compression force is given in BS EN 1993-1-8. The design model assumes a uniform bearing strength on the supporting material over the entirety of an effective area. The effective area comprises the cross section of the column member, extended by an additional bearing width all around the column profile – including inside the profile of a hollow section.

The additional bearing width, *c*, is established by setting the design compression equal to the resistance of the bearing area. The required thickness of the plate can then be established assuming that the bearing width acts as a cantilever subject to a uniform load.

A reduction is effective area is not usually necessary for bolt holes up to 50 mm diameter.

Typical effective areas are shown in Figure 2.1.

The individual areas around the flanges and web cannot overlap, which may happen with UC sections (Figure 2.1 (b)). Similarly, if the effective area is larger than the physical plate dimension in one or more locations, the inadmissible area is discounted and the additional bearing width, *c*, uniformly increased in other areas to compensate.

If the base is resisting a bending moment, it may be assumed that the effective area in compression is associated with one flange only, as shown in Figure 2.2.

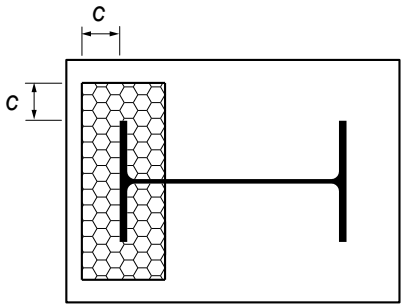


Figure 2.2 Compression in one flange only

2.2 DESIGN PROCEDURES

The design checks for a column base subjected to compression are:

- Check 1 – Required area
- Check 2 – Effective area
- Check 3 – Plate thickness
- Check 4 – Welds

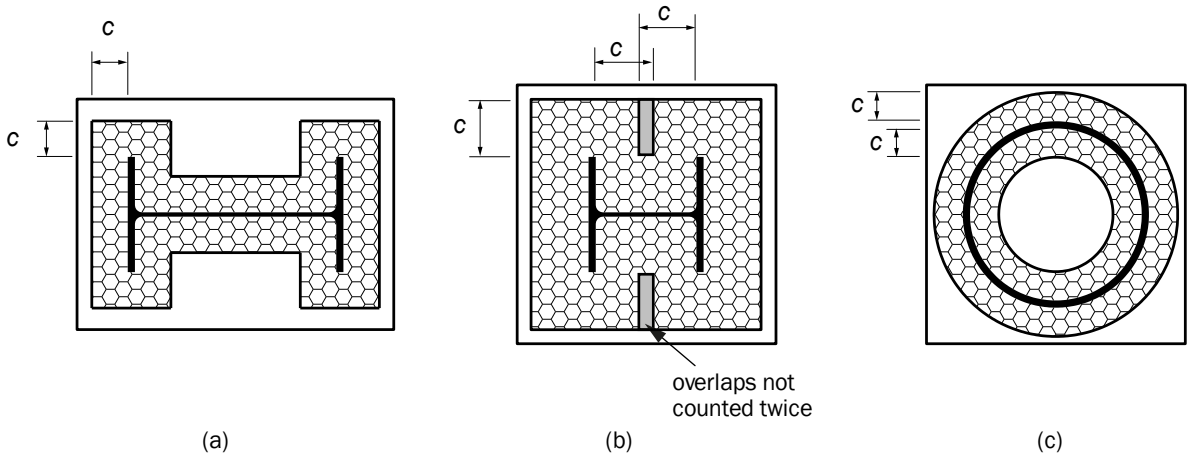
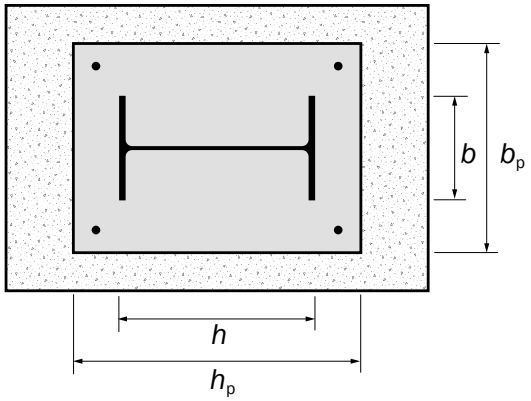


Figure 2.1 Column profile and effective area

CHECK 1 – REQUIRED AREA



Basic requirement

$A_p \geq A_{req}$

A_p is the area of base plate
$= h_p b_p$ for rectangular plates

A_{req} is the required area of base plate $= N_{Ed}/f_{jd}$

where:

f_{jd} $= \beta_j \alpha f_{cd}$

β_j may be taken as 2/3 (see Note 1 below)

α is a coefficient which accounts for dispersion of the concentrated force within the foundation (see Note 2 below)

If $\beta_j = 2/3$ and $\alpha = 1.5$ then $f_{jd} = f_{cd}$

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

α_{cc} $= 0.85$ (NA to BS EN 1992-1-1[22])

γ_c $= 1.5$ (NA to BS EN 1992-1-1)

Notes:

1. When determining the required area, the foundation material coefficient (β_j), may be taken as 2/3 assuming that:
   - The grout has a compressive strength at least equal to 0.2 f_{cd}, and
     - The grout is less than 50 mm thickness, or
     - The thickness of the grout is less than 0.2 h_p and 0.2 b_p.
   - The grout has a compressive strength at least equal to $2f_{cd}$, if over 50 mm thick.
2. Where the foundation's dimensions are unknown, but they will be orthodox (i.e. not narrow or shallow) it is reasonable to assume that the coefficient which accounts for diffusion of the concentrated force within the foundation $\alpha = 1.5$.

CHECK 1 – REQUIRED AREA (CONTINUED)

The coefficient α depends on the foundation depth and the distance between the plate and the edges of the foundation. If $\alpha = 1.5$ is assumed, the foundation depth should be at least 50% of the larger base plate dimension and all edge distances between the plate and the edge of the foundation should be at least 25% of the larger plate dimension. If more precision is required, $\alpha = \sqrt{A_{c1}/A_{c0}}$ as given in BS EN 1992-1-1.

As shown in Figure 2.3, if the edge distance of the foundation on all sides is larger than the maximum base plate dimension and the concrete is at least twice as thick as the maximum base plate dimension, $\alpha = 3$ and $f_{jd} = 2f_{cd}$.

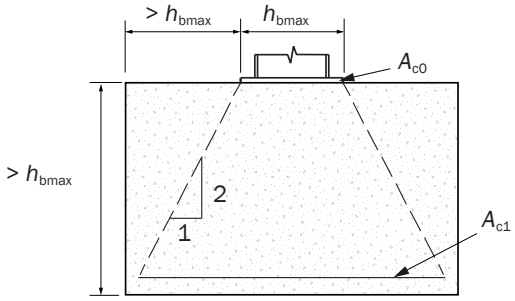
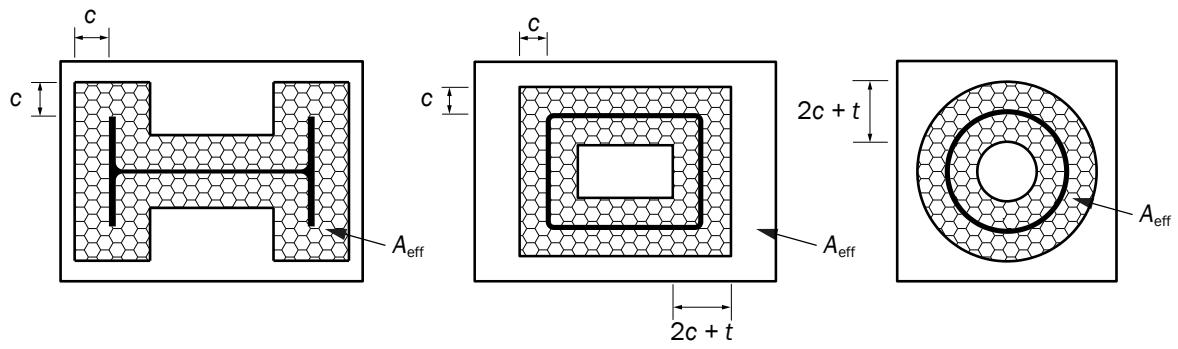


Figure 2.3 Foundation sizes to give maximum joint bearing strength

If the base is located on a support of limited dimensions, the area A_{c1} as shown in Figure 2.3 should be an area of identical shape to area A_{c0}, limited by the dimensions of the supporting structure and assuming a distribution of load at 1:2 as shown.

CHECK 2 – EFFECTIVE AREA



Basic requirement

$A_{req} \leq A_{eff}$

The additional bearing width c may be calculated from the following expressions. If there is overlap, or the dimensions of the base plate do not accommodate the additional bearing width, iteration may be necessary.

For a UB or UC $A_{eff} = 4c^2 + P_{col}\, c + A_{col}$

if $c > \dfrac{h - 2t_f}{2}$ then $A_{eff} = 4c^2 + 2(h + b)c + hb$

For a SHS or RHS $A_{eff} = P_{col}\,(t + 2c)$

if $c > \dfrac{b - 2t}{2}$ then $A_{eff} = 4c^2 + 2(h + b)c + hb$

For a CHS $A_{eff} = \pi\,(d - t)(t - 2c)$

if $c > \dfrac{d - 2t}{2}$ then $A_{eff} = \dfrac{\pi}{4}\,(d + 2c)^2$

where:

A_{req} is the required area of base plate (from Check 1)

A_{col} is the cross-sectional area of the column

h is the depth of the column

b is the width of the column

t is the thickness of the hollow section wall

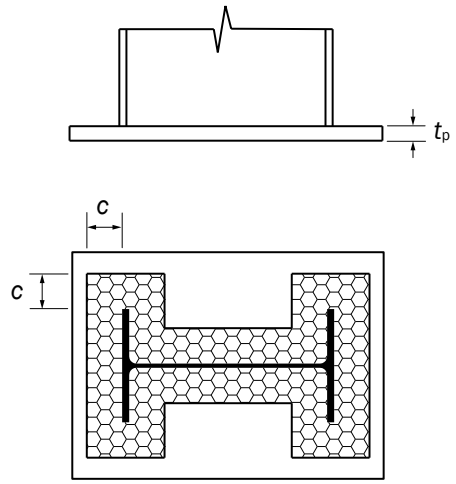
t_f is the thickness of the column flange

P_{col} is the column perimeter, which can be taken from section property tables, from the surface area per metre

For rigid bases, the effective area is located around the compressed flange.

If the dimension c is found to exceed the extent of the plate, the inadmissible area is neglected and the dimension c uniformly increased elsewhere to compensate.

CHECK 3 – PLATE THICKNESS



Basic requirement

$t_p \geq t_{p,min}$

$$t_{p,min} = c\sqrt{\frac{3\gamma_{M0} f_{jd}}{f_{yp}}}$$

where:

f_{yp} is the design strength of the base plate, noting that the strength decreases as the thickness increases

f_{jd} is defined in Check 1

c is defined in Check 2

γ_{M0} is taken from the UK National Annex to BS EN 1993-1-1 and equal to 1.0

CHECK 4 – WELDS

If the entirety of the column is in net compression, the welds to the column web need only be designed for shear.

Basic requirement for welds subject to a longitudinal force

$F_{w,Ed} \leq F_{w,Rd}$

where:

$F_{w,Ed}$ is the design value of the weld force per unit length $= F_{L,Ed}/l_w$

$F_{L,Ed}$ is the design value of the longitudinal force (shear)

l_w is the total effective length of the welds around the element. Generally it is convenient to assume that as the web of the section resists any horizontal shear, the welds along the web transfer the shear to the base plate. If welds are not continuous, a deduction must be made at each end of the weld length, in accordance with clause 6.5.1(1) of BS EN 1993-1-8:2024 (clause 4.5.1(1) of BS EN 1993-1-8:2005)

$F_{w,Rd}$ is the design resistance per unit length $= f_{vw,d}\, a$

a is the weld throat thickness $= 0.7$ s

s is the weld leg length

$f_{vw,d}$ $= \dfrac{f_u}{\sqrt{3}} \dfrac{1}{\beta_w \gamma_{M2}}$

β_w is the correlation factor according to the strength of the weaker part taken as 0.85 for S275 and 0.9 for S355 steel (Table 6.1 of BS EN 1993-1-8:2024 and Table 4.1 of BS EN 1993-1-8:2005)

γ_{M2} is taken from the UK National Annex to BS EN 1993-1-8 and equal to 1.25

f_u is the ultimate strength of the weaker part joined

$= 410$ N/mm^2 for S275 steel

$= 470$ N/mm^2 for S355 steel

BASE TENSION (UPLIFT)

3.1 INTRODUCTION

Most column bases in typical buildings will be subject to compression and shear. At the foot of bracing locations the base could be under significant net tension and shear, despite the contribution of axial compression. When bases are in tension, the holding down bolts transfer the force to the concrete substructure. The combination of shear and tension on the holding down bolts can be particularly onerous.

Base details subject to tension (either uplift or from a bending moment) may be stiffened, but thicker base plates are generally preferred to simplify fabrication.

3.2 DESIGN ASSUMPTIONS

BS EN 1993-1-8 presents an equivalent T-stub model to idealise the tension zone of a bolted joint. The T-stub comprises a web in tension, a flange in bending and bolts in tension, as shown in Figure 3.1. By calculating an appropriate length of equivalent T-stub, the resistance of the equivalent T-stub is taken to be equal to the resistance of the actual connection.

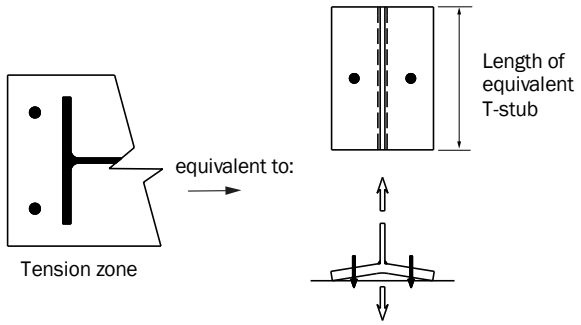


Figure 3.1 Equivalent T-stub model

The resistance of the equivalent T-stub is the lesser of:

- The resistance of the bolts – in this case the holding down bolts;
- The resistance of the "flange" in bending – in this case the base plate;
- The resistance of the "web" in tension – in this case the flange of the column.

BS EN 1993-1-8 recognises that prying may or may not occur in base details. In this guide, the beneficial effects of prying are neglected in the design of the base plate.

3.2.1 Resistance of holding down bolts in tension

The resistance of the holding down bolts is the lesser of the tensile resistance of the bolt itself and relevant modes of failure in the foundation.

3.2.2 Bolt resistance

The design tension resistance of the holding down bolt $F_{t,Rd}$ is given by:

$$F_{t,Rd} = \frac{0.9 f_{ub} A_s}{\gamma_{M2}}$$

where:

f_{ub} is the ultimate tensile strength of the holding down bolt

A_s is the tensile area of the holding down bolt

Design tension resistances for property class 8.8 holding down bolts are given in Table 3.1.

Bolt diameter	Design resistance (kN)
M20	141
M24	203
M30	323

Table 3.1 Design tension resistance of property class 8.8 holding down bolts

3.2.3 Pull-out resistance of holding down bolts

In the normal condition of holding down bolts cast in conical sleeves or cylindrical formers, not affected by proximity to a foundation edge, the resistance is based on the pull-out of the anchor plates embedded within the foundation. Attention is also drawn to clause A.18.1 of BS EN 1993-1-8:2024 which refers to other potential modes of failure, described in BS EN 1992-4.

In the case of anchors cast solidly into the concrete, the bond along the embedded length can be utilised. Threaded reinforcement may be used as embedded anchors.

If the tension resistance of an anchor embedded in a foundation is reliant on bond alone (i.e. there is no anchor plate), the design rules in BS EN 1992-1-1 limit the scope of application to bars with a maximum yield strength of 600 N/mm^2 (i.e. property class 4.6 or 6.8 bolts). If galvanised or plated anchors are used, bond should not be assumed for transmitting the tensile forces to the concrete.

When holding down bolts are cast in sleeves, the tension is assumed to be resisted by shear stress on the surface of a cone around each holding down bolt. Unless the holding down bolts are widely spaced, the conical surface may overlap. Guidance is available[23] which gives the area of overlapping cones. For initial design, a shear stress of 0.4 N/mm^2 may be used for typical grades of concrete.

Individual anchor plates are generally square and of the sizes given in Table 3.2. Individual anchor plates are commonly used but, when necessary, more elaborate anchorage systems such as welded channel sections or angles can be designed. If a combined anchor plate for a group of holding down bolts is used as an aid to maintain bolt location, large holes should be provided to facilitate concrete placing.

	Bolt diameter (property class 8.8)		
	M20	**M24**	**M30**
Length (mm)	300 (minimum) 375 450	375 450 600	450 600 750
Anchor plate (mm)	100 × 100	120 × 120	150 × 150
Thickness (mm)	15	20	25

Table 3.2 Recommended sizes of holding down bolts and anchor plates

A fabricated frame embedded within the foundation during casting may be used both to anchor all the holding down bolts and to keep them in the correct relative positions.

The design resistance of the anchorage should be based on determination of resistance to punching shear in accordance with BS EN 1992-1-1. The design procedure in this publication is based on BS EN 1992-1-1.

Punching shear is considered at a basic control perimeter a distance outside the loaded area as shown in Figure 3.2.

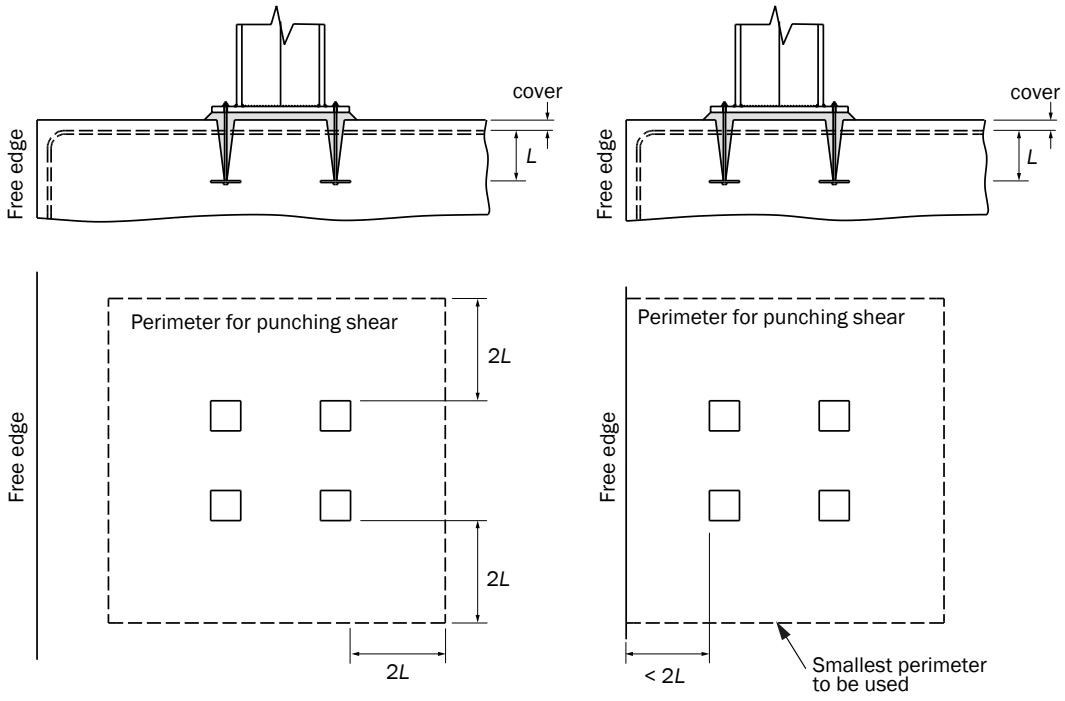


Figure 3.2 Control perimeter for a bolt group

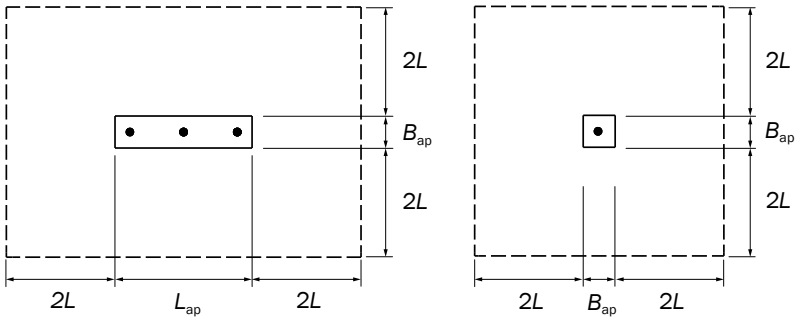


Figure 3.3 Basic control perimeter for single and multiple bolt group

The perimeter is located at a distance equal to twice the effective depth of a concrete slab, which for a column base is taken as the length of the holding down bolts below the concrete cover. If the edge of the base lies within the punching control perimeter then the base designer should verify that there is adequate anchored vertical reinforcement to resist the tension along the edge.

In most cases, the control perimeters of individual holding down bolts will overlap. When the control perimeters overlap, the resistance of the anchorage should be checked as a group with a single perimeter as shown in Figure 3.3, even if the anchor plates are not connected. If the plate is very large, the control perimeter around individual holding down bolts or groups of holding down bolts may govern, as shown in Figure 3.3.

Within Figure 3.2 and Figure 3.3:

L is the depth of the anchor bolts below the concrete cover

B_{ap} is the breadth of an anchor plate

L_{ap} is the length of an anchor plate

As an initial estimate, the shear resistance of the control perimeter may be based on a strength of 0.4 N/mm^2, for typical concrete grades in reinforced concrete bases without shear reinforcement.

3.3 DESIGN PROCEDURES

The design checks for a column base plate subjected to tension are:

- Check 1 – Bolt tension and T-stub
- Check 2 – Welds
- Check 3 – Anchorage

CHECK 1 – BOLT TENSION AND T-STUB

Basic requirement

$F_{t,Rd} \geq N_T$

N_T is the tensile force in the T-stub (For an I or H section this is taken as the net tensile force in the column flange)

The design resistance in bending is given by:

$F_{t,Rd} = \min (F_{t,pl,Rd}, F_{t,b,Rd})$

Resistance of base plate in bending

The design resistance of the base plate in bending is given by:

$$F_{t,pl,Rd} = \frac{2M_{pl,Rd}}{m_x}$$

Prying should not be relied on when the column is bearing on cementitious grout or on concrete. The above expression neglects any beneficial effects of prying.

where:

$$M_{pl,Rd} = \frac{025\Sigma l_{eff} t_p^2 f_{y,p}}{\gamma_{M0}}$$

l_{eff} is the effective length of the equivalent T-stub

t_p is the thickness of the base plate

$f_{y,p}$ is the design strength of the base plate

m_x is the distance from the bolt centreline to the fillet weld to the column flange (measured to a distance into the fillet weld equal to 20% of its leg length – see Figure 3.4)

γ_{M0} = 1.0 as given in the UK NA to BS EN 1993-1-1

The effective length of the T-stub l_{eff} can be determined from Table 3.3. If the corner holding down bolts are located outside the tips of the column flanges, the designer should verify whether the yield line patterns shown in the table are still appropriate.

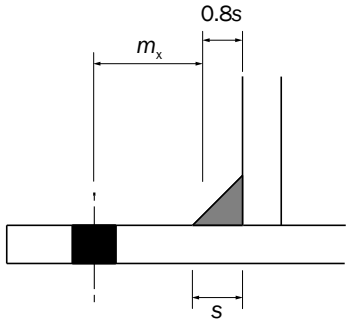


Figure 3.4 m_x **dimension**

CHECK 1 – BOLT TENSION AND T-STUB (CONTINUED)

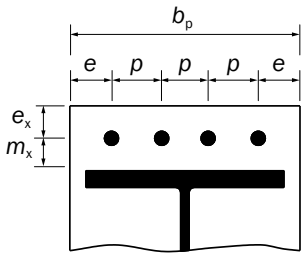
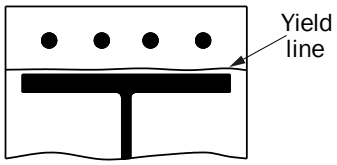
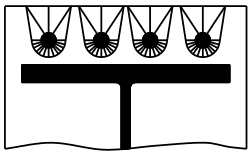
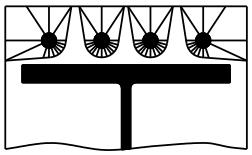
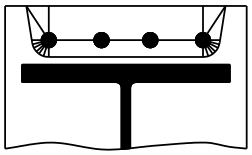
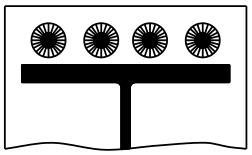
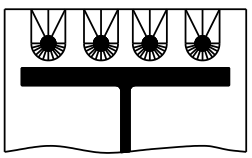
Single row of *n* holding down bolts	
	$l_{eff} = \frac{b_p}{2}$
	$l_{eff} = \frac{n}{2}(4m_x + 1.25e_x)$
	$l_{eff} = 2m_x + 0.625e_x + e + (n-2)(2m_x + 0.625e_x)$
	$l_{eff} = 2m_x + 0.625e_x + \frac{(n-1)p}{2}$
	$l_{eff} = n\pi m_x$
	$l_{eff} = \frac{n}{2}(\pi m_x + 2e_x)$

Table 3.3 Effective length for base plate T-stubs

CHECK 1 – BOLT TENSION AND T-STUB (CONTINUED)

Resistance of holding down bolts in tension

With a single row of holding down bolts, the design resistance is given by:

$F_{t,b,Rd} = n\, F_{t,Rd}$

where:

n is the number of holding down bolts in the equivalent T-stub

$F_{t,Rd}$ is the design tension resistance of a single bolt, as given in Table 3.1

CHECK 2 – WELDS

It is generally appropriate to assume that all the tension is carried by the flanges of the section and that horizontal shear is carried by the web. The welds to the flange should be designed as transverse welds and the web welds as longitudinal welds. The design of welds subject to a longitudinal force is covered in Section 2, Check 4.

Basic requirement for welds subject to a transverse force

$F_{w,Ed} \leq F_{w,Rd}$

where:

$F_{w,Ed}$ is the design value of the weld force per unit length $= F_{T,Ed} / l_w$

$F_{T,Ed}$ Is the design value of the transverse force

l_w is the total nominal length of the welds around the element (e.g. around the tension flange). If welds are not continuous, the nominal length must be reduced by twice the throat thickness, in accordance with clause 6.5.1(1) of BS EN 1993-1-8:2024 (4.5.1(1) of BS EN 1993-1-8:2005)

$F_{w,Rd}$ is the design weld resistance per unit length

$= 1.225\, a\, f_{vw,d}$ (for a transverse weld between elements at 90°)

a is the effective throat thickness of a fillet weld $= 0.7$ s

s is the leg length of a fillet weld

$f_{vw,d}$ $= \dfrac{f_u}{\sqrt{3}} \dfrac{1}{\beta_w \gamma_{M2}}$

β_w is the correlation factor according to the strength of the weaker part taken as 0.85 for S275 and 0.9 for S355 steel

γ_{M2} is taken from the UK National Annex to BS EN 1993-1-8 and equal to 1.25

f_u is the ultimate strength of the weaker part joined

$= 410$ N/mm^2 for S275 steel

$= 470$ N/mm^2 for S355 steel

The verification of welds subject to a longitudinal force is covered in Section 2.2.4

If welds are designed as subject to both transverse and longitudinal forces, the verification may be completed using either the simple method (conservative) of Section 3.3.5 or the directional method of Section 3.3.6.

CHECK 2 – WELDS (CONTINUED)

Simple approach for welds in combined shear and tension

$F_{w,Ed} \leq F_{w,Rd}$

$F_{w,Ed} = (F_{L,Ed}^2 + F_{T,Ed}^2)^{0.5}$

where:

$F_{w,Rd}$ is the design weld resistance per unit length $= a\, f_{vw,d}$

$F_{L,Ed}$ is the design value of the longitudinal force per unit length

$F_{T,Ed}$ is the design value of the transverse force per unit length

Directional approach for welds in combined shear and tension

$$a \geq \frac{\sqrt{3}\beta_w \gamma_{M2}}{f_u}\left(\frac{F_{T,Ed}^2}{K^2} + F_{L,Ed}^2\right)^{0.5} \text{ and } a \geq \frac{\gamma_{M2} F_{T,Ed} \sin\theta}{0.9 f_u}$$

where:

a is the effective throat thickness of a fillet weld

$K = \sqrt{\frac{3}{(1 + 2\cos\theta^2)}} = 1.225$ (for elements connected at 90°)

θ is the angle between the throat and the applied transverse force, commonly 45° for elements connected at 90°.

All other variables as previously defined.

CHECK 3 – ANCHORAGE

The anchorage should be verified by the designer responsible for the foundations.

Basic requirement

$V_{Rd,cs} > V_{Ed}$

where:

V_{Ed} is the design shear force, taken as the total tension force in the holding down bolts within the control perimeter

$V_{Rd,cs}$ is the resistance to punching shear determined in accordance with BS EN 1992-1-1

BASE SHEAR

4.1 INTRODUCTION

There are three principal ways of transferring shear from column base plates into the foundation:

- Friction between the base plate and grout (in limited circumstances the resistance of the holding down bolts may be combined with friction).
- Shear and bearing of the holding down bolts.
- Bearing of a shear stub welded to the underside of the base plate.
- Bearing of the base and column when cast within a pocket.

In common UK practice, oversize holes are provided in the base plate. If oversize holes are used, unless special measures are taken, it cannot be assumed that all the holding down bolts are effective in transferring shear.

Horizontal forces may be:

- Parallel to the web of a section, in which case it is assumed that the column web transfers shear to the base plate.
- Parallel to the flanges of a section, when it is assumed that one or both flanges transfers shear to the base plate.

In the common case of a bracing member intersecting with the base, the horizontal component of force may be transferred directly to the base plate via a gusset plate if the plate and welds are designed to transfer this force. In other arrangements, the horizontal component of force in the bracing member will contribute to the shear in the column web (or if the bracing is in the minor axis, will contribute to the shear in the flanges).

The vertical component of a bracing force is assumed to be shared between the flanges and web(s) of the column in proportion to their areas. Unless the centreline of the bracing and the centreline of the column intersect at the underside of the base plate, the resulting eccentric moment is treated as a couple, further modifying the forces in the flanges.

4.2 FRICTION

In the permanent design condition, friction between the base and the grout may be sufficient to resist the base shear. The coefficient of friction should be taken as 0.3[1].

The contribution of the holding down bolts can only be added to the design friction resistance if the holes in the base plate are not oversize, or if other measures are taken (Section 4.3) to ensure that the holding down bolts are in direct bearing with the base plate.

If oversize holes are specified without any special measures (which is the usual case in the UK), the contribution of the holding down bolts cannot be added to the design friction resistance.

4.3 HOLDING DOWN BOLTS DESIGN RESISTANCE

In oversize holes, if no special measures are taken, it cannot be assumed that base shear is shared equally by all holding down bolts.

If oversize holes are used, a simple approach in orthodox construction (such as braced frames or portal frames) is to assume that only half of the holding down bolts are utilised in transferring shear.

To utilise all holding down bolts in the transfer of base shear, options are:

- Use a base plate with standard clearance holes (2 mm larger than the bolt diameter for holding down bolts up to M24 and 3 mm if larger). If this option is adopted, casting the holding down bolts with precision is necessary and the use of a rigid template to hold the tops of the holding down bolts in the correct position is strongly recommended.
- Fill the annulus around the holding down bolt with resin and verify the bearing resistance in accordance with clause 5.9.4 of BS EN 1993-1-8:2024 (clause 3.6.2 of BS EN 1993-1-8:2005).
- After erection, weld cover plates with normal clearance holes to the top of the base plate. The surface finish of the base plate to facilitate welding and corrosion protection after welding should be considered. Welding of galvanised steel is not recommended. The cover plates must be thick enough to develop the required design bearing resistance.

4.3.1 Holding down bolts in shear

According to clause A.20 of BS EN 1993-1-8:2024, the design shear resistance of a holding down bolt is given by:

$$F_{vb,Rd} = \frac{(\alpha_{bc} f_{ub} A_s)}{\gamma_{M2}}$$

where:

α_{bc} $= 0.44 - 0.0003 f_{yb}$

f_{yb} is the yield strength of the holding down bolt

f_{ub} is the ultimate tensile strength of the holding down bolt

A_s is the tensile stress area of the holding down bolt

Clause 6.2.2(7) of BS EN 1993-1-8:2005 has similar provisions although the shear resistance of a holding down bolt is the lower of the above requirement and the design bolt resistance according to clause 3.6.1. The above expression including α_{bc} will always be critical, as recognised in BS EN 1993-1-8:2024.

4.3.2 Bearing at holding down bolt holes

The design bearing resistance of the holding down bolt holes should be calculated in the same way as ordinary bolts. It is highly unlikely that the design bearing resistance will be critical. If cover plates are used, the design bearing resistance must be calculated as the cover plates are likely to be significantly thinner than the base plate.

Having allowed for bolt deformation (Section 4.3.1) no bearing check on the concrete or grout is required.

4.3.3 Resin filling

Though uncommon in the UK, the annulus around the holding down bolts may be filled with resin. To allow for inaccurate filling to level, it is recommended to assume the annulus has only been filled to 80% of the plate thickness. The design resistance of the bearing resistance of the resin may be calculated in accordance with clause 5.9.4 of BS EN 1993-1-8:2024 (clause 3.6.2.2(5) of BS EN 1993-1-8:2005). If the annulus around holding down bolts is filled with resin, all holding down bolts may be assumed to contribute to the design shear resistance, in combination with friction.

4.4 SHEAR STUB

If a base is subject to significant shear, a shear stub may be welded to the underside of the base, located in a pocket in the foundation, as shown in Figure 4.1. The pocket is filled with non-shrink grout after the column is in its final position.

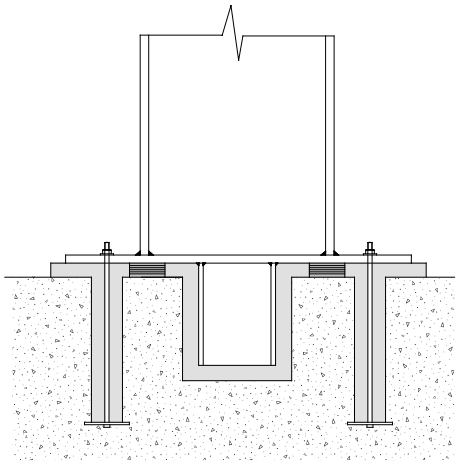


Figure 4.1 Base with shear stub

A shear stub (or shear key) typically consists of a short length of a steel Section welded to the underside of the base plate.

The horizontal shear from the bracing and any shear in the column, produces a reaction between the shear stub and the surrounding grout. This reaction produces a secondary moment M_{sec} as shown in Figure 4.2, which must be added to any other moments acting on the base.

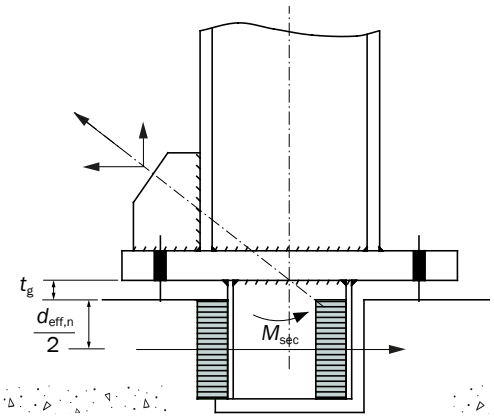


Figure 4.2 Base with stub subject to shear

The following assumptions are made in the design model:

- Both embedded flanges of an I-section stub are assumed to provide equal horizontal resistance to the horizontal component of the bracing force.
- A uniform bearing stress on the grout around the stub is assumed, as shown in Figure 4.2.
- The effective depth of the stub, $d_{eff,n}$, is taken as the depth embedded in the foundation, neglecting the grout thickness, t_g.

- It is assumed that bending of the stub flanges need not be considered. To support the assumption, the thickness of the flange should be no less than 1/20 of the width of the stub, usually requiring the selection of a UC profile.

Welding a stub to the underside of the base plate will increase the stiffness of the joint. To support the assumed characteristics of a nominally pinned base, the stub should be as small as possible.

4.4.1 Secondary moment M_{sec}

The secondary moment M_{sec} is treated as a couple producing tension and compression in the flanges of the stub as illustrated in Figure 4.3. The secondary moment M_{sec} is added to the design forces and moments (if any) on the base, producing additional compression on one side of the joint and reduced compression or additional tension in the holding down bolts on the opposing side of the joint. Guidance on designing bases for bending moments is given in Section 5.

The forces and bending moment which develop in the stub must be transferred through the base plate into the column.

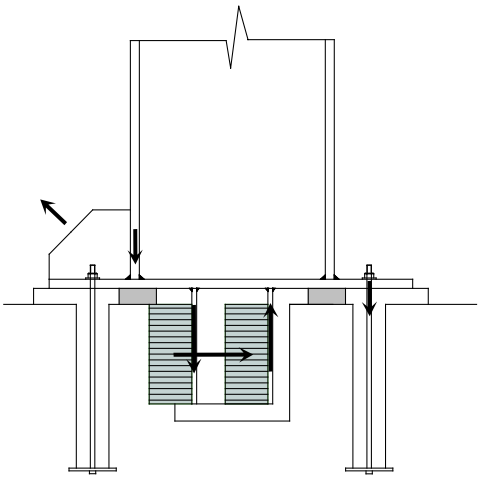


Figure 4.3 Shear stub model – forces due to secondary moment

It is conservative to assume that the forces in the flanges of the stub have a local effect on the column above the base plate.

If the flanges of the shear stub and column are in close proximity, it may be assumed that the force in the flange of the shear stub is transferred directly into the flange of the column. In the more general case of a wide column and narrow shear stub as shown in Figure 4.4, the force in the flange of the shear stub is transferred into the web of the column, requiring:

- a verification of the web of the column, which may also be subject to shear stress, and
- a verification of the effective width of the flange of the shear stub.

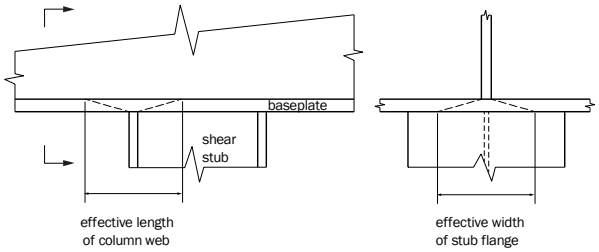


Figure 4.4 Shear stub not aligned with column flanges

In the usual arrangement with the gusset plate welded directly to the base plate, as shown in Figure 4.5, the horizontal welds between the gusset plate and the base plate may be designed for the full horizontal component for the bracing force. If the horizontal shear in the bracing is assumed to be transferred to the base plate, it may be assumed that the horizontal component of the bracing force has no impact on the shear stress in the column web.

If it is assumed some proportion of the horizontal component of the bracing force is transferred to the column, that force must be included as a shear stress in the column web verification.

A Von Mises check should be completed, taking the longitudinal stress $\sigma_{x,Ed}$ as the sum of the axial stress in the column and the local stress distributed from the flange of the shear stub. The shear stress, τ_{Ed} is the shear stress in the column (if any) which may be assumed to be distributed uniformly over a shear area equal to $h \times t_w$.

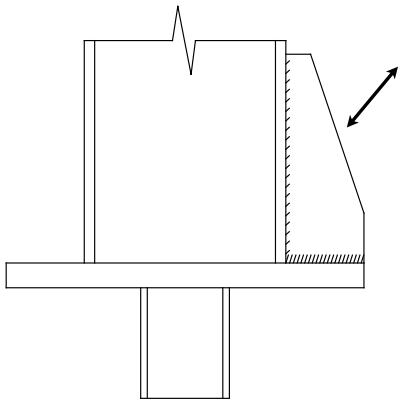


Figure 4.5 Gusset plate welded to base plate

In addition to verifying the stub itself and the weld, supplementary design checks are required on:

- Any additional compression assumed to act on the line of the column flange.
- Any additional tension assumed to develop in the holding down bolts.

The additional compression and tension forces which develop due to the secondary moment should be considered in combination with the axial force in the column, leading to a net reduction or increase in the forces allocated to the column flanges.

4.5 DESIGN PROCEDURES

The design checks for a column base subjected to shear are as follows:

- Check 1 – Base friction
- Check 2 – Holding down bolt in shear
- Check 3 – Welds
- Check 4 – Shear stub
- Check 5 – Supplementary checks

CHECK 1 – BASE FRICTION

Basic requirement

$V_{c,Ed} \leq F_{f,Rd}$

where:

$V_{c,Ed}$ is the design shear force

$F_{f,Rd}$ $= C_{f,d}\, N_{c,Ed}$

$N_{c,Ed}$ is the column design compressive load

$C_{f,d}$ is the coefficient of friction between the base plate and the grout layer

The coefficient of friction between the base plate (with no surface treatment) and the cementitious grout layer may be conservatively taken as 0.3. Tests in accordance with BS EN 1990[24] Annex D are required to determine an accurate coefficient value or a value for other types of grout.

CHECK 2 – HOLDING DOWN BOLT IN SHEAR

Basic requirement

$V_{c,Ed} \leq nF_{vb,Rd}$

where:

$V_{c,Ed}$ is the design shear force

$F_{vb,Rd}$ is the design shear resistance of a holding down bolt.

$= \frac{(\alpha_{bc} f_{ub} A_s)}{\gamma_{M2}}$

α_{bc} $= 0.44 - 0.0003\, f_{yb}$

A_s is the tensile stress area of the bolt

f_{ub} is the ultimate strength of the holding down bolt

f_{yb} is the yield strength of the holding down bolt, where 235 N/mm^2 $\leq f_{yb} \leq$ 640 N/mm^2

γ_{M2} is taken from the UK National Annex to BS EN 1993-1-8 and equal to 1.25

n is the number of holding down bolts resisting shear

Combined bolt shear and friction

Combining friction and the resistance of the holding down bolts is only permitted if:

- The holes in the base plates are normal clearance diameter, or
- The annulus around the holding down bolts are filled with resin, or
- Washer plates with standard diameter holes are welded to the base plate after erection.

$V_{c,Ed} \leq F_{v,Rd}$

where:

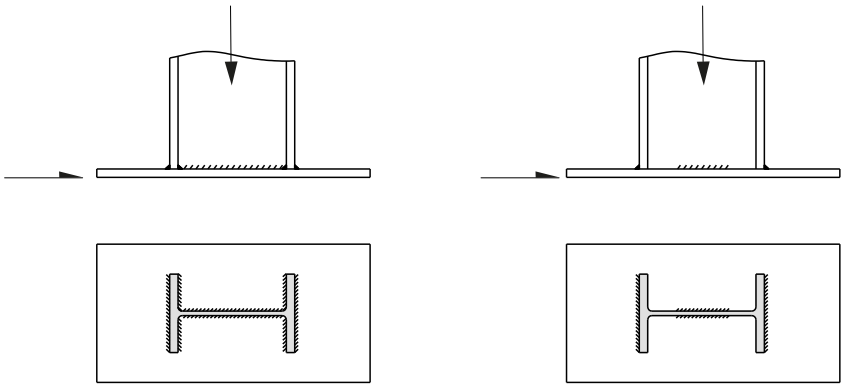
$F_{v,Rd}$ $= F_{f,Rd} + n\, F_{vb,Rd}$

$F_{f,Rd}$ is the design friction resistance from Check 1

$F_{vb,Rd}$ is the design shear resistance of a holding down bolt

n is the number of holding down bolts in the base plate treated with one of the special arrangements identified above.

CHECK 3 – WELDS



Basic requirement for shear

$V_{Ed} \leq F_{w,Rd}\, l_{w,eff}$

$F_{w,Rd}$ is the resistance of a fillet weld per unit length

$= a\, f_{vw,d}$

where:

a is the weld throat thickness $= 0.7$ s

$f_{vw,d}$ $= \dfrac{f_u}{\sqrt{3}} \dfrac{1}{\beta_w \gamma_{M2}}$

β_w is the correlation factor according to the strength of the weaker part taken as 0.85 for S275 and 0.9 for S355 steel

γ_{M2} is taken from the UK National Annex to BS EN 1993-1-8 and equal to 1.25

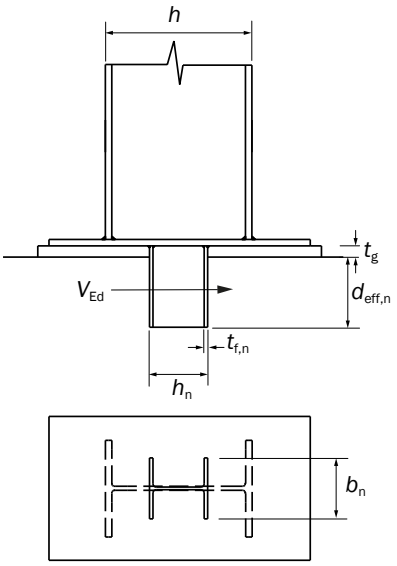
l_w is the total nominal length of the welds in direction of shear – normally the full depth of each side of the web

f_u is the ultimate strength of the weaker part joined

$= 410$ N/mm^2 for S275 steel

$= 470$ N/mm^2 for S355 steel

CHECK 4 – SHEAR RESISTANCE OF STUB



Concrete or grout resistance

$V_{Ed} \leq V_{n,c,Rd}$

where:

V_{Ed} is the design value of the shear force

$V_{n,c,Rd}$ $= 2b_n\, d_{eff,n}\, f_{cd}$ (assuming a stub with two flanges)

f_{cd} is the design compressive strength of concrete (or grout) whichever is weaker. Grout should normally be specified to be stronger than the concrete.

$f_{cd} = \frac{f_{ck}}{\gamma_c}$ (see Section 2, Check 1)

b_n is the width of the shear stub, perpendicular to the line of action of the shear force

h_n is the height of the shear stub, parallel to the line of action of the shear force

t_g is the thickness of the grout below the base plate

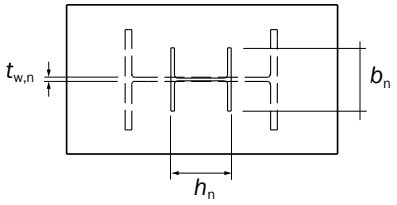
$t_{f,n}$ is the thickness of the flange of the shear stub

$t_{w,n}$ is the thickness of the web of the shear stub

$d_{eff,n}$ is the effective depth of the shear stub (which is the depth embedded in the foundation, neglecting the thickness of the grout)

h is the height of the column, measured parallel to the line of action of the shear force

CHECK 4 – SHEAR RESISTANCE OF STUB CONT...



Shear resistance of the stub

$V_{Ed} \leq V_{n,Rd}$

where:

V_{Ed} is the design value of the shear force

$V_{n,Rd} = \dfrac{A_{vn} f_{u,n}}{\sqrt{3}\gamma_{M0}}$

A_{vn} is the shear area of the shear stub. This may be calculated from BS EN 1993-1-1[18] or approximated as $h_n \times t_{w,n}$

$f_{u,n}$ is the ultimate strength of the shear stub

Bending resistance of stub

The secondary moment M_{sec} in the stub is decomposed to forces in the flanges of the stub, which may be limited by the effective breadth of the flange.

$M_{sec} \leq M_{n,Ed}$

where:

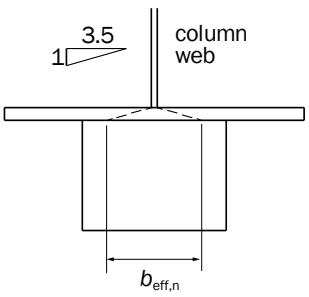
$M_{n,Rd} = b_{eff,n} \times t_{f,n} \times f_{y,n} \times (h_n - t_{f,n})$

$b_{eff,n}$ is the effective breadth of the stub flange, assuming a spread from the column web at 1:3.5 through the base plate, but $b_{eff,n} \leq b_n$

$f_{y,n}$ is the design strength of the shear stub

h_n is the overall depth of the shear stub

$t_{f,n}$ is the thickness of the stub flange



CHECK 4 – SHEAR RESISTANCE OF STUB (CONTINUED)

Weld resistance (stub to underside of base plate)

The secondary moment M_{sec} should be calculated by taking moments about the underside of the base plate, on the centreline of the column.

The moment on the shear stub, $M_{sec} = V_{Ed} \times \left(t_g + \frac{d_{eff,n}}{2}\right)$

The resultant force in each flange of the stub is given by $\frac{M_{sec}}{h_n - t_{fn}}$

Stub flange welds

The transverse weld around each flange should be sized to accommodate the force in the flange.

$F_{w,Ed} \leq F_{w,Rd}$

$F_{w,Ed}$ is the design value of the weld force per unit length, given by $\frac{M_{sec}}{(h_n - t_{fn}) \times 2(b_{eff,n})}$

where:

h_n is the section height of the shear stub

$b_{eff,n}$ is the effective width from 4.5.7

$t_{f,n}$ is the flange thickness of the stub

$F_{w,Rd}$ is the design weld resistance per unit length

$= 1.225\ a\ f_{vw,d}$ (for a transverse weld between elements at 90°)

a is the throat thickness of a fillet weld

$= 0.7$ s

s is the leg length of a fillet weld

$f_{vw,d} = \frac{f_u}{\sqrt{3}} \frac{1}{\beta_w \gamma_{M2}}$

Stub web welds

The weld along the web should be sized to accommodate the design shear.

$F_{w,Ed} \leq F_{w,Rd}$

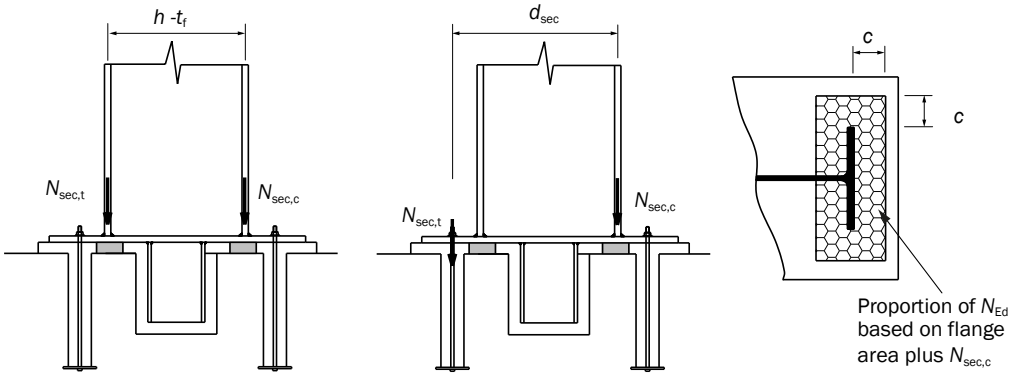
$F_{w,Ed}$ is the design value of the weld force per unit length, given by $V_{Ed} / 2(h_n - 2t_{f,n})$

$F_{w,Rd}$ is the design weld resistance per unit length

$= a\ f_{vw,d}$ (for a longitudinal weld)

All other notation as previously defined.

CHECK 5 – SUPPLEMENTARY CHECKS



Additional net tension in holding down bolts

Additional net compression

Supplementary forces to be considered in the base design

The secondary moment M_{sec} is combined with any other bending moments on the base and the joint verified as a moment-resisting base.

The vertical component of a bracing force is assumed to be divided uniformly across the column cross section. Forces in elements of the cross section may be determined in proportion to their areas.

If the vertical component of the bracing force leads to additional tension in a flange, the holding down bolts must be verified for the increased force.

If the vertical component of the bracing force results in additional compression in a flange the bearing resistance of the concrete must be verified. An effective area around the flange may be assumed when verifying the required thickness of base plate.

The total force in a flange may be taken as the summation of the axial force allocated to the flange on the basis of the flange area, N_{direct}, plus the contribution from the vertical component of the bracing force, $N_{brace,V}$, plus the secondary force N_{sec} which results from any moment due to bracing eccentricity. The direction of the forces should be respected when calculating the net total.

$$N_{tot} = N_{direct} + N_{brace,V} + N_{sec}$$

If the total compression N_{tot} exceeds N_{direct}, the T-stub in compression around the flange should be verified.

$$A_{req} = \frac{N_{tot}}{f_{jd}}$$

$$A_{eff} = (2c + b)(2c + t_f)$$

The additional bearing width c is found by setting $A_{req} = A_{eff}$ and solving for c.

The required base plate thickness, t_p is given by:

$$t_p \geq c\sqrt{\frac{3\gamma_{M0} f_{jd}}{f_{y,p}}}$$

CHECK 5 – SUPPLEMENTARY CHECKS (CONTINUED)

where:

$f_{y,p}$ is the design strength of the base plate, noting that the strength decreases as the thickness increases

N_{tot} is the design axial compression in the flange

b is the width of the column

t_f is the thickness of the column flange

Verification of web subject to local forces

The distribution of load from the flanges of a shear stub into the column web may assume a 1:3.5 spread through the base plate from the flanges of the stub.

It is conservative to add the longitudinal stress in the column to the additional stress assumed to be distributed from the flange of the shear stub. The shear stress may be neglected if the base details have been arranged such that the column web is not required to transfer shear. If the column web is subject to shear, a Von Mises interaction may be used to verifiy the combined longitudinal and shear stresses in the zones indicated in Figure 4.6, such that:

$$\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)^2 + 3\left(\frac{\tau_{Ed}}{f_y/\gamma_{M0}}\right)^2 \leq 1$$

where:

$\sigma_{x,Ed}$ is the longitudinal stress, being the sum of the axial stress in the column and the local stress distributed from the flange of the shear stub

τ_{Ed} is the shear stress in the column (if any) which may be assumed to be distributed uniformly over a shear area equal to $h \times t_w$

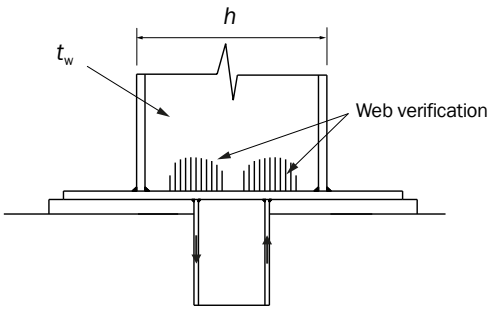


Figure 4.6 Local effects due to forces in shear stub flanges

4.6 BASES WITH BRACING ON THE MINOR AXIS

The common arrangement is for bracing on the minor axis to be coincident with the minor axis of the column section, as shown in Figure 4.7. It is recommended that the bracing be set out the align with the column centreline at the underside of the base plate to avoid introducing bending moment into the base.

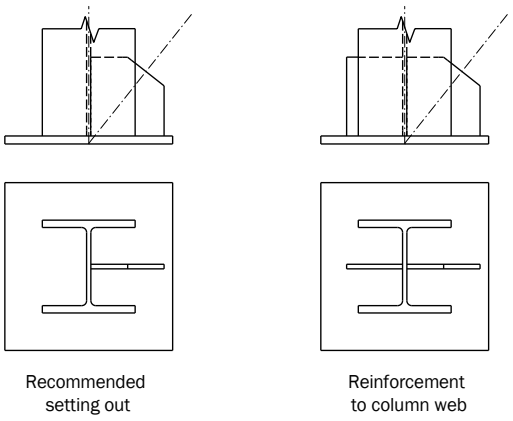


Figure 4.7 Bracing on minor axis

It is generally convenient to design the welds between the gusset plate and the base plate for the horizontal component of the bracing force. The vertical welds to the column web should then be designed for the vertical component of the bracing force. If horizontal force is assumed to be transferred to the web, recommended practice is to provide reinforcement on the opposing side of the column web, similar in size to the gusset.

If a shear stub must be added to accommodate significant horizontal shears, the orientation of the stub is generally perpendicular to the column above, as shown in Figure 4.8. This orientation results in a minor axis moment on the base and demands careful consideration of how the forces in the stub flanges are transferred into the joint.

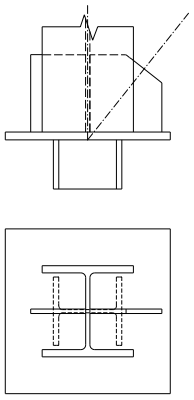


Figure 4.8 Bracing on minor axis with shear stub

Recommendations for bases with open sections subject to minor axis moments are given in Section 5.3.

The moment in the stub must be transferred to the flanges of the column above. Depending on the relative sizes of the column and stub, it may be possible to demonstrate that the forces are transferred directly if there is sufficient overlap at the tips of each member. Alternatively, stiffeners may be provided to transfer the forces in the stub flange to the column flanges, as shown in Figure 4.9.

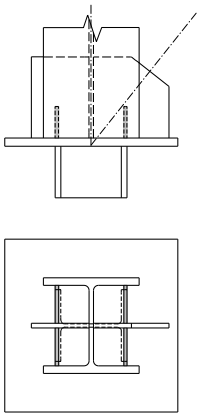


Figure 4.9 Bracing on minor axis with shear stub and stiffeners

BASES IN COMBINED AXIAL FORCE AND MOMENT

5.1 INTRODUCTION

This section covers the design of unstiffened base plates. Other arrangements may be chosen as illustrated in Figure 5.1 which:

- Distribute compression over a larger effective area, with a T-stub around each element.
- Provide a longer lever arm to holding down bolts in tension.
- Strengthen the base plate in the tension zone.

Guidance on practical sizing of stiffeners is given in Section 1.11.

5.2 DESIGN ASSUMPTIONS

The design bending moment on a base is the combination of any moment determined from the frame analysis, any moment which results from the setting out of a bracing force and any moment from a shear stub under the base.

The bending moment on the base is calculated at the underside of the base plate, on the centreline of the column.

5.2.1 Base moment from bracing force

If the bracing is set out to intersect on the centreline of the column at the underside of the base plate ($e = 0$), there is no resulting moment on the base. If the bracing is set out to any other point, a bending moment is produced which is equal to the bracing force multiplied by its eccentricity to the neutral setting out point, as shown in Figure 5.2.

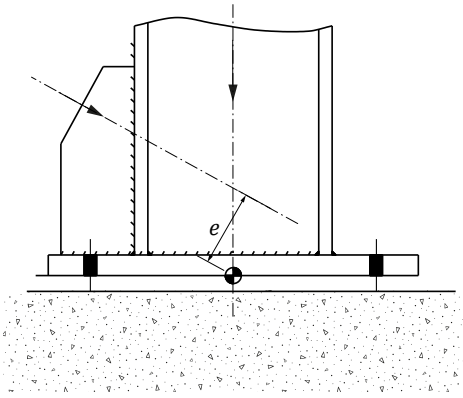


Figure 5.2 Eccentric bracing setting out

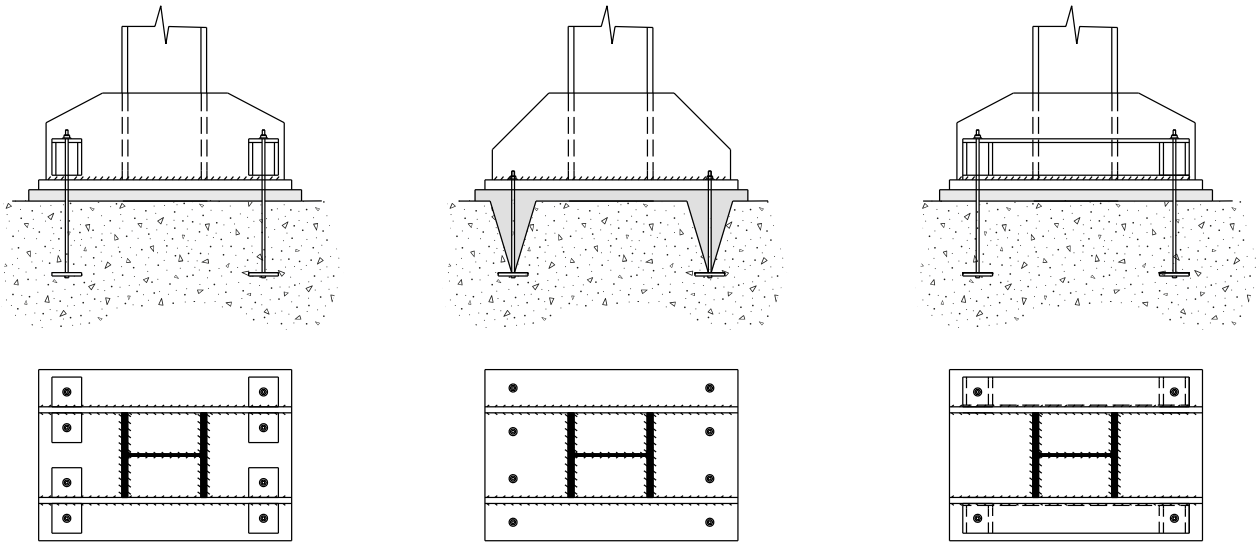


Figure 5.1 Reinforced base details

5.2.2 Base with a shear stub

The secondary moment M_{sec} (see Section 4.5) is additional to any other moments on the base.

5.2.3 Design model

For combined axial force and major axis bending at a column base, the design model in BS EN 1993-1-8 assumes that resistance is provided by two equivalent T-stubs in the base plate:

- One in tension, involving the holding down bolts and base plate in bending.
- One in compression, based on a compression zone in the concrete.

Generally, it is assumed that the effective area in compression is concentric with the compression flange. A larger moment resistance may be obtained by assuming the compression zone is partly or wholly outside the column flange, provided that the base plate is designed for the eccentricity. The range of situations is shown in Figure 5.3.

When bases are subject to combined axial load and bending moment, BS EN 1993-1-8 proposes a very simple model with modest differences depending on the relative magnitudes of the axial load and bending moment.

Three situations are recognised in BS EN 1993-1 8:

- "Dominant compression force", when both flanges of the column are in net compression.
- "Dominant tension force" when, both flanges of the column are in net tension.
- "Dominant bending moment", when one flange of the column is in compression and one in tension.

To determine which situation is dominant, the axial force in the column is assumed to be divided equally between the two flanges. The applied moment is distributed as two opposing forces separated by a lever arm equal to $h - t_f$.

After determining which situation is dominant, the force and moment is distributed in accordance with Figure 5.4.

If compression or tension is dominant:

- The axial force in the column is divided equally.
- The moment is decomposed to additional compression or tension forces by dividing the moment by the lever arm, z, as shown in Figure 5.4.

If the bending moment is dominant, the distribution of the axial force in the column depends on the position of the column centreline with respect to the lever arm, z, as shown in Figure 5.4.

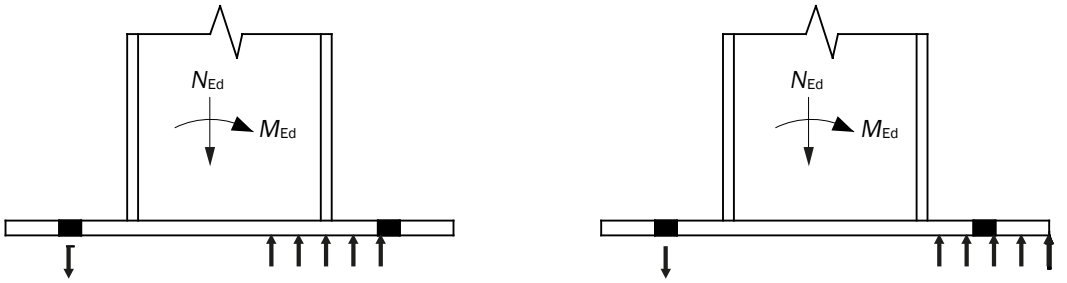
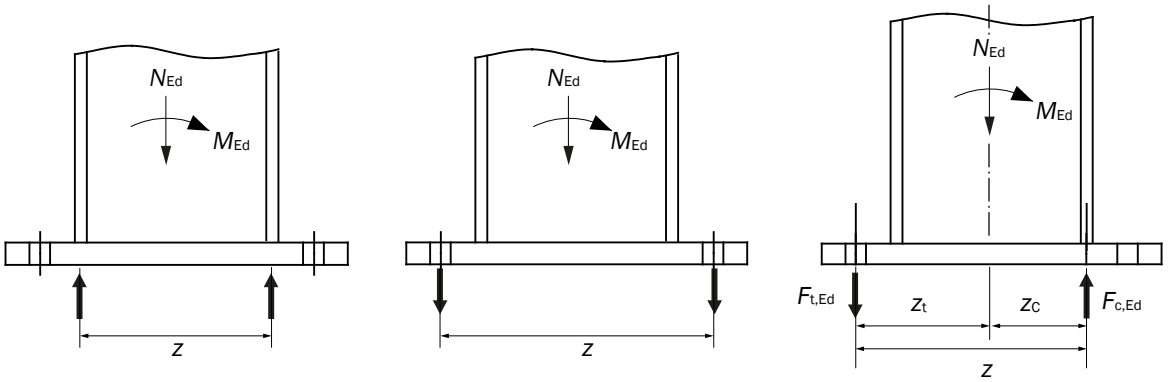


Figure 5.3 Concentric and eccentric compression zones



Dominant compression force | Dominant tension force | Dominant bending moment

Figure 5.4 Lever arm *z*

If the bending moment is dominant, the compression force, $F_{c,Ed}$ is given by:

$$F_{c,Ed} = N_{Ed}\frac{z_T}{z} + \frac{M_{Ed}}{z}$$

and the tension force $F_{t,Ed}$ is given by:

$$F_{t,Ed} = N_{Ed}\frac{z_C}{z} + \frac{M_{Ed}}{z}$$

where z_T and z_C are shown in Figure 5.4 for the dominant bending moment case.

5.3 MINOR AXIS MOMENT

BS EN 1993-1-8 contains no information on the design of bases to open sections subject to a minor axis moment. In principle, the guidance in BS EN 1993-1-8 may be adapted to accommodate minor axis moments. The compression zone may be assumed to be centred on each flange tip, surrounding a length of flange equal to its thickness, as shown in Figure 5.5.

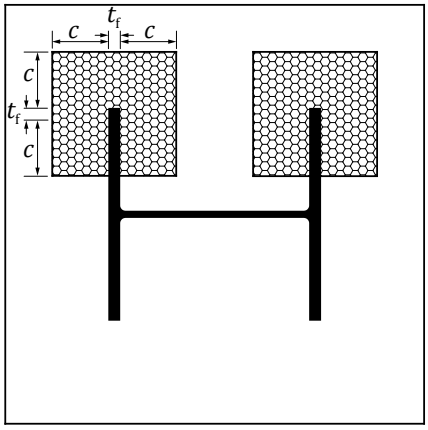


Figure 5.5 Effective compression area for minor axis bending

Resistance to tension (if any) is provided by holding down bolts on the tension side of the base detail. The yield line patterns from Table 3.3 should be carefully selected, as several patterns rely on the presence of a flange perpendicular to the direction of the applied moment – which does not exist when open sections are subject to a minor axis bending moment.

In the common case of holding down bolts located in the corners of the base plate, a yield line pattern extending across the corner of the plate, as illustrated in Figure 5.6, should be included in the patterns to be considered.

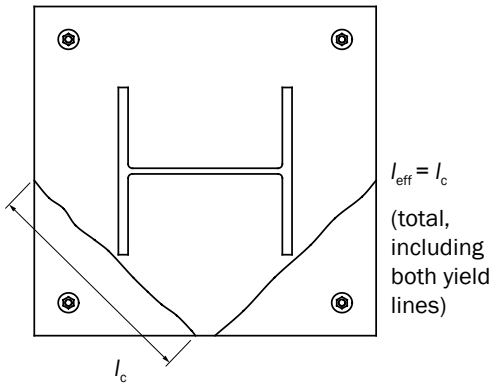


Figure 5.6 Yield line pattern for holding down bolts in plate corners

For more significant minor axis moments, it may be beneficial to weld plates across the toes of the section, as shown in Figure 5.7, which may then be treated as if they were flanges in both compression and tension zones.

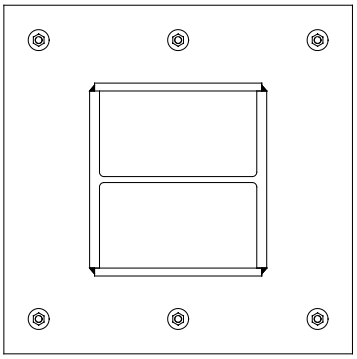


Figure 5.7 Plate reinforcement across column toes

An alternative approach, outside the scope of this publication, is to model the base detail in software which undertakes a finite element analysis of the joint.

5.4 BI-AXIAL MOMENT

Reference 20 suggests that when subject to biaxial bending, an acceptable design is obtained when:

$$\left(\frac{M_{Ed,y}}{M_{Rd,y}}\right)^2 + \left(\frac{M_{Ed,z}}{M_{Rd,z}}\right)^2 < 1$$

An alternative approach, outside the scope of this publication, is to model the base detail in software which undertakes a finite element analysis of the joint.

5.5 DESIGN PROCEDURES

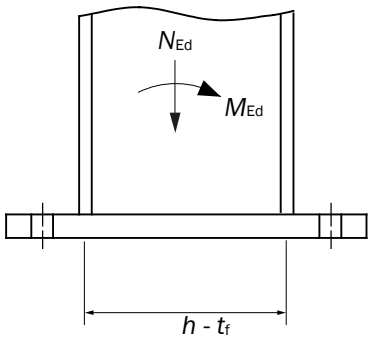
The design checks for a column base subjected to a moment in the major axis are as follows:

- Check 1 – Determine the forces in the equivalent T-stubs
- Check 2 – Resistance of the compression zone
- Check 3 – Resistance of the tension zone
- Check 4 – Resistance of welds in tension

CHECK 1 – FORCES IN THE EQUIVALENT T-STUBS

To determine if the axial force or moment is dominant the force in each flange is calculated, assuming the axial force is distributed equally between the flanges and the moment is resisted by a couple in the flanges.

In the following guidance the design moment M_{Ed} is the summation of the moment resulting from the frame analysis, any moment due to eccentric bracing setting out and any moment due to a shear stub welded below the base plate.



Respecting the sign convention as drawn above (force and moment are positive), the forces in the flanges are given by:

$$N_L = \frac{N_{Ed}}{2} - \frac{M_{Ed}}{(h - t_f)}$$

(as drawn, left flange)

$$N_R = \frac{N_{Ed}}{2} + \frac{M_{Ed}}{(h - t_f)}$$

(as drawn, right flange)

The values of N_L and N_R determine if the compression tension or bending is dominant.

If compression is dominant, the forces in the two T-stubs flanges are given by:

$$F_{c,Ed} = \frac{N_{Ed}}{2} + \frac{M_{Ed}}{z}$$

$$F_{t,Ed} = \frac{N_{Ed}}{2} - \frac{M_{Ed}}{z}$$

If tension is dominant, the forces in the two T-stubs flanges are given by:

$$F_{c,Ed} = \frac{-N_{Ed}}{2} + \frac{M_{Ed}}{z}$$

$$F_{t,Ed} = \frac{-N_{Ed}}{2} - \frac{M_{Ed}}{z}$$

CHECK 1 – FORCES IN THE EQUIVALENT T-STUBS (CONTINUED)

If the bending moment is dominant. the forces in the two T-stubs flanges are given by:

$$F_{c,Ed} = \frac{M_{Ed}}{z} + N_{Ed}\frac{z_T}{z}$$

$$F_{t,Ed} = \frac{M_{Ed}}{z} - N_{Ed}\frac{z_C}{z}$$

In each case, the appropriate lever arm is selected from Figure 5.4.

Additional force from bracing

If the vertical component of force in the bracing is $N_{brace,V}$ then the force in the flange is:

$$N_{brace,V} \times \frac{b_c \times t_f}{A_g}$$

where:

b_c is the width of the section

t_f is the thickness of the column flange

A_g is the gross cross sectional area of the column

Additional force from the secondary moment (if any)

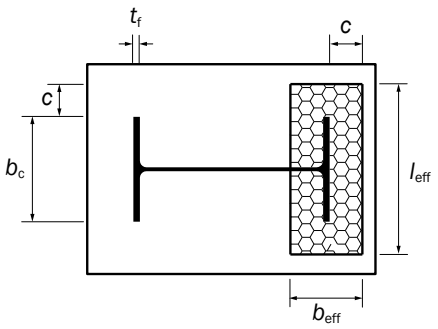
If eccentricity of the bracing produces a secondary moment, $M_{sec,brace}$ the force in the column flange is:

$$N_{sec} = \frac{M_{sec,brace}}{h - t_f}$$

CHECK 2 – RESISTANCE OF THE COMPRESSION ZONE

The required thickness of the base plate is based on an effective area subject to a uniform pressure equal to the design strength of the concrete. The effective area is generally assumed to be concentric with the compression flange. If the compression is assumed to be eccentric to the compression flange, the compression and tension forces should be re-evaluated based on revised lever arms.

If the base plate is not of sufficient size to accommodate the calculated additional bearing width, *c*, inadmissible areas are neglected and the additional bearing width *c* increased around other parts of the section perimeter to compensate.



The design compression resistance of the foundation is given by:

$F_{C,Rd} = f_{jd}\, b_{eff}\, l_{eff}$

f_{jd} is the design strength of the joint as described in Section 2.2

$= \beta_j\, \alpha\, f_{cd}$

$b_{eff} = 2c + t_c$ (unless limited by plate dimensions)

$l_{eff} = 2c + b_c$ (unless limited by plate dimensions)

The minimum plate thickness is given by:

$$t_p = c\sqrt{\frac{3\gamma_{M0} f_{jd}}{f_{yp}}}$$

Other variables are as previously described.

CHECK 3 – RESISTANCE IN THE TENSION ZONE

The resistance of the holding down bolts, and the holding down bolts in combination with bending of the base plate should be verified as described in Section 3.2.

CHECK 4 – RESISTANCE OF WELDS IN TENSION

The design force for the weld may be taken as

$$a \geq \frac{\sqrt{3}\beta_w \gamma_{M2}}{f_u}\left(\frac{P_T}{K}\right) \text{ but } a \geq \frac{\gamma_{M2} P_T \sin\theta}{0.9 f_u}$$

where:

a is the throat thickness

K $= 1.225$ when the weld is transverse and $\theta = 45°$

P_T is the transverse force on the weld, in N/mm.

All other variables are as previously defined.

REFERENCES

[1] BS EN 1993-1-8:2024
Eurocode 3 – Design of steel structures Part 1-8 Joints
BSI, 2024

[2] UK National Annex to BS EN 1993-1-8:2024 (not yet published)

[3] BS EN 1993-1-8:2005
Eurocode 3: Design of steel structures- Part 1-8: Design of joints
BSI, 2005

[4] BS EN 1992-4:2018
Eurocode 2. Design of concrete structures – Design of fastenings for use in concrete
BSI, 2018

[5] NA to BS EN 1993-1-8:2005
UK National Annex to Eurocode 3: Design of steel structures Part 1-8: Design of joints
BSI, 2008

[6] National Structural Steelwork Specification for Building Construction
7th Edition, 1st Revision
BCSA, 2023

[7] BS 7419:2012 Specification for holding down bolts
BSI, 2012

[8] BS EN ISO 4042:2022
Fasteners. Electroplated coating systems
BSI, 2022

[9] BS EN ISO 10684:2004
Fasteners. Hot dip galvanised coatings
BSI, 2003

[10] BS 7371-8:2011
Coatings on metal fasteners – Specification for sherardized coatings
BSI, 2011

[11] Brown, D. G. & Cosgrove, T. C
Brittle fracture: Selection of steel sub-grade to BS EN 1993-1-10 (P419)
SCI, 2017

[12] BS EN 1993-1-10:2025
Eurocode 3. Design of steel structures – Material toughness and through-thickness properties
BSI, 2025

[13] PD 6695-1-10:2009
Recommendations for the design of structures to BS EN 1993-1-10
BSI, 2009

[14] BS EN ISO 7091:2000
Plain washers. Normal series. Product grade C
BSI, 2000

[15] BS 4320:1968
Specification for metal washers for general engineering purposes. Metric series
BSI, 1968 (withdrawn 2021)

[16] BS EN 1090-2:2018+A1:2024
Execution of steel structures and aluminium structures – Technical requirements for steel structures
BSI, 2024

[17] BS EN 1992-1-1:2023
Eurocode 2. Design of concrete structures – General rules and rules for buildings, bridges and civil engineering structures
BSI, 2023

[18] BS EN 1993-1-1:2022
Eurocode 3. Design of steel structures – General rules and rules for buildings
BSI, 2023

[19] BCSA Guide to steel erection in windy conditions, Publication 39/05,
BCSA, 2005

[20] Design Guide 1 Base Connection Design for Steel Structures Third Edition
AISC, 2024

[21] NA to BS EN 1993-1-1:2022
UK National Annex to Eurocode 3: Design of steel structures Part 1-1: General rules and rules for buildings
BSI, 2026

[22] NA+A2:14 to BS EN 1992-1-1:2004+A1:2014
UK National Annex to Eurocode 2. Design of concrete structures – General rules and rules for buildings
BSI, 2005

[23] Simpson, D. Holding Down bolts to Eurocode 2
Concrete Advice No. 42
Concrete Society, 2017

[24] BS EN 1990-1:2023+A1:2026
Eurocode - Basis of structural design and geotechnical design. Part1: New structures
BSI, 2026

WORKED EXAMPLE 1

Steel Knowledge CALCULATION SHEET	Job No.: CDS102		
	Job Title: Design of column bases		
	Subject: Column subjected to axial compression and shear		
	Client: BCSA	Made by: BZB	Date: Jun 2024
		Checked by: DGB	Date: Feb 2025

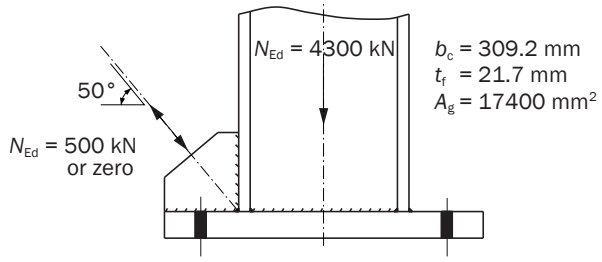
1 COLUMN SUBJECTED TO AXIAL COMPRESSION AND SHEAR (NO SHEAR STUB)

1.1 Base details

Column	305 × 305 × 137 UC Grade S355
Base plate	Grade S355
Concrete grade	C30/37
Design compression force in column	N_{Ed} = 4300 kN
Axial force in bracing member	N_{Ed} = ± 500 kN, or zero

In this example, the setting out of the vertical bracing has been modified so that the bracing gusset plate can be compact and supported on two edges. This eccentricity introduces a moment which has been allowed for in the design of the base plate.

If the bracing is set out to intersect with the column centreline at the underside of the base plate, no moment need be allowed for.

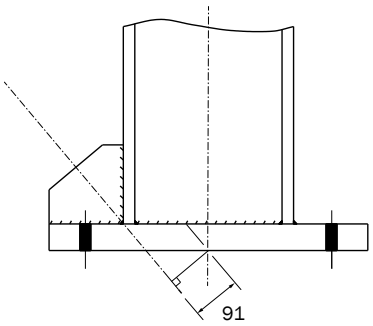


Vertical component of bracing force = ±500 sin 50 = ±383 kN

Horizontal component of bracing force = ±500 cos 50 = ±321 kN

The additional force in a column flange, $N_{brace,V} = \pm 383 \times \frac{309.2 \times 21.7}{17400} = \pm 148$ kN

Assuming a 50 mm base plate, the line of action of the bracing force is 91 mm from the centreline of the column at the underside of the base plate.



The additional moment due to eccentricity of bracing $= 500 \times 10^3 \times 91 \times 10^{-6} = 45.5$ kNm

The additional force in a flange due to the moment,

$$N_{sec} = \frac{\pm 45.5}{h - t_f} = \frac{\pm 45.5 \times 10^6}{320.5 - 21.7} \times 10^{-3} = \pm 152 \text{ kN}$$

The axial force in the column may be assumed to be distributed across the cross section in proportion to the area of the element.

The axial force in a column flange, $N_{direct} = 4300 \times \frac{309.2 \times 21.7}{17400} = 1658$ kN

With compression in the bracing member, the maximum compression in a flange

$= 1658 + 148 + 152 = 1958$ kN

With tension in the bracing member, the minimum net compression in a flange

$= 1658 - 148 - 152 = 1358$ kN (no uplift)

accompanied in each case with 321 kN shear on the base

1.2 Base in compression

With zero force in the bracing member, the design compression in the column is 4300 kN and the design compression force in one flange is 1658 kN

With compression in the bracing, the maximum design compression force in one flange $= 1958$ kN

The compressive stress in the most heavily loaded flange is conservatively assumed to be uniform across the entire column cross section to determine an equivalent axial compression.

Equivalent axial force $= \frac{1958 \times 10^3}{b \times t_f} \times A_{col} = \frac{1958 \times 10^3}{309.2 \times 21.7} \times 17400 \times 10^{-3} = 5081$ kN

1.2.1 Check 1 – Required area

Design compressive strength of concrete

$$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c} = \frac{0.85 \times 30}{1.5} = 17 \text{ N/mm}^2$$

$$f_{jd} = \alpha \beta_j f_{cd} = \frac{1.5 \times 2}{3} \times 17 = 17 \text{ N/mm}^2$$

Assuming
$\alpha = 1.5$
$\beta_j = 2/3$

1.2.2 Check 2 – Effective area

Area required $A_{req} = \frac{N_{Ed}}{f_{jd}} = \frac{5081 \times 10^3}{17} = 299000\,mm^2$

Effective area $A_{eff} = 4c^2 + c\,P_{col} + A_{col}$

Column perimeter $P_{col} = 1820$ mm

Area of column $A_{col} = 17400\ mm^2$

$A_{eff} = 4c^2 + 1820c + 17400$

$A_{eff} = A_{req}$

therefore $299000 = 4c^2 + 1820c + 17400$

$0 = 4c^2 + 1820c - 297260$

solving, $c = 128$ mm

To check that there is no overlap c must be less than half the depth between flanges:

$$\frac{h - 2t_f}{2} = \frac{320.5 - 2 \times 21.7}{2} = 138.6 \text{ mm} > 128 \text{ mm}$$

The assumption of no overlap is correct.

Minimum size of baseplate:

$h + 2c = 320.5 + 2 \times 128 = 577$ mm

$b + 2c = 309.2 + 2 \times 128 = 568$ mm

Adopt 600 x 600 baseplate.

Required thickness, $t_p \geq c\sqrt{\frac{3\gamma_{M0} f_{jd}}{f_{yp}}}$

Anticipating the thickness may exceed 40 mm, $f_y = 335\ N/mm^2$

$$t_p \geq 128\sqrt{\frac{3 \times 1.0 \times 17}{335}} = 49.9 \text{ mm}$$

Adopt a 50 mm base plate.

1.3 Base in shear

Design shear force $V_{Ed} = 321$ kN

1.3.1 Plate Friction

Assuming $C_{f,d} = 0.3$ (BS EN 1993-1-8:2024)

Minimum design value of compressive force in the column,

$N_{c,Ed} = 4300 - 383 = 3917$ kN

Base plate friction

$F_{f,Rd} = C_{f,d}\ N_{c,Ed} = 0.3 \times 3917 = 1175$ kN > 321 kN, OK

Since in this case the frictional resistance is more than the applied shear force, no contribution from the holding down bolts is required.

The four holding down bolts should be verified for the temporary condition.

1.3.2 Welds

The column is assumed to be in direct bearing with the base plate, so no weld design for the compression force is required. The longitudinal welds to the web must be verified for the applied shear force of 321 kN.

Length of weld = 246 × 2 = 492 mm

Note no reduction is required at the ends of the weld run, as the welds are continuous around the root radii and flanges.

Adopt 6 mm leg length fillet weld all round Blue Book

Load/mm = 321 / 492 = 0.65 kN/mm

Resistance of a 6 mm leg length longitudinal weld in S355 = 1.01 kN/mm, OK

1.3.3 Alternative design shear resistance, neglecting friction

If the design friction resistance were insufficient, the holding down bolts may be used in combination with friction, but only if the holes in the base plate are not oversize.

If friction were to be neglected, the holding down bolts may be utilised to carry the design shear. If the holes are oversize (the usual condition in the UK), it is recommended that only half the holding down bolts be assumed to be effective in transferring shear.

In this example, as the holes are oversize, the resistance of only two holding down bolts is verified.

Assume the holding down bolts are M24, property class 8.8.

Shear resistance per bolt, $F_{vb,Rd} = \frac{\alpha_{bc} f_{ub} A_s}{\gamma_{M2}}$

where $\alpha_{bc} = 0.44 - 0.0003 f_{yb} = 0.44 - 0.0003 \times 0.8 \times 800 = 0.25$

$$F_{vb,Rd} = \frac{\alpha_{bc} f_{ub} A_s}{\gamma_{M2}} = \frac{0.25 \times 800 \times 353}{1.25} \times 10^{-3} = 56.5 \text{ kN}$$

In this case, with holding down bolts in oversize holes, two holding down bolts provide a resistance of 113 kN, which is unsatisfactory.

Try M30, property class 8.8 holding down bolts.

$$F_{vb,Rd} = \frac{\alpha_{bc} f_{ub} A_s}{\gamma_{M2}} = \frac{0.25 \times 800 \times 561}{1.25} \times 10^{-3} = 89.8 \text{ kN}$$

Four M30 holding down bolts would provide a resistance of 359 kN, but to utilise the contribution from all four holding down bolts, the holes in the base plate would need to be normal clearance holes, or the annulus around the holding down bolts filled with resin, or washer plates with normal holes welded to the top of the base plate after erection.

WORKED EXAMPLE 2

Steel Knowledge CALCULATION SHEET	Job No.: CDS102		
	Job Title: Design of column bases		
	Subject: Column subjected to axial compression and shear (with shear stub)		
	Client: BCSA	Made by: BZB	Date: Jun 2024
		Checked by: DGB	Date: Feb 2025

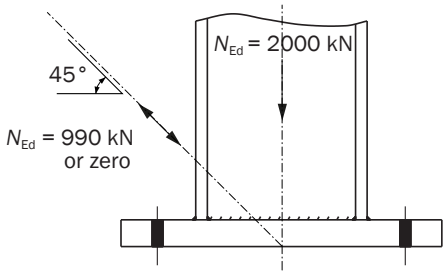
1 COLUMN SUBJECTED TO AXIAL COMPRESSION AND SHEAR (WITH SHEAR STUB)

1.1 Base details

Check the column base for the following:

Column	305 × 305 × 137 UC Grade S355
Base plate	Grade S355
Concrete grade	C30/37
Design compression force in column	N_{Ed} = 2000 kN
Axial force in bracing member	N_{Ed} = ± 990 kN, or zero

In this example, the setting out assumes that the bracing intersects with the underside of the base on the centreline of the column. There is no eccentricity in this case and the bracing does not produce a bending moment in the base. Note that the base will be designed for a moment arising from the eccentricity of the horizontal reaction on the shear stub.



From the bracing:

Vertical component = ±990 sin 45 = ±700 kN

Horizontal component = ±990 cos 45 = ±700 kN

The axial load in a column flange may be determined on the basis of the cross sectional area of a flange.

$$N_{direct} = 2000 \times \frac{309.2 \times 21.7}{17400} = 771 \text{ kN}$$

The additional force in a column flange, $N_{brace,V} = \pm 700 \times \frac{309.2 \times 21.7}{17400} = \pm 270$ kN

The bracing is not eccentric, so there is no additional moment and no additional force in the column flanges.

With compression in the bracing member, maximum net compression in one flange

$= 771 + 270 = 1041$ kN

With tension in the bracing member, minimum net compression

$= 771 - 270 = 501$ kN (no uplift)

accompanied in each case with 700 kN shear

1.2 Base in compression

With zero force in the bracing member, the design compression in the column is 771 kN

Maximum design compression force in one flange $N_{Ed} = 1041$ kN

The compressive stress in the most heavily loaded flange is conservatively assumed to be uniform across the entire column cross section to determine an equivalent axial compression.

Equivalent axial force $= \frac{1041 \times 10^3}{b \times t_f} \times A_{col} = \frac{1041 \times 10^3}{309.2 \times 21.7} \times 17400 \times 10^{-3} = 2700$ kN

1.2.1 Check 1 – Required area

Design compressive strength of concrete

$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c} = \frac{0.85 \times 30}{1.5} = 17$ N/mm²

$f_{jd} = \alpha \beta_j f_{cd} = \frac{1.5 \times 2}{3} \times 17 = 17$ N/mm²

Assuming
$\alpha = 1.5$
$\beta_j = 2/3$

1.2.2 Check 2 – Effective area

Area required $A_{req} = \frac{N_{Ed}}{f_{jd}} = \frac{2700 \times 10^3}{17} = 159000$ mm²

To calculate the effective area, assume first that there is no overlap.

Effective area $A_{eff} = 4c^2 + c\,P_{col} + A_{col}$

Column perimeter $P_{col} = 1820$ mm

Area of column $A_{col} = 17400$ mm²

$A_{eff} = 4c^2 + 1820c + 17400$

$A_{eff} = A_{req}$

therefore $159000 = 4c^2 + 1820c + 17400$

$0 = 4c^2 + 1820c - 141600$

solving, $c = 68$ mm

By inspection, there is no overlap between the flanges and the effective area falls within the area of the base plate.

Adopt 550 x 550 baseplate.

1.3 Base in shear

Design shear force $V_{Ed} = 700$ kN

1.3.1 Plate Friction

Minimum compression $= 2000 - 700 = 1300$ kN

Base plate friction $F_{f,Rd} = C_{f,d} N_{Ed} = 0.3 \times 1300 = 390$ kN

1.3.2 Bolt shear

Assume the holding down bolts are M24, property class 8.8.

Shear resistance per bolt, $F_{vb,Rd} = \frac{\alpha_{bc} f_{ub} A_s}{\gamma_{M2}}$

where $\alpha_{bc} = 0.44 - 0.0003 f_{yb} = 0.44 - 0.0003 \times 0.8 \times 800 = 0.25$

$F_{vb,Rd} = \frac{\alpha_{bc} f_{ub} A_s}{\gamma_{M2}} = \frac{0.25 \times 800 \times 353}{1.25} \times 10^{-3} = 56.5$ kN

If the holding down bolts were in ordinary holes, or used oversize holes and had washer plates welded after erection, the contribution from four holding down bolts would be:

$4 \times 56.5 = 226$ kN

The total resistance of friction and holding down bolts

$= 390 + 226 = 616$ kN, which is insufficient.

The use of more holding down bolts or larger holding down bolts could be considered and verified. In this instance a shear stub welded to the underside of the base is preferred.

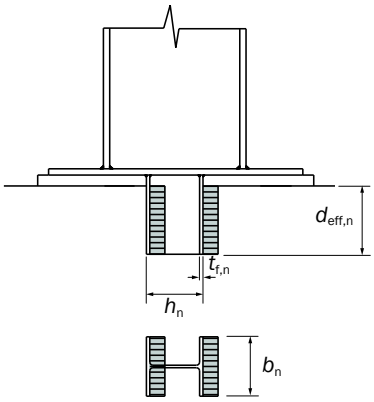
In practice, ordinary holes in base plates are not preferred, nor is welding washer plates after erection, meaning that the friction resistance cannot be combined with the bolt resistance.

If the holding down bolts in oversize holes are utilised in transferring shear, unless special measures are taken, it may be assumed that half of the holding down bolts are in bearing and are effective in transferring shear. In this example, many holding down bolts would be required, making a base stub a preferred solution.

1.4 Shear stub

1.4.1 Initial sizing

If a universal column section is selected for the shear stub, both flanges are assumed to transfer load.



As an initial estimate, if $d_{eff,n} = b_n$ then

$d_{eff,n}^2 \times 17 \times 2 > 700 \times 10^3$

$d_{eff,n} > 143$ mm

A 152 UC section, with 150 depth within the pocket is an initial choice.

1.4.2 Shear and bending resistance of the shear stub

The shear resistance of the stub must exceed 700 kN

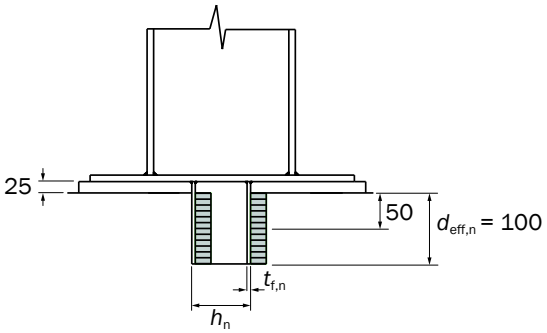
This exceeds the resistance of all S355 152 UC sections.

Try 254 UC 132, since $V_{c,Rd} = 918$ kN > 700 kN, OK — Blue Book

As the shear stub is 254 mm wide, the minimum effective embedment, $d_{eff,n}$

$$= \frac{700 \times 10^3}{2 \times 254 \times 17} = 81 \text{ mm}$$

Assume 100 mm effective embedment.



1.5 Secondary moment M_{sec}

Assuming a 25 mm grout layer, and 100 mm effective embedment within the pocket, the resulting moment M_{sec} at the underside of the base plate is given by:

$$M_{sec} = 700 \times 10^3 \times \left(25 + \frac{100}{2}\right) \times 10^{-6} = 52.5 \text{ kNm}$$

For a 254 UC 132 in S355, $M_{c,y,Rd} = 645$ kNm, > 52.5 kNm, OK — Blue Book

Although the applied shear is greater than half the plastic shear resistance of the stub, by inspection the reduction in moment resistance is not significant.

1.5.1 Welds between shear stub and underside of base plate

Axial force in each flange of the shear stub

$$= \frac{52.5 \times 10^6}{276.3 - 25.3} \times 10^{-3} = 209 \text{ kN}$$

Flange weld (a transverse weld)

$$\text{Force in weld} = \frac{209}{2 \times 254} = 0.41 \text{ kN/mm}$$

Web weld (a longitudinal weld)

$$\text{Force in weld} = \frac{700}{2 \times 226} = 1.55 \text{ kN/mm}$$

Adopt 10 mm leg length fillet weld all round: — Blue Book
Longitudinal capacity = 1.69 kN/mm
Transverse capacity = 2.07 kN/mm

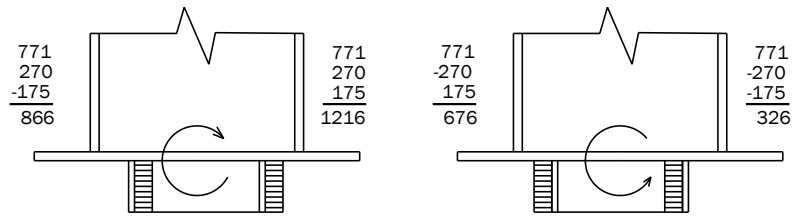
1.6 Bending moment

The secondary moment of 52.5 kNm is added to any other moments on the base. In this example, there are no other bending moments, so the total moment to be considered becomes 52.5 kNm, in combination with the axial compression of 2000 kN.

If the force in the bracing is compression, the secondary moment reduces the compression in the flange to which the bracing is attached and increases the compression in the opposite flange.

If the force in the bracing is tension, the secondary moment increases the compression in the flange to which the bracing is attached and reduces the compression in the opposite flange.

The two situations, with the resulting flange forces, are shown below.



There is no net tension. The maximum compression in a flange is 1216 kN, larger than the earlier design value of 1041 kN, so a further verification is required.

1.6.1 Check 1 – Required area

Design compressive strength of concrete:

Assuming
$\alpha = 1.5$
$\beta_j = 2/3$

$$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c} = \frac{0.85 \times 30}{1.5} = 17 \text{ N/mm}^2$$

$$f_{jd} = \alpha\beta_j f_{cd} = \frac{1.5 \times 2}{3} \times 17 = 17 \text{ N/mm}^2$$

The compressive stress in the most heavily loaded flange is conservatively assumed to be uniform across the entire column cross section to determine an equivalent axial compression.

$$\text{Equivalent axial force} = \frac{1216 \times 10^3}{b \times t_f} \times A_{col} = \frac{1216 \times 10^3}{309.2 \times 21.7} \times 17400 \times 10^{-3} = 3153 \text{ kN}$$

$$A_{req} = \frac{N_{Ed}}{f_{jd}} = \frac{3153 \times 10^3}{17} = 185500 \text{ mm}^2$$

1.6.2 Check 2 – Effective area

$A_{eff} = A_{req}$

therefore $185500 = 4c^2 + 662c + 6710$

$0 = 4c^2 + 662c - 178790$

solving, $c = 83$ mm

1.6.3 Check 3 – Plate thickness

Required thickness, $t_p \geq c\sqrt{\frac{3\gamma_{M0} f_{jd}}{f_{yp}}}$

Anticipating that the thickness may exceed 16 mm, $f_y = 345$ N/mm^2

$$t_p \geq 83\sqrt{\frac{3 \times 1.0 \times 17}{345}} = 32 \text{ mm}$$

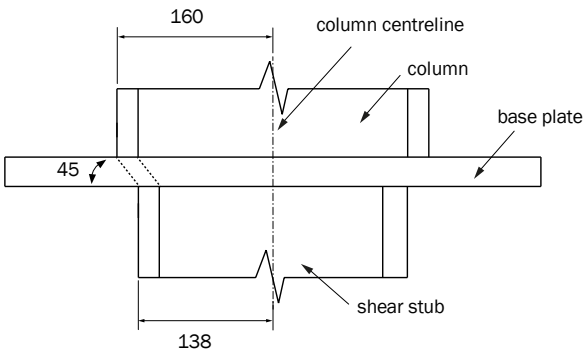
Adopt a 35 mm base plate

Minimum length of base plate $= 320 + 2 \times 83 = 486$ mm

Adopt 550 × 550 plate (primarily to provide at least 100 mm around the section to locate the holding down bolts)

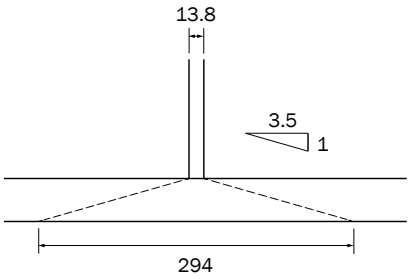
1.6.4 Verification of column web above the shear stub flanges

In this particular example, the alignment of the flanges of the column and the flanges of the shear stub are so close that the forces in the flanges of the shear stub may be transferred directly to the flanges of the column.



This example continues by demonstrating the calculation process for a much larger column where the web must be verified. The calculations assume a column web 13.8 mm thick and a 40 mm base plate.

Effective breadth of shear stub flange

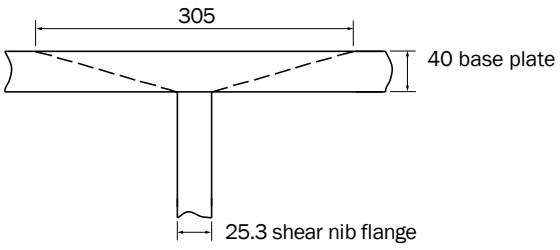


Effective breadth $= 13.8 + 7 \times 40 = 294$ mm

As the flange of the shear stub is 261 mm wide, < 294 mm, the entire width is effective.

Distribution of shear stub flange force into the column web

The distribution of force into the column web may be assumed at 1:3.5 through the base plate.



Effective length $= 25.3 + 40 \times 3.5 \times 2 = 305$ mm

Column web

Direct stress from force in shear stub flange, distributed through the base plate

$$= \frac{209 \times 10^3}{305 \times 13.8} = 50 \text{ N/mm}^2$$

Axial stress in the column web $= \frac{2000 \times 10^3}{17400} = 115$ N/mm^2

Total longitudinal stress, $\sigma_{e,Ed} = 50 + 115 = 165$ N/mm^2

In this example it is assumed that the gusset plate connection has been designed assuming that the horizontal welds to the base plate transfer the horizontal component of the bracing force.

In these circumstances, shear stress in the web can be neglected.

The verification is simply that $\sigma_{e,Ed} < f_y$, which is satisfied.

Column web weld to base plate

In this example, the longitudinal force in web weld is zero, since all the shear force is assumed to be transferred to the base plate via the gusset plate and welds.

Transverse force on weld, local to the flanges of the stub,

$$F_{w,T,Ed} = \frac{209 \times 10^3}{2 \times 305} = 343 \text{ N/mm}$$

For welds in S355, $\beta_w = 0.9$

Then minimum throat dimension,

$$a = \frac{\sqrt{3} \times 0.9 \times 1.25}{470}\left(\left(\frac{343}{1.225}\right)^2 + 0^2\right)^{0.5} = 1.2 \text{ mm}$$

but greater than

$$\frac{1.25 \times 343 \times \sin 45}{0.9 \times 470} = 0.72 \text{ mm}$$

Adopt 6 mm leg length fillet weld (throat = 4.24 mm) all round the section.

Note that as there is no net tension, the weld around the flanges is not designed since the compression forces are assumed to be transferred in direct bearing. Minimum 6 mm welds around the flanges are adopted.

The weld resistance in the temporary case must be verified.

WORKED EXAMPLE 3

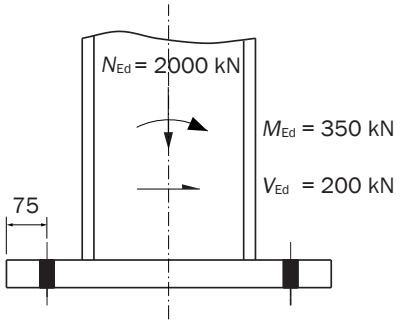
Steel Knowledge CALCULATION SHEET	Job No.: CDS102		
	Job Title: Design of column bases		
	Subject: Column subjected to combined axial compression and bending moment		
	Client: BCSA	Made by: BZB	Date: Jun 2024
		Checked by: DGB	Date: Feb 2025

1 COLUMN SUBJECTED TO COMBINED AXIAL COMPRESSION AND BENDING MOMENT

1.1 Base details

Column	305 × 305 × 137 UC Grade S355
Base plate	Try 600 × 600 Grade S355
Concrete grade	C30/37
Design compression force	N_{Ed} = 2000 kN
Design shear force	V_{Ed} = 200 kN
Design bending moment	M_{Ed} = 350 kNm

It should be noted that the temporary case may be more onerous than the final design condition.



The design situation depends on the resultant forces in each flange.

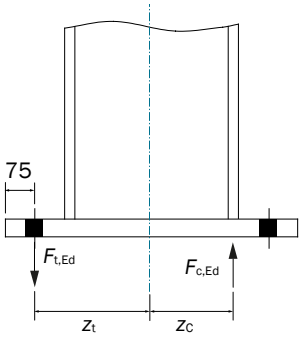
As drawn, the forces in the left and right flanges are:

$$N_{L,f} = \frac{N_{Ed}}{2} - \frac{M_{Ed}}{(h - t_f)} = \frac{2000}{2} - \frac{350 \times 10^3}{(320.5 - 21.7} = -171.3 \text{ kN}$$

$$N_{R,f} = \frac{N_{Ed}}{2} + \frac{M_{Ed}}{(h - t_f)} = \frac{2000}{2} - \frac{350 \times 10^3}{(320.5 - 21.7} = 2171.3 \text{ kN}$$

As drawn, the left side is in tension and the right side is in compression, The bending moment is therefore dominant.

1.1.1 Tension and compression forces



The lever arms z_t and z_c are given by:

$z_t = (600 - 2 \times 75) / 2 = 225$ mm

$z_c = (320.5 - 21.7) / 2 = 149.4$ mm

$z = 225 + 149 = 374$ mm

The forces in tension and compression are:

$$F_{T,Ed} = \frac{M_{Ed}}{z} - N_{Ed}\frac{z_C}{z} = \frac{350 \times 10^3}{374} - 2000 \times \frac{149}{374} = 137 \text{ kN}$$

$$F_{C,Ed} = \frac{M_{Ed}}{z} + N_{Ed}\frac{z_T}{z} = \frac{350 \times 10^3}{374} + 2000 \times \frac{225}{374} = 2139 \text{ kN}$$

1.2 Resistance of the compression T-stub

1.2.1 Compressive resistance of the concrete

The effective bearing area around the column flange is given by:

$A_{eff} = (b_c + 2c)(t_f + 2c) = (309.2 + 2_c)(21.7 + 2_c)$

$A_{eff} = (309.2 + 2c)(21.7 + 2c)$

$A_{eff} = 4c^2 + 662c + 6710$

$$A_{req} = \frac{2139 \times 10^3}{17} = 126000 \text{ mm}^2$$

$126000 = 4c^2 + 662c + 6710$

$0 = 4c^2 + 662c - 119290$

Solving, $c = 109$ mm

Check for sufficient plate outstand.

$$\frac{600 - 320.5}{2} = 140\text{mm} > 109\text{mm, OK}$$

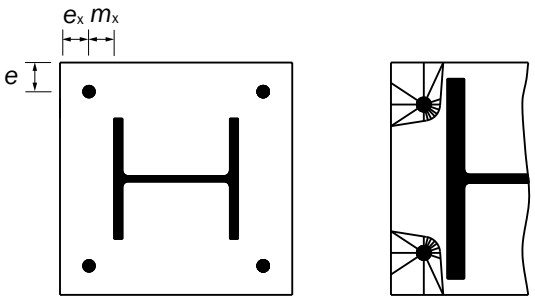
Anticipating $40 < t_p < 63$, then $f_y = 335$ N/mm^2

Plate thickness,

$$t_p \geq c\sqrt{\frac{3\gamma_{M0} f_{jd}}{f_y}} = 109 \times \sqrt{\frac{3 \times 1.0 \times 17}{335}} \quad = 42.5 \text{ mm, adopt 50 mm base plate.}$$

1.3 Resistance of the tension side

With only two holding down bolts on the tension side as shown, by inspection (in this particular case) the critical yield line pattern involves yield lines extending to both edges adjacent to the corner.



Assuming an 8 mm leg fillet weld to the flange,

$m_x = 300 - 75 - 0.5 \times 320.5 - 0.8 \times 8 = 58$ mm

From Table 3.3:

$l_{eff} = 2m_x + 0.625e_x + e = 2 \times 58 + 0.625 \times 75 + 75 = 238$ mm

$$M_{pl,Rd} = \frac{025\Sigma l_{eff} t_p^2 f_{y,p}}{\gamma_{M0}} \quad \frac{0.25 \times 238 \times 50^2 \times 335}{1.0} \times 10^{-6} = 49.8 \text{ kNm}$$

$$F_{t,pl,Rd} = \frac{2M_{pl,Rd}}{m_x} = \frac{2 \times 49.8 \times 10^6}{58} \times 10^{-3} = 1718 \text{ kN}, > 137 \text{ kN, OK}$$

With 2 No. M24 property class 8.8 holding down bolts,

$F_{t,b,Rd} = n\, F_{t,Rd} = 2 \times 203 = 813.2$ kN

Blue Book

$F_{t,R,Rd} = 406$ kN > 137 kN, OK

The resistance of the anchor plates to pull out must also be verified.

1.3.1 Resistance of weld in tension

Assuming the tension force is resisted by (in this instance) the weld on the outside of the column flange,

Load on weld $= 137/309 = 0.44$ kN/mm

For an 8 mm leg length fillet weld in S355, $F_{w,T,Rd} = 1.75$ kN/mm (a transverse weld)

1.4 Shear resistance

1.4.1 Plate friction and holding down bolts

If the axial compression and bending moment are always concurrent, then the design value of compressive force in the column, $N_{c,Ed} = 2171$ kN

Conservatively assuming that the compression force may apply independently of the bending moment, the design value of compressive force in the column,

$N_{c,Ed} = 1000$ kN

Assuming $C_{f,d} = 0.3$ (BS EN 1993-1-8:2024)

Base plate friction $F_{f,Rd} = C_{f,d}\ N_{c,Ed} = 0.3 \times 1000 = 300$ kN > 200 kN, OK

In this instance, the holding down bolts need not be assumed to transfer shear.

1.4.2 Weld in shear

Assuming the shear force is resistance by the two welds along the web of the UC,

Load on weld $= 200/(2 \times 247) = 0.41$ kN/mm

For an 8 mm leg length fillet weld in S355, $P_{L,Rd} = 1.4$ kN/mm (a longitudinal weld) — Blue Book

WORKED EXAMPLE 4

SCI Steel Knowledge CALCULATION SHEET	Job No.: CDS102		
	Job Title: Design of column bases		
	Subject: Verification of base detail in the temporary condition		
	Client: BCSA	Made by: DGB	Date: Feb 2025
		Checked by: BZB	Date: March 2026

1 VERIFICATION OF BASE DETAIL IN THE TEMPORARY CONDITION

1.1 Base details (as Example 3)

Column	305 × 305 × 137 UC Grade S355
Base plate	600 × 600 × 50, S355
Holding down bolts	M24, property Class 8.8
Column height	12 m
Concrete grade	C30/37 (but only 7 days old)

1.2 Design loading

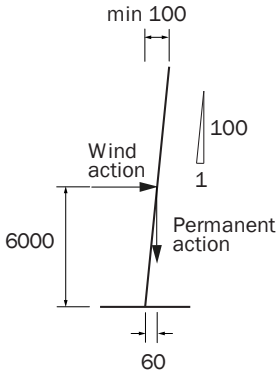
The column is assumed to be inclined at 1 in 100, with a minimum 100 mm offset at the top.

The peak velocity pressure has been determined as 0.56 kN/m^2. This exceeds the recommended minimum of 0.55 kN/m^2.

The peak velocity pressure included a probability factor p_{prob} = 0.85, based on a 5 year return period.

A dynamic amplifier of 1.2 is applied to the calculation of the overturning moment due to wind.

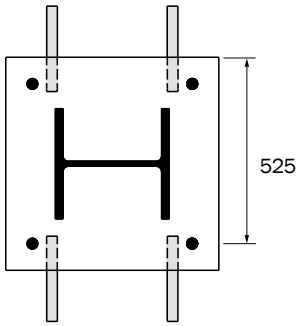
The overturning moment leads to tension on one side of the base and compression on the other.



Column self weight = 137 × 12 × 9.81 × 10^{-3} = 16.1 kN

From BS EN 1991-1-4, the force coefficient $c_{f,0}$ has been determined to be 2.1, based on $d = b$.

Design value of the bending moment at the base
$= 1.35 \times 16.1 \times 0.06 + 1.2 \times 1.5 \times 2.1 \times 0.56 \times 0.32 \times 12 \times 6 = 50.1$ kNm



Design tension in each bolt $= \frac{50.1}{2 \times 0.525} = 47.8$ kN

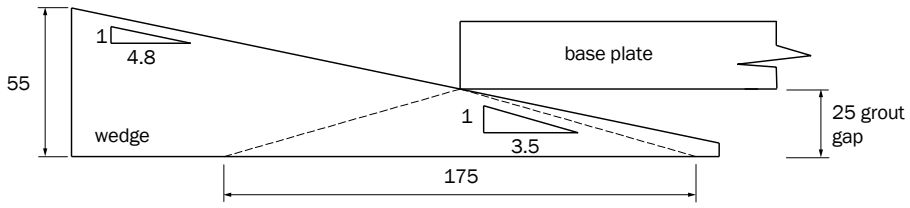
Assuming 2 No. M24 property class 8.8 holding down bolts, positioned as Example 3 in a 50 mm, S355 base plate, the resistance of each bolt is 203 kN > 47.8 kN, OK

The resistance of the anchor plate to pull out in the 7 day concrete must also be verified.

Resolving vertically, the total compression is given by:

$F_{c,Edd} = 2 \times 47.8 + 1.35 \times 16.1 = 117.3$ kN, or 58.7 kN per wedge

With a 25 mm grout gap, and a wedge detailed as shown, the length of wedge bearing on the concrete is taken as 175 mm



The 7 day strength is assumed to be 2/3 of the cylinder strength,

$\frac{2}{3} \times 30 = 20$ N/mm^2

Therefore, $f_{jd} = 0.85 \times \frac{20}{1.5} = 11$ N/mm^2

If the wedges are fabricated from 40 mm thick material, the design resistance in bearing under each wedge $= 11 \times 40 \times 175 \times 10^{-3} = 77$ kN > 58.7 kN, OK

The resistance in the orthogonal direction should also be verified. Assuming the wind is at 45° to the orthogonal directions will generally be a more onerous situation. Additional wedges may be introduced on the sides of the base plate parallel to the column flanges. The foundation must have sufficient resistance to accommodate the overturning moment.

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JOINTS IN STEEL CONSTRUCTION: COLUMN BASES

This publication covers the design of steel bases for nominally pinned and moment-resisting bases, including the verification of shear stubs. The design guidance is in accordance with BS EN 1993-1-8:2024. Numerical examples are included which demonstrate the design procedures for different types of base.

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